# Comprehensive Evaluation of Wisconsin Roundabouts
# Volume 2: Traffic Safety

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By

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16. Abstract


While roundabouts are still fairly new in the U.S. and Wisconsin, their safety benefits have been studied with varied results. For this study, researchers analyzed 24 Wisconsin roundabouts that were built in 2007 or before. Three years of before and after crash data were gathered as well as geometric and volume data. An Empirical Bayes (EB) analysis was used to examine the safety benefits for total crashes and injury (K, A, B, C) crashes. A simple before-and-after crash analysis was also completed to analyze specific types of injury crashes for each roundabout.

The EB analysis was performed using Safety Performance Functions (SPFs) from both the Highway Safety Manual (HSM) and Wisconsin specific data. The results using HSM and Wisconsin values were very similar, supporting the consistency between SPFs. Considering only the HSM SPFs, researchers found mixed results for total crash frequency but a significant decrease in crash severity. Wisconsin roundabouts had an unbiased estimate of a 9 percent decrease in total crashes with a standard error of 6 percent. National numbers similarly show decreases in total crashes. Wisconsin roundabouts showed a significant 52 percent decrease in injury crashes. Roundabouts nationwide are also experiencing a significant decrease in severe crashes.

When evaluating predictor variables, the speed limit of the approaches did not show a significant impact on the safety of the roundabout. While multi-lane roundabouts seem to be safer than single lane roundabouts when considering combined injury crashes, single lane roundabouts saw the largest decrease in total crashes. Two-way stop controlled (TWSC) intersection conversion to a roundabout had the highest safety benefit as compared to all-way stop controlled (AWSC) and signalized intersections.

Using an observational (simple) before-and-after crash analysis, researchers found that two sites in which fatal crashes were found in the before condition observed no fatal crashes after the installation of a roundabout. No sites experienced a fatal crash after a roundabout was installed. For all injury (A, B, and C) crashes, the number of locations with a reduction in these crash types were greater than the number of locations with increases in these crash types. Similarly, the magnitude of decrease in injury crashes is higher than the magnitude of increase. When examining crash type, researchers found that roundabouts changed the crash types that occur at intersections from more severe crash types like angle and head-on to less severe crash types such as sideswipe and single vehicle crashes.


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# CHAPTER 1 INTRODUCTION

In a recently published Federal Highway Administration (FHWA) memo, roundabouts are considered as one of the proven safety improvements (1). The memo is based on extensive safety research studies conducted overseas and at various locations in the United States (U.S.). Though the history of roundabout implementation in the U.S. is relatively short compared with Great Britain and other European counties, roundabouts have demonstrated a consistent trend in reducing crashes, especially in reducing severe injury crashes. A recent study of roundabouts in the U.S. identified crash reductions of approximately 35 percent for all crash types and 76 percent for fatal and injury crashes when an intersection was converted from a signal or stop control to a roundabout (2). The reason behind the large improvement in safety records at these locations lies in the design features of roundabouts that reduce conflict points as well as vehicular speeds. Roundabouts prohibit vehicles from making left-turning movements and all vehicles circulate counter-clockwise around a raised central island at a relatively low speed. The entering vehicles yield to vehicles in the roundabout, thus reducing all left-turning related crashes such as head-on or angle crashes which can result in serious injury outcomes. Lowered travel speeds also reduce the collision impact, thus reducing the crash consequence. Other design features that help to improve safety or facilitate safe movements are detailed in the FHWA Roundabout Information Guide (3).

Since the first modern roundabout was constructed, many safety evaluation studies have been conducted to quantitatively assess the safety benefits of this new intersection control strategy. The studies range from observational before-and-after to meta-analysis. However, these studies frequently show considerable differences in roundabout safety performance (2, 4, 5). Many factors can contribute to this disparity, and can be generally grouped into three categories: 1) driving population, 2) site choice, and 3) evaluation methodologies.

Though roundabouts are, by design, safer than other intersection control strategies, the safety benefits may be compromised by driver behavior. A less desirable design as well as inappropriate signage and pavement marking can also compromise the safety benefits. Roundabouts demand a high level of driver compliance to the traffic signs and judgment towards traffic conditions such as reducing their speed when approaching the roundabouts, judging a safe gap correctly, and yielding to the vehicles in the roundabouts. Roundabouts also require drivers to process more information than traditional intersections, especially in lane choice, because the lanes are not straight or perpendicular to other approaches but curved. The additional work load while driving may lead to a wrong lane choice, which contributes to same direction sideswipe crashes in the circulatory lanes. Site choice may also be critical because some roundabouts are constructed due to operational benefits of increasing capacity, reducing delay, improving flow continuity, environmental considerations, and others. For these roundabouts, safety benefits are not apparent. If the design of roundabout fails to consider particular user groups (pedestrians, bicyclists, visually impaired users, etc.) and special vehicle types (large trucks), safety may also be jeopardized if these populations are prevalent (6, 7). Daniels, et al. found that the variation in crash rates are mainly driven by the traffic exposure as well as vulnerable road users, who are more frequently involved in crashes at roundabouts than expected based on a sample of 90 roundabouts in Flanders, Belgium (5). Consistent data collection and evaluation methodology provide a comparable basis for the studies conducted at different times and from different areas. When performing an unbiased safety evaluation, the keys to success are data collection and selection of appropriate evaluation methodologies. Accurate data lay the foundation of a meaningful evaluation. Data collection needs to be designed for the purpose of the evaluation and more importantly, the roundabout related crashes, not just the crashes occurring at or near the roundabout. The evaluation methodologies should overcome data issues such as regression-to-the-mean, novelty effects, and others due to the short-term observations (4).

The present study is motivated by the need for a thorough and comprehensive observational before-and-after roundabout safety evaluation in Wisconsin. The first roundabout in Wisconsin was built and opened to traffic in 1999. Currently, there are approximately 200 roundabouts on the state trunk and local roads network with another 100 being planned by the end of 2015 construction season. Figure 1 shows the locations of roundabouts in Wisconsin that were built in 2008 or before. The objectives of this study are to develop unbiased, comprehensive evaluation methodologies, quantify the safety of roundabouts of various conditions, and support informed decision-making.

The report is organized into six chapters. This first chapter has presented the research problem and needs, along with the study objectives and roundabouts selected for evaluation. The second chapter presents a review of the literature. The third chapter describes the methodology. The frameworks for both the simple before-and-after analysis as well as the Empirical Bayes analysis are described. The fourth chapter explains the data collection and processing in detail. The fifth chapter presents the results and analyses. The last chapter presents the research conclusions.

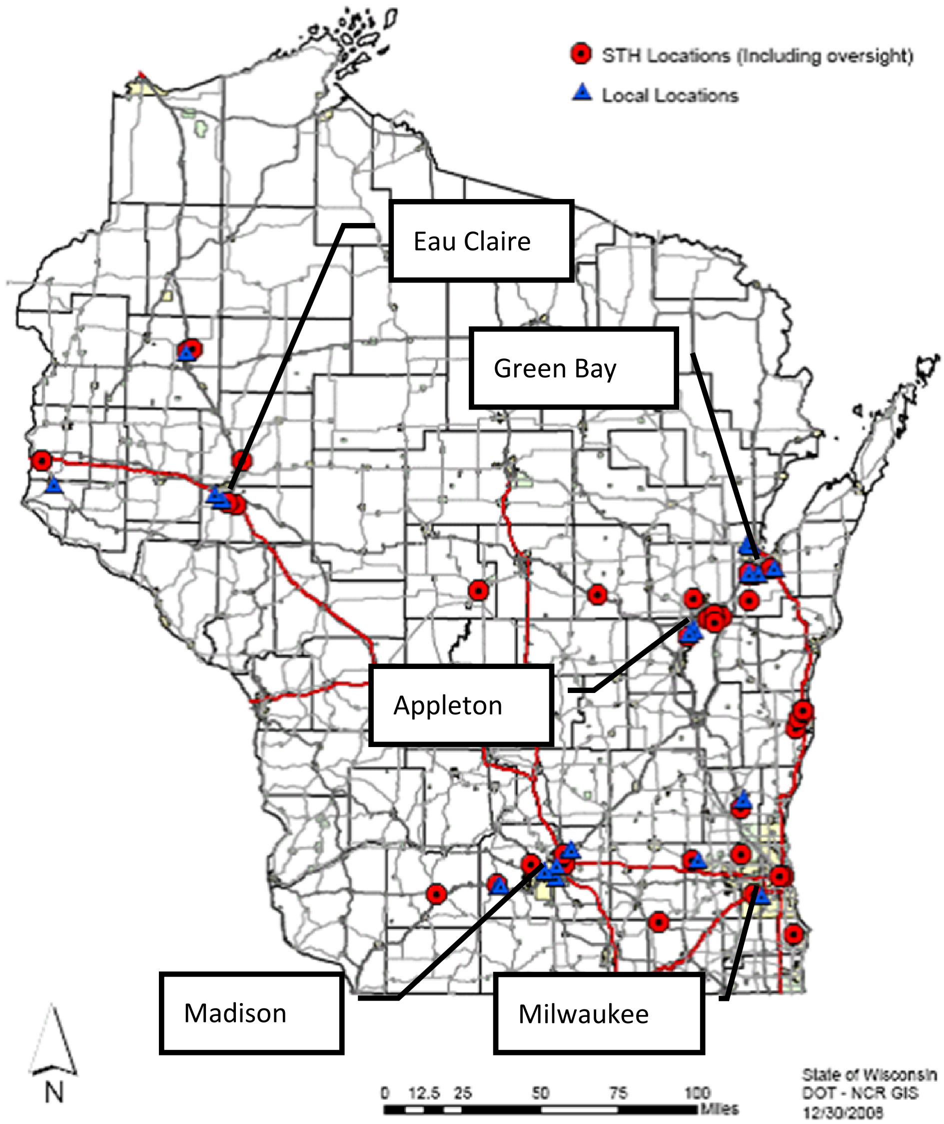


**Figure 1 Statewide Distribution of Wisconsin Roundabouts that were built in 2008.**

# CHAPTER 2 LITERATURE REVIEW

A number of research studies have shown that roundabouts are successful in reducing both crash frequency and severity. Most of the studies have been conducted internationally demonstrating that roundabouts are effective in improving safety at intersections. Robinson reported a reduction between 45 percent and 87 percent in the number of injury crashes in Australia, 57 percent and 78 percent in France, 25 percent and 39 percent in the United Kingdom (U.K.), and 51 percent in the U.S. (*16*). Furthermore, Retting reported a 76 percent reduction in injury crashes in the U.S. (*17*). A study conducted in seven U.S. states: Colorado, Florida, Kansas, Maine, Maryland, South Carolina, and Vermont, where a total of 23 intersections were converted to roundabouts, used state-of-the-art Empirical Bayes (EB) analysis to conduct before-and-after studies (4). The results revealed a 40 percent reduction in all crash severities and an overall reduction of 90 percent in injury crashes. The recently published Highway Safety Manual (HSM) includes the potential crash effects of converting a signalized intersection or a stop-controlled intersection into a modern roundabout by multiplying crash modification factors (CMFs) (*18*). Even though the CMFs vary in numbers considerably, the implications are self-evident in that roundabouts can significantly reduce crash severity and frequency.

A detailed review of the literature, looking at the effects of roundabouts on reduction of specific crash types and severity, shows several international studies but few U.S. studies. A research study in Belgium showed a 34 percent reduction in injury crashes with substantial differences related to speed limits (*19*). Other studies report that reduction in crash severity is more prominent for higher speed than lower speed intersections, or at an intersection without a traffic signal (*20, 21*). Persaud studied roundabout safety using the EB approach to account for both regression-to-the-mean and traffic volume changes that occur after converting intersections to roundabouts (4). The results showed a statistically significant reduction of 40 percent for all crash severities combined and an 80 percent decrease for all injury crashes based on a total of 23 intersection locations. In a meta-regression analysis using 28 studies, Elvik found slightly smaller safety effects when roundabouts replaced previously signalized intersections instead of intersections controlled by yield signs (59 percent instead of 64 percent for fatal and 46 percent instead of 53 percent for serious injury crashes) (*13*). The report concluded that roundabouts are clearly effective in reducing injuries and fatal crashes, but high uncertainties were found for property damage only crashes. A crucial point is that Elvik recognized that only 3 out of 28 studies used before-and-after studies with EB methodology while the rest used the simple before-and-after or cross sectional studies.

The abovementioned studies clearly demonstrate a reduction in crashes at locations where roundabouts were constructed. However, the large variability in the results suggests that safety improvements are highly localized and dependent upon the specific characteristics of each location. Therefore, there is a need to further study the safety impacts of roundabouts as new data becomes available at new roundabout locations.

The most definitive guide on roundabouts in the U.S. has been the NCHRP Report 572, which explicitly mentions the need for continuing research to gain further understanding of roundabout safety (2). The NCHRP Report 572 conducted an extensive review of the safety and operational aspects of roundabouts in the U.S. based on available nationwide data and recommended various crash prediction models for different roundabout characteristics, the specifics of which can be found in the report (2). The report also provides details on international studies such as research conducted in the 1980s by the Transportation Research group at the University of Southampton, U.K. on more than 80 four-legged roundabouts to establish safety models considering traffic volume and roundabout geometry (*22*). The study used generalized linear models to calculate crash frequency and severity models. Relationships were also established between the various geometric features of roundabouts and their effects on crash occurrences, details of which can be found in Appendix A of NCHRP Report 572 (4).

Roundabouts also change the type of crashes and manner of collisions, which is an important indicator of the consequence. Mandavilli conducted a thorough analysis of 238 crashes occurring at 38 roundabouts in Maryland by classifying crashes by movement and roundabout location (*23*). Four types of crashes were found to be a significant portion of the total: run-off-road, rear-end at entry, entering-circulating, and sideswipe same direction (in circulation). The observations were consistent with other studies even though the order of the crash type varied (*14*, *16, 22*). A recent study in Wisconsin showed that although crash severity decreased significantly at roundabout locations, the results were mixed for crash frequency (*24*).

Although most of the evidence suggests that roundabouts provide clear safety benefits, in terms of reduction in crash frequency and severity, there are several issues in regards to the safety of the road users who are vulnerable and/or visually impaired. There have been very few crash-based safety studies of those specific road users' at roundabouts because of sample size issues due to the low number of motor vehicle collisions involving such users. Especially for pedestrians with sensory or mobility impairments, pedestrian behavior and safety at roundabouts are not well understood. A study by Ashmead analyzed actual street crossings by sighted and visually impaired pedestrians at a two-lane urban roundabout under high and low traffic conditions (*25*). The research showed that visually impaired pedestrians took three times longer to cross than sighted participants. Some crossing required interventions because of danger to the participants. Wall found that crossings became increasingly difficult as vehicular volume increased and multilane roundabouts were more challenging than single-lane roundabouts in ensuring safe pedestrian access (*26*). On the other hand, Harkey and Carter studied the interactions between motorist and pedestrians at roundabouts as part of an NCHRP project and reported no substantial safety problems for non-motorists at roundabouts based on conflicts or collisions (*27*).

A potential solution to the safety issues facing vulnerable and visually impaired road users at roundabouts is the introduction of a signalized roundabout. There is very limited research in this regards and the potential for queue spillback into the circulating lane and delays to vehicular traffic is not well understood. Researchers at the Traffic Operations and Safety (TOPS) Laboratory have taken a national lead in this research effort. Research findings have shown several roundabout and traffic signal combinations that have had minimal impact on traffic operations yet provide a safe crossing environment for pedestrians and bicycles (*39, 40, 41*). Research papers on this topic have been included in Appendix D for additional reference. Note that this research has been recognized by the Transportation Research Board as a significant contribution to traffic safety and recognized through several awards.

Modern roundabouts have been promoted as a good alternative to conventional intersections due to their superior safety performance. In the past decade, the number of roundabouts in the U.S. has grown rapidly along with public interest. Numerous studies have followed and investigated roundabouts of all types and under different prevailing conditions. While successful evaluations and positive results have been reported in other states, it is unclear how these can be translated to the roundabouts in Wisconsin. It has also been acknowledged that the design and operations of a roundabout may have varying safety impacts on different road users.

# CHAPTER 3 METHODOLOGY

## Simple Before-and-After Study

The safety benefit of a treatment can be measured by a before-and-after study that calculates either the difference between or the ratio of the targeted crash frequency or rate before and after the implementation of the remedial measures, given by:

Change in safety: $\Delta$=B-A or

Ratio (also called the index of effectiveness): $\varepsilon$=B/A

Where:

B = the number of crashes occurred in the before period without the conversion, and

A = the number of crashes in the after period.

If only the number of crashes observed during the before and after analysis time period is used, the method is an observational before-and-after analysis or a "simple" before-and-after analysis. Simple before-and-after study means all the changes in crash frequency and severity can be attributed to the safety treatment. In general, positive value for the change in safety or a ratio of greater than one indicates desirable safety outcome.

## Empirical Bayes (EB) Before-and-After Study

The simple before-and-after comparison assumes that changes can be solely attributed to the safety improvement and everything else remains unchanged before and after the improvement, which is often not true. Therefore, a traffic volume adjustment is frequently deployed to normalize for differences in traffic volume between before and after periods. Moreover, the difference or ratio computed directly from the observed crash counts or rates between before and after periods may be biased as a result of regression-to-the-mean (RTM). RTM effect, or bias-by-selection, is a phenomenon that repeated measures of the data in a long run drifts towards the mean value. Due to this natural fluctuation, an extreme observation will usually be followed by a less extreme observation without any intervention. Locations slated for safety treatments usually have high crash counts, rates, or severities. A simple before-and-after analysis may inflate the countermeasure effectiveness by including the difference caused by RTM. Hauer suggested using the expected number of crashes that would have occurred in the after period had the countermeasure not been implemented as "B", which is the expected mean of a conditional (gamma) distribution of the long-term crash average of a location given the observed short-term crash history. The expected mean can be formulated as the weighted average of a predicted number of crashes and site-specific crash history as follows (9).

$$E=W\times\mu+(1-W)N \tag{1}$$

Where:

$W = \frac{1}{1+\mu Yk}$ =Weight of Prediction

E = Expected Crash Count (Estimate of Long Term Mean over Y years)

N = Observed Crashes (over Y years)

μ = Predicted Number of Crashes (SPF Calculated Value for Y years)

Y = Number of Years in Study

k = Overdispersion Parameter

The methodology of estimating the expected number of crashes is called EB analysis and the before-and-after comparison using the expected number of crashes that would have occurred in the after period without the conversion as B is called EB before-and-after analysis. Note that in the actual calculation, B is the expected average number of crashes in the after period. Any change in the traffic volume (AADT) or analysis time period needs to be factored into the comparison. An adjustment factor as shown in Equation 2 can account for these changes.

$$r_i = \left(\frac{SPF_{After}}{SPF_{Before}}\right)\left(\frac{Years_{After}}{Years_{Before}}\right) \tag{2}$$

Multiplying the 'r' factor to the EB expected number of crashes offers a correct estimate of the number of crashes that would have happened during the after time period had the treatment not been implemented.

The procedure is listed as follows:

1) Estimate EB expected average crashes in the before period for the intersection;
2) Estimate EB expected average crashes in the after period for the intersection through a traffic exposure adjustment factor $r_i$ (B);
3) Observe average crashes in the after period for the roundabout (A);
4) Calculate the change in safety by (B-A) or the safety effectiveness index (B/A); and
5) Estimate the confidence interval of the change in safety or the safety effectiveness based on all the sites evaluated.

The safety performance can be computed for individual roundabouts. When each roundabout shows varying performance, the difference or the ratio of the total number of crashes before and after the roundabout construction can provide a quantifiable mean (average) safety performance measure as well as the variance of the measurement from an overall perspective. The confidence intervals suggest the statistical significance of the estimated safety performance or indicate how certain the evaluation will be. The magnitude of the variance can be used to assist in managing the risk of investment.

The ratio or odds ratio (HSM 2010) of before and after crashes, also called index of effectiveness is formulated as in Equation 3 (*11*):

$$\theta = \frac{\sum_{\text{all sites}} N_{\text{observed,A}}}{\sum_{\text{all sites}} N_{\text{expected,A}}} \tag{3}$$

If the value of θ is smaller than 1, fewer crashes are observed than expected and the safety treatment is positive in reducing crashes. Otherwise, the safety effect is negative, which means the treatment is not effective in reducing crashes. An approximate unbiased estimate θ is given by Equation 4 (9):

$$\varepsilon = \frac{\theta}{1 + \frac{Var\left(\sum_{\text{all sites}} N_{\text{expected, A}}\right)}{\left(\sum_{\text{all sites}} N_{\text{expcted, A}}\right)^2}} \text{ where } Var\left(\sum_{\text{all sites}} N_{\text{expected, A}}\right) = \sum_{\text{all sites}}\left[r_i^2 N_{\text{expected, B}}\left(1 - w_{i,B}\right)^2\right] \tag{4}$$

Similar to θ, if the value of ε is smaller than 1, fewer crashes occur than expected. Safety effectiveness can be calculated as 100×(1-ε). A positive percentage indicates a net percentage reduction in crashes.

The variance of the unbiased estimate of θ in Equation 5 measures the estimate precision.

$$Var(\varepsilon) = \frac{\theta^2\left[\frac{1}{\sum N_{\text{observed,A}}} + \frac{Var\left(\sum_{\text{all sites}} N_{\text{expected, A}}\right)}{\left(\sum_{\text{all sites}} N_{\text{expcted, A}}\right)^2}\right]}{\left[1 + \frac{Var\left(\sum_{\text{all sites}} N_{\text{expected, A}}\right)}{\left(\sum_{\text{all sites}} N_{\text{expcted, A}}\right)^2}\right]} \tag{5}$$

$$SE(\varepsilon) = \sqrt{Var(\varepsilon)}$$

A simpler safety effectiveness measurement is the difference between expected before and after crashes given by $\Delta = \sum_{\text{all sites}}\left(N_{\text{expected, A}} - N_{\text{observed,A}}\right)$. A positive value means that observed number of crashes is fewer than expected, suggesting a net crash reduction.

The variance of Δ is given by Equation 6:

$$Var(\Delta) = Var\left(\sum_{\text{all sites}} N_{\text{expected, A}}\right) + Var\left(\sum_{\text{all sites}} N_{\text{observed, A}}\right) = Var\left(\sum_{\text{all sites}} N_{\text{expected, A}}\right) + \sum_{\text{all sites}} N_{\text{observed, A}} \quad (6)$$

$$SE(\Delta) = \sqrt{Var(\Delta)}$$

**Safety Performance Function**

A safety performance function (SPF) describes the relationship between the predicted number of crashes (dependent variable) and a set of crash contributing factors (independent variables). The state-of-the-practice distribution considered for modeling crashes is Poisson-gamma (or negative binomial (NB)) (*27, 28, 29, 30, 31, 32, 33*). Poisson-gamma models can account for over dispersion of the crash data which, if not properly considered, may lead to estimation inefficiency and inference errors. In highway safety applications the number of crashes ($N_i$) at a site 'i' is assumed to follow a Poisson distribution independently over other sites.

$$N_i | \mu_i \sim \text{Poisson}(\mu_i) \qquad i=1,2,\ldots,n \quad (7)$$

The log function used to link the mean number of crash counts with all possible covariates and unstructured errors is defined as:

$$\mu_i = (traffic\ exposure)^{\alpha} \exp(X_i\beta) \exp(e_i) \quad i = 1,2, \ldots n \quad (8)$$

If $\exp(e_i)$ is assumed to have a gamma distribution with a mean equal to 1 and variance equal to $k$ for all $i$ and independent of all the explanatory variables, the crash count derived from this Poisson-gamma process follows a negative binomial (NB) distribution which can account for overdispersion in the crash data. If we use $f(.)$ to represent the function of $(traffic\ exposure)^{\alpha} \exp(X_i\beta)$, the mean (μ) and variance (V) of the NB distribution are expressed as $f(.)$ and $f(.)[1+f(.)k]$, with $k$ as the overdispersion parameter (*33, 34, 35*). The NB distribution has a slightly complicated form to estimate the probability of a crash count using the following Equation 9:

$$f(N_i; k, \mu_i) = \frac{\Gamma(N_i + 1/k)}{\Gamma(1/k) N_i!} \left(\frac{1}{\mu_i k + 1}\right)^{1/k} \left(\frac{\mu_i k}{1 + \mu_i k}\right)^{N_i} \quad (9)$$

Where $\mu_i = (traffic\ exposure)^{\alpha} \exp(X_i\beta)$ and $X_i$ is the vector of predictor variables. $\beta$ and k are predictor variable and overdispersion factor, respectively, which can be estimated via maximum likelihood estimate (MLE) in statistical software packages such as GENMOD in SAS (*36*). Note that the formula for traffic exposure depends on the highway facility evaluated: if it is for a roadway segment, 100 million vehicle miles traveled is recommended or for an intersection or a roundabout, one million daily entering vehicles are recommended.

## Novelty Effect

In the context of human factors, the novelty effect is the tendency for performance to initially improve or reduce when new technology is instituted, not because of any actual improvement or reduction in learning or achievement. In the context of traffic control strategies, the novelty effect exists for any new modifications in the driving environment because of the time taken for drivers to be familiar with the changes or additions. However, the scale of the novelty effect varies by treatment. For a new traffic signal timing or phasing, signing or pavement marking improvements, the novelty effects may be negligible because either a driver has experienced the treatment somewhere else or the changes are too small to be noticed, but for a roundabout such assumption is not always true. One of the biggest oppositions to the construction of roundabouts is that it may be a daunting driving task to some motorists. The learning experience for drivers may be longer at roundabouts, which may compromise the roundabout safety performance. Note that the significant novelty effect induced by any new traffic control devices will have impacts on evaluation. Accounting for novelty effect will help to correctly quantify the roundabout safety effectiveness during the early stage. Moreover, it is possible that the novelty effect, a by-product of roundabout design to the drivers who have never experienced it before, can be mitigated via effective educational and outreach programs. Hence, it is necessary to estimate the magnitude of the novelty effect.

The novelty effect can be measured either by conducting a longitudinal driver interview or by observing driver erratic maneuvers over a certain time period. A driver survey was not considered for this study because of the difficultly in selecting cross-cutting subjects (drivers) and because the timing for conducting a longitudinal survey (at the inception of a roundabout) is no longer available at the time of the study. The reported crashes were used as a surrogate measure for quantifying the duration and magnitude of the novelty effect. In order to capture the novelty effect, a time index was used as a distinct variable in the analysis. Crashes were aggregated during each time period and indexed by time. A declining trend can be observed if there are detrimental effects on crashes associated with drivers' experience with roundabouts.

In this study, the history of the total number of crashes (TOTAL) and total injury crashes (KABC) during the immediate three-year time period after the 30 roundabout constructions were plotted against the nth quarter (3-month). There were 262 crashes, including 41 injury crashes, which took place at 24 roundabouts during the study time period. No novelty effect for the sampled roundabouts is observed as seen by the lack of temporal pattern as shown in Figure 2.

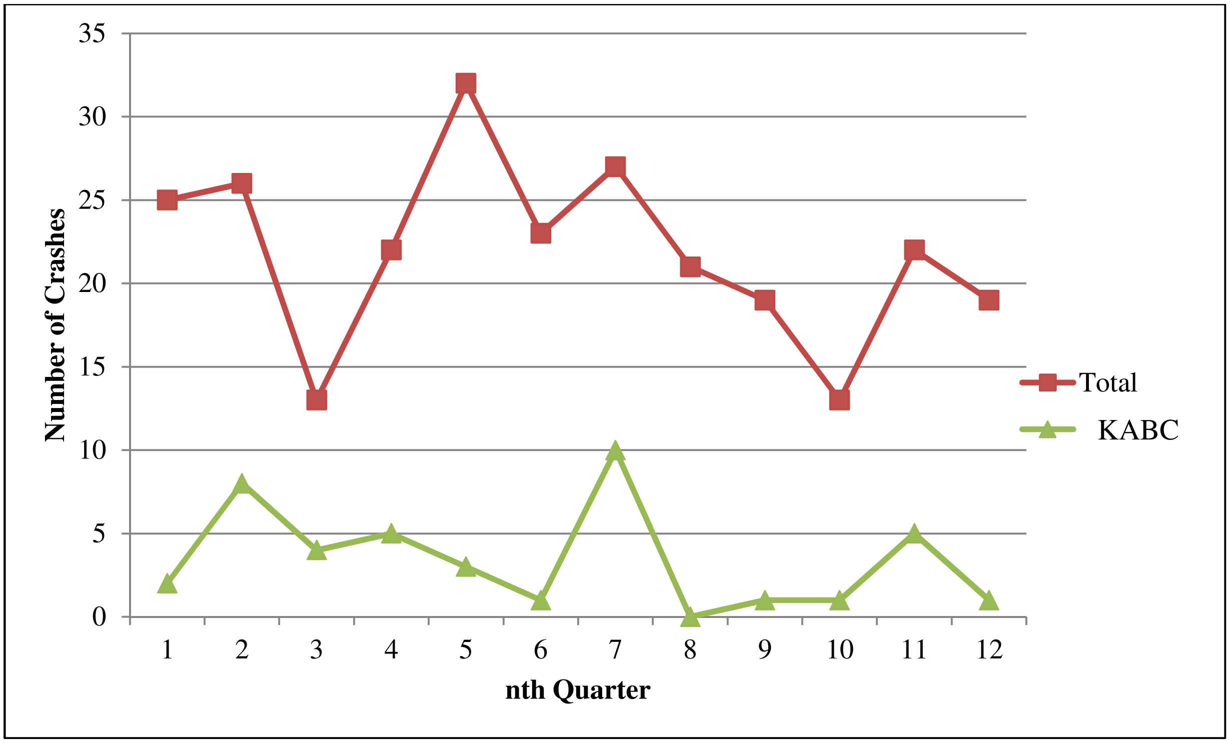


**Figure 2 Post-Construction Roundabout Crashes.**

The novelty effect can potentially be confounded by the RTM effect, which makes distinguishing the two difficult. The difference between the RTM and novelty effect is that the RTM bias is a natural statistical variation which is present all the time while novelty effect is biased because the unfamiliarity with traffic control strategies will diminish after certain time period. Without an apparent sign of novelty effect, separating the two will be a great challenge. Another noteworthy issue is the roundabout open date which, in this study, is the date the roundabout opened to traffic. The construction completion will always be after the open-to-traffic date. There will typically be top soil, seeding, and other miscellaneous work such as the completing of the curb and gutter work or paving that is finished after the open-to-traffic date. Sometimes projects are carried over to the next year to complete the loose ends before project closeout.

## CHAPTER 4 DATA COLLECTION AND PROCESSING

In order to perform a meaningful before-and-after comparison, a minimum of three years of data are required. Hence, all 30 roundabouts under Wisconsin Department of Transportation (WisDOT) oversight that were built in 2007 or before were included in the study. Figure 3 shows the locations of the thirty roundabouts included in the study.

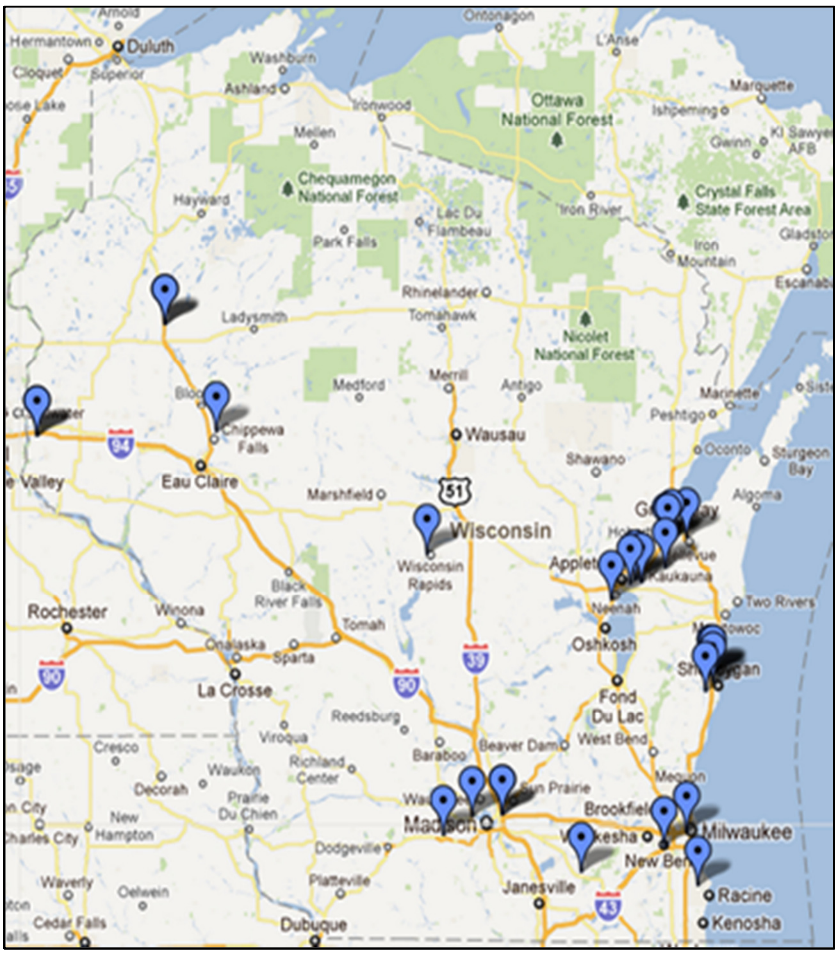


**Figure 3 Locations of Roundabouts used for this study.**

### Crash Data

Crash data for the roundabout locations were retrieved from the WisTransPortal from January 1, 1994 to December 31, 2010 (*15*). Relevant crash information was gathered based on the time when a crash occurred and the study area defined for a particular roundabout. Three-year before and three-year after crashes were collected for each roundabout. Crashes that occurred during the construction year were excluded. Crash location is defined not only by the address but also by the police definition as "intersection related", i.e., a crash is caused by the activity related to the operations of the intersection. Not limited to the intersection junction or circulatory area, the data collection allows for crashes occurring on roundabout approaches due to speeding or sudden stop or slowing down to also be included. A detailed manual review of each Wisconsin crash report forms (MV4000) was also conducted for all queried crash data using police narratives and diagrams to differentiate whether or not crashes were truly roundabout crashes or related to roundabout operations.

The importance of manually reviewing each MV4000 police report cannot be underestimated. This helps to distinguish crashes occurring at nearby intersections from those occurring at the roundabout. One example is the roundabout of CTH A and CTH JJ in Outagamie County. As displayed in Figure 4, there is another intersection also named CTH A and CTH JJ northwest of the roundabout. Both intersections are three-legged and yield-sign controlled on CTH JJ. Without referring to the actual diagram in the police report, it would be impossible to tell one from another. More popular situations are crashes occurring at roundabout proximities due to a driveway or other activities that are irrelevant to the roundabout. Once again easy decisions can be made based on clearly scripted diagrams. However, the quality of diagrams varies from report to report. In general, an electronically filed crash report with a roundabout template will remind or help the officer in choosing the proper intersection configurations. Another example of using a police report diagram is to distinguish crashes occurring between two interchange ramp roundabouts as exhibited in Figure 5. In this study, when there are no effective ways to separate crashes occurring at one interchange ramp roundabout from the other, the crashes were split between the two ramp terminals.

Based on the discussion with Wisconsin Department of Transportation engineers, crashes occurring during the construction year of the roundabout were excluded from the study to minimize the effects of construction activities and other complications such as being partially open to traffic during the construction. All 30 roundabouts studied had a three-year before period and a three-year after period of crash history. A three-year before and after period was used to obtain a statistically reliable result. Six roundabouts were omitted due to either a lack of post-construction data or unique geometry, specifically:

- Four roundabouts were newly constructed intersections and had no historic crashes;
- One roundabout combined several closely spaced intersections; and
- One roundabout had significant changes during the study period, occurring until 2009.

Detailed information about the roundabouts is shown in Appendix A.

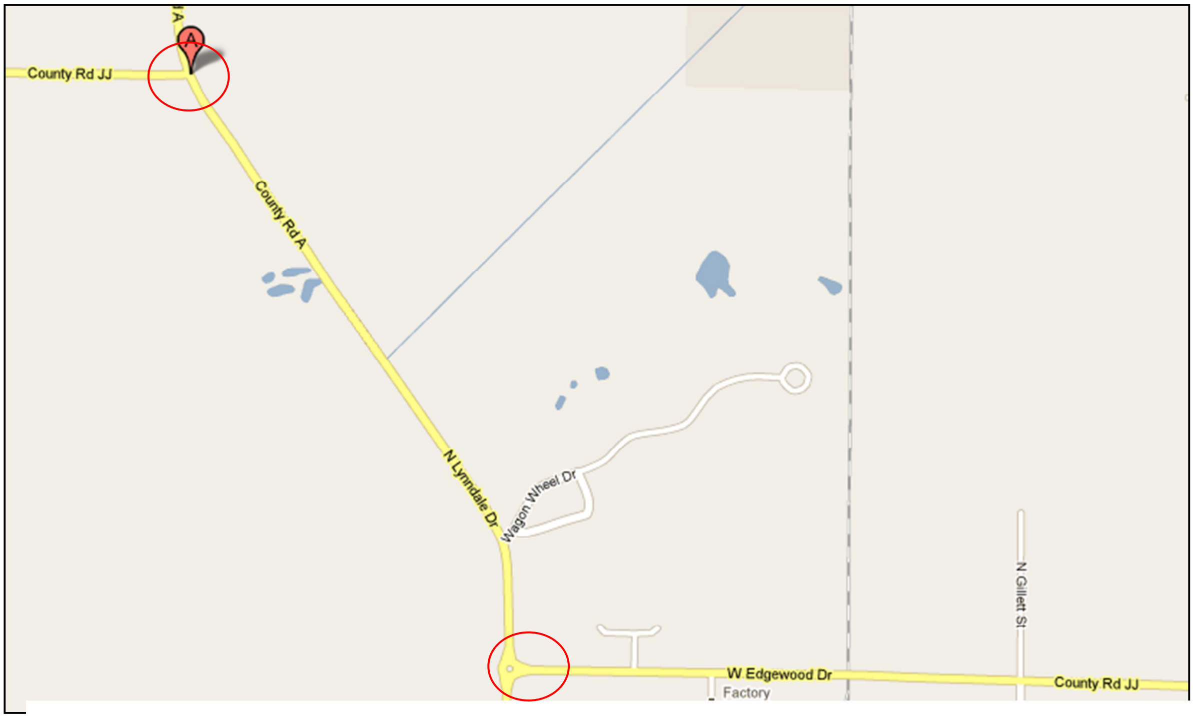


**Figure 4 Example of Roundabout Locations.**


**Figure 5 Example of Roundabouts at Ramp Terminals.**

## Geometric and Traffic Data

Important roundabout design features include the number of approaches, speed limit, number of circulating lanes, lane width, inscribed circle diameter (ICD), center island diameter (CID), and the total AADT. The AADT at a roundabout was defined as the sum of AADT on each approach entering the roundabout. Traffic volume information was primarily collected from the Wisconsin Highway Traffic Volume Data which is published by WisDOT annually (*37*). For the roundabouts with missing AADT, individual traffic counts were conducted.

In general, researchers observed in the dataset that three-legged roundabouts carried less traffic than four-legged roundabouts. The three-legged roundabouts had an AADT range of 5,850 to 23,300 vehicles per day (vpd) with an average of 14,200 vpd, while the four-legged roundabouts had a range of 4,100 to 48,100 vpd with an average of 17,565 vpd. Similarly, single-lane roundabouts had lower traffic volumes than multi-lane roundabouts. In the roundabouts observed for this research study, the AADT for the single-lane roundabouts ranged from 6,000 to 21,900 vpd with an average of 12,595 vpd. For the multi-lane roundabouts, AADT ranged from 4,100 to 48,100 vpd with an average of 20,170 vpd.

In addition in current AADT levels, the intersection configuration and traffic data before roundabout conversion were collected, including AADT, number of intersection approach legs, number of major roadway lanes, existence of major roadway median, speed limit, and more importantly, the traffic control type before the roundabout conversion.

WisDOT Region and area type were collected. The area type was determined urban if the municipality where the roundabout was located had a population greater than 5000. The characteristics of the 24 roundabouts are listed in Table 1. Detailed characteristics about each roundabout are listed in Appendix A.

**TABLE 1 Characteristics of Modern Roundabouts in the Scope of the Study**

| Characteristics | Number | Percentage |
|---|---|---|
| **Area Type*** | | |
| Urban | 16 | 66.7% |
| Rural | 8 | 33.3% |
| **Number of circulating lanes** | | |
| 1 | 12 | 50.0% |
| 2 | 12 | 50.0% |
| **Previous intersection traffic control** | | |
| No control/Yield (NC) | 2 | 8% |
| Two-way Stop Controlled (TWSC) | 12 | 50% |
| All-way Stop Controlled (AWSC) | 5 | 21% |
| Signalized | 5 | 21% |
| **WisDOT Region** | | |
| NC | 1 | 4.1% |
| NE | 13 | 54.2% |
| NW | 3 | 12.5% |
| SE | 3 | 12.5% |
| SW | 4 | 16.7% |

# CHAPTER 5 RESULTS

## Simple Before-and-After Analysis

Since individual injury levels cannot be analyzed using EB analysis, a simple before-after analysis was completed. This analysis did not take into consideration any RTM effects but was helpful in evaluating individual injury levels. Table 2 shows the observed crash statistics for all 24 study roundabouts in the before and after period. The frequency is classified by crash outcome (K, A, B, C, and PDO). For the roundabouts at interchange ramp terminals, crash reports were verified manually to assign the crash to one of the roundabouts. However, in some instances, this was not possible due to the unavailability of the crash report or lack of a collision diagram/narrative in the crash report. In such cases, the crash was assigned as 0.5 to each of the roundabouts at the ramp terminals. Consequently, four of the evaluated roundabouts show non-integer crash frequencies.

**TABLE 2 Before and After Crash Data**

| Roundabout | WisDOT Region | Before | | | | | | After | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | K | A | B | C | PDO | Total | K | A | B | C | PDO | Total |
| STH 54/Gaynor St/17th St | NC | | | 2 | 6 | 9 | 17 | | | | | 20 | 20 |
| CTH F/S. Ninth St. | NE | | | 1 | | | 1 | | | 1 | 2 | 1 | 4 |
| CTH F/Suburban Dr. | NE | | | | | 2 | 2 | | | | | | |
| STH 32/57 and STH 96 | NE | | | 1 | | 6 | 7 | | | 1 | 1 | 6 | 8 |
| STH 141 / Allouez Ave | NE | | 1 | 1 | 1 | 9 | 12 | | | 1 | 2 | 8 | 11 |
| STH 32/STH 57 Broadway | NE | | | | 1 | 8 | 9 | | | | 3 | 40 | 43 |
| STH 55/CTH KK | NE | 1 | 1 | 4 | 5 | 9 | 20 | | | 1 | | 4 | 5 |
| Lake Park/Plank Rd (CTH LP/CTH P) | NE | | | | | | | | | 1 | | 2 | 3 |
| CTH N / Emons Road | NE | | | 1 | 1 | | 2 | | | 2 | | 3 | 5 |
| STH 28/32 (high speed) | NE | | | 1 | 1 | 6 | 8 | | | | 1 | 10 | 11 |
| STH 42/ I-43, Interchange Ramps (West)* | NE | | | 1 | 1 | 7.5 | 9.5 | | | 1 | 3.5 | 8 | 12.5 |
| STH 42/ I-43, Interchange Ramps (East)* | NE | | | 1 | 1 | 13.5 | 15.5 | | 2 | 1 | 0.5 | 12 | 15.5 |
| STH 42/Vanguard, Wal-Mart entrance | NE | | | | 1 | 1 | 2 | | | | | 8 | 8 |
| Breezewood ln/Tullar Rd | NE | | | | 2 | 2 | 4 | | | | | 6 | 6 |
| US 53 ramps and CTH O (West)* | NW | | | | 1 | 9.5 | 10.5 | | | 1 | 0.5 | 2 | 3.5 |
| US 53 ramps and CTH O (East)* | NW | | | | | 5.5 | 5.5 | | | 1 | 1.5 | 2 | 4.5 |
| STH 124/CTH S | NW | | 1 | 2 | 8 | 5 | 16 | | | | 1 | 5 | 6 |
| Canal St/25th Ave | SE | | | | | 1 | 1 | | | | 2 | 11 | 13 |
| STH 38/CTH K | SE | | | 3 | 6 | 19 | 28 | | 1 | | 1 | 18 | 20 |
| Elkhorn Rd (Bus 12)/Bluff Rd/Clay St | SE | | 1 | | | 2 | 3 | | | | | 3 | 3 |
| STH 78/STH 92, 8th St, Springdale, CTH ID | SW | | | 1 | | 13 | 14 | | | | | 11 | 11 |
| Thompson and Commercial (North) | SW | 1 | 1 | 3 | 7 | 8 | 20 | | 1 | | 6 | 32 | 39 |
| Thompson and STH 30 (South) | SW | | | 1 | 4 | 8 | 13 | | | | 1 | 7 | 8 |
| Old STH 12/Parmenter | SW | | | 3 | 1 | 5 | 9 | | | | | 2 | 2 |
| Grand Total | | 2 | 5 | 26 | 47 | 149 | 229 | 0 | 4 | 11 | 26 | 221 | 262 |

*Crashes at interchange ramp terminals that could not be ascribed to a particular roundabout were assigned as 0.5 to each roundabout.TABLE 3 shows the number of locations with increase, no change, or decrease in crashes between before and after periods. There were no fatal (K) crashes in the after period. The two sites with fatal (K) crashes in the before period did not experience fatal crashes after the roundabout was installed. For all injury (A, B, and C) crashes, the number of locations with reduced crashes was greater than the number of locations with increased crashes. The magnitude of decrease in injury crashes was higher than the magnitude of increase. For PDO and total crashes the number of locations with increases in crashes was 12 as opposed to 10 locations with decreases in crashes. Overall, roundabouts in Wisconsin had a marked decrease in fatal and injury crashes, but a significant increase in PDO crashes, both results consistent with other national and international studies.

**TABLE 3 Simple Before-and-After Crash Trend**

| Change in Crashes | Number of locations | | | | | |
|---|---|---|---|---|---|---|
| | K | A | B | C | PDO | Total Crashes |
| Increase | 0 | 2 | 4 | 7 | 12 | 12 |
| No change | 22 | 18 | 11 | 5 | 2 | 2 |
| Decrease | 2 | 4 | 9 | 12 | 10 | 10 |

The following summarizes the trends for fatal and injury crashes as observed in Table 2.

- Fatal (K) crashes: Two of 24 locations had two fatal crashes in before period. None in after period. No location had an increase in fatal crashes.
- Incapacitating (A) crashes: Five of 24 locations had crashes in the before period. Four locations had none in the after period and one location remained unchanged. Two locations observed crashes increasing from zero to one and two, respectively.
- Non-Incapacitating (B) crashes: 12 of 24 locations had crashes in the before period. Of the 12, eight locations had no crashes in after period, one location reduced from four to one, three locations did not change, and one location observed crashes increasing from one to two. Three locations found crashes increasing from zero to one.
- Possible Injury (C) crashes: 16 of 24 locations had crashes in the before period. Of the 16, six locations had no crashes in the after period, one location did not change, six locations found crashes reduced from 27 to 10 in total, and for the remaining three locations crashes increased from one to two, three, and 3.5, respectively. Four locations found crashes increasing from zero to one, 1.5 or two, respectively.

Table 4 shows the distribution of crash types and severity in the before and after period for all 24 roundabouts. The crash type is based on the manner of collision (“MNRCOLL”) field in the crash report. Angle (ANGL) and head-on (HEAD) crashes were reduced from 119 to 54 and from four to one, respectively. Sideswipe-opposite crashes reduced from seven to one. Rear-end crashes (REAR) were similar in the before (55) and after (54) periods. No collision (NO C) and sideswipe-same direction (SSS) crashes increased from 35 to 77 and 10 to 75, respectively. Therefore, roundabouts can change the crash type distribution at intersections.

**TABLE 4 Distribution of Crash Types and Severity for All Roundabouts**

| Collision Type | Before | | | | | | After | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | K | A | B | C | PD | Total | K | A | B | C | PD | Total |
| ANGL | 1 | 4 | 16 | 26 | 72 | 119 | | 2 | | 3 | 49 | 54 |
| HEAD | | | 2 | 2 | | 4 | | | | | 1 | 1 |
| NO C | 1 | 1 | 2 | 2 | 29 | 35 | | | 9 | 4 | 64 | 77 |
| REAR | | | 5 | 13 | 37 | 55 | | 1 | 1 | 13 | 39 | 54 |
| SSOP | | | | 3 | 3 | 6 | | | | | 1 | 1 |
| SSS | | | 1 | 1 | 8 | 10 | | 1 | 1 | 6 | 67 | 75 |
| Total | 2 | 5 | 26 | 47 | 149 | 229 | | 4 | 11 | 26 | 221 | 262 |

## EB Analysis with HSM Safety Performance Function (SPF)

SPFs can be found for a variety of highway facilities and intersection types in HSM. Appropriate SPFs were identified using the pre-roundabout intersection geometric characteristics (number of legs, number of lanes), area setting (urban, rural), as well as traffic control types (Yield, TWSC, AWSC, Signalized). SPFs can be developed for different types of crashes based on the purpose of the evaluation. The standard SPF is formulated as the equation of expected annual crash count μ in Equation 11:

$$\mu = \mathrm{a}(Major\ AADT)^{\mathrm{b}}(Minor\ AADT)^{\mathrm{c}} \tag{11}$$

Coefficients a, b, c, and an overdispersion factor k can be estimated via maximum likelihood estimation (MLE). A reference of SPFs applied in the study is listed in Appendix B.

To facilitate the EB analysis, procedures were coded to a spreadsheet with all the key parameters. Tables 5 and 6 show the results for total number of crashes and injury (K, A, B, and C) crashes, respectively. The first two columns are intersection IDs and descriptions; followed by observed crashes during the three-year before period, the expected number of crashes during the three-year after period, and the observed number of crashes during the three-year after period. The last two columns are before-and-after comparison results with the shaded cells and negative values suggesting an increase in crashes after building the roundabout.

**TABLE 5 EB Analysis for Total Crashes**

| Location | Intersection Type | Observed Total Crashes-Before | Expected Crashes –EB-After | Observed Total Crashes-After | B-A | B-A [% Reduction=100(B-A)/B] |
|---|---|---|---|---|---|---|
| STH 54/Gaynor St/17th St | 4Urb4ST | 17.00 | 12.732 | 20.00 | -7.27 | -57.09 |
| CTH F/S. Ninth St. | 2Urb4ST | 1.00 | 2.306 | 4.00 | -1.69 | -73.47 |
| CTH F/Suburban Dr. | 2Urb4ST | 2.00 | 2.247 | 0.00 | 2.25 | 100.00 |
| STH 32/57 and STH 96 | 2Urb4STALL | 7.00 | 6.711 | 8.00 | -1.29 | -19.21 |
| STH 141 / Allouez Ave | 2Rur4ST | 12.00 | 22.636 | 11.00 | 11.64 | 51.41 |
| STH 32/STH 57 Broadway | 4Urb4SG | 9.00 | 18.874 | 43.00 | -24.13 | -127.82 |
| STH 55/CTH KK | 2Rur4ST | 20.00 | 16.745 | 5.00 | 11.74 | 70.14 |
| Lake Park/Plank Rd (CTH LP/CTH P) | 2Urb4ST | 0.00 | 1.588 | 3.00 | -1.41 | -88.93 |
| CTH N / Emons Road | 2Rur4ST | 2.00 | 9.269 | 5.00 | 4.27 | 46.06 |
| STH 28/32 | 2Rur4ST | 8.00 | 8.607 | 11.00 | -2.39 | -27.81 |
| STH 42/ I-43, Interchange Ramps (West) | 4Rur4SG | 9.50 | 25.718 | 12.50 | 13.22 | 51.40 |
| STH 42/ I-43, Interchange Ramps (East) | 4Rur4SG | 15.50 | 25.820 | 15.50 | 10.32 | 39.97 |
| STH 42/Vanguard, Wal-Mart entrance | 4Rur4SG | 2.00 | 12.080 | 8.00 | 4.08 | 33.78 |
| Breezewood ln/Tullar Rd | 2Urb4YD | 4.00 | 4.437 | 6.00 | -1.56 | -35.23 |
| US 53 ramps and CTH O (West) | 2Urb4ST | 10.50 | 8.930 | 3.50 | 5.43 | 60.80 |
| US 53 ramps and CTH O (East) | 2Urb4ST | 5.50 | 6.590 | 4.50 | 2.09 | 31.71 |
| STH 124/CTH S | 2Rur4ST | 16.00 | 14.583 | 6.00 | 8.58 | 58.85 |
| Canal St/25th Ave | 4Urb3STALL | 1.00 | 3.274 | 13.00 | -9.73 | -297.01 |
| STH 38/CTH K | 4Urb3ST | 28.00 | 17.586 | 20.00 | -2.41 | -13.73 |
| Elkhorn Rd (Bus 12)/Bluff Rd/Clay St | 4Urb4YD | 3.00 | 2.811 | 3.00 | -0.19 | -6.74 |
| STH 78/STH 92, 8th St, Springdale, CTH ID | 4Urb4SG | 14.00 | 11.788 | 11.00 | 0.79 | 6.68 |
| Thompson and Commercial (North) | 4Urb4STALL | 20.00 | 27.289 | 39.00 | -11.71 | -42.91 |
| Thompson and STH 30 (South) | 4Urb3STALL | 13.00 | 17.154 | 8.00 | 9.15 | 53.36 |
| Old STH 12/Parmenter | 4Urb4STALL | 9.00 | 8.430 | 2.00 | 6.43 | 76.27 |

**TABLE 6 EB Analysis for Fatal and Injury Crashes**

| Location | Intersection Type | Observed Fatal and Injury Crashes-Before | Expected Crashes –EB-After | Observed Fatal and Injury Crashes-After | B-A | B-A [% Reduction=100(B-A)/B] |
|---|---|---|---|---|---|---|
| STH 54/Gaynor St/17th St | 4Urb4ST | 8.00 | 5.273 | 0.00 | 5.27 | 100.00 |
| CTH F/S. Ninth St. | 2Urb4ST | 1.00 | 1.449 | 3.00 | -1.55 | -107.00 |
| CTH F/Suburban Dr. | 2Urb4ST | 0.00 | 0.694 | 0.00 | 0.69 | 100.00 |
| STH 32/57 and STH 96 | 2Urb4STALL | 1.00 | 1.300 | 2.00 | -0.70 | -53.90 |
| STH 141 / Allouez Ave | 2Rur4ST | 3.00 | 2.973 | 3.00 | -0.03 | -0.92 |
| STH 32/STH 57 Broadway | 4Urb4SG | 1.00 | 5.751 | 3.00 | 2.75 | 47.83 |
| STH 55/CTH KK (high speed) | 2Rur4ST | 11.00 | 2.809 | 1.00 | 1.81 | 64.40 |
| Lake Park/Plank Rd (CTH LP/CTH P) | 2Urb4ST | 0.00 | 0.936 | 1.00 | -0.06 | -6.87 |
| CTH N / Emons Road | 2Rur4ST | 2.00 | 2.743 | 2.00 | 0.74 | 27.10 |
| STH 28/32 (high speed) | 2Rur4ST | 2.00 | 1.993 | 1.00 | 0.99 | 49.83 |
| STH 42/ I-43, Interchange Ramps (West) | 4Rur4SG | 2.00 | 8.894 | 4.50 | 4.39 | 49.40 |
| STH 42/ I-43, Interchange Ramps (East) | 4Rur4SG | 2.00 | 6.996 | 3.50 | 3.50 | 49.97 |
| STH 42/Vanguard, Wal-Mart entrance | 4Rur4SG | 1.00 | 7.921 | 0.00 | 7.92 | 100.00 |
| Breezewood ln/Tullar Rd | 2Urb4YD | 2.00 | 2.170 | 0.00 | 2.17 | 100.00 |
| US 53 ramps and CTH O (West) | 2Urb4ST | 1.00 | 1.463 | 1.50 | -0.04 | -2.55 |
| US 53 ramps and CTH O (East) | 2Urb4ST | 0.00 | 1.329 | 2.50 | -1.17 | -88.10 |
| STH 124/CTH S | 2Rur4ST | 11.00 | 2.146 | 1.00 | 1.15 | 53.40 |
| Canal St/25th Ave | 4Urb3STALL | 0.00 | 1.431 | 2.00 | -0.57 | -39.78 |
| STH 38/CTH K | 4Urb3ST | 9.00 | 3.897 | 2.00 | 1.90 | 48.68 |
| Elkhorn Rd (Bus 12)/Bluff Rd/Clay St | 4Urb4YD | 1.00 | 1.086 | 0.00 | 1.09 | 100.00 |
| STH 78/STH 92, 8th St, Springdale, CTH ID | 4Urb4SG | 1.00 | 2.054 | 0.00 | 2.05 | 100.00 |
| Thompson and Commercial (North) | 4Urb4STALL | 12.00 | 9.644 | 7.00 | 2.64 | 27.42 |
| Thompson and STH 30 (South) | 4Urb3STALL | 5.00 | 6.819 | 1.00 | 5.82 | 85.34 |
| Old STH 12/Parmenter | 4Urb4STALL | 4.00 | 2.865 | 0.00 | 2.87 | 100.00 |

A few findings are highlighted from the EB results for the 24 roundabout locations in Tables 5 and 6.

1. Mixed results for total crash frequency
   - 13 locations (54 percent) showed a decrease in crash frequency, but 11 locations (46 percent) showed an increase.
   - On average, the increase was 1.93 crashes per year and the decrease was 2.31 crashes per year.

- Nationally, a 35 percent reduction was observed for all crash types while Wisconsin roundabouts had an unbiased 9 percent decrease in all crash types with a standard error of 6 percent (2).

2. Significant decrease in crash severity
   - 17 locations (71 percent) had a decrease in fatal (K) and injury (A, B, and C) crashes.
   - 7 locations did not have any fatal and injury crashes after the roundabout conversion compared with only three locations before.
   - The average decrease in fatal and injury crashes was 0.94 crashes per year.
   - A decrease of 52% for injury crashes. Roundabouts nationwide are also experiencing a significant decrease in severe crashes.
3. A detailed review of the locations that have experienced an increase in all crashes or injury crashes reveals that the majority of the sites have only incurred slight crash increases. A small percentage of roundabouts experienced significant increases in crash frequency and severity and contribute substantially to the summary statistics.
   - The largest increase of injury crashes for a single site (CTH F/S. Ninth St.) was 0.52 crashes per year.
   - Among the 11 locations with increased crash records, STH 32/STH 57 Broadway, Canal St/25th Ave, Thompson and Commercial, contribute 38 percent, 15 percent and 18 percent of all increases in the total number of crashes, respectively. All combined, the three locations contribute 71 percent of the crash increases.

In summary, all of the roundabouts have shown promising safety improvements in all crash severity levels. Though mixed results were observed in total crash frequency after the roundabout conversion, a small portion (27 percent) of the locations accounted for a significant portion (71 percent) of the total increase. A significant decrease (52 percent) was observed for injury crashes. Even for the roundabouts with an increase in injury crashes, the average annual increase was 0.2 with the largest annual increase being 0.5.

Crash reductions observed at Wisconsin roundabouts were not as high as reported in other studies (*13*). However, it is premature to conclude that the safety benefits of Wisconsin roundabouts are not equally or more effective than those in other states without a good understanding of the differences in the data and study methodologies. Several locations had a relatively low number of crashes and injury severity crashes before being converted to a roundabout. Operational and non-engineering reasons, rather than safety, were the motivation for constructing roundabouts at these locations. In spite of the varying safety performance across individual roundabouts being evaluated, the level of improvement can be associated with the prevailing geometric characteristics and traffic conditions prior to roundabout construction. The following analysis was focused on three aspects: speed, number of lanes, and traffic control strategies.

***Speed and Roundabout Safety***

Speed is considered as one of the major contributing factors to roundabout crashes especially when the speed differentiation between the roadway and roundabout is relatively large. Due to the circulatory design of a roundabout, drivers may have difficulties in negotiating curves, especially during inclement weather. Yield-controlled approaches may be ignored by inattentive drivers, especially under high speed conditions. Insufficient conspicuity of the roundabout due to the lack of proper traffic signing, pavement marking, and landscaping may be contributing to a driver's inability to effectively regulate their choice of speed entering and circulating the roundabout. Other design exceptions or limitations in entry deflection or flare can compromise a driver's decision making. There are perceived concerns with constructing roundabouts on roadways or corridors with high-speed approaches posted at 45 mph or greater. Attempts have been made to identify speed effect on crash occurrence, including the speed-based models developed in NCHRP Report 572 (2). The models however were concluded as inadequate to address the relationship between speed and crashes.

Table 7 shows that among the 24 roundabout locations in Wisconsin, 11 have at least one approach with the posted speed limit of 45 mph or higher. The roundabouts with high-speed approaches experienced a crash reduction of 34 percent while the roundabouts with low-speed approaches experienced a 10 percent crash increase. Seven out of 11 high-speed roundabouts have decreased crash rates compared to 6 out of 13 of the roundabouts with low-speed approaches. If only fatal and injury crashes are considered, high-speed and low-speed roundabouts are similar. A 53 percent reduction in injury crashes was observed at low-speed roundabouts, while a 49 percent reduction in injury crashes was observed at high-speed roundabouts. However, the number of roundabouts in which injury crashes decreased is different: 6 out of 11 high-speed roundabouts experienced decreases in injury crashes compared to 11 out of 13 of the low-speed roundabouts. The Wisconsin roundabout analysis was consistent with a recent study by Ritchie and Lenters in which they concluded that roundabouts are the most appropriate control for intersections with high-speed approaches (*12*).

**TABLE 7 Speed and Roundabout Safety**

| | | Low-speed | High-speed |
|---|---|---|---|
| | Number of RABs | 13 | 11 |
| Total Crashes | RABs with Increased Crashes | 7 | 4 |
| | RABs with Decreased Crashes | 6 | 7 |
| | Total Expected Crashes | 164 | 124 |
| | Total Observed Crashes | 180 | 82 |
| | % of Changes | 9.94% | -34.13% |
| KABC Crashes | RABs with Increased Crashes | 2 | 5 |
| | RABs with Decreased Crashes | 11 | 6 |
| | Total Expected Crashes | 55 | 30 |
| | Total Observed Crashes | 26 | 15 |
| | % of Changes | -52.73% | -49.38% |

***Single-lane versus Multi-lane***

As shown in Table 8, five of the single-lane roundabout locations experienced a decrease in all crashes. Six of the multi-lane roundabout locations experienced a decrease in all crashes. While both single and multi-lane roundabouts experienced some crash increase at several sites, total crashes were different. Multi-lane roundabouts had a 6 percent increase in total crashes; conversely, single-lane roundabouts had a nearly 36 percent reduction in all crashes. The opposite was observed when examining fatal and injury crashes. Only one out of twelve multi-lane roundabouts experienced an increase in injury crashes compared with six out of twelve of the single-lane roundabouts. Considering injury crashes, multi-lane roundabouts had an overall reduction of 63 percent while single-lane roundabouts showed an 18 percent reduction.

**TABLE 8 Roundabout Safety Performance by Number of Lanes**

| | | Single-lane | Multi-lane |
|---|---|---|---|
| | Number of RABs | 12 | 12 |
| Total Crashes | RABs with Increased Crashes | 5 | 6 |
| | RABs with Decreased Crashes | 7 | 6 |
| | Total Expected Crashes | 105 | 184 |
| | Total Observed Crashes | 67 | 195 |
| | % of Changes | -35.98% | 6.23% |
| KABC Crashes | RABs with Increased Crashes | 6 | 1 |
| | RABs with Decreased Crashes | 6 | 11 |
| | Total Expected Crashes | 22 | 63 |
| | Total Observed Crashes | 18 | 23 |
| | % of Changes | -18.20% | -63.28% |

**Traffic Control Strategies**

The NCHRP Report 572 study reported reductions of approximately 35 percent for all crashes and 76 percent for injury crashes when an intersection was converted to a roundabout from a signal or stop control (2). The safety benefits however vary considerably among traffic control alternatives, including Yield, two-way stop controlled (TWSC), all-way stop controlled (AWSC), and signal control. The highest safety effectiveness was recorded when an intersection was converted from TWSC as shown in Table 9. Mixed results were found for other control types. The safety performance is insignificant when converting an AWSC to a roundabout due to its low volume conditions. The conversion from a signalized intersection to a roundabout requires more considerations such as left-turning volume, left-turn storage space, and the space between intersections because the safety benefits are conditional to these unique situations.

The conversion of signalized intersections gained marginal safety improvements while more crashes occurred at roundabout locations converted from No Control/Yield and AWSC. From the injury crash perspective, benefits were scored for traffic control of all types. Though injury crashes increased at a few locations with previous control types as TWSC and AWSC, the overall crash reduction is positive.

**TABLE 9 Roundabout Safety Performance by Traffic Control Type**

| | | No Control/Yield | TWSC | AWSC | Signalized |
|---|---|---|---|---|---|
| | Number of RABs | 2 | 12 | 5 | 5 |
| Total Crashes | RABs with Increased Crashes | 2 | 5 | 3 | 1 |
| | RABs with Decreased Crashes | 0 | 7 | 2 | 4 |
| | Total Expected Crashes | 7 | 124 | 63 | 94 |
| | Total Observed Crashes | 9 | 93 | 70 | 90 |
| | % of Changes | 24.18% | -24.89% | 11.36% | -4.54% |
| KABC Crashes | RABs with Increased Crashes | 0 | 5 | 2 | 0 |
| | RABs with Decreased Crashes | 2 | 7 | 3 | 5 |
| | Total Expected Crashes | 3 | 28 | 22 | 32 |
| | Total Observed Crashes | 0 | 18 | 12 | 11 |
| | % of Changes | -100.00% | -35.03% | -45.60% | -65.21% |

## EB Analysis with Wisconsin Intersection SPFs

A SPF is one of the most important components in EB method used for correcting the short-term crash counts. The SPFs from the HSM offer the state-of-the-art crash prediction methodologies and guidance on developing appropriate performance models. Since the SPFs are equations that estimate the expected average crash frequency as a function of traffic volume and roadway characteristics, these estimates depend on the data from which they are developed. It is suggested that SPFs in HSM be calibrated before applying directly to estimate local safety performances.

Following the HSM procedures, five intersection SPFs were developed using the crash data collected from 3,202 intersections in Wisconsin between 2001 and 2003 (*38*). The five models are for four-legged signalized intersections, four-legged unsignalized intersection with median, four-legged unsignalized intersections without median, three-legged unsignalized intersections on two-lane highways, and three-legged unsignalized intersections on four-lane highways. Though the key variables in the Wisconsin SPFs are somewhat different from the HSM SPFs, they serve the same purpose — predicting the expected number of crashes.

Replacing the HSM SPFs with the Wisconsin SPFs, the same analysis was repeated for all crashes and fatal and injury crashes, as well as safety performance in the aspects of approach speed, number of lanes, and traffic control before. In general, the results from the Wisconsin data are very close to the HSM counterparts. In several safety metrics, EB analysis with the Wisconsin SPFs suggests more benefits after the roundabout conversion. Major findings concluded are listed as below with detailed statistics in Appendix C.

1. Significant decrease in crash severity;
    - 20 locations (83 percent) had a decrease in injury (KABC) crashes.

- The average annual decrease was more than one crash/year.
- Only four sites had a slight increase in injury crashes and the increase is merely 0.13 crashes per year.
- The unbiased estimate of the injury crash reduction was 61 percent with a 6 percent standard deviation.

2. Both high-speed and low-speed roundabouts experienced a higher reduction in injury crashes;
    - Injury crashes are reduced 68 percent at high-speed roundabouts.
    - Injury crashes are reduced 54 percent at all low-speed roundabouts.
3. Significant injury reductions also occurred at both single- and multi-lane roundabouts;
    - Injury crash reduction at single-lane roundabouts was 55 percent.
    - Injury crash reduction at multi-lane roundabouts was 64 percent.
4. Opposite changes in total crashes were observed between roundabout groups;
    - High-speed roundabouts had an overall reduction while low-speed roundabouts have an overall increase. The disparity was consistent with the EB analysis with HSM SPFs.
    - Multi-lane roundabouts had an undesirable 12 percent increase in crashes while single-lane ones had a moderate 30 percent decrease.
5. TWSC, AWSC, and signalized intersections had a similar level of injury crash reduction after the conversion; and
    - Injury crash reduction at TWSC intersections was 63 percent.
    - Injury crash reduction at AWSC intersections was 51 percent.
    - Injury crash reduction at Signalized intersections was 59 percent.
6. Changes in total number of crashes vary considerably between traffic control alternatives.
    - TWSC had the largest reduction of 22 percent.
    - No Control or Yield had a marginal reduction of 1.38 crashes.
    - AWSC and Signalized intersections had an increase of 23.5 percent and 5.5 percent, respectively.

# CHAPTER 6 CONCLUSIONS

While roundabouts are still fairly new in the U.S. and Wisconsin, their safety benefits have yielded varied results. For this study, researchers analyzed 24 Wisconsin roundabouts that were built in 2007 or before. Three years of before and after crash data were gathered as well as geometric and volume data. An observational (simple) before and after crash analysis was completed to analyze specific types of injury crashes for each roundabout. An EB analysis was used to examine the safety benefits for total crashes and injury (K, A, B, C) crashes.

**Simple Before-and-After Analysis**

Fatal (K) crashes were eliminated at two sites and none of the sites experienced any fatal crashes in the after period. For all injury (A, B, and C) crashes, the number of locations with reduced crashes is greater than the number of locations with increased crashes. The magnitude of decrease in injury crashes is higher than the magnitude of increase. For PDO and total crashes, 12 locations observed increases in the number of crashes as opposed to 10 locations which experienced decreases. Overall, roundabouts in Wisconsin had a marked decrease in fatal and injury crashes and a significant increase in PDO crashes, which has been observed in other national and international studies. The increase is PDO crashes which weighted the overall increase in crashes at several locations can be less attributed to a safety issue and more attributed to drivers understanding the navigational requirements of roundabouts.

When examining crash type, researchers found that roundabouts can modified the crash types that occur at intersections from more severe crash types to less severe crash types. Angle and head-on crashes were reduced from 119 to 54 and from four to one, respectively. Sideswipe-opposite crashes were reduced from seven to one. Rear-end crashes were similar in the before (55) and after (54) periods. No collision and sideswipe-same direction crashes increased from 35 to 77 and ten to 75, respectively. Again, these crash types may be more reflective of driver's navigational limitations than inherent safety issues with the roundabouts.

**Empirical Bayes Crash Analysis**

The EB analysis was performed using SPFs from both the HSM and Wisconsin specific data. The results using both SPF values were very similar. The HSM SPFs were used resulting in the following:

- Mixed results for total crash frequency
    - 13 locations (54 percent) had a decrease in crash frequency;,11 locations (46 percent) showed an increase.
    - On average, the increase was 1.93 crashes per year and the decrease was 2.31 crashes per year.
    - Nationally, a 35 percent reduction was observed for all crashes while Wisconsin experienced a 9 percent reduction (2).

- Significant decrease in crash severity
    - 17 locations (71 percent) had a decrease in fatal (K) and injury (A, B, and C) crashes.
    - 7 locations did not have any combined fatal and injury crashes after the roundabout conversion compared with only 3 locations before.
    - No location observed fatal crashes in the after period.
    - The average decrease in fatal and injury crashes was 0.94 crashes per year.
    - A decrease of 52% for injury crashes. Roundabouts nationwide are also experiencing a significant decrease in severe crashes.

When looking at predictor variables, the speed limit of the approaches does not appear to contribute to safety issues. In other words, the posted speed limit on the approach did not have a significant effect on safety. While multi-lane roundabouts seem to be safer than single lane roundabouts when looking at fatal and injury crashes, single lane roundabouts saw a larger decrease in total crashes. TWSC conversions had the highest safety benefit as compared to AWSC and signalized.

## Future Research

This research study has shown that the majority of roundabouts installations have led to improvements in traffic safety. Nevertheless, this initial study was a sample of the roundabouts that existed in 2007 with some limitations on the availability of data. Additional research to include the much larger number of roundabouts that exist today and build on the sample size may allow for increased variable sensitivity in determining the optimal attributes of roundabout applications throughout Wisconsin.

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# Appendix A- WI Roundabout Data

## TABLE 10 Characteristics of Each Roundabout in the Study

| | | | | | | REQUIRED (*for Intersection Safety Performance Function Base Conditions*) | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | AADT-Before | | | AADT-After | | | | | | | |
| STD # | Roundabout Name (Streets) | Region | County | Years of Before Crash Data | Year of After Crash Data | Roundabout Open to the Traffic | AADT-Major | AADT-Minor | AADT-Total | AADT-Major | AADT-Minor | AADT-Total | Number of Lanes | Area type | Number of Legs | Intersection Traffic Control-Before | High Speed |
| 2 | **17/2ND AND GAYNOR** | NC | Wood | 2001-2003 | 2005-2007 | 9/15/2004 | 17,875 | 3,875 | 21,750 | 12,500 | 5,600 | 18,100 | 2 | 3 | 4 | 2 | |
| 4 | **MATTHEW AND NINTH AND CTH F (SCHEURING)** | NE | Brown | 2001-2003 | 2005-2007 | 11/1/2004 | 8,700 | 3,300 | 12,000 | 10,100 | 1,800 | 11,900 | 1 | 3 | 4 | 2 | |
| 5 | **SUBURBAN AND CTH F (SCHUERING)** | NE | Brown | 2001-2003 | 2005-2007 | 11/1/2004 | 7,400 | 3,250 | 10,650 | 6,450 | 1,800 | 8,250 | 1 | 3 | 4 | 2 | |
| 6 | **STH 32/57 (GREENLEAF) & STH 96 (DAY)** | NE | Brown | 2004-2006 | 2008-2010 | 8/31/2007 | 7,250 | 3,500 | 10,750 | 7,250 | 3,500 | 10,750 | 1 | 3 | 4 | 3 | Y |
| 7 | **USH 141 & ALLOUEZ AVE** | NE | Brown | 2004-2006 | 2008-2010 | 10/1/2007 | 7,000 | 1,700 | 8,700 | 10,100 | 3,500 | 13,600 | 1 | 1 | 4 | 2 | Y |
| 8 | **STH 32/57 (CLAUDE ALLOUEZ) & BROADWAY** | NE | Brown | 2004-2006 | 2008-2010 | 7/12/2007 | 32,500 | 15,600 | 48,100 | 50,000 | 24,900 | 74,900 | 2 | 3 | 4 | 4 | |
| 9 | **STH 55 & CTH KK** | NE | Calumet | 2003-2005 | 2007-2009 | 8/3/2006 | 10,300 | 4,500 | 14,800 | 8,950 | 3,650 | 12,600 | 1 | 1 | 4 | 2 | Y |
| 10 | **CTH LP/LAKE PARK & CTH P/PLANK** | NE | Calumet | 2004-2006 | 2008-2010 | 11/1/2007 | 8,250 | 3,850 | 12,100 | 7,300 | 4,450 | 11,750 | 1 | 2 | 4 | 2 | Y |
| 12 | **CTH N / Emons Road** | NE | Outagamie | 2004-2006 | 2008-2010 | 8/31/2007 | 7,800 | 800 | 8,600 | 12,200 | 2,100 | 14,300 | 1 | 1 | 4 | 2 | |
| 15 | **STH 28 & STH 32** | NE | Sheboygan | 2003-2005 | 2007-2009 | 9/1/2006 | 8,350 | 2,950 | 11,300 | 5,300 | 3,750 | 9,050 | 1 | 1 | 4 | 2 | Y |
| 17 | **STH 42 & I 43 RAMPS (West)*** | NE | Sheboygan | 2004-2006 | 2008-2010 | 11/2/2007 | 10,700 | 1,300 | 12,000 | 23,000 | 3,000 | 26,000 | 2 | 1 | 4 | 4 | |
| 41 | **STH 42 & I 43 RAMPS (East)*** | NE | Sheboygan | 2004-2006 | 2008-2010 | 11/2/2007 | 15,100 | 4,700 | 19,800 | 20,000 | 8,076 | 28,076 | 2 | 1 | 4 | 4 | |
| 18 | **STH 42 & VANGUARD Wal-Mart Entrance** | NE | Sheboygan | 2004-2006 | 2008-2010 | 11/3/2007 | 11,600 | 1,500 | 13,100 | 20,000 | 7,000 | 27,000 | 2 | 1 | 4 | 4 | Y |
| 19 | **BREEZEWOOD & TULLAR** | NE | Winnebago | 2002-2004 | 2006-2008 | 9/15/2005 | 13,000 | 4,800 | 17,800 | 11,350 | 4,800 | 16,150 | 1 | 3 | 4 | 1 | |
| 20 | **USH 53 & CTH O RAMPS (west)*** | NW | Baron | 2003-2005 | 2007-2009 | 6/1/2006 | 7,300 | 3,100 | 10,400 | 7,770 | 3,299 | 11,069 | 1 | 3 | 4 | 2 | Y |
| 21 | **USH 53 & CTH O RAMPS (east)*** | NW | Baron | 2003-2005 | 2007-2009 | 6/1/2006 | 12,850 | 2,600 | 15,450 | 14,200 | 3,000 | 17,200 | 1 | 3 | 4 | 2 | Y |
| 22 | **STH 124 & CTH S** | NW | Chippewa | 2003-2005 | 2006-2008 | 10/15/2005 | 8,250 | 3,600 | 11,850 | 5,100 | 4,700 | 9,800 | 1 | 1 | 4 | 2 | Y |
| 27 | **CANAL AND 25TH ST.** | SE | Milwaukee | 2003-2005 | 2006-2008 | 9/15/2005 | 13,600 | 10,500 | 24,100 | 18,400 | 4,900 | 23,300 | 2 | 3 | 3 | 3 | |
| 28 | **STH 38 & CTH K(NORTHWESTERN)** | SE | Racine | 2004-2006 | 2008-2010 | 11/15/2007 | 14,200 | 2,300 | 16,500 | 8,960 | 4,160 | 13,120 | 2 | 3 | 3 | 2 | Y |
| 29 | **Elkhorn Rd (Bus 12)/Bluff Rd/Clay St** | SE | Walworth | 2004-2006 | 2008-2010 | 10/15/2007 | 10,050 | 2,100 | 12,150 | 7,100 | 2,100 | 9,200 | 2 | 2 | 4 | 1 | Y |
| 35 | **STH 92/8TH AND STH 92/SPRINGDALE** | SW | Dane | 2001-2003 | 2005-2007 | 4/27/2004 | 20,000 | 8,000 | 28,000 | 17,400 | 6,650 | 24,050 | 2 | 3 | 4 | 4 | |
| 36 | **THOMPSON AND COMMERICAL** | SW | Dane | 2001-2003 | 2005-2007 | 10/18/2004 | 14,000 | 4,108 | 18,108 | 15,500 | 9,600 | 25,100 | 2 | 3 | 4 | 3 | |
| 37 | **THOMPSON AND STH 30** | SW | Dane | 2001-2003 | 2005-2007 | 10/18/2004 | 9,695 | 4,284 | 13,979 | 13,575 | 3,300 | 16,875 | 2 | 3 | 3 | 3 | |
| 38 | **USH 12 RAMP & PARMENTER** | SW | Dane | 2003-2005 | 2007-2009 | 10/15/2006 | 10,200 | 4,500 | 14,700 | 9,000 | 5,950 | 14,950 | 2 | 3 | 4 | 3 | Y |

Area Type: 1. Rural, 2. Suburban, 3. Urban

Intersection Traffic Control Before: 1. No Control or yield, 2. Minor Road Stop Control, 3. All-Way Stop Control, 4. Signalized Intersection

**Table 11 Roundabout Locations NOT included in the Safety Study**

| Roundabout Location | City | WisDOT Region | Reason Not Included in Study |
|---|---|---|---|
| Lake Park & Kensington | Appleton | NE | Intersection did not exist previously, no before data |
| CTH O (Superior) & Wilgus/STH 40 | Sheboygan | NE | Intersection did not exist previously, no before data |
| STH 35 & Hanley (west ramp terminal) | Hudson | NW | Original intersection was at-grade, while new configuration is grade-separated making comparison not possible |
| STH 35 & Hanley (east ramp terminal) | Hudson | NW | Original intersection was at-grade, while new configuration is grade-separated making comparison not possible |
| 5th/6th Street & Florida | Milwaukee | SE | Original intersection was several closely spaced intersections and new configuration is a roundabout, making comparison difficult |
| I-43 NB Off Ramp/Moorland Road | New Berlin | SE | Location had modifications made through 2009 |

## Appendix B- HSM SPFs and Results

**TABLE 12 Safety Performance Functions Used in HSM EB Analysis**

| Intersection Type | Total Crashes | | | | | Fatal and Injury Crashes | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | a | B | C | k | Source | a | b | c | k | Source |
| 2Rur3ST | -9.86 | 0.79 | 0.49 | 0.54 | HSM 10-18 | -9.35 | 0.71 | 0.21 | 1.23 | SA 1.2 |
| 2Rur4ST | -8.56 | 0.6 | 0.61 | 0.24 | HSM 10-19 | -9.36 | 0.66 | 0.4 | 0.001 | SA 1.2 |
| 2Urb4ST | -8.9 | 0.82 | 0.25 | 0.4 | HSM 12-30 | -11.13 | 0.93 | 0.28 | 0.48 | HSM 12-30 |
| 2Urb4STALL | -12.37 | 1.22 | 0.27 | 0.47 | SA 1.2 | -10.02 | 1.27 | -0.22 | 0.89 | SA 1.2 |
| 2Urb4YD | -8.9 | 0.82 | 0.25 | 0.4 | HSM 12-30 (URB4ST) | -11.13 | 0.93 | 0.28 | 0.48 | HSM 12-30 (URB4ST) |
| 4Rur4SG | -7.182 | 0.722 | 0.337 | 0.277 | HSM 11-22 | -6.393 | 0.638 | 0.232 | 0.218 | HSM 11-22 |
| 4Rur4ST | -10.008 | 0.848 | 0.448 | 0.494 | HSM 11-22 | -11.554 | 0.888 | 0.525 | 0.742 | HSM 11-22 |
| 4Urb3ST | -13.36 | 1.11 | 0.41 | 0.8 | HSM 12-30 | -14.01 | 1.16 | 0.3 | 0.69 | HSM 12-30 |
| 4Urb3STALL | -12.37 | 1.22 | 0.27 | 0.47 | SA 1.2 | -10.02 | 1.27 | -0.22 | 0.89 | |
| 4Urb4SG | -10.99 | 1.07 | 0.23 | 0.39 | HSM 12-30 | -13.14 | 1.18 | 0.22 | 0.33 | HSM 12-30 |
| 4Urb4ST | -8.9 | 0.82 | 0.25 | 0.4 | HSM 12-30 | -11.13 | 0.93 | 0.28 | 0.48 | HSM 12-30 |
| 4Urb4STALL | -12.37 | 1.22 | 0.27 | 0.47 | SA 1.2 | -10.02 | 1.27 | -0.22 | 0.89 | SA 1.2 |
| 4Urb4YD | -8.9 | 0.82 | 0.25 | 0.4 | HSM 12-30 (URB4ST) | -11.13 | 0.93 | 0.28 | 0.48 | HSM 12-30 (URB4ST) |

# TABLE 13 EB Analysis Total Crashes

| Site # | Location | Intersection Type | AADT Major-Before | AADT Minor-Before | SPF Predicted Annual Crashes | Overdispersion Factor k | Years in the Before Period | Observed Total Crashes-Before | Expected Crashes-EB | AADT Major-After | AADT Minor-After | SPF Predicted Annual Crashes After | Adjustment Factor Due to SPF Before and After | Years in the After Period | Expected Crashes -EB Adjusted by AADT and Years in After Period | Observed Total Crashes-After | B-A | B-A [ % Reduction=100(B-A)/B] |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 2 | STH 54/Gaynor St/17th St | 4Urb4ST | 17,875 | 3,875 | 3.301 | 0.400 | 3 | 17.00 | 5.190 | 12,500 | 5,600 | 2.700 | 0.818 | 3 | 12.732 | 20.00 | -7.27 | -57.09 |
| 4 | CTH F/S. Ninth St. | 2Urb4ST | 8,700 | 3,300 | 1.757 | 0.400 | 3 | 1.00 | 0.791 | 10,100 | 1,800 | 1.707 | 0.971 | 3 | 2.306 | 4.00 | -1.69 | -73.47 |
| 5 | CTH F/Suburban Dr. | 2Urb4ST | 7,400 | 3,250 | 1.533 | 0.400 | 3 | 2.00 | 0.972 | 6,450 | 1,800 | 1.182 | 0.771 | 3 | 2.247 | 0.00 | 2.25 | 100.00 |
| 6 | STH 32/57 and STH 96 | 2Urb4STALL | 7,250 | 3,500 | 1.969 | 0.470 | 3 | 7.00 | 2.237 | 7,250 | 3,500 | 1.969 | 1.000 | 3 | 6.711 | 8.00 | -1.29 | -19.21 |
| 7 | STH 141 / Allouez Ave | 2Rur4ST | 7,000 | 1,700 | 3.631 | 0.240 | 3 | 12.00 | 3.898 | 10,100 | 3,500 | 7.029 | 1.936 | 3 | 22.636 | 11.00 | 11.64 | 51.41 |
| 8 | STH 32/STH 57 Broadway | 4Urb4SG | 32,500 | 15,600 | 10.453 | 0.390 | 3 | 9.00 | 3.563 | 50,000 | 24,900 | 18.456 | 1.766 | 3 | 18.874 | 43.00 | -24.13 | -127.82 |
| 9 | STH 55/CTH KK | 2Rur4ST | 10,300 | 4,500 | 8.291 | 0.240 | 3 | 20.00 | 6.900 | 8,950 | 3,650 | 6.707 | 0.809 | 3 | 16.745 | 5.00 | 11.74 | 70.14 |
| 10 | Lake Park/Plank Rd (CTH LP/CTH P) | 2Urb4ST | 8,250 | 3,850 | 1.748 | 0.400 | 3 | 0.00 | 0.564 | 7,300 | 4,450 | 1.640 | 0.938 | 3 | 1.588 | 3.00 | -1.41 | -88.93 |
| 12 | CTH N / Emons Road | 2Rur4ST | 7,800 | 800 | 2.447 | 0.240 | 3 | 2.00 | 1.311 | 12,200 | 2,100 | 5.765 | 2.356 | 3 | 9.269 | 5.00 | 4.27 | 46.06 |
| 15 | STH 28/32 (high speed) | 2Rur4ST | 8,350 | 2,950 | 5.650 | 0.240 | 3 | 8.00 | 3.255 | 5,300 | 3,750 | 4.979 | 0.881 | 3 | 8.607 | 11.00 | -2.39 | -27.81 |
| 17 | STH 42/ I-43, Interchange Ramps (West) | 4Rur4SG | 10,700 | 1,300 | 6.911 | 0.277 | 3 | 9.50 | 3.722 | 23,000 | 3,000 | 15.917 | 2.303 | 3 | 25.718 | 12.50 | 13.22 | 51.40 |
| 41 | STH 42/ I-43, Interchange Ramps (East) | 4Rur4SG | 15,100 | 4,700 | 13.665 | 0.277 | 3 | 15.50 | 5.854 | 20,000 | 8,076 | 20.089 | 1.470 | 3 | 25.820 | 15.50 | 10.32 | 39.97 |
| 18 | STH 42/Vanguard, Wal-Mart entrance | 4Rur4SG | 11,600 | 1,500 | 7.687 | 0.277 | 3 | 2.00 | 1.617 | 20,000 | 7,000 | 19.144 | 2.490 | 3 | 12.080 | 8.00 | 4.08 | 33.78 |
| 19 | Breezewood ln/Tullar Rd | 2Urb4YD | 13,000 | 4,800 | 2.682 | 0.400 | 3 | 4.00 | 1.653 | 11,350 | 4,800 | 2.400 | 0.895 | 3 | 4.437 | 6.00 | -1.56 | -35.23 |
| 20 | US 53 ramps and CTH O (West) | 2Urb4ST | 7,300 | 3,100 | 1.498 | 0.400 | 3 | 10.50 | 2.784 | 7,770 | 3,299 | 1.601 | 1.069 | 3 | 8.930 | 3.50 | 5.43 | 60.80 |
| 21 | US 53 ramps and CTH O (East) | 2Urb4ST | 12,850 | 2,600 | 2.279 | 0.400 | 3 | 5.50 | 1.953 | 14,200 | 3,000 | 2.564 | 1.125 | 3 | 6.590 | 4.50 | 2.09 | 31.71 |
| 22 | STH 124/CTH S | 2Rur4ST | 8,250 | 3,600 | 6.334 | 0.240 | 3 | 16.00 | 5.513 | 5,100 | 4,700 | 5.584 | 0.882 | 3 | 14.583 | 6.00 | 8.58 | 58.85 |
| 27 | Canal St/25th Ave | 4Urb3STALL | 13,600 | 10,500 | 5.707 | 0.470 | 3 | 1.00 | 0.927 | 18,400 | 4,900 | 6.718 | 1.177 | 3 | 3.274 | 13.00 | -9.73 | -297.01 |
| 28 | STH 38/CTH K | 4Urb3ST | 14,200 | 2,300 | 1.532 | 0.800 | 3 | 28.00 | 7.665 | 8,960 | 4,160 | 1.171 | 0.765 | 3 | 17.586 | 20.00 | -2.41 | -13.73 |
| 29 | Elkhorn Rd (Bus 12)/Bluff Rd/Clay St | 4Urb4YD | 10,050 | 2,100 | 1.766 | 0.400 | 3 | 3.00 | 1.246 | 7,100 | 2,100 | 1.329 | 0.752 | 3 | 2.811 | 3.00 | -0.19 | -6.74 |
| 35 | STH 78/STH 92, 8th St, Springdale, CTH ID | 4Urb4SG | 20,000 | 8,000 | 5.332 | 0.390 | 3 | 14.00 | 4.759 | 17,400 | 6,650 | 4.403 | 0.826 | 3 | 11.788 | 11.00 | 0.79 | 6.68 |
| 36 | Thompson and Commercial (North) | 4Urb4STALL | 14,000 | 4,108 | 4.589 | 0.470 | 3 | 20.00 | 6.389 | 15,500 | 9,600 | 6.534 | 1.424 | 3 | 27.289 | 39.00 | -11.71 | -42.91 |
| 37 | Thompson and STH 30 (South) | 4Urb3STALL | 9,695 | 4,284 | 2.965 | 0.470 | 3 | 13.00 | 4.069 | 13,575 | 3,300 | 4.166 | 1.405 | 3 | 17.154 | 8.00 | 9.15 | 53.36 |
| 38 | Old STH 12/Parmenter | 4Urb4STALL | 10,200 | 4,500 | 3.196 | 0.470 | 3 | 9.00 | 3.036 | 9,000 | 5,950 | 2.959 | 0.926 | 3 | 8.430 | 2.00 | 6.43 | 76.27 |

# TABLE 14  HSM EB Analysis Fatal and Injury Crashes

| Site # | Location | Intersection Type | AADT Major-Before | AADT Minor-Before | SPF Predicted Annual Crashes | Overdispersion Factor k | Years in the Before Period | Observed Fatal and Injury Crashes-Before | Expected Crashes-EB | AADT Major-After | AADT Minor-After | SPF Predicted Annual Crashes After | Adjustment Factor Due to SPF Before and After | Years in the After Period | Expected Crashes -EB Adjusted by AADT and Years in After Period | Observed Fatal and Injury Crashes-After | B-A | B-A [% Reduction=100(B-A)/B] |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 2 | STH 54/Gaynor St/17th St | 4Urb4ST | 17,875 | 3,875 | 1.335 | 0.480 | 3 | 8.00 | 2.21 | 12,500 | 5,600 | 1.062 | 0.7949 | 3 | 5.273 | 0.00 | 5.273 | 100.00 |
| 4 | CTH F/S. Ninth St. | 2Urb4ST | 8,700 | 3,300 | 0.653 | 0.480 | 3 | 1.00 | 0.50 | 10,100 | 1,800 | 0.634 | 0.9695 | 3 | 1.449 | 3.00 | -1.551 | -107.00 |
| 5 | CTH F/Suburban Dr. | 2Urb4ST | 7,400 | 3,250 | 0.560 | 0.480 | 3 | 0.00 | 0.31 | 6,450 | 1,800 | 0.418 | 0.7459 | 3 | 0.694 | 0.00 | 0.694 | 100.00 |
| 6 | STH 32/57 and STH 96 | 2Urb4STALL | 7,250 | 3,500 | 0.591 | 0.890 | 3 | 1.00 | 0.43 | 7,250 | 3,500 | 0.591 | 1 | 3 | 1.300 | 2.00 | -0.700 | -53.90 |
| 7 | STH 141 / Allouez Ave | 2Rur4ST | 7,000 | 1,700 | 0.582 | 0.001 | 3 | 3.00 | 0.58 | 10,100 | 3,500 | 0.990 | 1.7003 | 3 | 2.973 | 3.00 | -0.027 | -0.92 |
| 8 | STH 32/STH 57 Broadway | 4Urb4SG | 32,500 | 15,600 | 3.466 | 0.330 | 3 | 1.00 | 1.04 | 50,000 | 24,900 | 6.387 | 1.8426 | 3 | 5.751 | 3.00 | 2.751 | 47.83 |
| 9 | STH 55/CTH KK (high speed) | 2Rur4ST | 10,300 | 4,500 | 1.109 | 0.001 | 3 | 11.00 | 1.12 | 8,950 | 3,650 | 0.929 | 0.8382 | 3 | 2.809 | 1.00 | 1.809 | 64.40 |
| 10 | Lake Park/Plank Rd (CTH LP/CTH P) | 2Urb4ST | 8,250 | 3,850 | 0.649 | 0.480 | 3 | 0.00 | 0.34 | 7,300 | 4,450 | 0.604 | 0.9294 | 3 | 0.936 | 1.00 | -0.064 | -6.87 |
| 12 | CTH N / Emons Road | 2Rur4ST | 7,800 | 800 | 0.462 | 0.001 | 3 | 2.00 | 0.46 | 12,200 | 2,100 | 0.914 | 1.9764 | 3 | 2.743 | 2.00 | 0.743 | 27.10 |
| 15 | STH 28/32 (high speed) | 2Rur4ST | 8,350 | 2,950 | 0.815 | 0.001 | 3 | 2.00 | 0.81 | 5,300 | 3,750 | 0.665 | 0.8154 | 3 | 1.993 | 1.00 | 0.993 | 49.83 |
| 17 | STH 42/ I-43, Interchange Ramps (West) | 4Rur4SG | 10,700 | 1,300 | 3.287 | 0.218 | 3 | 2.00 | 1.50 | 23,000 | 3,000 | 6.502 | 1.9783 | 3 | 8.894 | 4.50 | 4.394 | 49.40 |
| 41 | STH 42/ I-43, Interchange Ramps (East) | 4Rur4SG | 15,100 | 4,700 | 5.517 | 0.218 | 3 | 2.00 | 1.72 | 20,000 | 8,076 | 7.483 | 1.3565 | 3 | 6.996 | 3.50 | 3.496 | 49.97 |
| 18 | STH 42/Vanguard, Wal-Mart entrance | 4Rur4SG | 11,600 | 1,500 | 3.577 | 0.218 | 3 | 1.00 | 1.30 | 20,000 | 7,000 | 7.239 | 2.0237 | 3 | 7.921 | 0.00 | 7.921 | 100.00 |
| 19 | Breezewood ln/Tullar Rd | 2Urb4YD | 13,000 | 4,800 | 1.054 | 0.480 | 3 | 2.00 | 0.82 | 11,350 | 4,800 | 0.929 | 0.8814 | 3 | 2.170 | 0.00 | 2.170 | 100.00 |
| 20 | US 53 ramps and CTH O (West) | 2Urb4ST | 7,300 | 3,100 | 0.545 | 0.480 | 3 | 1.00 | 0.45 | 7,770 | 3,299 | 0.588 | 1.0784 | 3 | 1.463 | 1.50 | -0.037 | -2.55 |
| 21 | US 53 ramps and CTH O (East) | 2Urb4ST | 12,850 | 2,600 | 0.879 | 0.480 | 3 | 0.00 | 0.39 | 14,200 | 3,000 | 1.004 | 1.1422 | 3 | 1.329 | 2.50 | -1.171 | -88.10 |
| 22 | STH 124/CTH S | 2Rur4ST | 8,250 | 3,600 | 0.876 | 0.001 | 3 | 11.00 | 0.88 | 5,100 | 4,700 | 0.709 | 0.8099 | 3 | 2.146 | 1.00 | 1.146 | 53.40 |
| 27 | Canal St/25th Ave | 4Urb3STALL | 13,600 | 10,500 | 1.031 | 0.890 | 3 | 0.00 | 0.27 | 18,400 | 4,900 | 1.790 | 1.736 | 3 | 1.431 | 2.00 | -0.569 | -39.78 |
| 28 | STH 38/CTH K | 4Urb3ST | 14,200 | 2,300 | 0.550 | 0.690 | 3 | 9.00 | 1.86 | 8,960 | 4,160 | 0.385 | 0.7002 | 3 | 3.897 | 2.00 | 1.897 | 48.68 |
| 29 | Elkhorn Rd (Bus 12)/Bluff Rd/Clay St | 4Urb4YD | 10,050 | 2,100 | 0.658 | 0.480 | 3 | 1.00 | 0.50 | 7,100 | 2,100 | 0.477 | 0.7239 | 3 | 1.086 | 0.00 | 1.086 | 100.00 |
| 35 | STH 78/STH 92, 8th St, Springdale, CTH ID | 4Urb4SG | 20,000 | 8,000 | 1.688 | 0.330 | 3 | 1.00 | 0.84 | 17,400 | 6,650 | 1.375 | 0.8147 | 3 | 2.054 | 0.00 | 2.054 | 100.00 |
| 36 | Thompson and Commercial (North) | 4Urb4STALL | 14,000 | 4,108 | 1.315 | 0.890 | 3 | 12.00 | 3.40 | 15,500 | 9,600 | 1.242 | 0.9441 | 3 | 9.644 | 7.00 | 2.644 | 27.42 |
| 37 | Thompson and STH 30 (South) | 4Urb3STALL | 9,695 | 4,284 | 0.817 | 0.890 | 3 | 5.00 | 1.40 | 13,575 | 3,300 | 1.327 | 1.624 | 3 | 6.819 | 1.00 | 5.819 | 85.34 |
| 38 | Old STH 12/Parmenter | 4Urb4STALL | 10,200 | 4,500 | 0.862 | 0.890 | 3 | 4.00 | 1.19 | 9,000 | 5,950 | 0.692 | 0.8022 | 3 | 2.865 | 0.00 | 2.865 | 100.00 |

## Appendix C- Wisconsin SPFs and Results

**TABLE 15 Wisconsin Specific SPF**

| Intersection type | Sample Size | Number of Approaches | Traffic Control | Number of Lanes | Divided | AADT Range | Model |
|---|---|---|---|---|---|---|---|
| 4S | 686 | 4 | Signalized | 2 or 4 | Both | [3100,82075] | crashes/year=exp(-7.22)(AADT)$^{0.924}$, k=0.2131 |
| | | | | | | | (F+I) crashes/year= exp(-9.1951)(AADT)$^{1.0259}$ k=0.2415 |
| 4USD | 220 | 4 | Unsignalized | 2 or 4 | Divided | [1000, 54000] | crashes/year=exp(-2.5753)*(AADT)$^{0.4081}$, k=0.2677 |
| | | | | | | | (F+I) crashes/year =exp(-3.5040)*(AADT)$^{0.4216}$, k=0.3087 |
| 4USUD | 442 | 4 | Unsignalized | 2 or 4 | Undivided | [1055,37600] | Crashes/year=exp(-5.4529)(AADT)$^{0.7048}$ k=0.3236 |
| | | | | | | | (F+I) crashes/year=exp(-4.8312)(AADT)$^{0.5437}$ k=0.4659 |
| 3US2L | 190 | 3 | Unsignalized | 2 | Both | [485,30600] | Crashes/year=exp(-6.1337)(AADT)$^{0.7386}$ k=0.3062 |
| | | | | | | | (F+I) crashes/year=exp(-6.0989)(AADT)$^{0.6265}$ k=0.3917 |
| 3US4L | 106 | 3 | Unsignalized | 4 | Both | [1000,46750] | Crashes/year=exp(-2.3965)(AADT)$^{0.3696}$ k=0.3383 |
| | | | | | | | (F+I) crashes/year=exp(-3.5040)(AADT)$^{0.4216}$ k=0.4817 |

Note:

1. In "4S" intersections, though lane factor is statistically significant, the AIC value is no larger than the single variable AADT model for both total and (F+I) crashes.
2. Area type (U/R) is not statistically significant in any models
3. (F+I) crashes stand for fatal, type A, B, and C crashes. Only PDO crashes are excluded.
4. For any intersection types without a lane factor (2 or 4), the number of lane is not statistically significant.
5. All models are developed with NB distribution. Lagrange Multiplier statistics indicate the overdispersion factor k is statistically significant at 5 percent level of significance.

## TABLE 16 Wisconsin Specific EB Analysis for Total Crashes

| Site # | Location | Intersection Type | AADT-Before | SPF Predicted Annual Crashes | Overdispersion Factor k | Years in the Before Period | Observed Total Crashes-Before | Expected Crashes-EB | AADT-After | AADT Ratio | Adjustment Factor Due to AADT | Years in the After Period | Expected Crashes -EB Adjusted by AADT and Years in After Period | Observed Total Crashes-After | B-A | B-A [% Reduction=100(B-A)/B] |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 2 | STH 54/Gaynor St/17th St | ISET4USUD | 21750 | 4.885 | 0.324 | 3 | 17.00 | 5.531 | 18,100 | 0.832 | 0.879 | 3 | 14.577 | 20.00 | -5.423 | -37.204 |
| 4 | CTH F/S. Ninth St. | ISET4USD | 12000 | 3.518 | 0.268 | 3 | 1.00 | 1.166 | 11,900 | 0.992 | 0.997 | 3 | 3.486 | 4.00 | -0.514 | -14.758 |
| 5 | CTH F/Suburban Dr. | ISET4USUD | 10650 | 2.953 | 0.324 | 3 | 2.00 | 1.258 | 8,250 | 0.775 | 0.835 | 3 | 3.152 | 0.00 | 3.152 | 100.000 |
| 6 | STH 32/57 and STH 96 | ISET4USUD | 10750 | 2.973 | 0.324 | 3 | 7.00 | 2.498 | 10,750 | 1.000 | 1.000 | 3 | 7.494 | 8.00 | -0.506 | -6.756 |
| 7 | STH 141 / Allouez Ave | ISET4USUD | 8700 | 2.561 | 0.324 | 3 | 12.00 | 3.587 | 13,600 | 1.563 | 1.370 | 3 | 14.744 | 11.00 | 3.744 | 25.395 |
| 8 | STH 32/STH 57 Broadway | ISET4S | 48100 | 15.460 | 0.213 | 3 | 9.00 | 4.145 | 74,900 | 1.557 | 1.506 | 3 | 18.724 | 43.00 | -24.276 | -129.651 |
| 9 | STH 55/CTH KK | ISET4USUD | 14800 | 3.724 | 0.324 | 3 | 20.00 | 6.029 | 12,600 | 0.851 | 0.893 | 3 | 16.148 | 5.00 | 11.148 | 69.036 |
| 10 | Lake Park/Plank Rd (CTH LP/CTH P) | ISET4USUD | 12100 | 3.231 | 0.324 | 3 | 0.00 | 0.781 | 11,750 | 0.971 | 0.980 | 3 | 2.295 | 3.00 | -0.705 | -30.703 |
| 12 | CTH N / Emons Road | ISET4USUD | 8600 | 2.540 | 0.324 | 3 | 2.00 | 1.207 | 14,300 | 1.663 | 1.431 | 3 | 5.183 | 5.00 | 0.183 | 3.524 |
| 15 | STH 28/32 (high speed) | ISET4USUD | 11300 | 3.079 | 0.324 | 3 | 8.00 | 2.770 | 9,050 | 0.801 | 0.855 | 3 | 7.106 | 11.00 | -3.894 | -54.790 |
| 17 | STH 42/ I-43, Interchange Ramps (West) | ISET4S | 12000 | 4.286 | 0.213 | 3 | 9.50 | 3.466 | 26,000 | 2.167 | 2.043 | 3 | 21.244 | 12.50 | 8.744 | 41.160 |
| 41 | STH 42/ I-43, Interchange Ramps (East) | ISET4S | 19800 | 6.808 | 1.213 | 3 | 15.50 | 5.230 | 28,076 | 1.418 | 1.381 | 3 | 21.667 | 15.50 | 6.167 | 28.462 |
| 18 | STH 42/Vanguard, Wal-Mart entrance | ISET4USUD | 13100 | 4.648 | 0.213 | 3 | 2.00 | 1.670 | 27,000 | 2.061 | 1.951 | 3 | 9.771 | 8.00 | 1.771 | 18.124 |
| 19 | Breezewood ln/Tullar Rd | ISET4USUD | 17800 | 4.242 | 0.324 | 3 | 4.00 | 1.902 | 16,150 | 0.907 | 0.934 | 3 | 5.327 | 6.00 | -0.673 | -12.638 |
| 20 | US 53 ramps and CTH O (West) | ISET4USD | 10400 | 3.318 | 0.268 | 3 | 10.50 | 3.450 | 11,069 | 1.064 | 1.026 | 3 | 10.618 | 3.50 | 7.118 | 67.037 |
| 21 | US 53 ramps and CTH O (East) | ISET4USD | 15450 | 3.900 | 0.268 | 3 | 5.50 | 2.333 | 17,200 | 1.113 | 1.045 | 3 | 7.314 | 4.50 | 2.814 | 38.472 |
| 22 | STH 124/CTH S | ISET4USUD | 11850 | 3.184 | 0.324 | 3 | 16.00 | 4.808 | 9,800 | 0.827 | 0.875 | 3 | 12.617 | 6.00 | 6.617 | 52.444 |
| 27 | Canal St/25th Ave | ISET3US4LANE | 24100 | 3.791 | 0.338 | 3 | 1.00 | 1.047 | 23,300 | 0.967 | 0.988 | 3 | 3.101 | 13.00 | -9.899 | -319.217 |
| 28 | STH 38/CTH K | ISET3US4LANE | 16500 | 3.296 | 0.338 | 3 | 28.00 | 7.944 | 13,120 | 0.795 | 0.919 | 3 | 21.896 | 20.00 | 1.896 | 8.659 |
| 29 | Elkhorn Rd (Bus 12)/Bluff Rd/Clay St | ISET4USUD | 12150 | 3.241 | 0.324 | 3 | 3.00 | 1.540 | 9,200 | 0.757 | 0.822 | 3 | 3.799 | 3.00 | 0.799 | 21.026 |
| 35 | STH 78/STH 92, 8th St, Springdale, CTH ID | ISET4S | 28000 | 9.378 | 0.213 | 3 | 14.00 | 5.340 | 24,050 | 0.859 | 0.869 | 3 | 13.921 | 11.00 | 2.921 | 20.983 |
| 36 | Thompson and Commercial (North) | ISET4USUD | 18108 | 4.293 | 0.324 | 3 | 20.00 | 6.207 | 25,100 | 1.386 | 1.259 | 3 | 23.441 | 39.00 | -15.559 | -66.376 |
| 37 | Thompson and STH 30 (South) | ISET3US4LANE | 13979 | 3.100 | 0.338 | 3 | 13.00 | 4.036 | 16,875 | 1.207 | 1.072 | 3 | 12.980 | 8.00 | 4.980 | 38.368 |
| 38 | Old STH 12/Parmenter | ISET4USD | 14700 | 3.822 | 0.268 | 3 | 9.00 | 3.202 | 14,950 | 1.017 | 1.007 | 3 | 9.672 | 2.00 | 7.672 | 79.322 |

# TABLE 17 Wisconsin Specific EB Analysis for Fatal and Injury Crashes

| Site # | Location | Intersection Type | AADT-Before | SPF Predicted (Fatal+Injury) Crashes | Overdispersion Factor k | Years in the Before Period | Observed (Fatal+Injury) Crashes-Before | Expected (Fatal+Injury) Crashes-EB | AADT-After | AADT Ratio | Adjustment Factor Due to AADT | Years in the After Period | Expected (Fatal+Injury) Crashes -EB Adjusted by AADT and Years in After Period | Observed (Fatal+Injury) Crashes-After | B-A | B-A [% Reduction=100(B-A)/B] |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 2 | STH 54/Gaynor St/17th St | ISET4USUD | 21750 | 1.820 | 0.466 | 3 | 8.00 | 2.43 | 18,100 | 0.832 | 0.905 | 3 | 6.591 | 0.00 | 6.591 | 100.00 |
| 4 | CTH F/S. Ninth St. | ISET4USD | 12000 | 1.578 | 0.309 | 3 | 1.00 | 0.84 | 11,900 | 0.992 | 0.996 | 3 | 2.508 | 3.00 | -0.492 | -19.62 |
| 5 | CTH F/Suburban Dr. | ISET4USUD | 10650 | 1.235 | 0.466 | 3 | 0.00 | 0.45 | 8,250 | 0.775 | 0.870 | 3 | 1.183 | 0.00 | 1.183 | 100.00 |
| 6 | STH 32/57 and STH 96 | ISET4USUD | 10750 | 1.241 | 0.466 | 3 | 1.00 | 0.67 | 10,750 | 1.000 | 1.000 | 3 | 1.996 | 2.00 | -0.004 | -0.22 |
| 7 | STH 141 / Allouez Ave | ISET4USUD | 8700 | 1.106 | 0.466 | 3 | 3.00 | 1.04 | 13,600 | 1.563 | 1.275 | 3 | 3.984 | 3.00 | 0.984 | 24.70 |
| 8 | STH 32/STH 57 Broadway | ISET4S | 48100 | 6.457 | 0.242 | 3 | 1.00 | 1.41 | 74,900 | 1.557 | 1.575 | 3 | 6.671 | 3.00 | 3.671 | 55.03 |
| 9 | STH 55/CTH KK (high speed) | ISET4USUD | 14800 | 1.476 | 0.466 | 3 | 11.00 | 2.95 | 12,600 | 0.851 | 0.916 | 3 | 8.113 | 1.00 | 7.113 | 87.67 |
| 10 | Lake Park/Plank Rd (CTH LP/CTH P) | ISET4USUD | 12100 | 1.323 | 0.466 | 3 | 0.00 | 0.46 | 11,750 | 0.971 | 0.984 | 3 | 1.371 | 1.00 | 0.371 | 27.07 |
| 12 | CTH N / Emons Road | ISET4USUD | 8600 | 1.099 | 0.466 | 3 | 2.00 | 0.84 | 14,300 | 1.663 | 1.318 | 3 | 3.311 | 2.00 | 1.311 | 39.60 |
| 15 | STH 28/32 (high speed) | ISET4USUD | 11300 | 1.275 | 0.466 | 3 | 2.00 | 0.89 | 9,050 | 0.801 | 0.886 | 3 | 2.354 | 1.00 | 1.354 | 57.52 |
| 17 | STH 42/ I-43, Interchange Ramps (West) | ISET4S | 12000 | 1.554 | 0.242 | 3 | 2.00 | 1.08 | 26,000 | 2.167 | 2.210 | 3 | 7.189 | 4.50 | 2.689 | 37.40 |
| 41 | STH 42/ I-43, Interchange Ramps (East) | ISET4S | 19800 | 2.598 | 1.242 | 3 | 2.00 | 0.85 | 28,076 | 1.418 | 1.431 | 3 | 3.638 | 3.50 | 0.138 | 3.80 |
| 18 | STH 42/Vanguard, Wal-Mart entrance | ISET4S | 13100 | 1.700 | 0.242 | 3 | 1.00 | 0.95 | 27,000 | 2.061 | 2.100 | 3 | 5.959 | 0.00 | 5.959 | 100.00 |
| 19 | Breezewood ln/Tullar Rd | ISET4USUD | 17800 | 1.632 | 0.466 | 3 | 2.00 | 0.96 | 16,150 | 0.907 | 0.948 | 3 | 2.734 | 0.00 | 2.734 | 100.00 |
| 20 | US 53 ramps and CTH O (West) | ISET4USD | 10400 | 1.485 | 0.309 | 3 | 1.00 | 0.82 | 11,069 | 1.064 | 1.027 | 3 | 2.520 | 1.50 | 1.020 | 40.48 |
| 21 | US 53 ramps and CTH O (East) | ISET4USD | 15450 | 1.755 | 0.309 | 3 | 0.00 | 0.67 | 17,200 | 1.113 | 1.046 | 3 | 2.098 | 2.50 | -0.402 | -19.14 |
| 22 | STH 124/CTH S | ISET4USUD | 11850 | 1.308 | 0.466 | 3 | 11.00 | 2.83 | 9,800 | 0.827 | 0.902 | 3 | 7.665 | 1.00 | 6.665 | 86.95 |
| 27 | Canal St/25th Ave | ISET3US4LANE | 24100 | 1.393 | 0.482 | 3 | 0.00 | 0.46 | 23,300 | 0.967 | 0.987 | 3 | 1.369 | 2.00 | -0.631 | -46.06 |
| 28 | STH 38/CTH K | ISET3US4LANE | 16500 | 1.207 | 0.482 | 3 | 9.00 | 2.35 | 13,120 | 0.795 | 0.917 | 3 | 6.454 | 2.00 | 4.454 | 69.01 |
| 29 | Elkhorn Rd (Bus 12)/Bluff Rd/Clay St | ISET4USUD | 12150 | 1.326 | 0.466 | 3 | 1.00 | 0.68 | 9,200 | 0.757 | 0.860 | 3 | 1.757 | 0.00 | 1.757 | 100.00 |
| 35 | STH 78/STH 92, 8th St, Springdale, CTH ID | ISET4S | 28000 | 3.706 | 0.242 | 3 | 1.00 | 1.25 | 24,050 | 0.859 | 0.856 | 3 | 3.205 | 0.00 | 3.205 | 100.00 |
| 36 | Thompson and Commercial (North) | ISET4USUD | 18108 | 1.648 | 0.466 | 3 | 12.00 | 3.29 | 25,100 | 1.386 | 1.194 | 3 | 11.779 | 7.00 | 4.779 | 40.57 |
| 37 | Thompson and STH 30 (South) | ISET3US4LANE | 13979 | 1.133 | 0.482 | 3 | 5.00 | 1.46 | 16,875 | 1.207 | 1.074 | 3 | 4.718 | 1.00 | 3.718 | 78.80 |
| 38 | Old STH 12/Parmenter | ISET4USD | 14700 | 1.719 | 0.309 | 3 | 4.00 | 1.48 | 14,950 | 1.017 | 1.007 | 3 | 4.478 | 0.00 | 4.478 | 100.00 |

## Appendix D

1. Simulation Study of Access Management at Modern Roundabouts: Pedestrian Crosswalk Treatments
2. Multimodal Accessibility of Modern Roundabouts: Intelligent Management System Versus Common Signalization Scheme
3. Intelligent Traffic Signal System for Urban Isolated Intersections: Dynamic Pedestrian Accommodation

**Simulation Study of Access Management at Modern Roundabouts: Pedestrian Crosswalk Treatments**

By

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**ABSTRACT:**

A widespread emergence of modern roundabouts in North America has kindled a controversy about the pedestrian access issue. Almost uninterrupted traffic streams, ambient noises, and urban settings make it difficult for the visually impaired to perceive safe gaps only depending on auditory cues. In 2005, the United States Access Board released a revised draft guideline to call for the provision of "*A pedestrian activated traffic signal … for each segment of the crosswalk …*" to ensure access for vision-impaired pedestrians. Access Management Manual prescribes major transportation actions encompassing multimodal streets with sidewalks and adequate pedestrian refuges, without addressing the pedestrian access issue at roundabouts. In North America very few roundabouts were ever outfitted with pedestrian signals, while little research explored signalizing roundabouts for pedestrian access improvement.

This study quantitatively assessed the performance of four pedestrian signals at modern roundabouts under a spectrum of crosswalk layouts, signal installations, and operational conditions, aimed to provide access management community with an objective basis for identifying treatments to improve the roundabout accessibility. "Two-Stage" installation scheme is found more operationally efficient and with its presence no significant differences exist among three layouts. When "One-Stage" scheme operates, "Distant" layout reduces vehicle delay and queue length due to enlarged storage space. HAWK induces the least vehicle delay; while PUFFIN minimizes the pedestrian delay and fully protects pedestrians. These findings are informative to transportation policy-makers, planners, and practitioners in access management community which face the challenge of improving roundabout accessibility to those with impaired visions. Future research directions are identified.

**Key Words:**

Modern Roundabout, Access Management, Pedestrian Crosswalk, Traffic Microsimulation

## BACKGROUND

Since the 1990s, there has been a widespread emergence of modern roundabouts in many states and municipalities of the United States. The keen interest in modern roundabouts can be considerably attributable to their vast success in some European and Oceanian countries. In geometrics, a modern roundabout is an unsignalized intersection which includes a central island encircled by a single-/multiple-lane roadway. Vehicles entering the roundabout need yield to those already navigating on the circulatory lane(s). Its strong appeals can be ascribed to verified safety improvement, bettered circulation efficiency, lessened maintenance cost, and beautified aesthetic impact (*1*). France, leading the world with roughly 15,000 modern roundabouts, has been constructing roundabouts at a rate of 1,000 or so per year (*2*). The inventory in the United States, although expanding in recent years, remains relatively rather limited. As of 2010, an online database (*3*) records only over 1,000 modern roundabouts which maintain in active operations, in sharp contrast to over 40,000 in the rest of the world. Currently, a large number of roundabouts are under construction or in the planning phase in North America. Simultaneously, the flourishing growth of modern roundabouts has kindled a nationwide debate, as a response to the roundabout studies, over the pedestrian access issue (*4*). In previous studies, Ashmead et al. (*5*) found that roundabouts indeed create serious difficulties to the visually impaired, and Wall et al. (*6*) revealed the crossing becomes increasingly difficult as the conflicting vehicle volume rises and multilane facilities are more challenging than single-lane ones to ensure pedestrian-friendly accessibility. Guth et al. (*7*) showed that the crosswalk segment on outbound lane(s) is more hazardous than that on inbound lane(s).

Williams and Levinson (*8*) pointed out "safety, capacity, continuity and connectivity of the roadway network are key" in access management. Through safety research there is a clear link between access design and crash rates (*9*); the access management has major concerns for safety and mobility of a roadway system (*10,11,12*). The Access Management Manual prescribes major transportation actions which encompass multimodal streets with sidewalks and adequate pedestrian refuges, without addressing the pedestrian access issue at roundabouts (*13*). In 2002, the United States Access Board published "Draft Guideline for Accessible Public Rights of Way, Roundabout" which proposes pedestrian signals at all roundabout crosswalks. In 2005, the Access Board released a revised draft to call for the provision of "*A pedestrian activated traffic signal… for each segment of the crosswalk…*(*14*)" at multilane roundabouts to ensure safe access for vision-impaired pedestrians. Operationally, this provision induces an interruption to the vehicular flow continuity which is originally intended in roundabout design. Another critical issue is the enhanced likelihood that the yielding queue spills back into the circulatory lane(s), which has been identified by Inman and Davis (*15*) for some signalized roundabouts.

Various signals have been practically applied to roundabout crosswalks in Europe, Australia (*16*), and Africa (*17)*. However, only a few roundabouts were ever signalized for pedestrians in North America. Two single-lane roundabouts were signalized at university campuses (University of Utah, Salt Lake City; University of North Carolina, Charlotte); while one double-lane roundabout was signalized in Lake Worth of Florida (*15,16*). In western Quebec Canada, a double-lane roundabout at Gatineau possesses a staggered offset crossing with a pedestrian signal on one approach. Currently, only very little research explored the theme of signalizing roundabouts to improve pedestrian access. Rouphail et al. (*18*) found the introduction of a pedestrian-actuated signal adds delays to visually impaired pedestrians compared with sighted pedestrians who cross at unsignalized splitter islands. Schroeder et al. (*19*) investigated two signal alternatives at single-/double-lane roundabouts to make these facilities accessible to the visually impaired. Simulation results reveal the impact of signalization was maximized under oversaturated conditions, but the vehicle delay and the queuing effect can be alleviated through an innovative signal. Lu et al. (*20*) developed an artificially intelligent signal system to better the roundabout accessibility and simulation results show the new system outperforms an existing one in manifold aspects. Although roundabouts are rarely signalized for pedestrian access in the United States, the call from the Access Board and the absence of roundabouts in the Access Management Manual make it imperative for access management community to have more practice-oriented research regarding the roundabout accessibility for pedestrians.

## STUDY OBJECTIVES

Intuitively, the introduction of a pedestrian signal poses additional delays to vehicles. However, it is not easy to quantify projected impacts of a roundabout signalization. This study was aimed to quantitatively assess the performance of four pedestrian signals installed at typical single-/double-lane modern roundabouts where crosswalk geometric layouts and signal installation schemes varied under different traffic conditions. The objective was to provide the access management community with an objective basis for identifying potential crosswalk treatments to improve the roundabout accessibility especially for the visually impaired, seniors, and children, while maintaining good multimodal mobility. Four hypotheses were established. Firstly, operational impact of adding a pedestrian signal is significantly relevant to multimodal traffic intensities (More pedestrians increase the number of pedestrian signal actuations. With more vehicles, each signal actuation incurs a stronger impact upon vehicle delays and an enhanced likelihood of yielding queues' spilling back into the circulatory lane(s)). Secondly, the likelihood of queue spillback can be diminished by shifting the crosswalk segment on the outbound lane(s) farther away from the circulatory lane(s). Furthermore, the vehicle delay is directly proportional to the display length of the "RED" interval. It is anticipated that the signalization with shortened "RED" display can lessen the vehicle delay. Finally, since the pedestrian clearance interval (i.e., "Flashing DON'T WALK (FDW)") is timed per the crossing distance and a design walking speed, the reduction of the "FDW" (thus the "RED") can be achieved per the separate-segment-based installation scheme compared with the whole-crosswalk-based counterpart.

## STUDY METHODOLOGY

From an operational perspective, this study investigated how some crosswalk treatments, which result from variations in three dimensions (i.e., signalization options, geometric layouts, and installation schemes), affect multimodal performance measures under distinct traffic conditions. It is impracticable to scrutinize the performance of these treatments in a real-world context due to potential disruptions and hazards posed to smooth and safe operations if traffic control strategies change on site. Instead, a controllable in-lab platform renders a valid surrogate means by which treatments can be implemented and evaluated in a quantifiable way.

### Study Platform

Traffic simulation is characterized by cost-effectiveness, unobtrusiveness, risk-free nature, and high-speed computation, which makes itself indispensable in the repertoire of transportation researchers. It exports various performance measures some of which are intractable in the field. Most importantly, it empowers researchers with full control over a myriad of operational and geometric factors of interest, and this offers the unique opportunity to evaluate the effectiveness of different study scenarios prior to deployments. VISSIM, a microsimulation program, is widely applied to model diverse transportation facilities due to its multimodal modeling capability, adequate detector function, self-customizable control algorithm, and convenient run-time control (*21*). VISSIM models were found valid for real-world freeways (*22*), urban networks (*23*), crosswalks (*24*), intersections, arterials, and roundabouts (*18,19,20*). It can mimic vehicle-yielding behaviors at roundabout entries, and its link-connector structure is flexible in modeling unique geometries. Hence, VISSIM was employed herein as a quantifiable study platform.

### Signalization Options

In practice, the conventional pedestrian-actuated (PA) signal is widely employed to serve pedestrians in the United States, while the HAWK (High Intensity Activated CrossWalK), also known as Pedestrian Hybrid Beacon, had been experimentally installed at mid-block locations in Tucson Arizona, Portland Oregon, and other cities (*25,26*), before it was approved by the Federal Highway Administration (FHWA) as an official traffic control device added to the MUTCD Chapter 4F which prescribes its application, design, and operation (*27*). PELICAN (PEdestrian LIght CONtrolled) and PUFFIN (Pedestrian User-Friendly INterface) signals have been vastly deployed in Europe and Oceania to manage mid-block crosswalks and, sometimes, roundabouts (*17,28,29*). In the United States, several transportation agencies recently published local guidelines for field deployments of PELICAN and PUFFIN signals, and this signifies their burgeoning deployments in American continent (*30*). FIGURE 1 illustrates the phasing sequences for four signals. Crossing pedestrians press the pushbutton mounted on a post beside the roadway to activate each signal into operations.

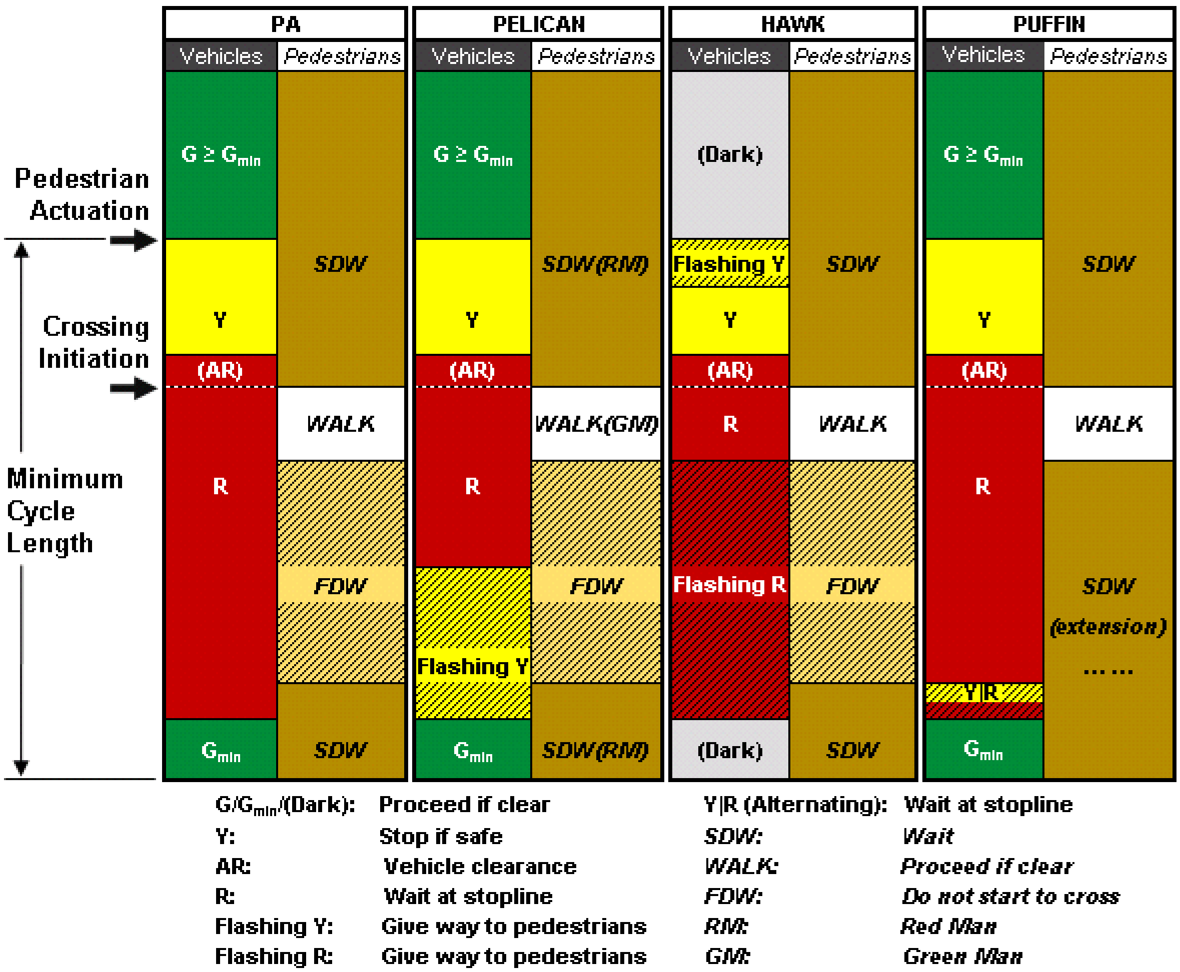



**FIGURE 1 Four pedestrian signal systems under study – phasing schemes comparison.**

*PA*

PA signals ramify into one type which is integrated into other signal phases (usually at intersections) and the other which operates independently. Both comply with relevant MUTCD design standards (*27*). Usually installed at mid-block points where vehicle traffic busily moves, the latter type studied herein can be timed to respond soon (or after a preset time length) once the pushbutton is pressed. PA has a standard set of vehicle signal displays composed of "Red", "Green", and "Yellow", while the pedestrian signal intervals encompass "WALK", "FDW", and "Steady DONT WALK (SDW)". All timing parameters are statically preset.

*PELICAN*

A substantial multitude of PELICAN installations remain in operations in Europe and Oceania (*28*). PELICAN has four intervals (i.e., “Red”, “Flashing Yellow”, “Green”, and “Yellow”) which are displayed in sequence for vehicle movements. Its “Flashing Yellow” permits drivers to proceed if all pedestrians vacate the crosswalk. Frequently, its pedestrian indications use the image of a walking "Green Man" and a stationary "Red Man". Pedestrians can cross only when the steady “Green Man” is illuminated. Afterwards, the flashing “Green Man” follows, which means any crossing must not be started despite the enough time for pedestrians on the crosswalk to leave safely. Then the “Red Man” starts and no pedestrians should be crossing at all.

*HAWK*

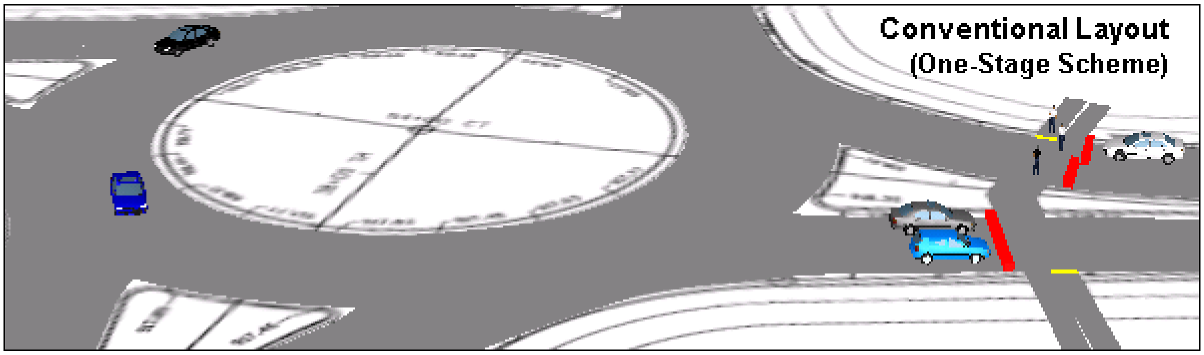
HAWK includes a “PEDESTRIANS” overhead sign and a sign instructing drivers to “STOP ON RED”. There is also a sign informing pedestrians on how to cross the street safely. Traditional signal displays operates in a different configuration. The vehicle signal remains dark for drivers unless activated by a pushbutton press. After activation, it launches “Flashing Yellow” and then “Steady Yellow” for a few seconds, alerting drivers to stop. Then, it displays a “Solid Red” which requires drivers to wait at the stop line. At this time, pedestrians receive a “WALK” indication. Afterwards, pedestrians are provided with a “FDW” indication and a countdown timer indicating the time left for crossing, and drivers see an “Alternating Flashing Red” display. During this period, drivers are required to stop or remain stopped until pedestrians have finished crossing the roadway, and then they may proceed cautiously when it is safe. Then, “SDW” follows for pedestrians and finally the vehicle signal reverts to dark. HAWK has been found to be associated with a statistically significant reduction in total crashes (*31*). It has appeared at an intersection without a standard traffic signal or in the middle of a long stretch of roadway in cities in Georgia, Minnesota, Virginia, Illinois, Arizona and Delaware and could increase in popularity due to a recent change in federal guidelines that allows HAWK to be installed without getting permission from the FHWA (*27*). Each HAWK system costs roughly $120,000 to install according to USA Today (*32*).

*PUFFIN*

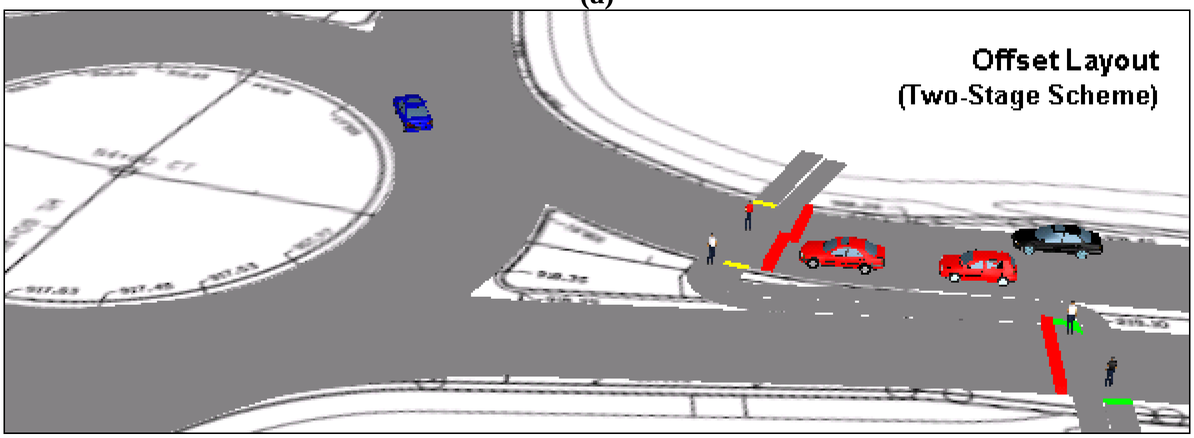
PUFFIN was viewed as an updated version of PELICAN. It has four vehicle signal intervals which iterate from “Red” to “Alternating Red & Yellow” then to “Green” and “Yellow”. The “Flashing Yellow” and “Flashing Green Man” intervals in PELICAN are omitted. This eradicates sources of pedestrians’ confusion and harassment sensed due to aggressive drivers (*29*). Three signals introduced above statically time the “FDW” using the crosswalk length and a design speed. From the safety perspective, this timing practice is unsustainable since vision-impaired people, seniors, and children induce considerable variability into the walking speed (*33,34,35,36*). Therefore, the static “FDW” duration cannot offer *all* pedestrians with *full* signal protection. Differently, PUFFIN uses on-crosswalk sensors to track pedestrians and the “SDW” (partially as the “FDW”) is dynamically adjusted to provide the pedestrian clearance time in real-time needs. So, PUFFIN produces a full sense of protection for pedestrians. The installation cost for PELICAN and PUFFIN systems ranges from $50,000 to $75,000, depending on the street width, the length of mast-arms, and the presence of center islands and landscaping (*37*).

## Geometric Layouts

A key issue is to design the geometric layouts for crosswalk segments on inbound and outbound lane(s). In the United States, the most seeable layout is to place the entire crosswalk across the splitter island, approximately one-vehicle length upstream from the entry yield line, which is termed “*Conventional* Layout” herein (FIGURE 2*a*). Due to the critical issue regarding the potential queue spillback into the circulatory lane(s), two other layouts were tested. One is to spatially shift the segment on outbound lane(s) farther away from the circular island, which is named “*Offset* Layout”: the segment is relocated by an offset of 80 feet from the entry yield line (FIGURE 2*b*). In operations, this can accommodate roughly four vehicles per lane before the rear end of a vehicle infringes upon the circulatory lane(s). The other one, "*Distant* Layout", pushes the whole crosswalk away from the circular island by a spacing of 120 feet, supplying the storage room for nearly six vehicles per lane (FIGURE 2*c*).




**(a)**

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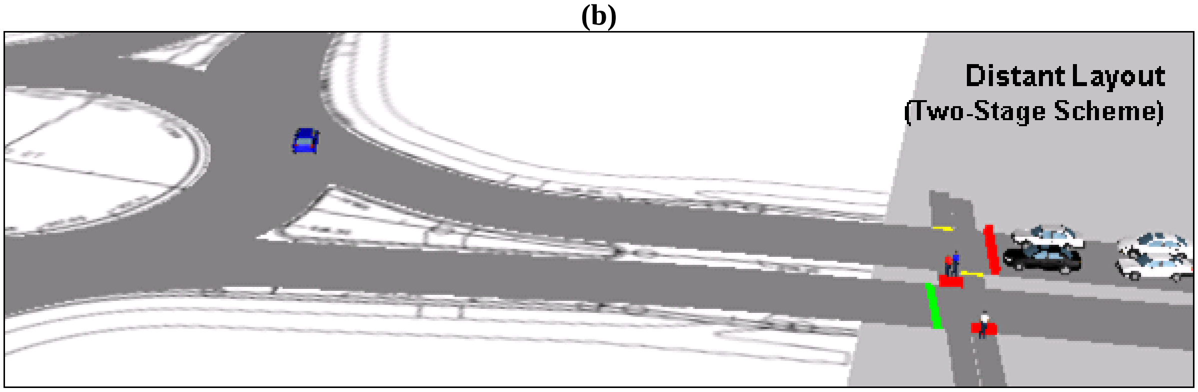



**(c)**

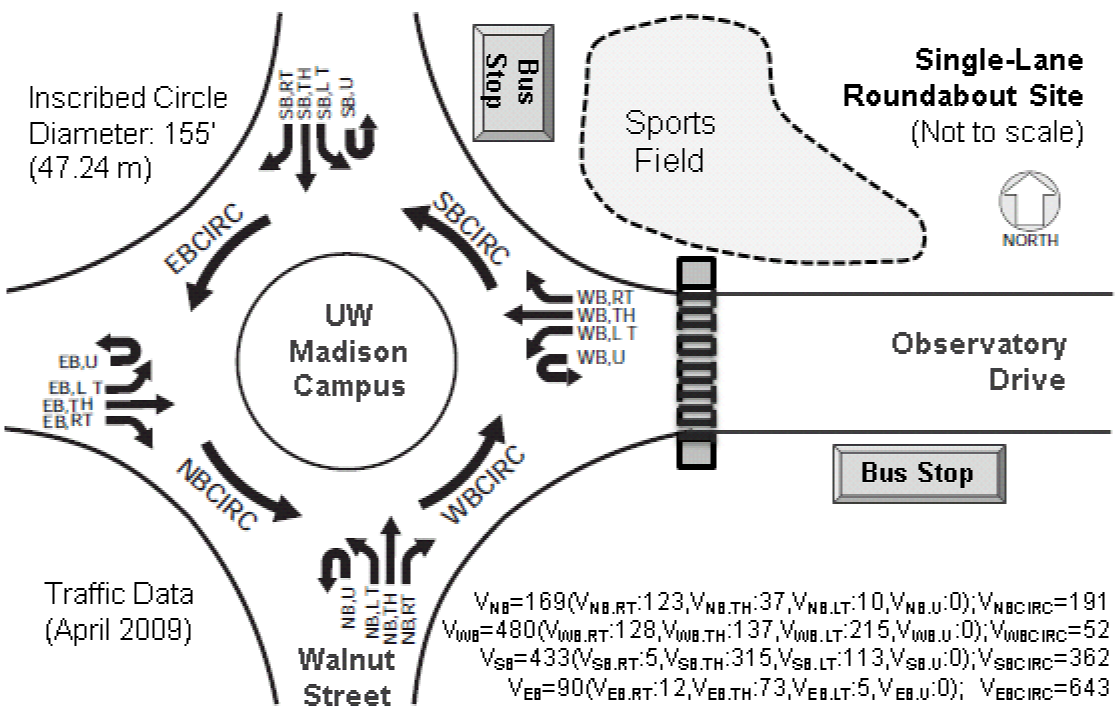
**FIGURE 2 Tested crosswalk layouts and installation schemes: *(a)* Conventional layout ("One-Stage" scheme), *(b)* Offset layout ("Two-Stage" scheme), and *(c)* Distant layout ("Two-Stage" scheme).**

### Installation Schemes

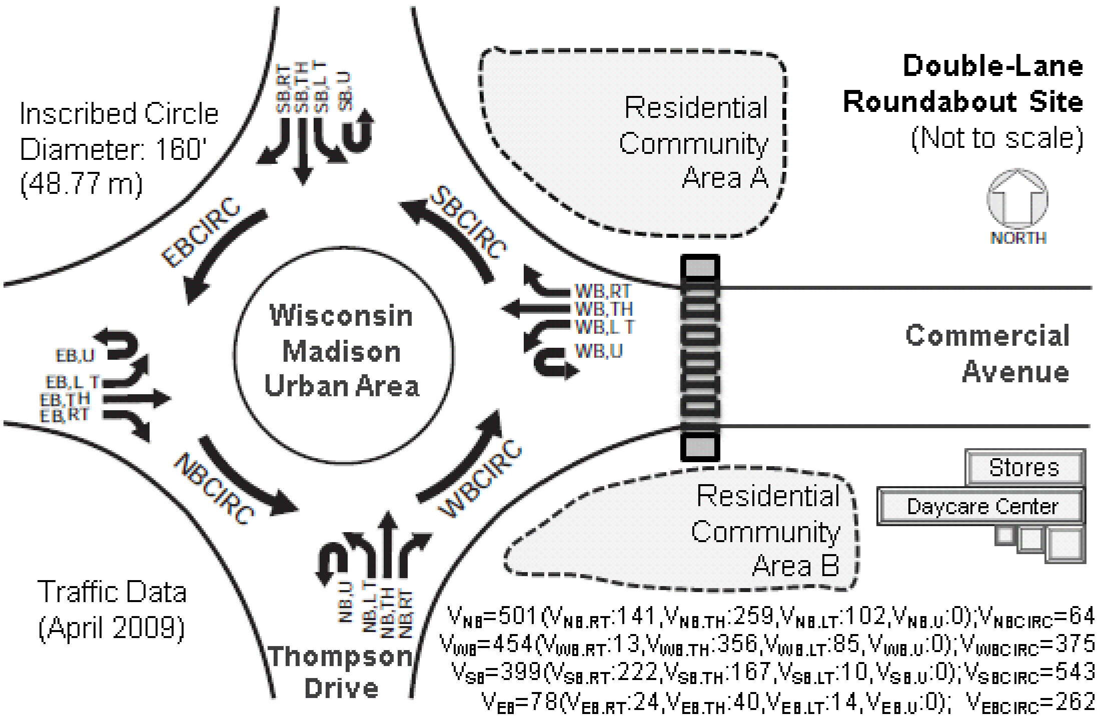
Since pedestrians cross both inbound and outbound lanes, whether to install separate signals for inbound and outbound lanes is important to roundabout signalization. With the "One-Stage" installation scheme, the same signal indication is valid for the whole crossing distance between roadway curbs and overrides both inbound and outbound lanes traversing a crosswalk, as shown in FIGURE 2*a*. With the "Two-Stage" scheme, inbound and outbound lanes are controlled separately and crossing pedestrians will wait midway on the splitter island or in the median area, as shown in FIGURE 2*b,c*. Obviously, it is unreasonable to use the “One-Stage” scheme in conjunction with the “Offset” layout due to the long walking distance which will definitely induce too much delay to all roundabout users.

**Field Sites**

Two modern roundabouts newly constructed in Madison Wisconsin were identified as the field sites to collect actual peak-hour traffic data used as the base volume. This study signalized the crosswalk at the approach on which the most intense vehicle volume and the highest prevailing speed exist, because such an approach produces the fewest safe crossable gaps for pedestrians. Field observations revealed there are serious accessibility issues at the single-lane roundabout (FIGURE 3*a*): during workdays, the heaviest commuting traffic and walking people densely flow on the westbound approach between two bus stops which yield a number of riders encompassing vision-impaired pedestrians and seniors; seasonal football events generate crowed traffic flows in which many pedestrians are present. At the double-lane roundabout, the westbound approach is located between two residential communities and in the proximity of some abutting properties (e.g., daycare center, stores, etc.). The most intense peak-hour flows move on northbound and westbound approaches; while vehicles on the latter have the highest prevailing speed which poses hazards to crossing pedestrians (FIGURE 3*b*). Future residential growth is expected to the east and will add more traffic to the westbound approach of this roundabout as that growth is realized.




**(a)**

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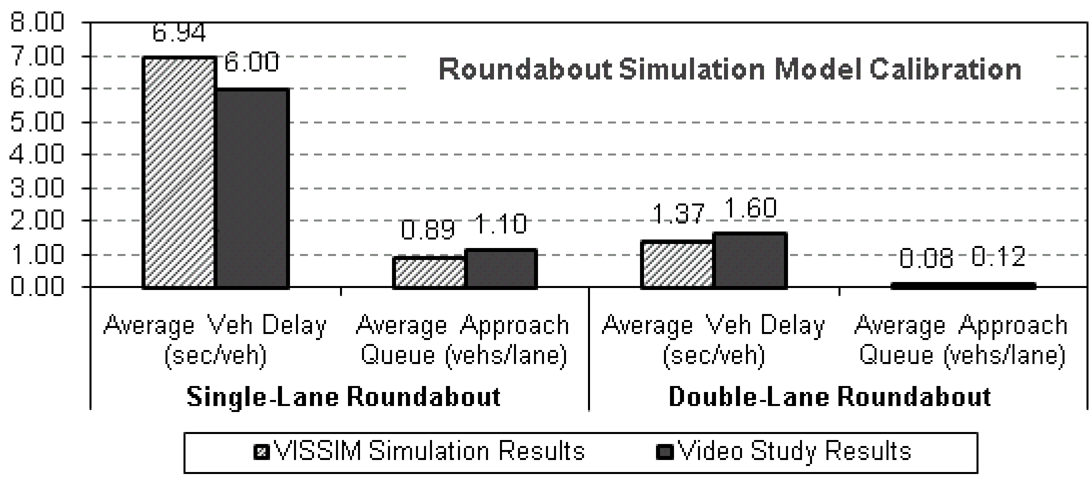
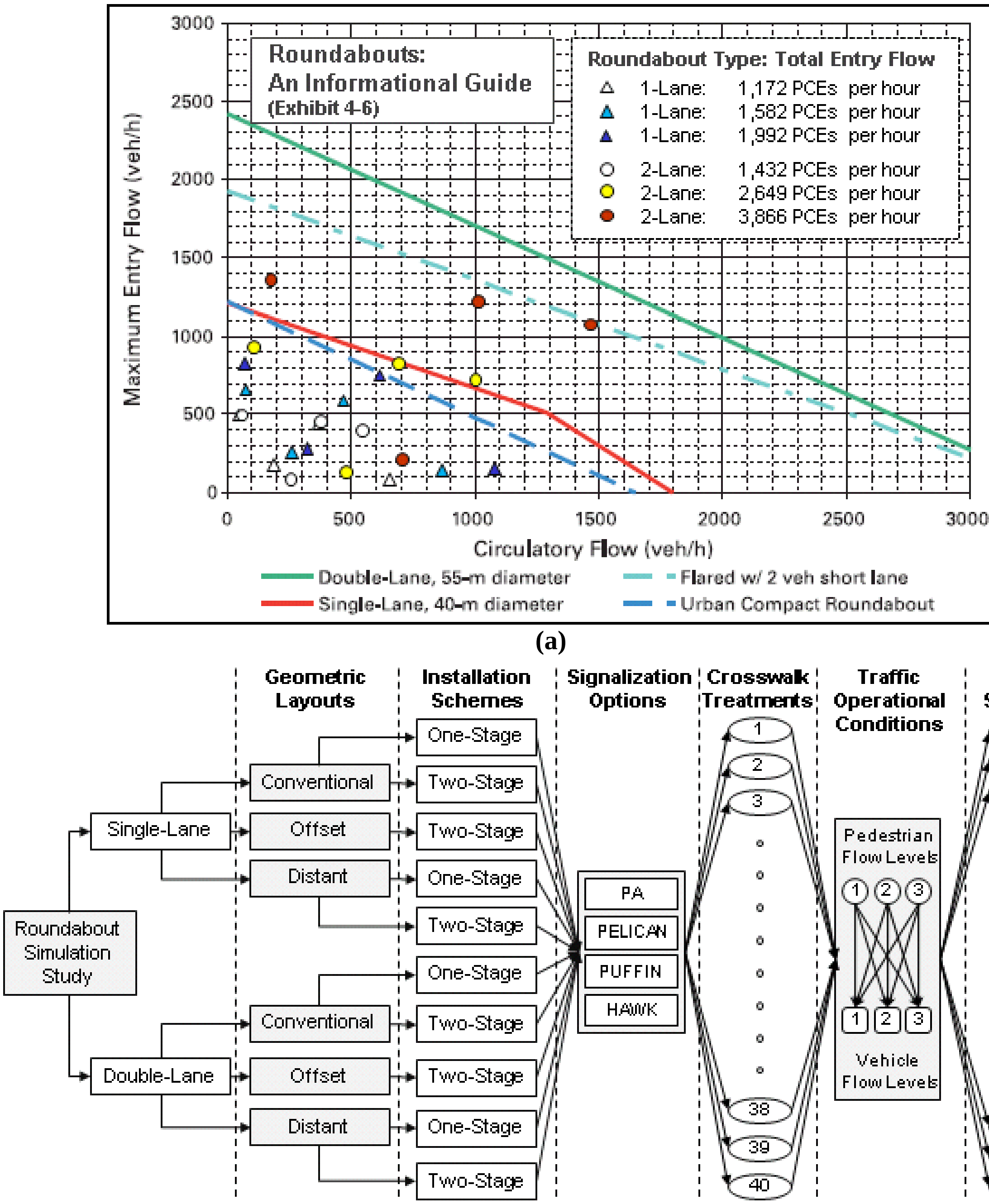



**(c)**

**FIGURE 3 Roundabout traffic volumes and simulation calibration: (*a*) actual peak-hour traffic volumes (V's) in PCEs (passenger car equivalents) calculated by the FHWA Informational Guide standard (*38*) for the single-lane site, (*b*) V's in PCEs by the FHWA Informational Guide standard for the double-lane site, and (*c*) VISSIM model calibration results.**

### Model Calibration

Simulation models were established with base volumes, turning percentages, and design speeds which are in compliance with the FHWA's Informational Guide (*38*). The vehicle-yielding behaviors were modeled consistently or closely with a documented example in which "The values used for minimal gap time, minimal headway and maximum speed have been determined through research. Thus for most applications these serve as a realistic base for most applications (*21*)". Vehicle speeds were calibrated using field data; proceeding speeds on approaches, entering speeds near the circulatory lane(s), and circulating speeds around islands were verified to have normal distributions. The model validation in the "zero-pedestrian" case was implemented via comparing average vehicle delays and average approach queues with those counterparts observed from real-world data: these video recordings of field sites were replayed repetitively to manually obtain the approximate measurements by means of intensive visual scrutiny, stop watch manipulations and data recording. The results displayed that vehicle delays and queues match field observations to an acceptable degree (FIGURE 3*c*) (*20*); while it was clearly realized by researchers that the observation sample size and the measurement method are limited so the validation could be refined with additional field data and the aid of sophisticated image-processing techniques.

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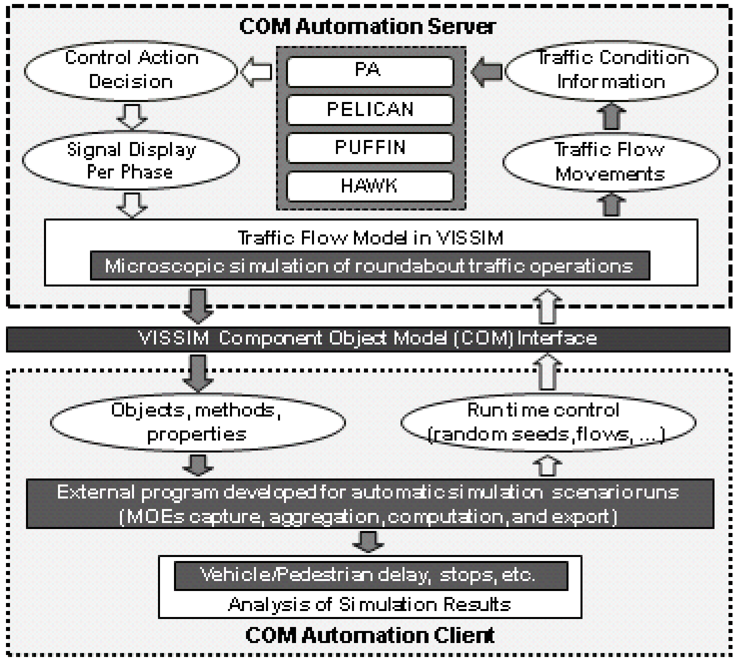



**(c)**

**FIGURE 4 Simulation experiment design and implementation: (*a*) entry volumes relative to theoretical capacity in FHWA Informational Guide (*38*), (*b*) simulation experiment design, and (*c*) run-time control and computation via VISSIM-based component object model (COM) automation.**

**Experiment Design**

The observed traffic volumes do not exceed the theoretical capacity for their sizes as cited in the FHWA's Informational Guide (*38*). To explore additional cases, base volumes were enlarged at a fixed growth rate to create additional scenarios which approach the maximum capacity. The FHWA's Informational Guide recommends that roundabouts should be designed to operate under 85% of the estimated capacity. Through a calculation in compliance with this Informational-Guide-based "85%" threshold, the single-lane roundabout base volumes were augmented by 35% and 70% to produce 1,582 and 1,992 PCEs per hour, while the double-lane roundabout base volumes were increased by 85% and 170% to obtain 2,649 and 3,866 PCEs per hour. Therefore, three vehicle intensity levels were established: "Existing Flow", "Approaching Capacity", and "Saturated Condition". FIGURE 4*a* depicts both base and enhanced volumes of two roundabouts superimposed upon the Guide's capacity figure.

Three pedestrian flow intensities were investigated: 12 ("Few"), 60 ("Some"), and 150 ("Many") pedestrians per hour (pph). These designed pedestrian flows do not suffice for the MUTCD Section 4C.05 Warrant 4 (*27*), because the main motivation of adding these signals is not to satisfy a MUTCD design warrant but to improve the roundabout accessibility for pedestrians. Roughly 15% of walking populations walk more slowly at a speed less than 3.5 feet per second (fps) (*39*). Therefore, the mean speed was set to 3.0 fps and a researcher-customized speed distribution, with minimum and maximum speeds equal to 1.0 and 8.0 fps respectively, was modeled to embody major findings in previous studies.

To consider all possible cases, each geometric layout was combined with "One-Stage" and "Two-Stage" installation schemes, except for the "Offset" layout which is only reasonable to be combined with the "Two-Stage" scheme. FIGURE 4*b* exhibits geometric layouts and installation schemes were combined with signalization options to generate 40 different pedestrian crosswalk treatments each of which was modeled with varied traffic conditions to create 360 different study scenarios.

**Basic Timing Settings**
PUFFIN's dynamic "SDW" provides *full* signal protection for *all* pedestrians some of whom walk at the minimum walking speed (1.0 fps). For other signals, higher walking speeds for timing "FDW" would leave slow pedestrians unprotected by signals when the "FDW" terminates and the vehicle signal turns green. Therefore, to protect all pedestrians under study and maintain the strict reasonableness in comparing four signals so that all study scenarios can have a uniform degree of signal protection for subject pedestrians, the static "FDW" for three other signals was timed with the crossing distance and minimum walking speed (1.0 fps) to guarantee adequate clearance time for *all* pedestrians modeled. "WALK" was uniformly 6.0 s based on relevant MUTCD recommendations. Minimum vehicle greens and "All-Red" intervals were universally set to 36 s and 1 s respectively. "Yellow" was set to 2.5 s for HAWK and 4.0 s for other signals. "Flashing Yellow" and "Alternating Red/Yellow" were set to 1.5 s for HAWK and 1.0 s for PUFFIN.

**Performance Measures**
One target of this study was to quantitatively identify the impact of pedestrian crosswalk treatments upon roundabout operations. Generic vehicle-based performance measures (e.g., average vehicle delay, average queue length, and average number of stops) were obtained by means of the "*pedestrian-induced*" effect which is defined as the difference between the measures generated at certain pedestrian volumes and its counterparts in the "Zero-Pedestrian" case (*19*). Here average number of stops was treated as the safety index: its increase implies more deceleration occurrences, which aggravates the potential for rear-end crashes and, from human factors perspective, makes drivers increasingly prone to commit incompliance with signals. Average pedestrian delay is defined as the difference between the actual travel time consumed in crossing a roundabout and minimum travel time (at a given walking speed without delays) across the pathway of interest.

**Simulation Data**
With different random seeds, 15 replications were simulated for each scenario to dampen stochastic variations resultant from underlying simulation models, which amounted to 5,400 runs. Each run lasted 3,600 simulation seconds. The first replication populated the model, and the last ran as the clear-out period. Then, data from thirteen replications in-between were collected. The data for performance measures were procured within an evaluation node surrounding roundabouts. Simulation runs for each treatment were implemented automatically. As a client in seamless dialogue with the VISSIM-based server, an external program extracted, aggregated, calculated, and finally output data to Excel spreadsheets during run time (FIGURE 4*c*).

## STUDY RESULTS

Study results for single-/double-lane roundabouts are reported by means of thirteen replications. FIGURE 5 and FIGURE 6 exhibit operational effects of pedestrian volume levels and vehicle flow intensities in conjunction with signalization options and geometric layouts. In each figure, subfigures *a* and *c* demonstrate "One-Stage" results for 8 treatments and 72 scenarios; while subfigures *b* and *d* exhibit "Two-Stage" results for 12 treatments and 108 scenarios. Each subfigure is plotted at a different scale.

**Pedestrian-Induced Vehicle Delay**
FIGURE 5 shows the pedestrian-induced vehicle delays at single-/double-lane roundabouts.

*Single-Lane Roundabout*
FIGURE 5*a-b* shows, when the vehicle volume is fixed at a specific level, vehicle delays are ubiquitously enhanced when crossing pedestrians increase incrementally from "Few" to "Some" and "Many". This

operational feature can be explained by the fact that more crossing demands will pose increased interruptions to vehicular circulations at roundabouts.

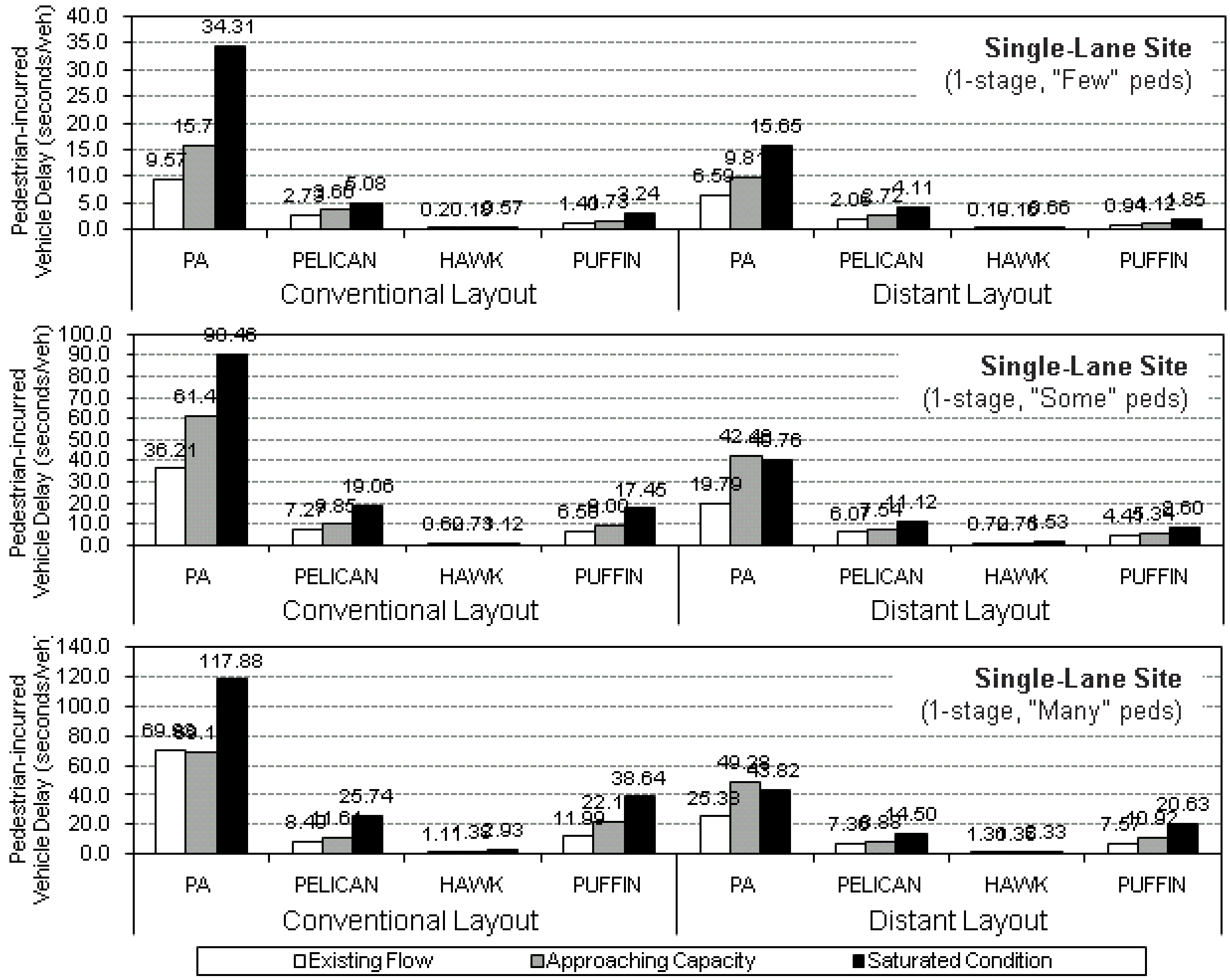
FIGURE 5*a* demonstrates, when the pedestrian flow level is specifically maintained, a roughly monotonic relationship exists between the vehicle volume and the vehicle delay for PELICAN, HAWK, and PUFFIN. For these three signals, the "Saturated Condition" yields the maximum vehicle delay. HAWK has the lowest vehicle delay compared with other signals under each operational condition. PA generates the highest vehicle delay in all study scenarios while PELICAN and PUFFIN have much lower vehicle delays relatively close to each other. Comparatively, the "Distant" layout exhibits potential advantages over the "Conventional" layout since vehicle delays from three signals (i.e., PA, PELICAN, and PUFFIN) are universally reduced when the "Conventional" layout changes to the "Distant" layout. FIGURE 5*b* shows, given "Some" or "Many" pedestrians, the vehicle delay has an approximately monotonic relationship with the vehicle volume for 12 treatments and the "Saturated Condition" produces the largest vehicle delay for each treatment. Under each operational condition, HAWK gives the lowest vehicle delay in comparison with other signals, regardless of crosswalk layouts. For most scenarios, there are no substantial differences in vehicle delays which are respectively produced by each signal at three layouts.

It is also observed in FIGURE 5*a* that all "One-Stage" vehicle delays are significantly larger than their "Two-Stage" counterparts in FIGURE 5*b*. This indicates that the "Two-Stage" installation scheme outperforms the "One-Stage" counterpart in operational efficiency, because the "Two-Stage" schemes have shorter "FDW" intervals which make vehicles wait for shorter time to traverse a roundabout.

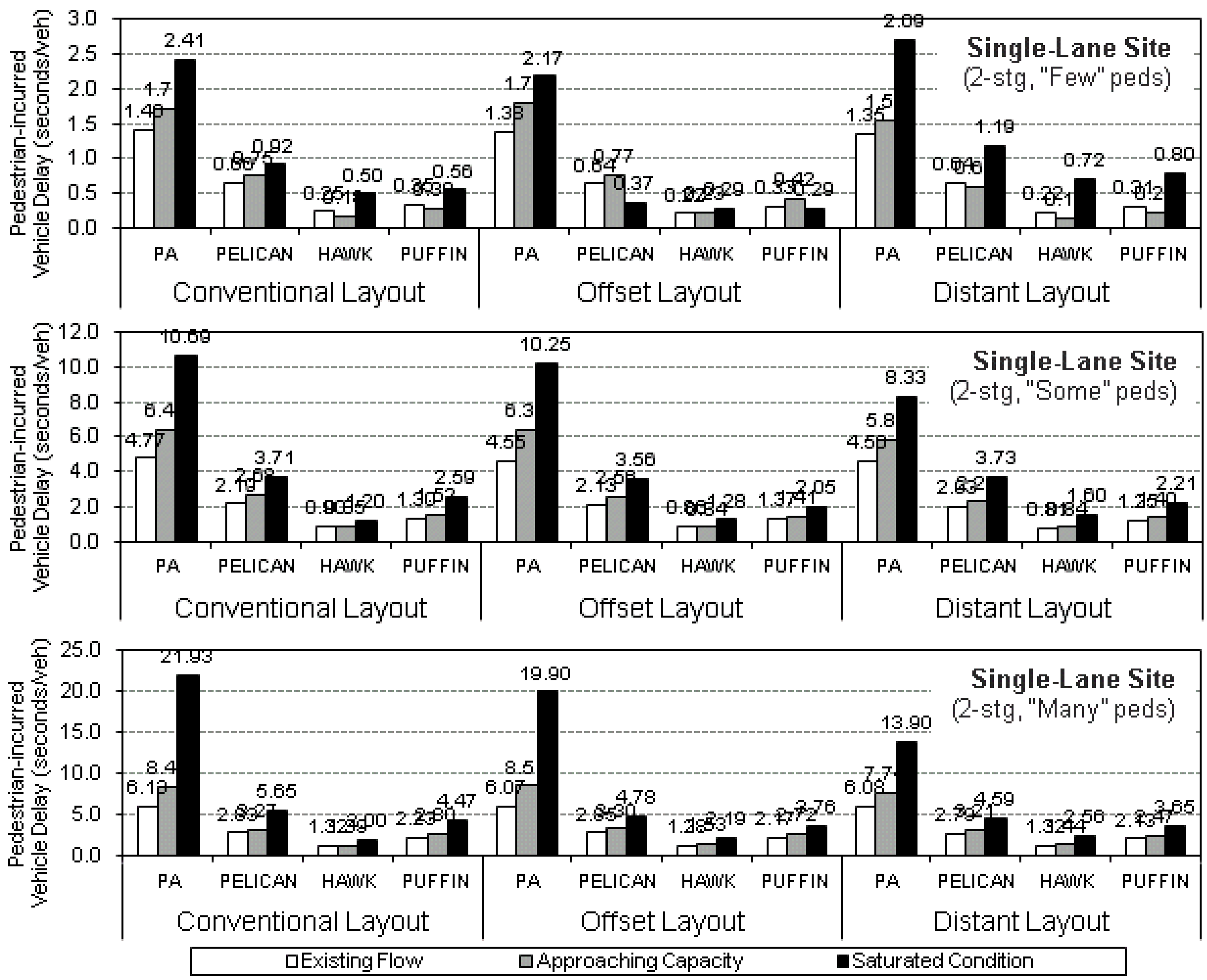
*Double-Lane Roundabout*

FIGURE 5*c-d* shows, under "Existing Flow" and "Approaching Capacity" conditions, vehicle delays are universally increased when pedestrians increase from "Few" and "Some" to "Many". This operational characteristic can be attributed to more frequent interruptions incurred by denser pedestrians to vehicle circulations. When the pedestrian intensity level is specifically fixed, vehicle delays become higher when the vehicle volume changes from "Existing Flow" to "Approaching Capacity" conditions. Interestingly, under "Saturated Condition", some treatments yield negative pedestrian-induced vehicle delays. In these scenarios, the presence of pedestrian signals make vehicle delays diminish. This phenomenon could be due to the observation that the pedestrian signal metering traffic on the busiest approach facilitates the entering vehicle streams at downstream roundabout approaches.

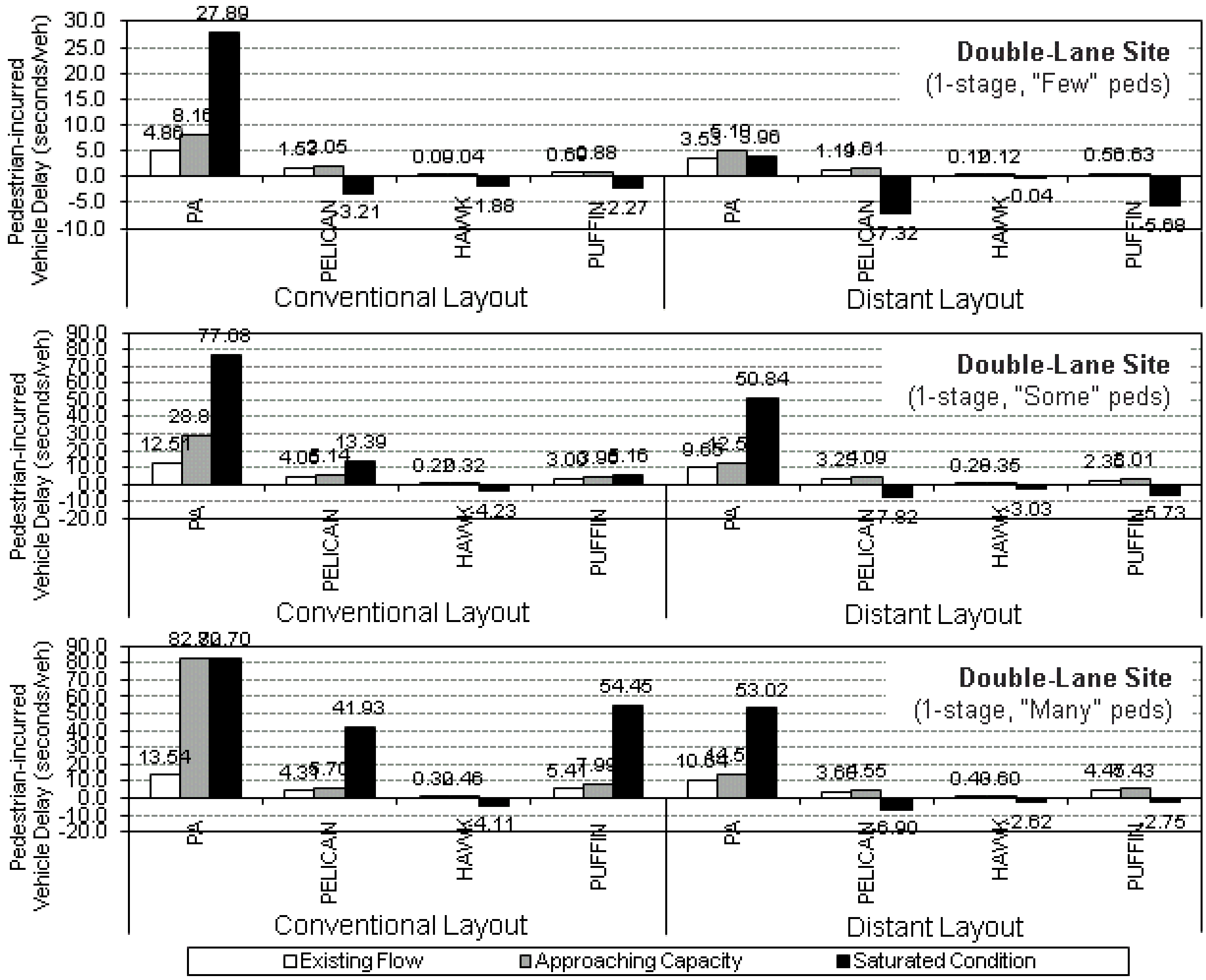
FIGURE 5*c-d* also reveals, under "Existing Flow" and "Approaching Capacity" conditions, HAWK generates the lowest vehicle delay compared with other signals, no matter which crosswalk layout is applied. PA generates the highest vehicle delay while PELICAN and PUFFIN have lower vehicle delays relatively close to each other. FIGURE 5*c* reveals that PA, PELICAN, and PUFFIN produce more vehicle delays at "Conventional" layout than those at "Distant" layout. FIGURE 5*d* shows that no substantial differences are found among vehicle delays produced at three layouts. For PA, PELICAN, and PUFFIN, "One-Stage" vehicle delays (FIGURE 5*c*) are significantly larger than "Two-Stage" counterparts (FIGURE 5*d*). For HAWK, the discrepancies between vehicle delays of both installation schemes are rather limited.


Single-Lane Site
(1-stage, "Few" peds)
(1-stage, "Some" peds)
(1-stage, "Many" peds)
Pedestrian-incurred Vehicle Delay (seconds/veh)
PA
PELICAN
HAWK
PUFFIN
Conventional Layout
Distant Layout
Existing Flow
Approaching Capacity
Saturated Condition

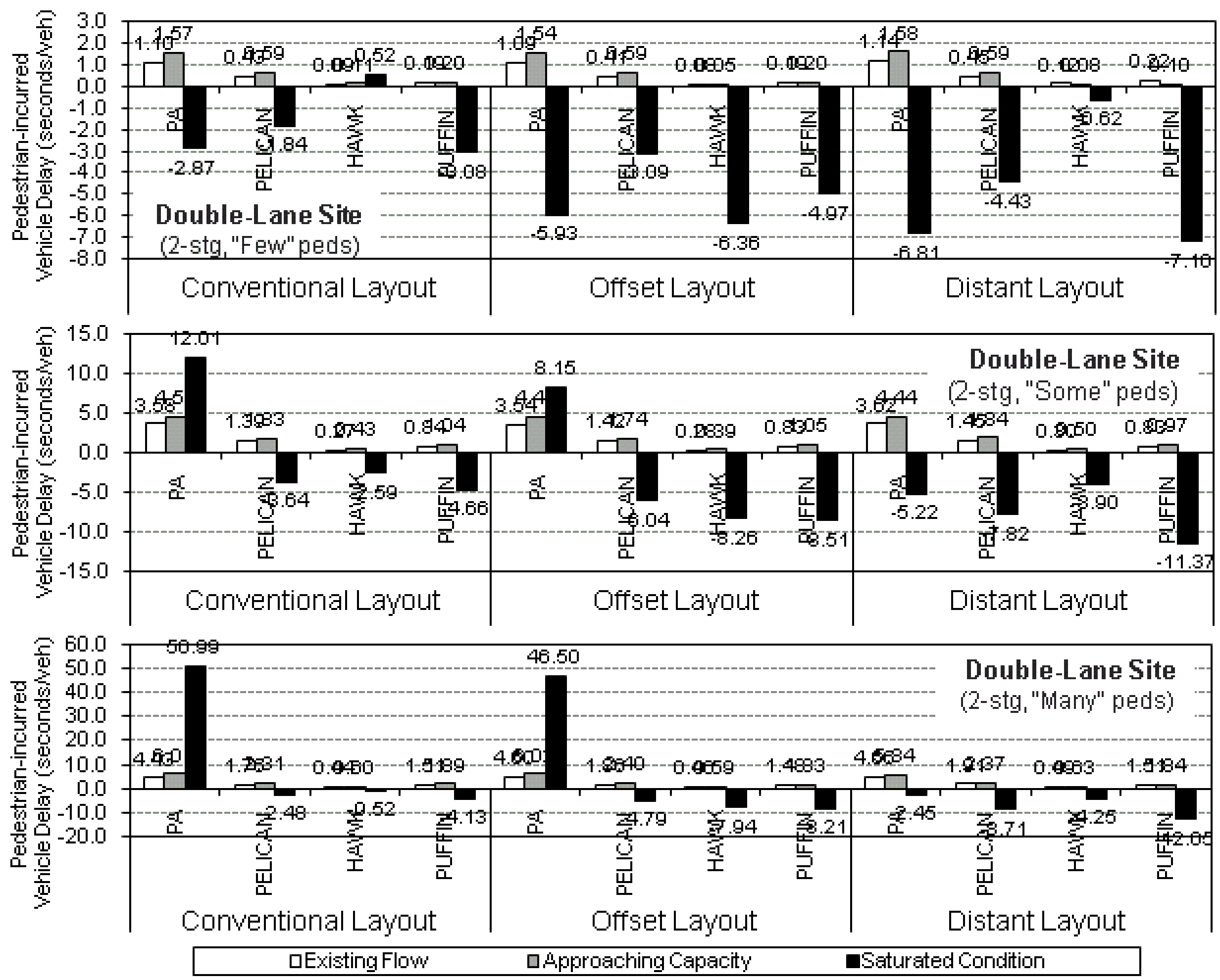
**(a)**


Single-Lane Site
(2-stg, "Few" peds)
Single-Lane Site
(2-stg, "Some" peds)
Single-Lane Site
(2-stg, "Many" peds)
Pedestrian-incurred Vehicle Delay (seconds/veh)
PA
PELICAN
HAWK
PUFFIN
Conventional Layout
Offset Layout
Distant Layout
Existing Flow
Approaching Capacity
Saturated Condition

**(b)**


Double-Lane Site
(1-stage, "Few" peds)
Double-Lane Site
(1-stage, "Some" peds)
Double-Lane Site
(1-stage, "Many" peds)
Pedestrian-incurred Vehicle Delay (seconds/veh)
PA
PELICAN
HAWK
PUFFIN
Conventional Layout
Distant Layout
Existing Flow
Approaching Capacity
Saturated Condition

**(c)**




**(d)**

**FIGURE 5 Pedestrian-induced vehicle delay: (*a*) single-lane roundabout and 1-stage scheme (12 pph, 60 pph, and 150 pph), (*b*) single-lane roundabout and 2-stage scheme (12 pph, 60 pph, and 150 pph), (*c*) double-lane roundabout and 1-stage scheme (12 pph, 60 pph, and 150 pph), and (*d*) double-lane roundabout and 2-stage scheme (12 pph, 60 pph, and 150 pph).**

**Pedestrian-Induced Queue Length**

FIGURE 6 exhibits the pedestrian-induced queue lengths generated at single-/double-lane roundabouts. The results are much similar to those of pedestrian-induced vehicle delays.

*Single-Lane Roundabout*

FIGURE 6*a-b* shows, at a specific vehicle intensity level, queue lengths are universally prolonged for all treatments when increasing crossing demands pose more disruptions to vehicle movements at roundabouts.

FIGURE 6*a* unveils, when the pedestrian flow intensity level is specifically maintained, there is a roughly monotonic relationship between vehicle volume and queue length for PELICAN, HAWK, and PUFFIN. For these three signals, the "Saturated Condition" yields the maximum queue length for each treatment. Compared with other three signals, the queue length from HAWK is the shortest in most cases no matter which crosswalk layout is employed. PA generates the longest queue length in almost all cases while PELICAN and PUFFIN generate much shorter queue lengths. Relatively, the "Distant" layout is better than the "Conventional" counterpart since queue lengths from three signals (i.e., PA, PELICAN, and PUFFIN) are shortened when the "Conventional" layout changes to the "Distant" one.

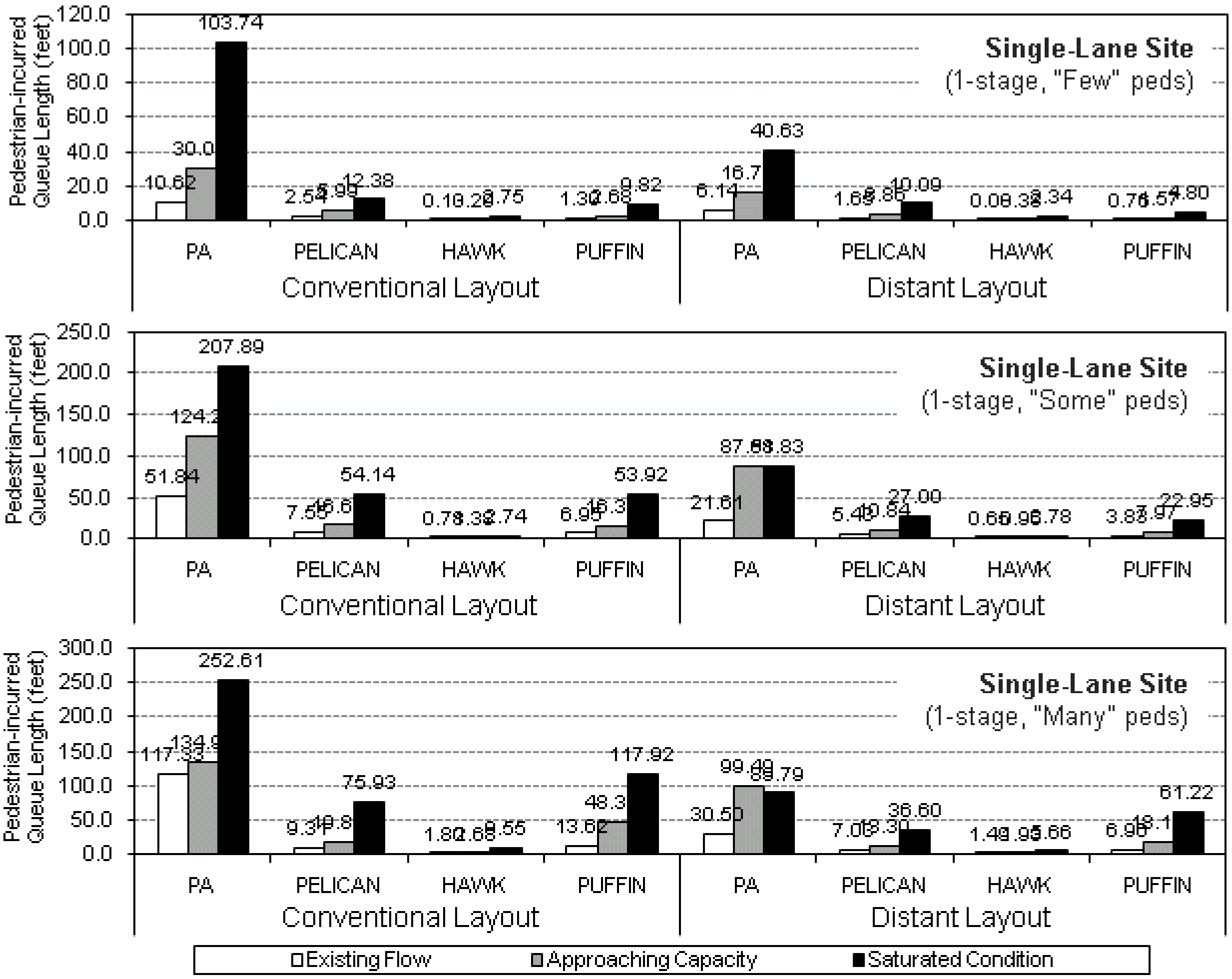
FIGURE 6*b* shows the queue length has a monotonic relationship with the vehicle volume and the "Saturated Condition" produces the longest queue for each treatment. Generally, HAWK produces the shortest queue length while PA has the longest. In most cases, the differences among three layouts do not pose significant distinctions in queue length produced by each signal. FIGURE 6*a* shows all "One-Stage" queue lengths are significantly longer than their "Two-Stage" counterparts shown in FIGURE 6*b*. This indicates that for vehicles the "Two-Stage" scheme is more operational efficient than the "One-Stage" alternative.

*Double-Lane Roundabout*

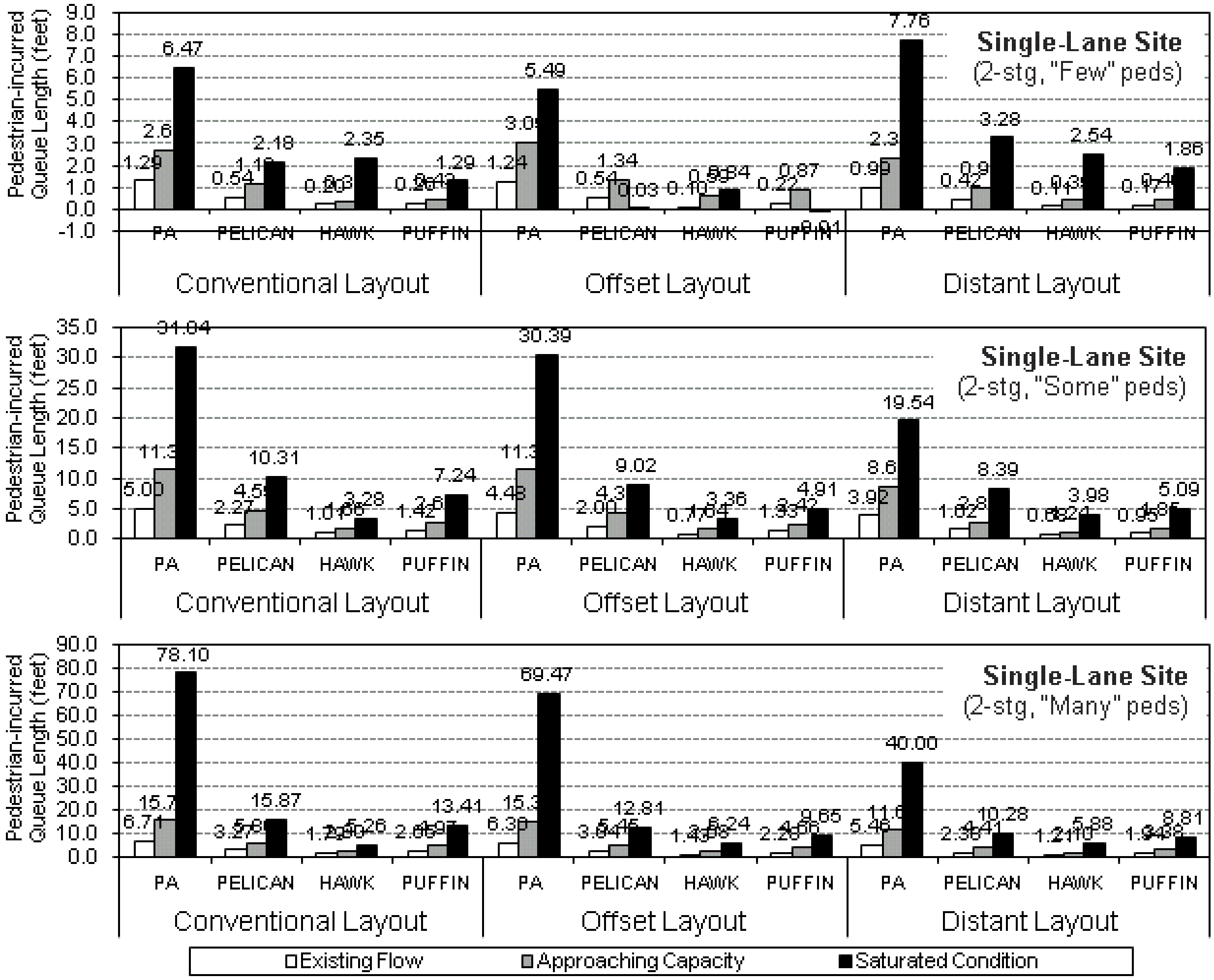
FIGURE 6*c-d* exhibits, under "Existing Flow" and "Approaching Capacity" conditions, queue lengths increase for all treatments when pedestrian flows are intensified from "Few" until "Many". HAWK generates the shortest queue length in comparison with PA, PELICAN and PUFFIN.

FIGURE 6*c-d* shows, if the pedestrian flow intensity is fixed at a specific level, the queue length increases when the vehicle volume changes from "Existing Flow" to "Approaching Capacity" condition. For PA, PELICAN, and PUFFIN, their "One-Stage" queue lengths (FIGURE 6*c*) are significantly longer than their "Two-Stage" counterparts (FIGURE 6*d*). The discrepancies between "One-Stage" and "Two-Stage" queue lengths given by HAWK are rather small. Interestingly, many treatments yield negative pedestrian-induced queue lengths under "Saturated Condition". In these cases, the introduction of pedestrian signal make queue lengths shrink, which could be ascribed to the metering effect of the pedestrian signal on the busiest approach. It also shows, under "Existing Flow" and "Approaching Capacity" conditions, HAWK has the shortest queue length compared with other signals. PA generates the largest queue length while PELICAN and PUFFIN have shorter queue lengths relatively close to each other.

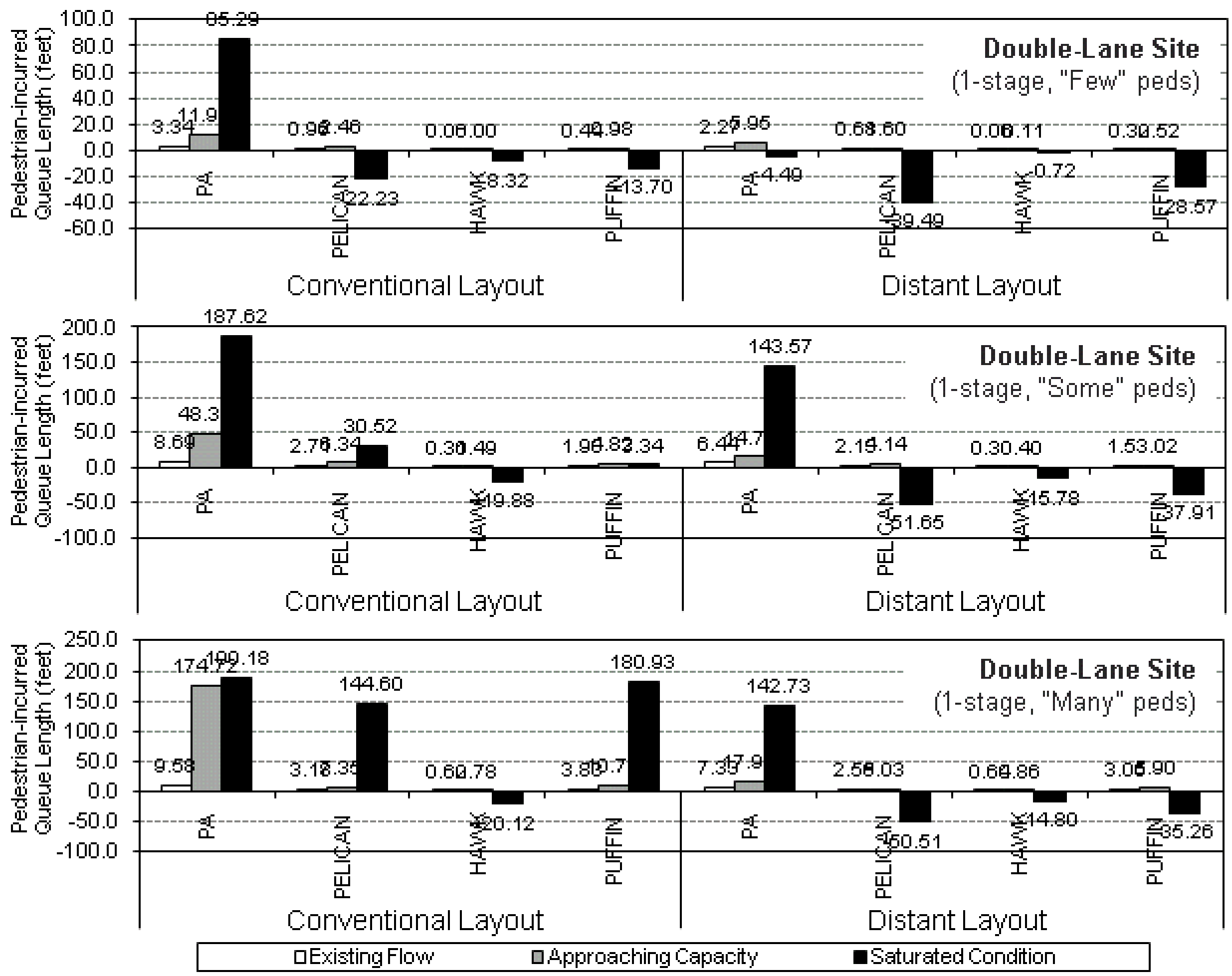
FIGURE 6*c* reveals PA, PELICAN, and PUFFIN produce longer queues at the "Conventional" layout than those at the "Distant" layout. FIGURE 6*d* shows there are no substantial differences among queue lengths generated at three layouts.


Pedestrian-incurred Queue Length (feet)
Single-Lane Site
(1-stage, "Few" peds)
(1-stage, "Some" peds)
(1-stage, "Many" peds)
PA
PELICAN
HAWK
PUFFIN
Conventional Layout
Distant Layout
Existing Flow
Approaching Capacity
Saturated Condition

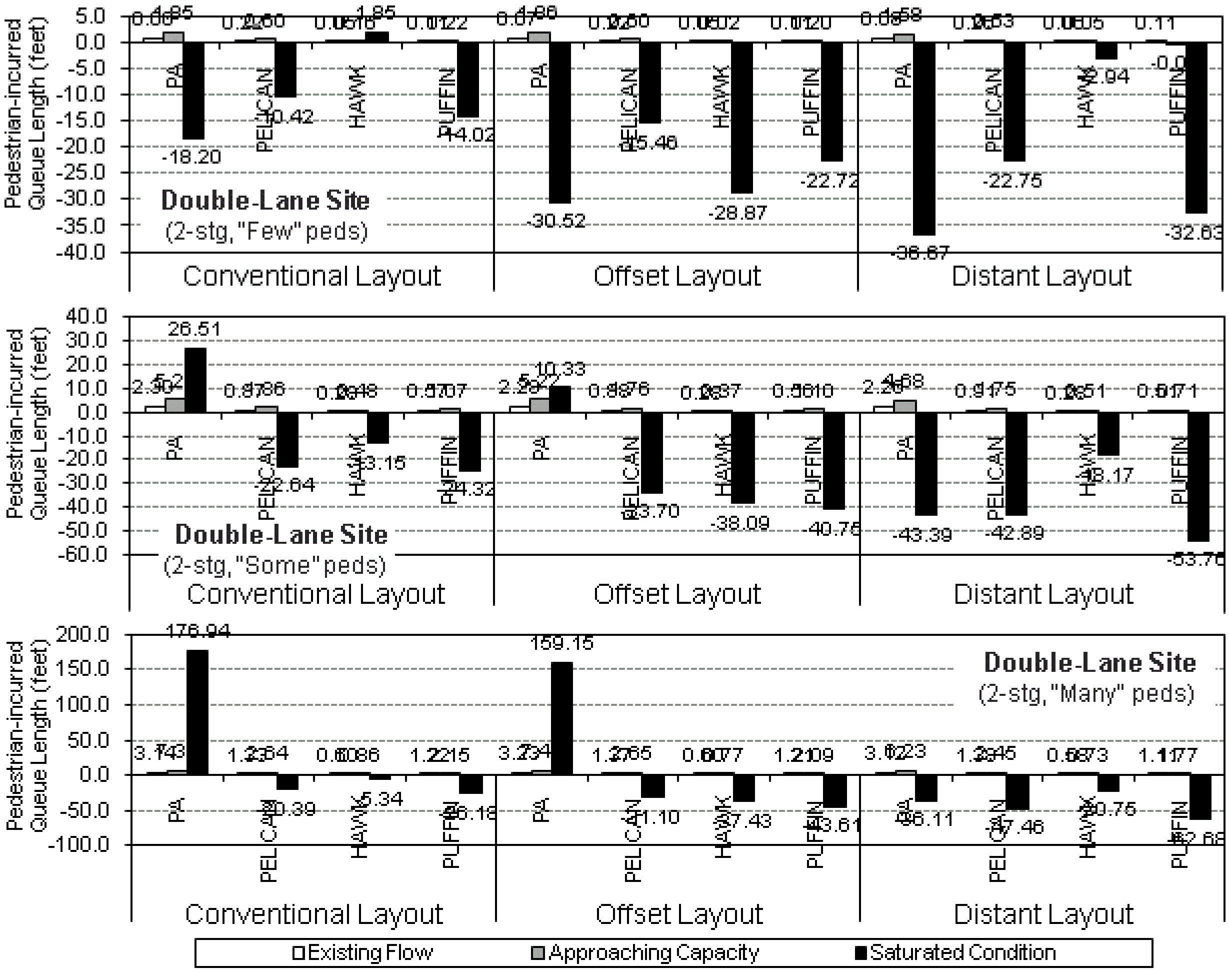
**(a)**


Single-Lane Site
(2-stg, "Few" peds)
(2-stg, "Some" peds)
(2-stg, "Many" peds)
Pedestrian-incurred Queue Length (feet)
PA
PELICAN
HAWK
PUFFIN
Conventional Layout
Offset Layout
Distant Layout
Existing Flow
Approaching Capacity
Saturated Condition

**(b)**


Double-Lane Site
(1-stage, "Few" peds)
(1-stage, "Some" peds)
(1-stage, "Many" peds)
Pedestrian-incurred Queue Length (feet)
PA
PELICAN
HAWK
PUFFIN
Conventional Layout
Distant Layout
Existing Flow
Approaching Capacity
Saturated Condition

**(c)**




**(d)**

**FIGURE 6 Pedestrian-induced queue length: (*a*) single-lane roundabout and 1-stage scheme (12 pph, 60 pph, and 150 pph), (*b*) single-lane roundabout and 2-stage scheme (12 pph, 60 pph, and 150 pph), (*c*) double-lane roundabout and 1-stage scheme (12 pph, 60 pph, and 150 pph), and (*d*) double-lane roundabout and 2-stage scheme (12 pph, 60 pph, and 150 pph).**

**Number of Stops**

The results for single-/double-lane roundabouts disclose very similar operational features to pedestrian-induced vehicle delays in major aspects. It could be inferred that the "Distant" layout and the "Two-Stage" scheme can be believed safer across most study scenarios, and the introduction of pedestrian signals renders motorized vehicles move more smoothly somewhere under "Saturated condition", which diminish the likelihood of vehicle-to-vehicle crashes. Additionally, HAWK and PUFFIN are believed safer than other signals for most treatments, while the latter is more advantageous because of its protective on-crosswalk pedestrian sensor.

**Pedestrian Delay**

Since four signals operate with a fixed length of minimum vehicle green, it is expected that pedestrian delays will be independent of traffic flow fluctuations. TABLE 1 indicates that, at both single-lane and double-lane roundabouts, when the pedestrian flow level is specifically maintained the pedestrian delay at a treatment keeps constant while the vehicle volume changes from "Existing Flow" until "Saturated Condition". At a specific vehicle flow level, when there are more crossing pedestrians the pedestrian delays consistently exceed those from fewer pedestrians. With more crossing pedestrians, it is more likely for a larger portion of pedestrian flows to arrive during the minimum green time and then wait for signal display. Therefore, more pedestrians are delayed by “minimum green” constraints.

PA, PELICAN, and HAWK generate equal pedestrian delays for a specific combination of geometric layout and installation scheme, which can be explained by the identical length of “FDW” timed for them. Additionally, the paired t-tests reveal that each of three signals has significantly higher pedestrian delay than that from PUFFIN, which means it should confidently believed that the dynamic pedestrian clearance time provided by PUFFIN significantly saves pedestrian waiting time.

It was originally expected that different geometric layouts produce variable pedestrian delays due to distinct pathway deflections. When the “Two-Stage” scheme is applied to single-lane and double-lane roundabouts, pedestrian delays from a specific signal fluctuate limitedly among three layouts given each of three pedestrian flow levels. When the “One-Stage” scheme is applied to single-lane and double-lane roundabouts and there are “Some” or “Many” pedestrians, pedestrian delays generated by a specific signal at “Conventional” layout are higher than those at “Distant” layout.

**TABLE 1 Pedestrian Delay (seconds) – Average of 13 Simulation Replications**

| **(a). "Existing Flow"** | | | | | |
|---|---|---|---|---|---|
| Crosswalk Treatment | (a-1). Single-Lane Roundabout Site (1,172 PCEs per hour) | | | | |
| | “Conventional” Layout | | “Offset” Layout | “Distant” Layout | |
| | IS-1 | IS-2 | IS-2 | IS-1 | IS-2 |
| | *(a-1-1). "Few" pedestrians (12 pph)* | | | | |
| PA | 19.12 | 20.03 | 19.68 | 20.87 | 18.90 |
| HAWK | 19.12 | 20.03 | 19.68 | 20.87 | 18.90 |
| PELICAN | 19.12 | 20.03 | 19.68 | 20.87 | 18.90 |
| PUFFIN | 9.95* | 15.88* | 15.36* | 9.85* | 15.08* |
| | *(a-1-2). "Some" pedestrians (60 pph)* | | | | |
| PA | 43.84 | 40.76 | 40.47 | 39.69 | 40.38 |
| HAWK | 43.84 | 40.76 | 40.47 | 39.69 | 40.38 |
| PELICAN | 43.84 | 40.76 | 40.47 | 39.69 | 40.38 |
| PUFFIN | 21.95* | 29.79* | 28.76* | 20.26* | 28.79* |
| | *(a-1-3). "Many" pedestrians (150 pph)* | | | | |
| PA | 49.04 | 50.33 | 51.09 | 45.08 | 52.27 |
| HAWK | 49.04 | 50.33 | 51.09 | 45.08 | 52.27 |
| PELICAN | 49.04 | 50.33 | 51.09 | 45.08 | 52.27 |
| PUFFIN | 33.60* | 40.84* | 40.47* | 28.97* | 39.69* |
| Crosswalk Treatment | (a-2). Double-Lane Roundabout Site (1,432 PCEs per hour) | | | | |
| | “Conventional” Layout | | “Offset” Layout | “Distant” Layout | |
| | IS-1 | IS-2 | IS-2 | IS-1 | IS2 |
| | *(a-2-1). "Few" pedestrians (12 pph)* | | | | |
| PA | 22.93 | 22.51 | 22.49 | 21.70 | 21.48 |
| HAWK | 22.93 | 22.51 | 22.49 | 21.70 | 21.48 |
| PELICAN | 22.93 | 22.51 | 22.49 | 21.70 | 21.48 |
| PUFFIN | 11.25* | 15.90* | 15.09* | 10.58* | 15.32* |
| | *(a-2-2). "Some" pedestrians (60 pph)* | | | | |
| PA | 45.40 | 49.30 | 47.12 | 41.09 | 48.19 |
| HAWK | 45.40 | 49.30 | 47.12 | 41.09 | 48.19 |
| PELICAN | 45.40 | 49.30 | 47.12 | 41.09 | 48.19 |
| PUFFIN | 23.29* | 31.24* | 29.98* | 20.71* | 29.59* |
| | *(a-2-3). "Many" pedestrians (150 pph)* | | | | |
| PA | 51.10 | 61.18 | 61.10 | 45.82 | 61.41 |
| HAWK | 51.10 | 61.18 | 61.10 | 45.82 | 61.41 |
| PELICAN | 51.10 | 61.18 | 61.10 | 45.82 | 61.41 |
| PUFFIN | 33.80* | 44.34* | 43.50* | 30.56* | 43.40* |

**TABLE 1 Pedestrian Delay (seconds) – Average of 13 Simulation Replications (*continued*)**

| **(b). "Approaching Capacity"** | | | | | |
|---|---|---|---|---|---|
| Crosswalk Treatment | (b-1). Single-Lane Roundabout Site (1,582 PCEs per hour) | | | | |
| | "Conventional" Layout | | "Offset" Layout | "Distant" Layout | |
| | IS-1 | IS-2 | IS-2 | IS-1 | IS-2 |
| | *(b-1-1). "Few" pedestrians (12 pph)* | | | | |
| PA | 19.12 | 20.03 | 19.68 | 20.87 | 18.90 |
| HAWK | 19.12 | 20.03 | 19.68 | 20.87 | 18.90 |
| PELICAN | 19.12 | 20.03 | 19.68 | 20.87 | 18.90 |
| PUFFIN | 9.95* | 15.88* | 15.36* | 9.85* | 15.08* |
| | *(b-1-2). "Some" pedestrians (60 pph)* | | | | |
| PA | 43.84 | 40.76 | 40.47 | 39.69 | 40.38 |
| HAWK | 43.84 | 40.76 | 40.47 | 39.69 | 40.38 |
| PELICAN | 43.84 | 40.76 | 40.47 | 39.69 | 40.38 |
| PUFFIN | 21.95* | 29.79* | 28.76* | 20.26* | 28.79* |
| | *(b-1-3). "Many" pedestrians (150 pph)* | | | | |
| PA | 49.04 | 50.33 | 51.09 | 45.08 | 52.27 |
| HAWK | 49.04 | 50.33 | 51.09 | 45.08 | 52.27 |
| PELICAN | 49.04 | 50.33 | 51.09 | 45.08 | 52.27 |
| PUFFIN | 33.60* | 40.84* | 40.47* | 28.97* | 39.69* |
| Crosswalk Treatment | (b-2). Double-Lane Roundabout Site (2,649 PCEs per hour) | | | | |
| | "Conventional" Layout | | "Offset" Layout | "Distant" Layout | |
| | IS-1 | IS-2 | IS-2 | IS-1 | IS-2 |
| | *(b-2-1). "Few" pedestrians (12 pph)* | | | | |
| PA | 22.93 | 22.51 | 22.49 | 21.70 | 21.48 |
| HAWK | 22.93 | 22.51 | 22.49 | 21.70 | 21.48 |
| PELICAN | 22.93 | 22.51 | 22.49 | 21.70 | 21.48 |
| PUFFIN | 11.25* | 15.90* | 15.09* | 10.58* | 15.32* |
| | *(b-2-2). "Some" pedestrians (60 pph)* | | | | |
| PA | 45.40 | 49.30 | 47.12 | 41.09 | 48.19 |
| HAWK | 45.40 | 49.30 | 47.12 | 41.09 | 48.19 |
| PELICAN | 45.40 | 49.30 | 47.12 | 41.09 | 48.19 |
| PUFFIN | 23.29* | 31.24* | 29.98* | 20.71* | 29.59* |
| | *(b-2-3). "Many" pedestrians (150 pph)* | | | | |
| PA | 51.10 | 61.18 | 61.10 | 45.82 | 61.41 |
| HAWK | 51.10 | 61.18 | 61.10 | 45.82 | 61.41 |
| PELICAN | 51.10 | 61.18 | 61.10 | 45.82 | 61.41 |
| PUFFIN | 33.80* | 44.34* | 43.50* | 30.56* | 43.40* |

**TABLE 1 Pedestrian Delay (seconds) – Average of 13 Simulation Replications (*continued*)**

| **(c). "Saturated Condition"** | | | | | |
|---|---|---|---|---|---|
| Crosswalk Treatment | (c-1). Single-Lane Roundabout Site (1,992 PCEs per hour) “Conventional” Layout | | “Offset” Layout | “Distant” Layout | |
| | IS-1 | IS-2 | IS-2 | IS-1 | IS-2 |
| | *(c-1-1). "Few" pedestrians (12 pph)* | | | | |
| PA | 19.12 | 20.03 | 19.68 | 20.87 | 18.90 |
| HAWK | 19.12 | 20.03 | 19.68 | 20.87 | 18.90 |
| PELICAN | 19.12 | 20.03 | 19.68 | 20.87 | 18.90 |
| PUFFIN | 9.95* | 15.88* | 15.36* | 9.85* | 15.08* |
| | *(c-1-2). "Some" pedestrians (60 pph)* | | | | |
| PA | 43.84 | 40.76 | 40.47 | 39.69 | 40.38 |
| HAWK | 43.84 | 40.76 | 40.47 | 39.69 | 40.38 |
| PELICAN | 43.84 | 40.76 | 40.47 | 39.69 | 40.38 |
| PUFFIN | 21.95* | 29.79* | 28.76* | 20.26* | 28.79* |
| | *(c-1-3). "Many" pedestrians (150 pph)* | | | | |
| PA | 49.04 | 50.33 | 51.09 | 45.08 | 52.27 |
| HAWK | 49.04 | 50.33 | 51.09 | 45.08 | 52.27 |
| PELICAN | 49.04 | 50.33 | 51.09 | 45.08 | 52.27 |
| PUFFIN | 33.60* | 40.84* | 40.47* | 28.97* | 39.69* |
| Crosswalk Treatment | (c-2). Double-Lane Roundabout Site (3,866 PCEs per hour) “Conventional” Layout | | “Offset” Layout | “Distant” Layout | |
| | IS-1 | IS-2 | IS-2 | IS-1 | IS-2 |
| | *(c-2-1). "Few" pedestrians (12 pph)* | | | | |
| PA | 22.93 | 22.51 | 22.49 | 21.70 | 21.48 |
| HAWK | 22.93 | 22.51 | 22.49 | 21.70 | 21.48 |
| PELICAN | 22.93 | 22.51 | 22.49 | 21.70 | 21.48 |
| PUFFIN | 11.25* | 15.90* | 15.09* | 10.58* | 15.32* |
| | *(c-2-2). "Some" pedestrians (60 pph)* | | | | |
| PA | 45.40 | 49.30 | 47.12 | 41.09 | 48.19 |
| HAWK | 45.40 | 49.30 | 47.12 | 41.09 | 48.19 |
| PELICAN | 45.40 | 49.30 | 47.12 | 41.09 | 48.19 |
| PUFFIN | 23.29* | 31.24* | 29.98* | 20.71* | 29.59* |
| | *(c-2-3). "Many" pedestrians (150 pph)* | | | | |
| PA | 51.10 | 61.18 | 61.10 | 45.82 | 61.41 |
| HAWK | 51.10 | 61.18 | 61.10 | 45.82 | 61.41 |
| PELICAN | 51.10 | 61.18 | 61.10 | 45.82 | 61.41 |
| PUFFIN | 33.80* | 44.34* | 43.50* | 30.56* | 43.40* |

NOTE: IS-1 – “One-Stage” installation scheme; IS-2 – “Two-Stage” installation scheme.
* Pedestrian delay from PUFFIN is significantly different from that of PA, PELICAN, or HAWK at $\alpha = 0.05$ by paired t-test, given a specific combination of geometric layout and installation scheme.

## CONCLUDING REMARKS

This simulation study assessed four pedestrian signals hypothetically installed at typical single-/double-lane modern roundabouts where crosswalk layouts and installation schemes varied under an array of operational conditions, which enabled the quantification of mutual interactions among pedestrian crossing behaviors and traffic operations. The aim was to objectively identify potential crosswalk treatments to improve the roundabout accessibility especially for the visually impaired, seniors, and children, while maintaining acceptable multimodal mobility. The study results suggest a non-monotonic relationship between the signalization effects and all levels of vehicle volumes. Vehicle delays appeared to be the largest as traffic volumes approach the roundabout capacities. It could also be concluded that: (a) “Two-Stage” installation scheme is more operationally efficient than the “One-Stage” counterpart; (b) There are no significant differences among three geometric layouts if they are used in combination with the “Two-

Stage" scheme. When the "One-Stage" scheme is employed, the "Distant" layout, compared with the "Conventional" layout, can reduce vehicle delays and queue lengths due to the enlarged vehicle storage space at the exit lane(s); (c) HAWK poses the least delays to vehicles for most study scenarios; while PUFFIN has the minimum pedestrian delays for all scenarios. These two signals are promising for roundabout signalization, while PUFFIN is believed to provide a better balance between pedestrian safety and operational efficiency; (d) An interesting finding is the addition of pedestrian signals to double-lane roundabouts is beneficial for the vehicle circulation when vehicle flows are saturated. The study findings are informative to transportation policy-makers, planners, and practitioners in access management community which face the challenge of improving the roundabout accessibility to pedestrians especially those with impaired visions.

## FUTURE RESEARCH

This study was focused on specific roundabouts which have a very busy approach with frequent crossing pedestrian flows, and thus only one signalization was applied to each site. For those roundabouts where heavy multimodal flows exist on two or more approaches, the effect of multiple signalizations on roundabout operations can be explored for more understandings. Simultaneously, we believe an all-leg signalization could be questionable since it becomes a mix between a roundabout and a signalized intersection. Due to random pedestrian arrivals, however, four independent pedestrian signals cannot be in coordinated operations. So, it is highly likely an all-leg signalization incurs additional disturbances and/or delays to traffic flows on the whole roundabout. For wide practical use, a sufficient number of field experiments and evaluations are essential to providing the latest knowledge, expertise, and experience for advances in the state-of-the-practice in roundabout access management and their integration into established planning, policy, design processes and documents.

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# Multimodal Accessibility of Modern Roundabouts

## Intelligent Management System Versus Common Signalization Scheme

George (Xiao-Zhao) Lu, Fang Guan, and David A. Noyce

**Modern roundabouts have become popular in North America during the past decade. This popularity can be attributed to their great success in Europe and Australia. There has been significant debate, however, over their accessibility for pedestrians. With almost uninterrupted traffic flows, roundabouts make it difficult for the visually impaired to determine safe gaps, as they rely on auditory cues alone. Such crossing is particularly complicated by ambient noises and circulating vehicles on busy urban roundabouts. Various pedestrian signals have been installed at roundabouts overseas. The United States Access Board published a draft guideline proposing pedestrian signals at all roundabout crossings to ensure access for the visually impaired. Roundabout operations can be a complex process of transporting multimodal travelers. There is increased interest in harnessing artificial intelligence to address issues to improve transportation systems. This research developed a crosswalk signal and introduced fuzzy logic control (FLC) into the signal timing to accommodate roundabout users. The system was assessed against the Pedestrian User-Friendly Intelligent (PUFFIN) crossings under varied geometries under different traffic conditions. The objective was to identify potential treatments for improving roundabout accessibility, safety, and efficiency. The results reveal that "distant" layout reduces vehicle delays and queue lengths when the FLC signal is applied, especially under saturated traffic conditions. From safety and operational perspectives, the FLC signal outperforms PUFFIN. The FLC signal implements the signal timing effectively, decreases pedestrian delay, and maintains adequate vehicle circulation. Multimodal traveler needs at a modern roundabout are satisfied in manifold ways.**

During the past decade, modern roundabouts have become more popular in many states and municipalities throughout North America. Their popularity can be attributed to their great success in Europe and Australia. Their attractiveness comes from proven safety benefits, enhanced operational efficiency, reduced maintenance cost, and strengthened aesthetic appeals (*1*). In geometry, a modern roundabout is an unsignalized intersection with a round island encircled by a roadway. In operations, vehicles entering the roundabout yield to vehicles simultaneously moving on the circulatory path.

The increasing prevalence of roundabouts generated significant debate over the multimodal accessibility. Past studies identified that roundabouts pose serious difficulties to visually impaired pedestrians (*2, 3*). Pedestrian crossing becomes increasingly difficult as vehicles increase, and multilane roundabouts are more challenging than single-lane facilities to ensure safe pedestrian access (*4*). Another study further verified that crossing segments on exit lanes is more difficult than that on entry lanes (*5*). In 2002, the United States Access Board published the "Draft Guideline for Accessible Public Rights of Way, Roundabout" to propose pedestrian signals at all roundabout crosswalks. Later, the revised draft was released to call for the provision of "A pedestrian-activated traffic signal . . . for each segment of the crosswalk . . ." (*6*) at multilane roundabouts to ensure access for the visually impaired. From an operational perspective, the trade-off for this provision is interruption of motorized vehicle flows. The enhanced likelihood that a yielding queue spills back into the circulatory roadway is also a critical issue that has been identified worldwide at roundabouts with existing signalization (*7*). Until 2009, only three roundabouts were outfitted with pedestrian signals in the United States: two single-lane roundabouts on university campuses (University of Utah, Salt Lake City, and University of North Carolina, Charlotte) and one double-lane roundabout in Lake Worth, Florida. In Gatineau, Canada, a double-lane roundabout shows a staggered offset crossing with a pedestrian signal on one approach. In contrast, varied signal systems have been installed at roundabouts in Europe, Australia (*3*), and South Africa (*8*). There is minimal literature relevant to signalizing roundabouts, but a study by Rouphail et al. (*9*) indicated that addition of a pedestrian-actuated signal to a roundabout incurred delays to visually impaired pedestrians compared with sighted pedestrians who cross at unsignalized splitter islands. Another study [Schroeder et al. (*10*)] explored signalization options to make single- and double-lane roundabouts accessible to the visually impaired. The signalization impact was found to be greatest as the vehicular volume approaches capacity, but vehicle delay and queue can be mitigated through innovative signal control logic.

While few roundabouts have pedestrian signals in North America, the call from the Access Board for pedestrian access implies that more research is expected in the transportation engineering community. Contemporary transportation professionals face increased challenges in offering safe, efficient, and reliable transportation systems. Adding to these challenges is the fact that transportation system operations

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are an inherently complex process consisting of manifold, often competing, or even conflicting objectives in a dynamic setting. "As the complexity of a system increases, our ability to make precise and yet significant statements about its behaviors diminishes, and significance and complexity become almost mutually exclusive characteristics," said Kosko (*11*), which implies some transportation problems are difficult to resolve using traditional methodologies. Gradually, there has been increased interest in employing artificial intelligence (AI) to address complex issues to improve safety, operations, and other aspects of transportation systems (*12, 13*).

## OBJECTIVES AND HYPOTHESES

Inspired by the methodological tendency toward the AI domain, this research developed a new signal system in which fuzzy logic control (FLC) was introduced into the signal timing for vehicles and pedestrians at modern roundabouts. The installation of a pedestrian signal is intuitively associated with additional delays to motorized vehicles, but it is difficult to quantify this effect. This research was intended to quantitatively assess the performance of the new system against a signalization scheme that is commonly used in Europe and Australia for midblock or roundabout crosswalks. The analysis includes evaluating different crosswalk geometries and signalization schemes given a spectrum of pedestrian and vehicle volumes at single- and double-lane roundabouts. The goal is to identify potential treatments that improve the roundabout accessibility especially for the visually impaired, seniors, and children, while maintaining a good service quality for vehicles. It is hypothesized: (*a*) The operational impact of adding a pedestrian signal is a function of vehicle and pedestrian flows. (Increasing pedestrians enhance the frequency of signal actuations. Given more vehicles, each actuation poses a more drastic impact to vehicle delays and increases the likelihood that yielding queues spill back into the circulatory path); (*b*) The risk of queue spillback can be lessened by shifting the crosswalk segment on outbound lane(s) further away from the round island; and (*c*) FLC is more effective, flexible, and adaptable than traditional controls in tackling a dynamic environment.

## STUDY METHODOLOGY

From an all-roundabout-user perspective, this research explored how the change of control affects certain performance measures in three dimensions: signalization strategy, crosswalk layout, and multimodal traffic intensity. It is infeasible to examine the performances of two signal systems in a real-world context due to disruptions and detriments to traffic safety and operations. Instead, a reliable in-lab test bed should be established as a surrogate means by which signal systems can be implemented and evaluated under a controllable condition.

### Study Environment

Traffic simulation is an indispensable tool for contemporary transportation professionals and researchers because of its cost-effectiveness, unobtrusiveness, risk-free nature, and computational efficiency. It yields extensive performance measures that fully reflect the operations. More importantly, it offers the unique opportunity to implement different traffic management strategies and evaluate their effectiveness under various traffic conditions prior to field deployment. VISSIM, a microsimulation program, is widely employed to model various facilities due to some technical advantages over other counterparts (e.g., traveler behavior modeling, detector functionality, control logic flexibility, and run time control) (*14*). VISSIM models have been used or calibrated to mimic real-world situations at freeways (*15*), urban networks (*16*), crosswalks or intersections (*17–19*), arterials, and roundabouts (*9, 10*). Its link-connector structure can flexibly model unique roundabout geometries. It can implement user-defined control strategies and emulate yielding behaviors by vehicles. Therefore, a VISSIM-based framework was established as a reliable and controllable platform for study.

### Crosswalk Geometry

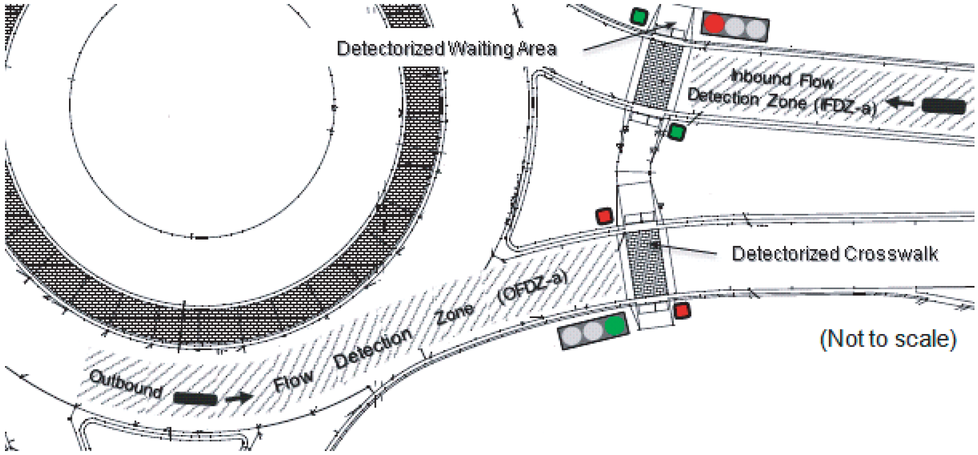
In North America, the most common crosswalk layout for a roundabout runs across the splitter island, about one vehicle length (20 ft) upstream of the yield line, which can be termed conventional layout (Figure 1*a*). Two other layouts were analyzed considering the key issue that vehicle queues may spill back into the circulatory path. One, offset layout, is to spatially shift the crosswalk segment on the exit leg further away from the circle: the exit-leg segment is "offset" by a distance of 80 ft from the circulatory lane(s), and approximately this distance accommodates four vehicles per lane before vehicles intrude into the circulatory path (Figure 1*b*). The other, distant layout, moves the entire crosswalk a distance of 120 ft away outward, which yields roughly six-vehicle queue storage per lane (Figure 1*c*).

### Signalization Alternatives

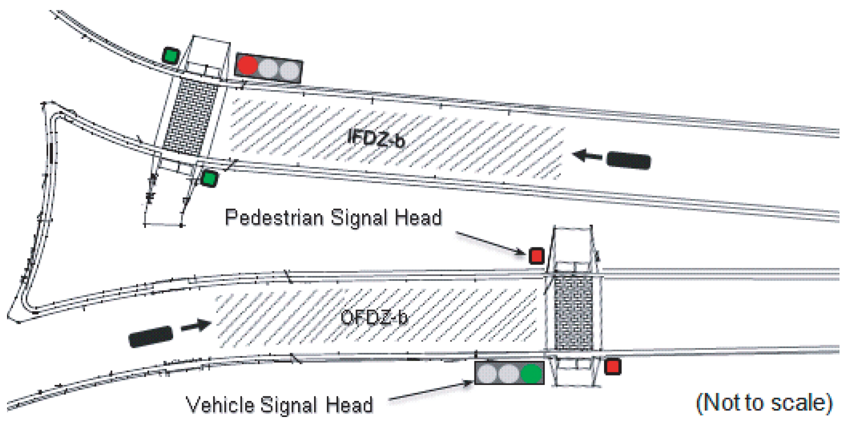
Some pedestrian signals installed in Europe include PEdestrian LIght CONtrol (PELICAN), Two CAN (TOUCAN), and Pedestrian User-Friendly Intelligent (PUFFIN) (*20, 21*). In the United States, a few local traffic management authorities provide the guidelines for considering these options to control mid-block crosswalks, which signifies the rising number of applications in North America (*22*). Conventionally, the pedestrian-actuated (PA) signal is used for crosswalks per the *Manual on Uniform Traffic Control Devices* (MUTCD) (*23*), while High-Intensity Activated CrossWalK (HAWK) is experimented at mid-block crosswalks in Tucson, Arizona; Portland, Oregon; and several other cities (*24, 25*).

#### *PUFFIN*

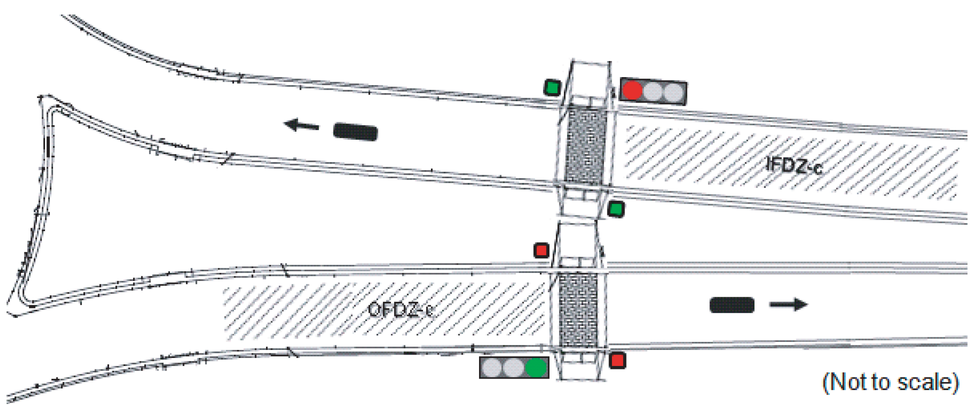
PA, PELICAN, TOUCAN, and HAWK statically time "Flashing DON'T WALK" (FDW) using the crossing distance and a design walking speed. From a safety perspective, this practice is questionable due to the variability in walking speeds among the visually impaired, the aging population, and the growing child mobility. Past studies in North America revealed that walking speeds vary considerably for different populations (*26–28*). Current pedestrian signals lack adequate "pedestrian friendliness" since their static FDW timing does not provide full signal protection for all pedestrians. Pedestrians can be exposed to yielding vehicles if insufficient FDW time is provided.




(a)




(b)




(c)

FIGURE 1 Crosswalk layouts and detection zones for input data collection: (*a*) conventional layout, (*b*) offset layout, and (*c*) distant layout.

In contrast, PUFFIN is adequate in "pedestrian friendliness" because in-crosswalk sensors adjust the pedestrian clearance time to offer the crossing time needed. This dynamic timing provides full protection to the visually impaired, seniors, and children (Figure 2*a*, Tables 1 and 2). However, PUFFIN omits safety elements or human factors in its control logic, without pursuing additional operational and safety objectives in dynamic operations.

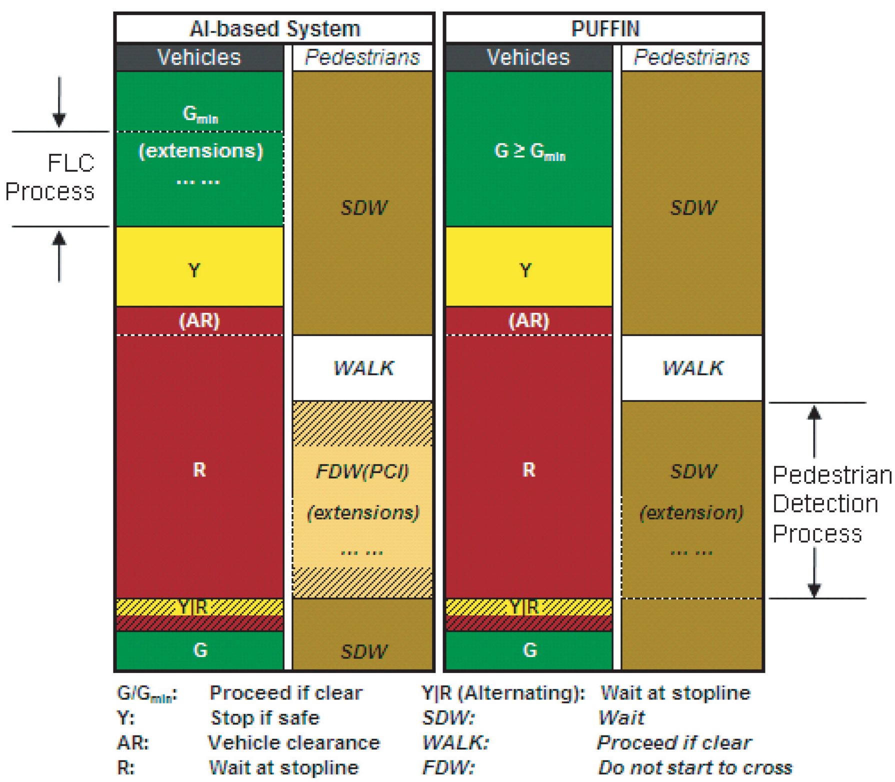
### An AI-Based System

An AI-based system signalizes a roundabout means to determine the optimal time for switching the right-of-way between vehicles and pedestrians. Any traffic signal control is a process of apportioning green time to conflicting facility users. The signal system evaluates ongoing traffic conditions with decision-making criteria to conduct appropriate adjustments in timing plans. In principle, signal control is a process of determining, at regular time intervals ($\Delta t$), whether to extend or terminate the current vehicle green.

In reality, a crossing guard is sometimes deployed at a facility (e.g., crosswalk near a school) for traffic management. The guard subjectively processes intuitive rules by evaluating ongoing and desired operations. For example, if he or she feels a pedestrian has been waiting for a "frustratingly long" time and upcoming vehicles are "sparsely" present, he or she "terminates" the right-of-way for vehicles and switches it to pedestrians. This manual control is effective, safe, adaptive, and robust for tackling dynamic traffic operations, because the human intelligence has unlimited flexibility in data processing, logical reasoning, and decision making. In this research, an FLC-based signal system was developed to artificially emulate the human intelligence of the guard. Following the signal control principle, the system compares traffic conditions during current and next phases to realize some competing objectives. It is different from existing systems that do not examine prevailing traffic conditions for all travelers.

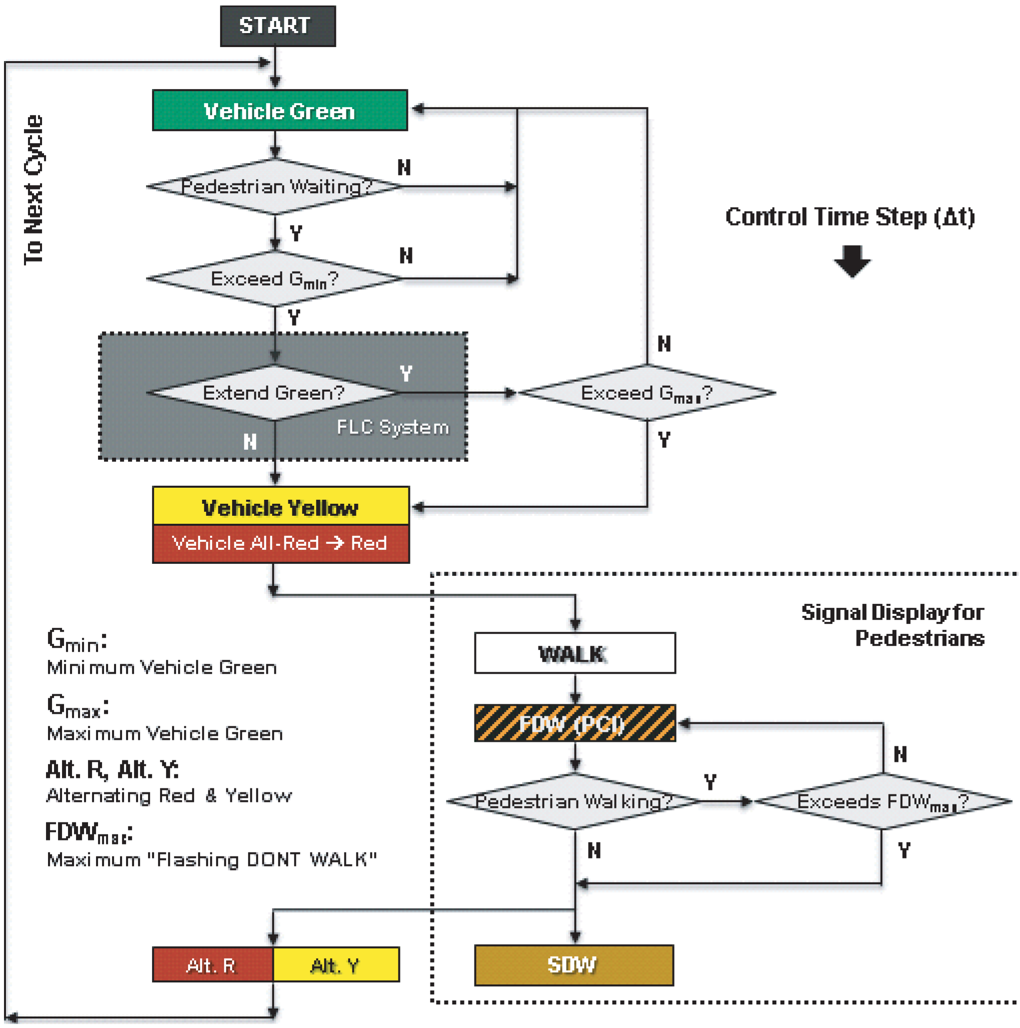
Figures 2*a* and *b* depict the phasing scheme and the control logic of the system. With pedestrians absent, the system displays the "idle phase" (vehicle green). With pedestrians present, it examines whether the minimum green is exceeded by the green already displayed. If so, it evaluates ongoing operations for all roundabout users and executes the fuzzy inference for control action. Once specified, decision-making criteria are triggered and the current green is either terminated or extended. This process is reiterated at time intervals until the right-of-way is switched or the maximum green is reached. When the green is terminated, "WALK" starts, then "FDW" follows. To offer the "pedestrian friendliness," the system displays dynamic "FDW" via on-crosswalk sensors and extends it up to its maximum for the slowest pedestrians. One second of "Alternating Red/Yellow," alerting drivers to possible pedestrians, is displayed before the vehicle green for the sake of the consistence with PUFFIN.

FLC has the ability to handle multiple objectives (*13*). Several objectives were set: (*a*) minimum delay to pedestrians—the wait time should be decreased as much as possible, (*b*) minimum delay to vehicles—vehicles should traverse a roundabout with the least possible delay, (*c*) maximum safety for all users, which is twofold embodied: first, to offer full signal protection for the visually impaired, seniors, and children who walk slowly, and second, to dissipate




(a)

FIGURE 2 AI-based pedestrian signals at roundabouts: (*a*) signal phasing schemes comparison.
*(continued)*




(b)

FIGURE 2 *(continued)* AI-based pedestrian signals at roundabouts: (*b*) fuzzy logic signal control flowchart.

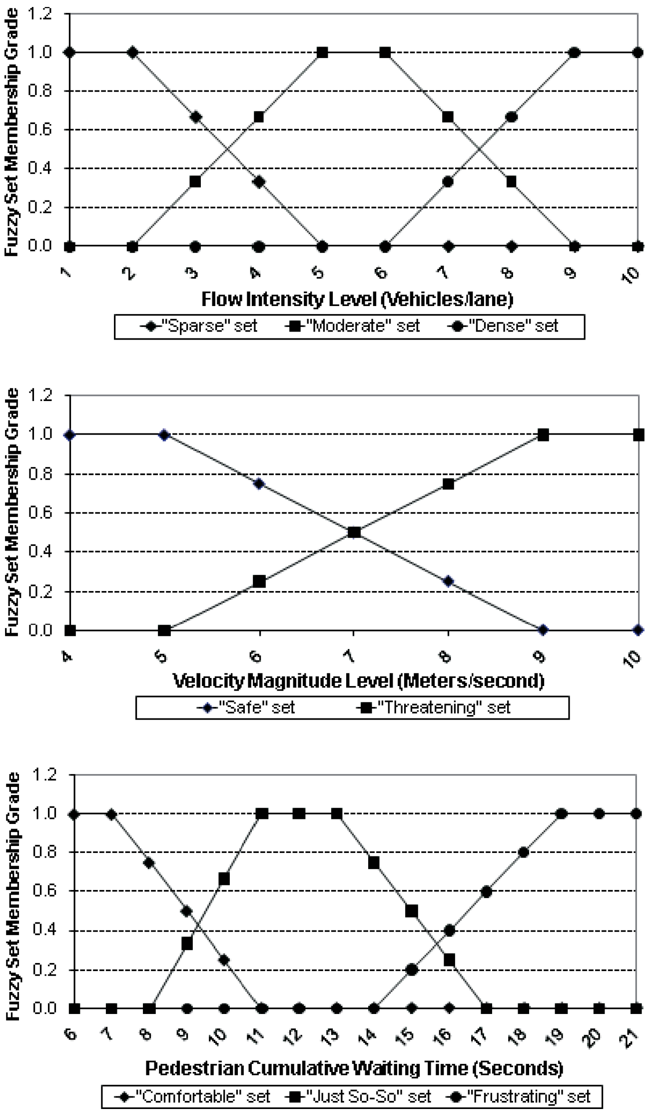
*(continued on next page)*

vehicles promptly, when they approach the crosswalk in a large volume and at a high speed, for safety consideration: to diminish the likelihood of rear-end collisions if green terminates abruptly.

A portion in Figure 3*d* illustrates key components of the FLC system: fuzzifier, inference engine, and defuzzifier.

**Fuzzifier** The membership function plays a key role in a fuzzifier, which transforms the following crisp input variables into these fuzzy sets to be processed by the inference engine:

- $CWT_{In}(t+\Delta t)$, $CWT_{Out}(t+\Delta t)$, $CWT_{Cir}(t+\Delta t)$. Definition: Maximum roadside-based Cumulative Waiting Time (seconds) already consumed by pedestrians, at time point $(t+\Delta t)$, who are waiting to cross Inbound, distant Outbound lane(s), or lane(s) near a Circulatory path. This input variable represents how long pedestrians have been waiting since the last change from "WALK" to "FDW," which reflects human factors in accommodating pedestrians. Pedestrian delays have operations- and safety-related implications. The longer a pedestrian has been delayed, the more likely he or she is to cross without an appropriate signal display.
- $FIL_{In}(t+\Delta t)$, $FIL_{Out}(t+\Delta t)$, $FIL_{Cir}(t+\Delta t)$. Definition: Average lane-based Flow Intensity Level (vehicles/lane) within $\Delta t$ for vehicles approaching or passing Inbound, Outbound, or Circulating lane(s). This input variable measures the number of vehicles within detection zones for signalized approach lanes and the circulatory roadway, which reflects ongoing vehicle flow intensity. Vehicle delays have both efficiency- and safety-related impacts. The more intense the flow intensity is, the more strongly the vehicles demand for green. Psychologically, the longer a motorist is delayed, the more likely he or she will become impatient or aggressive.
- $VML_{In}(t+\Delta t)$, $VML_{Out}(t+\Delta t)$, $VML_{Cir}(t+\Delta t)$. Definition: Maximum lane-based Velocity Magnitude Level (meters per second) within $\Delta t$ for vehicles approaching or passing Inbound, Outbound, or Circulating lane(s). This input variable reflects the threatening vehicle when it approaches the crosswalk. The vehicle speed addresses the safety issue: the faster the vehicles are moving, the more likely it is that an abrupt green termination incurs rear-end collisions.
- $SCA(t+\Delta t)$. Definition: Signal Control Action taken at the time point $(t+\Delta t)$. This output variable denotes the control actions on vehicle green: Extension or Termination.




(c)

**FIGURE 2** *(continued)* **AI-based pedestrian signals at roundabouts: (*c*) membership functions used in Rule Base A (Tables 1 and 2) of fuzzy logic signal system.**

**TABLE 1 Fuzzy Logic Inference Engines for Roundabout FLC Signals: Generic Format of Fuzzy Logic Rules**

| Fuzzy Rule | Premises (crisp inputs: $X = a$, $Y = b$, $Z = c$) | Consequences |
|---|---|---|
| Rule 1 | If {*X is* $a_1$} and {*Y is* $b_1$} and {*Z is* $c_1$} | Then {$E^a$ or $T^b$} |
| | . . . . . . . . . . . . . . . . . . | |
| Rule i | If {*X is* $a_i$} and {*Y is* $b_i$} and {*Z is* $c_i$} | Then {*E* or *T*} |
| | . . . . . . . . . . . . . . . . . . | |
| Rule n | If {*X is* $a_n$} and {*Y is* $b_n$} and {*Z is* $c_n$} | Then {*E* or *T*} |
| Crisp output | {E or T} | |
| Where | *X*, *Y*, *Z* = input (state) variables related to traffic conditions, *a*, *b*, *c* = values of input variables, and $a_i$, $b_i$, $c_i$ = natural language expressions for traffic conditions | |

[a]Terminate current vehicle green.
[b]Extend current vehicle green.

Hypothetically, $CWT_{In,Out,Cir}(t{+}\Delta t)$ was collected per near-crosswalk pushbuttons and waiting area sensors, while vehicle data were collected in six detection zones that cover inbound–outbound–circulating lane(s) (Figure 1): OFDZ-*a* collects $FIL_{Cir}(t + \Delta t)$ and $VML_{Cir}(t + \Delta t)$; OFDZ-*b* and OFDZ-*c* collect $FIL_{Out}(t + \Delta t)$ and $VML_{Out}(t + \Delta t)$; other three zones (IFDZ-*a*, IFDZ-*b*, and IFDZ-*c*) collect $FIL_{In}(t + \Delta t)$ and $VML_{In}(t + \Delta t)$. Some fuzzy sets were defined for each input variable (Tables 1 and 2). Trapezoid membership function was harnessed to avoid complicating the problem, which needs four parameters of trapezium breakpoints: Trapezoid $(x;m,n,p,q) = \max\{\min((x-m)/(n-m),1,(q-x)/(q-p),0)\}$. For different layouts, membership functions for $CWT(t + \Delta t)$ are segregated into "motorist friendly" and "pedestrian friendly" types, which emphasize the respective convenience to motorists and pedestrians. To capture the quantifiable feeling of all fuzzy sets that represent operational conditions, this research conducted field observations, quantitative recordings, and basic statistical analyses for subject roundabouts, as well as literature review (*12*). To approximate the function shapes that produce near-optimal simulation performances, trial-and-error methods were applied to a number of scenarios. As an example, Figure 2*c* delineates the membership functions in a rule base.

**Inference Engine** It "thinks" as a "human brain" through a set of rules that describe, in natural language, ongoing traffic conditions for current and next phases. Tables 1 and 2 show the generic structure of a rule base. The facts following "IF" and "THEN" are termed "premise" and "consequence," respectively, while "AND" is an "operator." The traditional inference draws a conclusion when a rule is an exact match between the input (*a*, *b*, *c*) and a premise ($a_i$, $b_i$, $c_i$). So, many rules are necessary to cover all possibilities. The output is singular and the decision-making process is characterized by its rigidness. PUFFIN lies in this realm; its timing mechanism performs in an inflexible way, due to the rigidity in maintaining specific parameters. Differently, the fuzzy inference makes a conclusion based on the similarity between the input (*a*, *b*, *c*) and premises ($a_1$, $b_1$, $c_1$; . . . ; $a_i$, $b_i$, $c_i$; . . . ; $a_n$, $b_n$, $c_n$). A one-to-one match is unnecessary and the extent of similarity dominates the degree of truth in consequence. With this paradigm, a specific input triggers multiple rules because the input and premises in triggered rules are represented by fuzzy sets and fuzzy relationships produced by set operations. Hence, different consequences from all activated rules are valid and they are aggregated for a final output space consisting of fuzzy control actions. To be defuzzified, the final output space is a compromise among these conclusions from all triggered rules. Essentially, all rules and conclusions are implicitly associated with procuring manifold, perhaps conflicting, objectives given numerous possibilities of ongoing traffic conditions. The decision-making mechanism is characterized by its flexibleness, which exhibits the robust and adaptive feature in pursuing multiple goals because the membership functions implicitly enclose an extensive scope of possibilities. Typically, the number of rules depends on the combination of fuzzy sets defined. Three layout-specific rule bases were established through "assimilating" the human intelligence of a crossing guard (Tables 1 and 2). Mamdani's method adopted herein is the most common approach for the aggregation process. It was from Zadeh's work on fuzzy algorithms for complex systems and decision processes (*29*), which was among the first control system built using fuzzy set theory by synthesizing a set of linguistic control rules obtained from experienced human operators (*30*).

**TABLE 2 Three Inference Rule Bases Established on Basis of Crosswalk Layouts**

| | If | | | Then |
|---|---|---|---|---|
| Rule Base A | Input (state) Variables $CWT_{In}(t+\Delta t)$ | $FIL_{In}(t+\Delta t)$ | $VML_{In}(t+\Delta t)$ | Output (control) Variable $SCA(t+\Delta t)$ |
| 1 | Comfortable | Sparse | Safe | T[a] |
| 2 | Comfortable | Sparse | Threatening | T |
| 3 | Comfortable | Moderate | Safe | E[b] |
| 4 | Comfortable | Moderate | Threatening | E |
| 5 | Comfortable | Dense | Safe | E |
| 6 | Comfortable | Dense | Threatening | E |
| 7 | Just so-so | Sparse | Safe | T |
| 8 | Just so-so | Sparse | Threatening | T |
| 9 | Just so-so | Moderate | Safe | T |
| 10 | Just so-so | Moderate | Threatening | E |
| 11 | Just so-so | Dense | Safe | E |
| 12 | Just so-so | Dense | Threatening | E |
| 13 | Frustrating | Sparse | Safe | T |
| 14 | Frustrating | Sparse | Threatening | T |
| 15 | Frustrating | Moderate | Safe | T |
| 16 | Frustrating | Moderate | Threatening | T |
| 17 | Frustrating | Dense | Safe | T |
| 18 | Frustrating | Dense | Threatening | E |
| Rule Base B | Input (state) Variables $CWT_{Out}(t+\Delta t)$ | $FIL_{Out}(t+\Delta t)$ | $VML_{Out}(t+\Delta t)$ | Output (control) Variable $SCA(t+\Delta t)$ |
| 1 | Comfortable | Sparse | Slow | T |
| 2 | Comfortable | Sparse | Fast | T |
| 3 | Comfortable | Moderate | Slow | E |
| 4 | Comfortable | Moderate | Fast | E |
| 5 | Comfortable | Dense | Slow | E |
| 6 | Comfortable | Dense | Fast | E |
| 7 | Just so-so | Sparse | Slow | T |
| 8 | Just so-so | Sparse | Fast | T |
| 9 | Just so-so | Moderate | Slow | E |
| 10 | Just so-so | Moderate | Fast | E |
| 11 | Just so-so | Dense | Slow | E |
| 12 | Just so-so | Dense | Fast | E |
| 13 | Frustrating | Sparse | Slow | T |
| 14 | Frustrating | Sparse | Fast | T |
| 15 | Frustrating | Moderate | Slow | T |
| 16 | Frustrating | Moderate | Fast | T |
| 17 | Frustrating | Dense | Slow | T |
| 18 | Frustrating | Dense | Fast | E |

*(continued on next page)*

**TABLE 2** *(continued)* **Three Inference Rule Bases Established on Basis of Crosswalk Layouts**

| | If | | | Then |
|---|---|---|---|---|
| Rule Base C | $CWT_{Cir}(t+\Delta t)$ | $FIL_{Cir}(t+\Delta t)$ | $VML_{Cir}(t+\Delta t)$ | $SCA(t+\Delta t)$ |
| 1 | Comfortable | Sparse | Slow | T |
| 2 | Comfortable | Sparse | Fast | T |
| 3 | Comfortable | Moderate | Slow | E |
| 4 | Comfortable | Moderate | Fast | E |
| 5 | Comfortable | Dense | Slow | E |
| 6 | Comfortable | Dense | Fast | E |
| 7 | Just so-so | Sparse | Slow | T |
| 8 | Just so-so | Sparse | Fast | T |
| 9 | Just so-so | Moderate | Slow | E |
| 10 | Just so-so | Moderate | Fast | E |
| 11 | Just so-so | Dense | Slow | E |
| 12 | Just so-so | Dense | Fast | E |
| 13 | Frustrating | Sparse | Slow | T |
| 14 | Frustrating | Sparse | Fast | T |
| 15 | Frustrating | Moderate | Slow | T |
| 16 | Frustrating | Moderate | Fast | T |
| 17 | Frustrating | Dense | Slow | E |
| 18 | Frustrating | Dense | Fast | E |

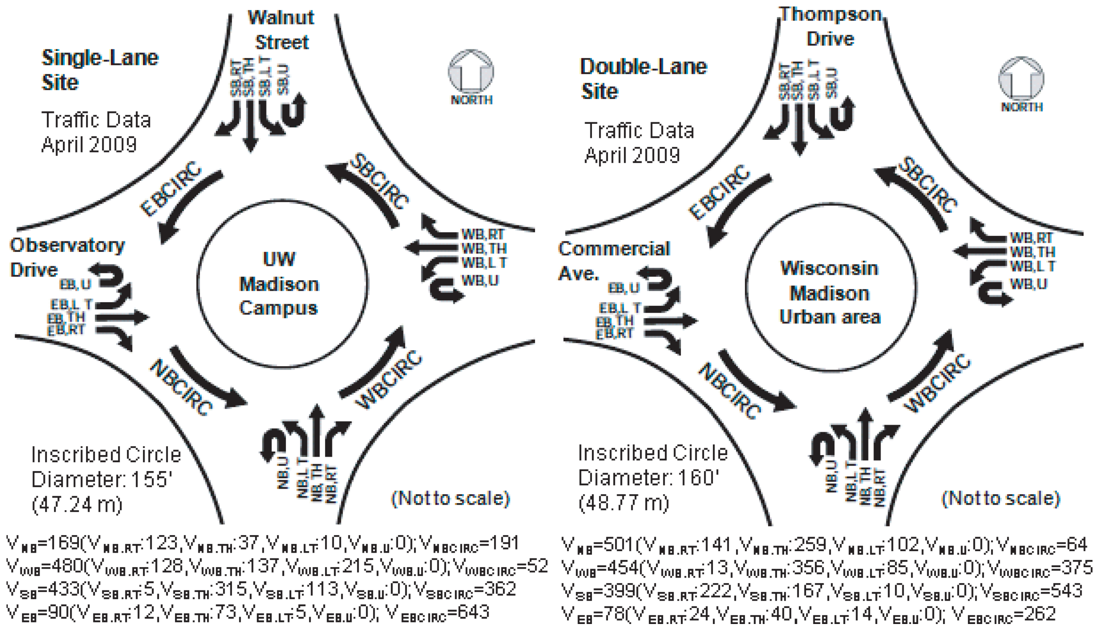
**Defuzzifier** It realizes a mapping from the output space of fuzzy control actions into a final output variable. Some defuzzifiers were developed to finalize the output. The most frequently employed mappings include "maximum criterion," "mean of maximum," and "center of gravity." Each has its own unique features suitable for different control problems (*31*). Traffic signal control has a binary characteristic: extension or termination. This means that the final output variable is in crisp form, so "maximum criterion" is the most appropriate herein in contrast to other defuzzifiers that transform the output space into a continuous variable.

## Traffic Flow Modeling

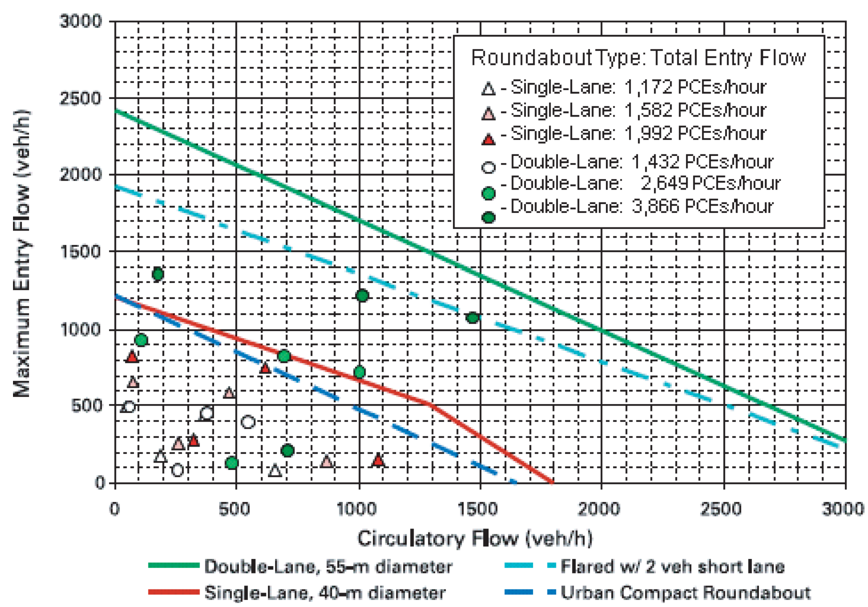
It is reasonable to signalize the crosswalk on the approach with the densest vehicle volumes and highest speeds, since such an approach generates the scarcest safe crossable gaps. Actual peak-hour traffic volumes collected at two modern roundabouts in Wisconsin were used as the base volumes (Figure 3*a*). The single-lane site has significant commuting traffic. Field observations uncovered pedestrian access issues: two bus stops in the vicinity yield a large number of riders, including vision-impaired pedestrians; seasonal football events generate massive pedestrian streams in which many seniors walk. The double-lane site is in proximity to a residential community in Madison. The peak-hour traffic is heavy and prevailing vehicle speeds are fast. The observed volumes are below the theoretical capacity for the respective size as cited in FHWA's roundabout guide (*32*). To investigate more cases, vehicle flow intensities were increased at a fixed growth rate to simulate scenarios closer to maximum capacity. The roundabout guide recommends that roundabouts be designed to operate at less than 85% of the estimated capacity. Through a guide-based calculation toward the 85% threshold, the single-lane volume was increased by 35% and 70% to achieve 1,582 and 1,992 PCEs/h, while 85% and 170% were applied to the double-lane volume to get 2,649 and 3,866 PCEs/h. Conceptually, three intensity levels (existing condition, approaching capacity, and saturated condition) were established. Figure 3*b* illustrates the base and enhanced volumes of two sites superimposed on the guide's capacity figure. Each scenario was analyzed at pedestrian flows of 0 (none), 12 (few), 60 (some), 150 (many) pedestrians/h. These pedestrian flows are less than the MUTCD Section 4C.05 Warrant 4 (*23*), since the primary motivation for installing these signals is not to suffice for a MUTCD warrant but to make roundabouts more accessible. Approximately 15% of pedestrians walk more slowly than 3.5 ft/s (*33*). So, the mean walking speed was set to 3.0 ft/s and a researcher-customized distribution was modeled (maximum/minimum speeds: 8.0/1.0 ft/s) to reflect past study findings.

## Model Calibration

VISSIM models were coded with observed volumes, turning movements, and geometric designs consistent with the FHWA roundabout guide (*32*). Vehicle speeds were calibrated from field data, which include speeds prevailing on inbound approaches, entering circles, and bypassing islands. Speeds are characterized by normal distributions. Minimum gap times, minimum headways, and maximum speeds have been determined through previously documented research results that "serve as a realistic base for most applications" (*14*). The yielding behaviors were modeled in compliance with two examples (*14*). Then, the performance of each model for the "zero-pedestrian" case was validated by contrasting average vehicle delays and approach queues with manual measurements from video recordings. The results demonstrated vehicle delays, and queues match video observations to a large extent (Figure 3*c*), although there was a limited sample of observations and the validation work could be improved with additional data.




(a)




(b)

FIGURE 3 Roundabout modeling and simulation experiments: (*a*) actual peak-hour traffic volumes (Vs) in passenger car equivalents (PCEs) calculated by FHWA Roundabout Guide standard (*32*) for both subject sites, (*b*) VISSIM model calibration results.

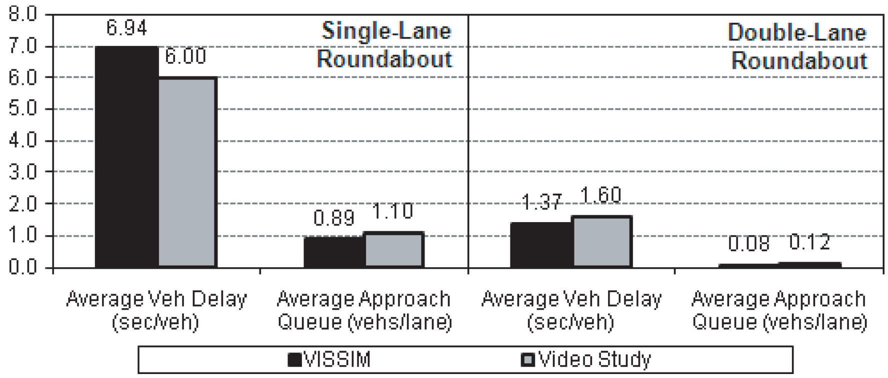
*(continued on next page)*

## Performance Measures

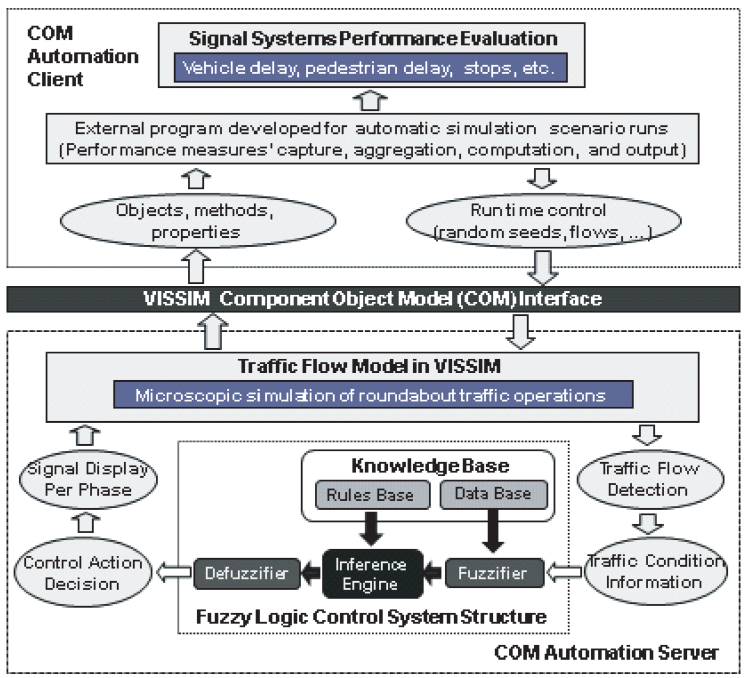
One major objective was to quantify the impact of pedestrians crossing at a signalized approach upon the roundaboutwide operations. These vehicle-related performance measures (i.e., average vehicle delay, average queue length, and average number of stops) were determined in terms of the "pedestrian-induced" effect, which was defined as the discrepancy between measures generated at certain pedestrian volumes and those at the "zero-pedestrian" base case. Average number of stops was viewed as a safety indicator: its increase signifies more frequent acceleration or deceleration occurrences, which intensify the potential for rear-end collisions. Average pedestrian delay was used to evaluate operational efficiency and safety, which is defined as the difference between actual travel time and minimum one (at given walking speed without delays) across the path of interest.

## Basic Timing Parameters

The time interval ($\Delta t$) lasted 1.0 s. With two-phase signals, all lanes in two directions were managed independently and a pedestrian may wait on the median. Minimum vehicle greens varied; "Yellow"




(c)




(d)

FIGURE 3 *(continued)* Roundabout modeling and simulation experiments: (*c*) entry volumes relative to theoretical capacity in FHWA Informational Guide, and (*d*) FLC system structure and run time control–computation via VISSIM-based COM automation.

and "All-Red" intervals displayed for 1 and 4 s, respectively. The MUTCD recommends 4 to 7 s for "WALK"; 6 s was used.

### Run-Time Control and Computation

Crosswalk layouts were combined with signal systems to yield six treatments, each of which was modeled with vehicle and pedestrian flow intensities to create 108 scenarios. Twelve simulation replications were implemented for each scenario to overcome stochastic variations from underlying models. There were 1,296 runs; each lasted 3,600 s. The data for performance measures were collected within an evaluation node surrounding roundabout models. During run time, 1,296 runs were carried out automatically and data were captured, aggregated, computed, and exported per an external program that was developed as a Component Object Model client in dialogue with the VISSIM-based server (Figure 3*d*).

## STUDY RESULTS

Figures 4 and 5 show the results regarding single- and double-lane roundabouts in conjunction with signalization schemes, crosswalk layouts, and pedestrian volume levels. Figures also illustrate the effects of vehicle flow intensities: existing condition, approaching capacity, and saturated condition. Results are reported by mean values of 12 replications.

### Pedestrian-Induced Vehicle Delay

The single-lane roundabout results in Figures 4*a*–*f* suggest, at a specific vehicle flow intensity, pedestrian-induced vehicle delays are consistently enhanced when pedestrians increase from "few" to "some" to "many." This demonstrates the operational effect of crossing pedestrians upon vehicle flow efficiency, which means more pedestrians

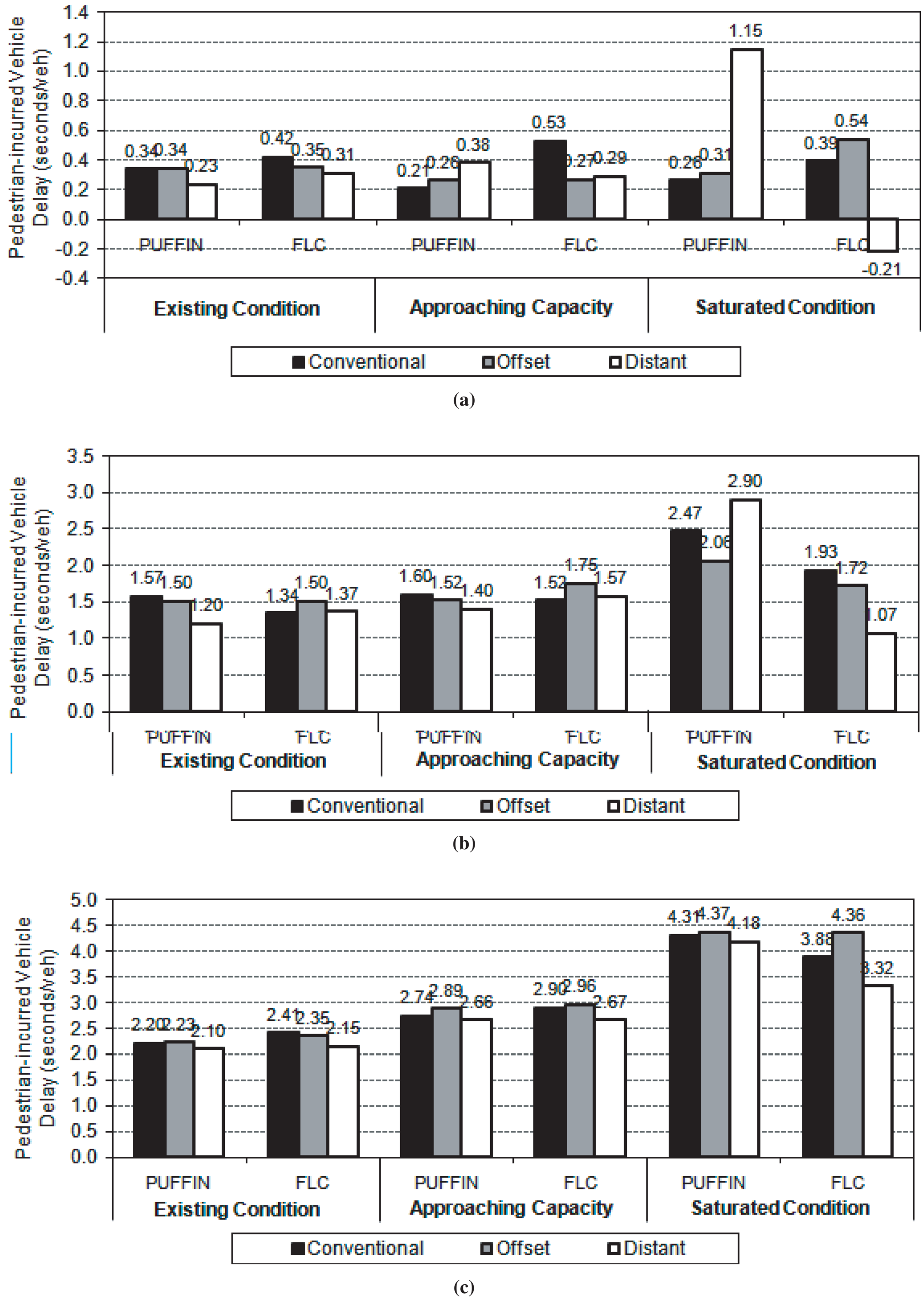
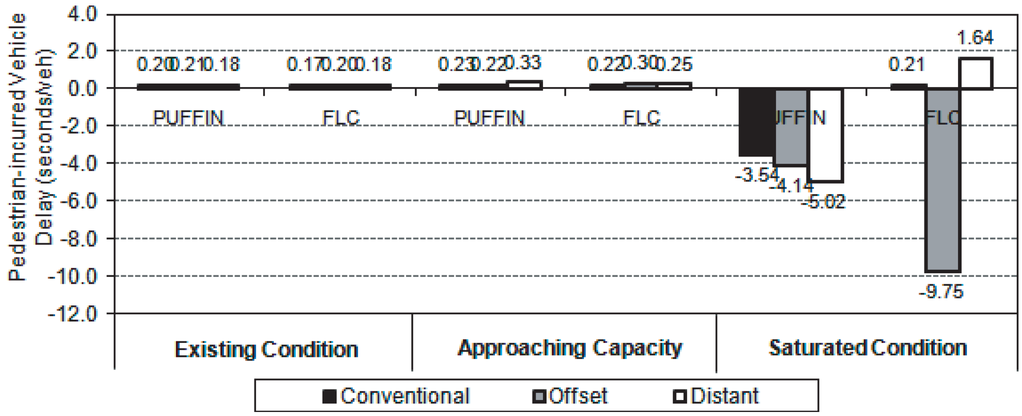



FIGURE 4 Pedestrian-induced vehicle delay: (*a*) single-lane roundabout—few pedestrians [12 pedestrians per hour (pph)], (*b*) single-lane roundabout—some pedestrians (60 pph), (*c*) single-lane roundabout—many pedestrians (150 pph).

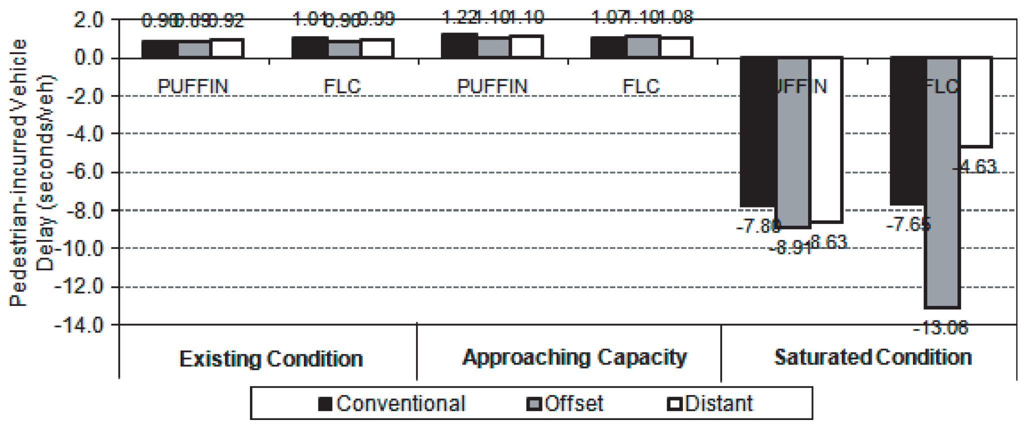
*(continued on next page)*

make the vehicle flow interruption occur more frequently. When the vehicle volume increases given "some" or "many" pedestrians, the delay impact of PUFFIN on vehicles, regarding each layout, gradually rises up to its maximum under "saturated condition." Given "many" pedestrians, for each signal there exists a roughly monotonic relationship between vehicle volumes and pedestrian-induced vehicle delays.

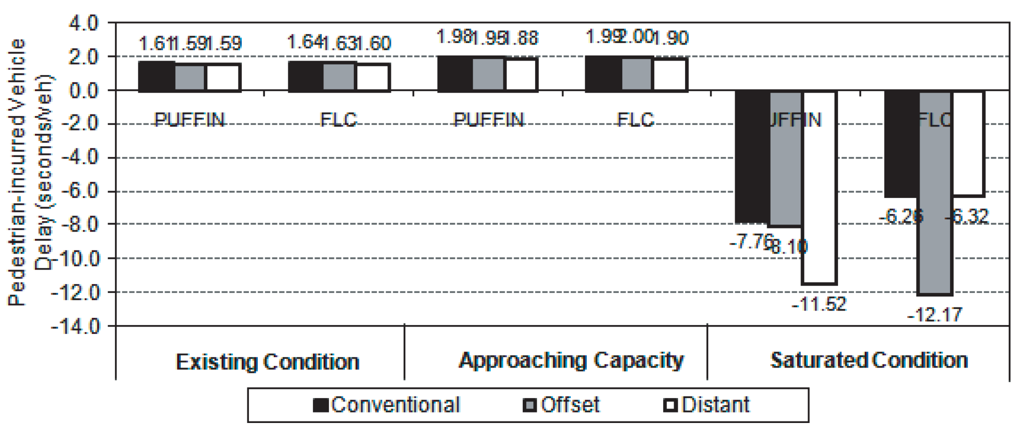
FLC system performance is better than, close, or roughly equal to PUFFIN across most pedestrian and vehicle flow levels, considering the magnitude of vehicle delays generated by the "zero-pedestrian" case (6.94 s/vehicle for "existing condition" and higher for larger vehicle volumes). Comparatively, the "distant" layouts show potential advantages at the single-lane roundabout, since their additional queue storages produce vehicle delays less than those




(d)




(e)

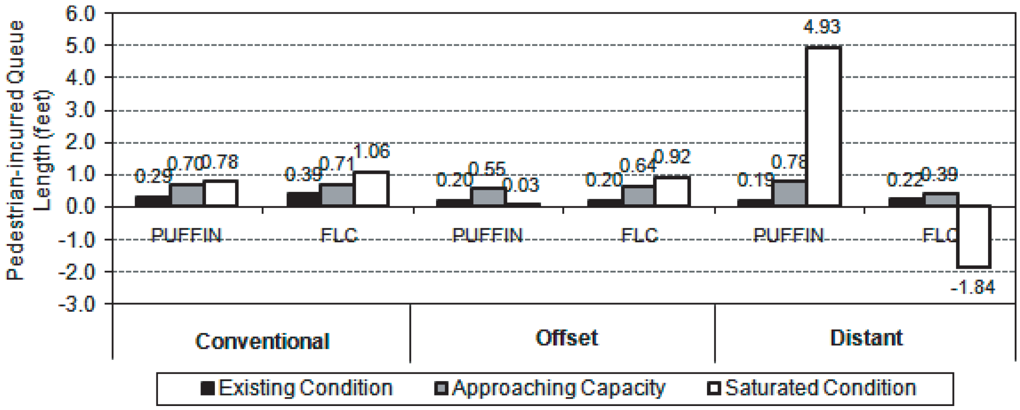



(f)

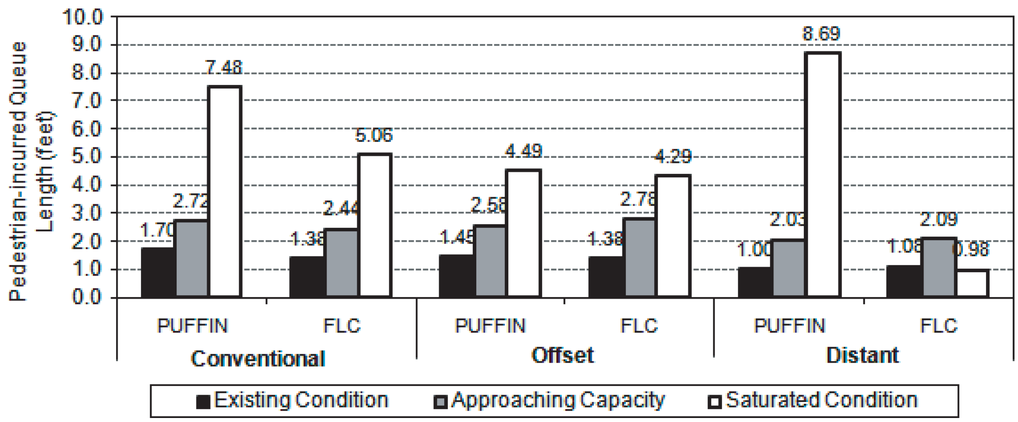
FIGURE 4 *(continued)* Pedestrian-induced vehicle delay: (*d*) double-lane roundabout—few pedestrians (12 pph), (*e*) double-lane roundabout—some pedestrians (60 pph), and (*f*) double-lane roundabout—many pedestrians (150 pph).

at other layouts across most scenarios, especially when the FLC signal operates under "saturated condition." Note that this layout is disadvantageous when PUFFIN works in two situations where pedestrians are "few" or "some" and vehicles are in "saturated condition."

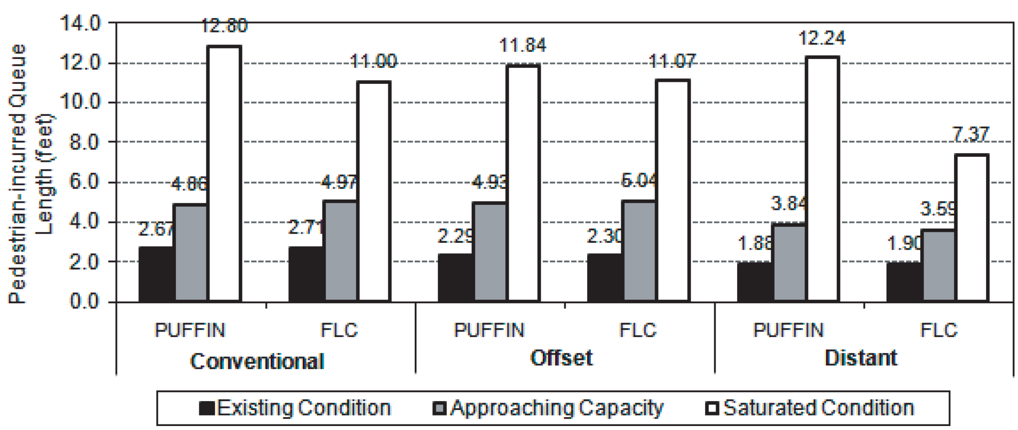
The results at the double-lane roundabout have similar characteristics to those at the single-lane site, except for an interesting observation that some scenarios yield negative vehicle delays under "saturated condition." In these scenarios, the presence of pedestrian actuations substantially decreases pedestrian-induced vehicle delays. This phenomenon could be the consequence of pedestrian signal metering traffic on the busiest approach, thus facilitating the entering vehicle flows at downstream roundabout approaches.




(a)




(b)



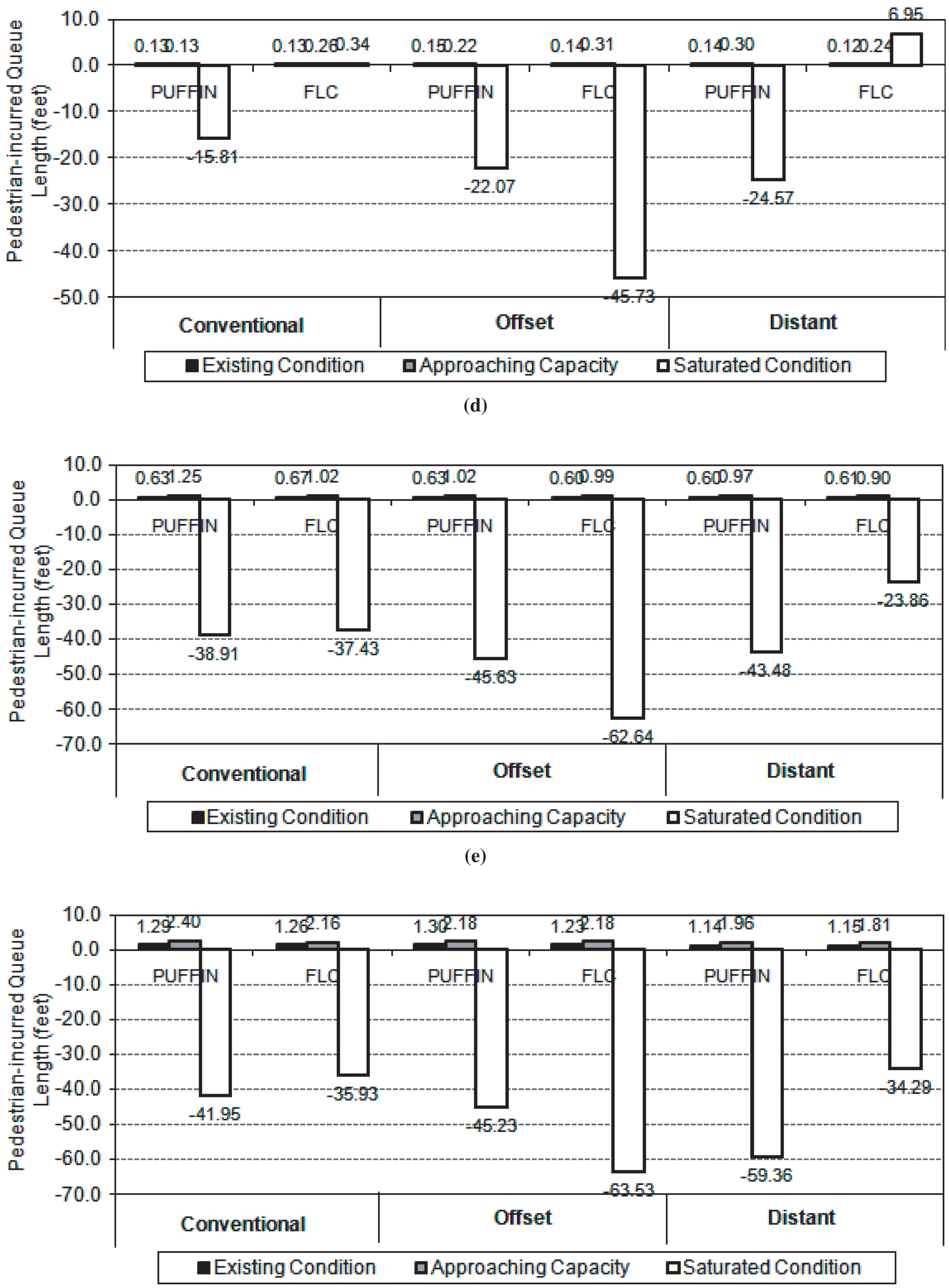

(c)

FIGURE 5 Pedestrian-induced queue length: (*a*) single-lane roundabout—few pedestrians (12 pph), (*b*) single-lane roundabout—some pedestrians (60 pph), (*c*) single-lane roundabout—many pedestrians (150 pph).

*(continued on next page)*

## Pedestrian-Induced Queue Length

The results in Figures 5*a* through *f* give the pedestrian-induced queue lengths. At specific vehicle intensity, queue lengths at the single-lane roundabout are prolonged if pedestrian volume increases. When the vehicle volume increases at a specific pedestrian flow, the influence of a pedestrian signal upon vehicle queues reaches, across most scenarios, its maximum under "saturated condition." There exists a nearly monotonic nexus between vehicle volumes and queue lengths regardless of signals and layouts, especially when there are "many" pedestrians. Given the magnitude of "zero-pedestrian" queue lengths, the FLC signal outperforms or resembles PUFFIN across most scenarios. The "distant" layout exhibits potential advantages at the single-lane roundabout when the FLC




(d)



(e)



(f)

FIGURE 5 *(continued)* Pedestrian-induced queue length: (*d*) double-lane roundabout—few pedestrians (12 pph), (*e*) double-lane roundabout—some pedestrians (60 pph), and (*f*) double-lane roundabout—many pedestrians (150 pph).

**TABLE 3 Pedestrian Delay (seconds): Average of 12 Simulation Replications, Existing Condition**

| | Single-Lane Roundabout (1,172 PCEs/h) | | Double-Lane Roundabout (1,432 PCEs/h) | |
|---|---|---|---|---|
| Crosswalk Layout | PUFFIN | FLC | PUFFIN | FLC |
| | (A-1-1). Few pedestrians (12 pph) | | (A-2-1). Few pedestrians (12 pph) | |
| Conventional | 14.29 (1.89) | 13.51 (1.95) | 14.99 (3.32) | 13.87 (1.91) |
| Offset | 13.64 (2.18) | 12.29 (1.83) | 13.65 (2.43) | 12.37 (1.88) |
| Distant | 14.26 (2.44) | 13.35 (1.60) | 15.37 (2.87) | 13.72 (1.54) |
| | (A-1-2). Some pedestrians (60 pph) | | (A-2-2). Some pedestrians (60 pph) | |
| Conventional | 30.61 (3.17)[a] | 26.83 (2.77)[a] | 31.42 (3.20)[a] | 24.71 (2.59)[a] |
| Offset | 28.59 (2.34)[a] | 24.40 (1.30)[a] | 30.36 (2.18)[a] | 25.30 (2.68)[a] |
| Distant | 30.97 (3.12)[a] | 23.34 (2.61)[a] | 29.33 (2.80)[a] | 24.18 (3.44)[a] |
| | (A-1-3). Many pedestrians (150 pph) | | (A-2-3). Many pedestrians (150 pph) | |
| Conventional | 40.12 (2.76)[a] | 35.29 (3.00)[a] | 43.66 (2.89)[a] | 37.34 (3.23)[a] |
| Offset | 38.76 (3.67)[a] | 33.31 (2.43)[a] | 43.07 (4.05)[a] | 36.60 (2.15)[a] |
| Distant | 41.51 (3.19)[a] | 34.24 (2.79)[a] | 43.76 (3.02)[a] | 38.24 (3.52)[a] |

NOTE: Standard error shown in parentheses.
[a]Delays under PUFFIN and FLC signal controls, which are significantly different at $\alpha = 0.05$ by $t$-test.

signal works, since this combination renders queue lengths shorter than those at other layouts across most scenarios.

There is an interesting observation for the double-lane roundabout: for all scenarios in which there are "some" and "many" pedestrians under "saturated condition," negative pedestrian-induced queue lengths are generated regardless of signals and layouts. The metering effect of pedestrian signal on vehicles could explain this phenomenon. Queue lengths become the shortest when the FLC signal operates at the "offset" layout.

## Number of Stops

The results for single- and double-lane roundabouts disclose similar operational characteristics to pedestrian-induced vehicle delays in basic aspects. It could be interpreted that the "distant" layout is safer across most scenarios, and the addition of pedestrian signals makes vehicles flow more smoothly somewhere under "saturated condition," which reduces the potential for vehicle-to-vehicle collisions.

## Pedestrian Delay

Tables 3–5 list average pedestrian delays. Because PUFFIN timing runs without vehicle green extensions, it is expected that the pedestrian delay is independent of traffic volume change, and this is verified by Tables 3–5. The results suggest that, at specific vehicle intensity, pedestrian delays given denser pedestrian volumes are consistently larger than those given lower pedestrian volumes, because pedestrians in denser volumes are more likely to arrive during the minimum green time. A comparison shows that pedestrian delays from the FLC signal are consistently lower than those from PUFFIN, even though the differences are not statistically significant for low pedestrian volumes. At higher pedestrian flow intensities, pedestrian delay savings from the FLC signal are more obvious, since more pedestrians are affected by "minimum green" constraints. Statistical $t$-tests of the differences confirm that, with relatively high pedestrian volumes, the FLC signal results in significantly lower pedestrian delays than those from PUFFIN. Different crosswalk geometries were expected to produce different pedestrian

**TABLE 4 Pedestrian Delay (seconds): Average of 12 Simulation Replications, Approaching Capacity**

| | Single-Lane Roundabout (1,582 PCEs/h) | | Double-Lane Roundabout (2,649 PCEs/h) | |
|---|---|---|---|---|
| Crosswalk Layout | PUFFIN | FLC | PUFFIN | FLC |
| | (B-1-1). Few pedestrians (12 pph) | | (B-2-1). Few pedestrians (12 pph) | |
| Conventional | 14.29 (1.89) | 13.59 (1.64) | 13.20 (3.59) | 13.90 (1.91) |
| Offset | 13.64 (2.18) | 12.61 (1.71) | 13.65 (2.43) | 12.41 (1.88) |
| Distant | 15.58 (4.75)[a] | 13.70 (1.74)[a] | 15.37 (2.87) | 13.71 (1.54) |
| | (B-1-2). Some pedestrians (60 pph) | | (B-2-2). Some pedestrians (60 pph) | |
| Conventional | 30.61 (3.17)[a] | 25.12 (2.39)[a] | 31.42 (3.20)[a] | 24.73 (2.59)[a] |
| Offset | 28.59 (2.34)[a] | 24.85 (1.34)[a] | 30.36 (2.18)[a] | 25.31 (2.68)[a] |
| Distant | 28.42 (1.55)[a] | 23.28 (2.42)[a] | 29.33 (2.80)[a] | 24.18 (3.44)[a] |
| | (B-1-3). Many pedestrians (150 pph) | | (B-2-3). Many pedestrians (150 pph) | |
| Conventional | 40.12 (2.76)[a] | 35.16 (3.65)[a] | 43.66 (2.89)[a] | 37.13 (3.23)[a] |
| Offset | 38.76 (3.67)[a] | 33.68 (2.56)[a] | 43.07 (4.05)[a] | 36.73 (2.15)[a] |
| Distant | 41.51 (3.19)[a] | 34.24 (2.77)[a] | 43.76 (3.02)[a] | 38.25 (3.52)[a] |

NOTE: Standard error shown in parentheses.
[a]Delays under PUFFIN and FLC signal controls, which are significantly different at $\alpha = 0.05$ by $t$-test.

**TABLE 5 Pedestrian Delay (seconds): Average of 12 Simulation Replications, Saturated Condition**

| | Single-Lane Roundabout (1,992 PCEs/h) | | Double-Lane Roundabout (3,866 PCEs/h) | |
|---|---|---|---|---|
| Crosswalk Layout | PUFFIN | FLC | PUFFIN | FLC |
| | (C-1-1). Few pedestrians (12 pph) | | (C-2-1). Few pedestrians (12 pph) | |
| Conventional | 14.29 (1.89) | 15.06 (2.39) | 15.44 (2.33) | 13.99 (1.86) |
| Offset | 13.64 (2.18) | 13.89 (2.21) | 13.65 (2.43) | 12.41 (1.88) |
| Distant | 15.58 (4.75) | 14.61 (1.87) | 15.37 (2.87) | 13.71 (1.54) |
| | (C-1-2). Some pedestrians (60 pph) | | (C-2-2). Some pedestrians (60 pph) | |
| Conventional | 30.61 (3.17)[a] | 26.10 (2.22)[a] | 31.42 (3.20)[a] | 24.78 (2.53)[a] |
| Offset | 28.59 (2.34)[a] | 24.97 (1.81)[a] | 30.36 (2.18)[a] | 25.39 (2.72)[a] |
| Distant | 28.42 (1.55)[a] | 24.09 (2.39)[a] | 29.33 (2.80)[a] | 24.22 (3.44)[a] |
| | (C-1-3). Many pedestrians (150 pph) | | (C-2-3). Many pedestrians (150 pph) | |
| Conventional | 40.12 (2.76)[a] | 34.90 (3.68)[a] | 43.66 (2.89)[a] | 37.33 (3.23)[a] |
| Offset | 38.76 (3.67)[a] | 33.99 (1.73)[a] | 43.07 (4.05)[a] | 36.58 (2.15)[a] |
| Distant | 41.51 (3.19)[a] | 34.72 (2.89)[a] | 43.76 (3.02)[a] | 37.83 (3.32)[a] |

NOTE: Standard error shown in parentheses.
[a]Delays under PUFFIN and FLC signal controls, which are significantly different at $\alpha = 0.05$ by *t*-test.

crossing times due to varied path deflections, but Tables 3 through 5 do not unveil significant differences among three layouts across most scenarios.

## CONCLUSIONS AND FUTURE DIRECTIONS

This research developed an AI-based roundabout management system, which was quantitatively compared with an existing signal system at varied geometries under different operational conditions. The analysis suggests a nonmonotonic relationship between signalization effects and all levels of vehicle volumes. Pedestrian-induced vehicle delays appear to be the largest as traffic volumes approach the roundabout capacity. Partly due to the vehicle storage space at roundabout exit lanes, the modified crosswalk geometry, "distant" layout, can reduce vehicle delays and queue lengths, especially when the FLC signal works under saturated traffic conditions. An interesting finding is, when vehicle flows are saturated, the addition of pedestrian signals to the double-lane roundabout results in lower vehicle delays than the absence of pedestrian signals, which could be explained by the metering effect of pedestrian signal on vehicles.

The results also reveal that FLC controls the signal timing effectively and outperforms PUFFIN from safety and operational perspectives, especially under saturated traffic conditions. It significantly decreases pedestrian delays and also maintains good vehicle service. Comprehensively, multimodal traveler needs are satisfied through increased pedestrian safety, decreased rear-end hazard, bettered operational efficiency, and diminished social cost; such a compromise is executed by a flexible decision-making mechanism implicitly embedded in the fuzzy logic system. The control algorithm and the parameter setting are straightforward, yet the system performance is adaptive to dynamic roundabout operations. Therefore, the merit in FLC is suitable for resolving complex transportation operation issues.

These findings are important for engineering practitioners faced with the task of improving roundabout accessibilities for pedestrians. The research also adds an impetus to developing AI-based signals for other multimodal transportation facilities. Future direction should include field test, validation, and deployment of FLC-based signal control strategies.

## ACKNOWLEDGMENTS

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**Intelligent Traffic Signal System for Urban Isolated Intersections: Dynamic Pedestrian Accommodation**

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**ABSTRACT:**

One critical issue facing traffic engineers is to optimize signalized intersections to improve multimodal safety and operations. Accommodating pedestrians at urban intersections is challenging as multimodal service demands compete highly on limited green time. Highway Capacity Manual prescribes the parallel vehicle green must exceed “WALK” plus “pedestrian clearance interval (PCI)” timed by a design walking speed. This static “PCI” timing is unsafe since seniors or children are usually slower than the design pedestrian. Furthermore, a vehicle flow issue arises when the prolonged “PCI” exceeds the operationally efficient parallel green: additional vehicle right-of-way, unnecessary for operational efficiency, preempts green time from a conflicting phase, increasing intersection-level queuing delays. It is necessary to achieve a tradeoff among competing multifaceted traveler needs. Fuzzy logic control (FLC) is proved more effective, flexbile, and robust than traditional techniques in handling competing objectives. With “dynamic PCI” concept, this research developed an intelligent signal system which realizes friendly pedestrian accommodation and incorporates FLC into fulfilling multifaceted vehicle needs. In simulation approach, this research quantified the potential benefits from the new system optimized by genetic algorithm, compared with the standard vehicle actuated controller (VAC).

Simulation experiments revealed the current countermeasure, which lowers the design speed for “PCI” timing to strengthen safety, was operationally inefficient. The existing “PCI” timing standard cannot offer safe crossing for all pedestrians, and the VAC omits multifaceted vehicle needs in control logic. The FLC system offers full pedestrian protection via dynamic “PCI” display and fulfills vehicle needs, which outperforms the VAC by realizing a reasonable compromise among competing objectives in controlling an isolated intersection in urban setting.

**Key Words:**

Intersection signal control, pedestrian safety, fuzzy logic, genetic algorithm, traffic simulation

## BACKGROUND

Contemporary transportation engineering community faces increasing challenges in providing the moving public with a safer, efficient, and reliable multimodal transportation system. Today's transportation problems have become increasingly complex as their scopes have been rapidly expanding far beyond the traditional realms, which are characterized by: "a large number of variables are involved; the parametric relationships among them are not well-understood; a large volume of incomplete data is involved, and the goals and constraints are many and the priorities among the stakeholders are unclear (*1*)". Philosophically, "as the complexity of a system increases, our ability to make precise and yet significant statements about its behaviors diminishes, and significance and complexity become almost mutually exclusive characteristics (*2*)", which implies a multitude of transportation problems are very difficult to resolve in traditional approaches. Artificial intelligence (AI) approaches are proved instrumental to modeling the behaviors of complex phenomena while rendering researchers with the latitude to "acknowledge some level of ignorance", which "provides the opportunity to examine the problem from different perspectives and compare the results (*1*)". There have been rising interests in applying AI methodology to addressing the complex issues of safety, operations, and other facets of transportation infrastructure systems (*3*).

Perhaps one of the most critical issues which challenge traffic engineers is to comprehensively optimize the performance of urban signalized intersections where motorized vehicles and non-motorized travelers are busily transported in dynamic operations. The signalization improvement has been recognized as one of the most cost-effective ways of mitigating roadway congestion and ameliorating multimodal transportation safety (*4*). In traffic engineering, the intersection control design pursues dual principal objectives: "(a) Ensure safety for all intersection users; (b) Promote efficient movement of all users through the intersection (*5*)". To achieve the coupled goals is not an easy task, since generally safety and efficiency are mutually competing (or even conflicting), rather than reinforcing or complementary, goals (*5*). Fuzzy logic control (FLC) has been proved more effective, flexbile, and robust than traditional techniques in tackling a complex system in which conflicting objectives, subjective perception, imprecise data, and vague decision-making criteria play critical roles (*1,6*). Hence, the variability and complexity in intersection signalization can be effectively modeled in a FLC-based way for improvement purposes.

The United States safety data indicate that a pedestrian was killed or injured in a traffic crash every 120 or 8 minutes respectively (*7*). Intersection crosswalks are particularly hazardous to seniors and children. In 2008, 35 percent of pedestrian fatalities among people 60 and older occurred at intersections, compared with 20 percent for those younger than 60 (*8*). Alarmed by the tragic casualties, nationwide cities are increasingly seeking for novel strategies to make pedestrian crosswalks safer (*9*). Although a plenty of intersection research endeavours have led to improved motorized traffic movement efficiency at signalized intersections, the impacts of walking speed variability upon intersection safety and operations and relevant problems remains inadequately researched.

## PROBLEM STATEMENT

At a signalized intersection, pedestrians are accommodated by preset phasing scheme and timing design. During a pedestrian phase, the "WALK" interval displays to release waiting pedestrians, then the "Flashing DON'T WALK (FDW)" interval, which functions as the "pedestrian clearance interval (PCI)", lasts for a predetermined duration. Finally, the "Steady DON'T WALK (SDW)" interval follows to prohibit crossing movements. The Highway Capacity Manual (HCM) prescribes the minimum crossing time requirement as follows (*10*):

$$G_p=3.2+L/S_p+2.7(N_{ped}/W_E), \quad \text{for} \quad W_E>10 \text{ feet} \tag{1}$$
$$G_p=3.2+L/S_p+0.27N_{ped}, \quad \text{for} \quad W_E\leq 10 \text{ feet} \tag{2}$$

Where: $G_p$ – Minimum pedestrian crossing time (seconds);

$L$ – Crosswalk length (feet);
$S_p$ – Average pedestrian walking speed (design speed) (feet per second (fps));
$N_{ped}$ – Number of pedestrians crossing per phase in a single crosswalk (pedestrians);
$W_E$ – Crosswalk width (feet).

Equations (1) and (2) allocate 3.2 s as the minimal reaction and start-up time to "WALK", and the last term allocates additional start-up time using pedestrian volumes. Once the "WALK" ends, pedestrians just starting to cross an intersection require the "PCI" for safe clearance. Importantly, the HCM regulates that the parallel vehicle green interval must equal or exceed the "WALK" plus the "PCI" (*10*). The length of "PCI", $L/S_p$, is calculated by a constant walking speed ($S_p$) which is critical in determining how much clearance time is actually given to crossing pedestrians. Histocial pedestrian studies suggested disparate design standards for various populations since the walking speed was found to fluctuate from 1.0 to 8.0 fps (*11,12,13*). Therefore, it is perilous to provide a uniform "PCI" length for a wheelchair user and a young runner. Yet this is how an existing traffic signal system operates at an intersection: the Manual on Uniform Traffic Control Devices (MUTCD) designates 3.5 fps as $S_p$ for "PCI" timing (*14*). The question of "What is the most appropriate $S_p$?" has kindled a nationwide debate, while the countermeasure has been implemented in terms of shortening $S_p$ to lengthen the "PCI" duration. MUTCD Section 4E.06 states "*Where … pedestrians who use wheelchairs, routinely use the crosswalk, a walking speed of less than 3.5 fps should be considered in determining the pedestrian clearance time* (*14*)", while no specific value is stipulated. It is plausible that this countermeasure can be effective to offer adequate crossing safety.

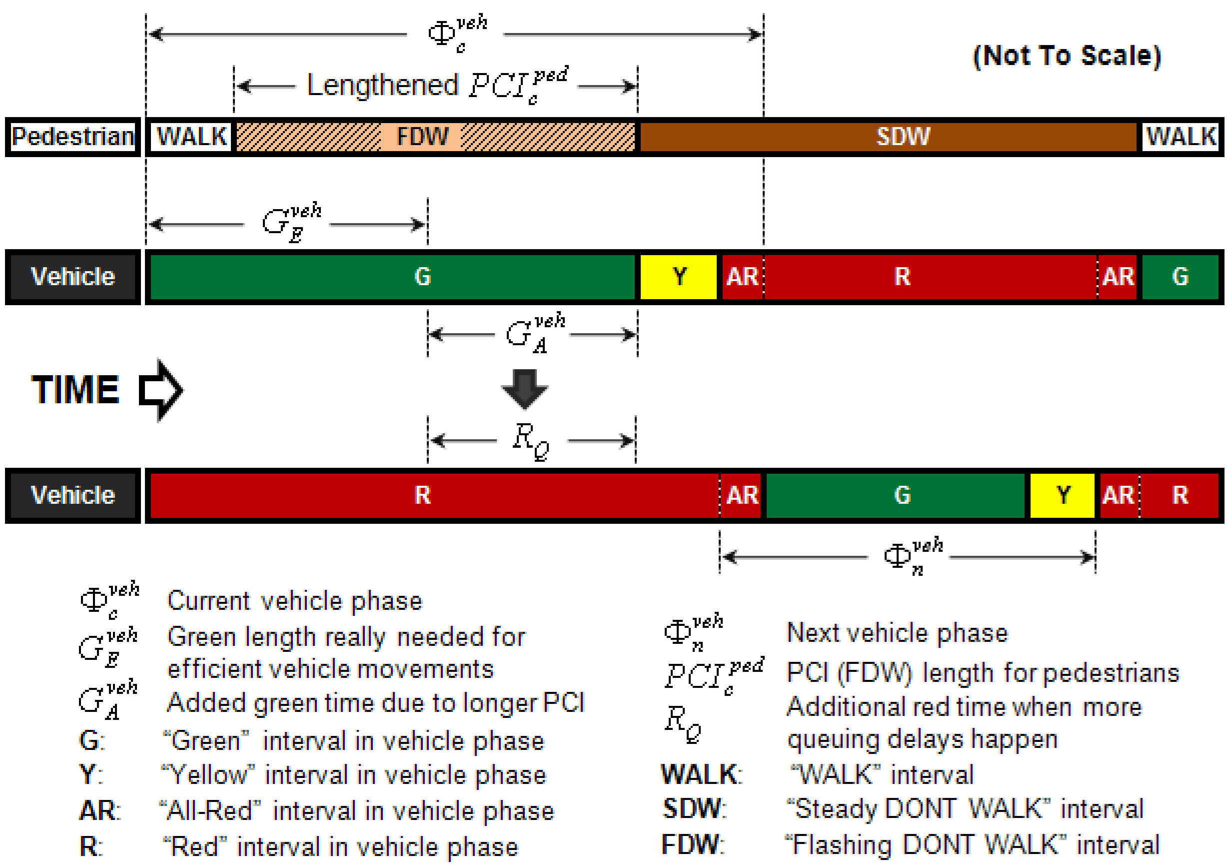



**FIGURE 1 Illustration of the crossing safety versus traffic efficiency problem.**

However, another problem arises. When the lengthened $PCI_c^{ped}$ exceeds the parallel green ($G_E^{veh}$) which is really required for efficient vehicle movements, the additional green time ($G_A^{veh}$) resulting from the prolongation is operationally surplus (FIGURE 1). The longer vehicle right-of-way produced by $G_A^{veh}$ preempts a certain amount of green time from the next phase $\Phi_n^{veh}$ and incurs $R_Q$ during which increased queuing delays are inflicted upon the *whole* intersection. Therefore, to provide a longer but *static* "PCI" for variable walking needs can pose a detriment to vehicle flow efficiency, leaving the intersection operations systematically sub-optimal. This inefficiency problem will enormously deteriorate if a tiny $S_p$ is used: once expeditious pedestrians vacate the crosswalks, the residual "PCI" time is purely idle and $G_A^{veh}$ is underused; simultaneously, queuing vehicles held by $R_Q$ are desirous of the right-of-way. Hence, it is reasonable to attain a compromise among the competing objectives which are pertinent to operational efficiency and safety needs for multimodal intersection users.

## RESEARCH OBJECTIVES

The problem aforementioned is inherently rooted in the difficulty of incorporating an instantaneously changeable $S_p$ into the "PCI" timing. Automated pedestrian detectors can provide significant operational and safety benefits when installed in conjunction with pedestrian pushbuttons at actuated traffic signals (*15*). Previous studies have evaluated a spectrum of potential pedestrian detection technologies including microwave (*15,17)*, active-/passive-infrared (*15,16*,*17*), video-imaging, ultrasonic, and piezoelectric sensors. In Europe, PUFFIN (pedestrian user-friendly intelligent) and PUSSYCAT (pedestrian urban safety system and comfort at traffic) utilize microwave and passive-infrared sensors to detect the presence of pedestrians on crosswalks (*18,19*). This research hypothesizes that similar sensors can capture the pedestrian presence to meet data input needs in intersection control, then the "PCI" duration can be *dynamic* to reflect the crossing time in instantaneous needs, mitigating the intersection-level vehicle flow inefficiency. Inspired by the methodological transition toward AI domain, this research developed a traffic signal system prototype which not only realizes a dynamic accommodation for crossing pedestrians but also incorporates FLC into fulfilling multifaceted needs for vehicles. This research also quantified the potential benefits from the new system which was optimized by genetic algorithm (GA), compared with a standard signal control widely deployed.

## INTELLIGENT SIGNAL SYSTEM

Applications of fuzzy sets and fuzzy logic to traffic signal controls origninated in 1970s. The first known attempt was made by Pappis and Mamdani (*20*) who conducted a simulation study of a fuzzy logic controller at an one-way signalized intersection. Chang and Shyu (*21*) produced a fuzzy expert system to evaluate whether a signal is required for an intersection. Kim (*22*) studied the fuzzy algorithms of isolated intersections and discussed the turning traffic problem. Niittymaki and Kikuchi (*23*) developed a fuzzy logic controller for a pedestrian crosswalk and the controller provided a pedestrian friendly control while keeping vehicle delays smaller than conventional controls. Trabia et al. (*24*) presented a fuzzy-logic-based adaptive signal controller for an intersection with conflicting movements. The controller produced fewer vehicle delays than the traffic-actuated controller. Murat and Gedizlioglu (*25*) developed a FLC-based signal model for isolated intersections and compared it with the traffic-actuated simulation and other specific vehicle-actuated models. They found the fuzzy model operationally outperformed previous models. Lu and Noyce (*26*) and Lu et al. (*27*) developed fuzzy signal systems to manage travelers at mid-block crosswalks and modern roundabouts. Their simulation experiments found the FLC-based signal timing effective and it outperformed a common system in manifold aspects. However, no previous research has holistically integrated a dynamic pedestrian flow control into a FLC-based intersection signal system to improve multimodal safety and operations, which motivated this research endeavour.

**Phasing Scheme**

To signalize an intersection means to determine the reasonable time for switching the right-of-way among conflicting traffic movements. In principle, traffic signal control is a decision-making process of determining, at time intervals (*Δt*), whether extend or terminate the current vehicle green, while guaranteeing safe pedestrian accommodation. In this research, the FLC implements a complex decision-making process for determining green termination. From a systems engineering perspective, if the logic in a phasing scheme is overly sophisticated, its interaction with the FLC process could be intractable and incur potential impairments to systematic optimization. In this philosophy, a sensible *balance* should be maintained between a simplification in phasing scheme and the complexity in decision-making process. FIGURE 2.*a* depicts a 4-phase scheme designed to offer the flexibility of skipping left-Turn (LT) phases if LT vehicles are absent. Usually right-turn-on-red is permitted for intersection signalization in North America. Green starts with (i) an exclusive LT phase (Φ1 or Φ3) followed by a Through (TH) phase (Φ2 or Φ4) or (ii) a TH phase (Φ2 or Φ4) followed by a LT phase (Φ3 or Φ1) or a TH phase (Φ4 or Φ2).

**Operational States**

The green control structure for most traffic signal systems is universial – minimum and maximum greens delimit an in-between period in which the control logic plays the role in "extension or termination" decision-making by examining the presences of specific operational states. At time steps, the FLC-based signal system identifies a specific operational state at time point (*T*+*Δt*) to navigate its logic flow. All operational states are defined here in terms of certain time-step-based variables for vehicle green display and constant parameters for signal timing limits, as follows.

(a). "Φ2 or Φ4 Min-Over" state occurs, when $t_{\Phi 2}^{G}(T+\Delta t) \geq G_{\Phi 2}^{Min}$ or $t_{\Phi 4}^{G}(T+\Delta t) \geq G_{\Phi 4}^{Min}$

(b). "Φ2 or Φ4 Max-Out" state occurs, when $t_{\Phi 2}^{G}(T+\Delta t) \geq G_{\Phi 2}^{Max}$ or $t_{\Phi 4}^{G}(T+\Delta t) \geq G_{\Phi 4}^{Max}$

(c). "Φ1 or Φ3 Green-Over" state occurs, when $t_{\Phi 1}^{G}(T+\Delta t) \geq G_{\Phi 1}$ or $t_{\Phi 3}^{G}(T+\Delta t) \geq G_{\Phi 3}$

(d). "Φ2 or Φ4 PCI Max-Out" state occurs, when $t_{\Phi 2}^{G}(T+\Delta t) \geq W_{\Phi 2} + PCI_{\Phi 2}^{Max}$ or $t_{\Phi 4}^{G}(T+\Delta t) \geq W_{\Phi 4} + PCI_{\Phi 4}^{Max}$

Where: $t_{\Phi 2}^{G}(T+\Delta t), t_{\Phi 4}^{G}(T+\Delta t)$: Green length already displayed for Φ2, Φ4 at (*T*+*Δt*);

$G_{\Phi 2}^{Min}, G_{\Phi 4}^{Min}$: Minimum timing limits for Φ2, Φ4 vehicle green display;

$G_{\Phi 2}^{Max}, G_{\Phi 4}^{Max}$: Maximum timing limits for Φ2, Φ4 vehicle green display;

$W_{\Phi 2}, W_{\Phi 4}$: "WALK" interval length in parallel with Φ2, Φ4 ($W_{\Phi 2} < G_{\Phi 2}^{Min}, W_{\Phi 4} < G_{\Phi 4}^{Min}$);

$t_{\Phi 1}^{G}(T+\Delta t), t_{\Phi 3}^{G}(T+\Delta t)$: Vehicle green length already displayed for Φ1, Φ3 at (*T*+*Δt*);

$G_{\Phi 1}, G_{\Phi 3}$: Vehicle green length preset for Φ1, Φ3;

$PCI_{\Phi 2}^{Max}, PCI_{\Phi 4}^{Max}$: Maximum timing limit for the "PCI" interval in parallel with Φ2, Φ4, and $G_{\Phi 2}^{Max} < PCI_{\Phi 2}^{Max} + W_{\Phi 2}$, $G_{\Phi 4}^{Max} < PCI_{\Phi 4}^{Max} + W_{\Phi 4}$.

**FLC Variables**

With the time step (*Δt*) incrementally proceeding, a FLC process operates to evaluate ongoing intersection operations through fuzzifying input variables below for the inference engine and the defuzzifier which make the decision, via output variables, about the control action on current vehicle green in Φ2 or Φ4.

*Input Variables*

(i). $x^{c}_{\Phi 2}(T+\Delta t), x^{c}_{\Phi 4}(T+\Delta t)$:

**Traffic intensity level** (vehicles/lane) which upcomes on TH lanes of the current phase (Φ2, Φ4) in the last time step (*Δt*).

The variables denote the average number of vehicles between paired detectors for TH lanes, which reflect the vehicle flow intensity in operations. They address the issue of operational efficiency in dissipating vehicles at an intersection. It should be reasonable to assume that the more intensely vehicle flows arrive the more desirously they demand the green signal display. Each variable has "Sparse", "Moderate", and "Intense" fuzzy sets.

(ii). $y^{c}_{\Phi 2}(T+\Delta t), y^{c}_{\Phi 4}(T+\Delta t)$:

**Vehicle discharge headway** (vehicles/second/lane) which ongoes on TH lanes of the current phase (Φ2, Φ4) in the last time step (*Δt*).

The variables represent the time length between adjacent vehicles being dissipated across the STOP line, which embody a safety-related factor in discharging vehicles. The larger value for opposite approaches was used. It is believed that the smaller the headway is the more probably vehicles are packed; the larger the headway is, the more probably a platoon is proceeding (*23*). Each variable has "Short" and "Long" fuzzy sets.

(iii). $z^{n}_{\Phi 3}(T+\Delta t), z^{n}_{\Phi 4}(T+\Delta t); z^{n}_{\Phi 1}(T+\Delta t), z^{n}_{\Phi 2}(T+\Delta t)$:

**Average queue length** (vehicles/lane) which accumulates for the next phase (Φ3, Φ4; Φ1, Φ2) in the last time step (*Δt*).

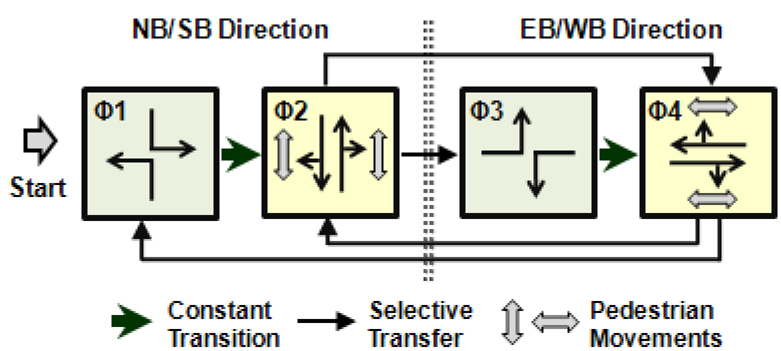
The variables quantify how many vehicles have been queuing in lane(s), which encompass an operational and safety-related element in serving vehicles. It should be inferable that the longer a vehicle has been waiting or queuing the more prone the driver is to commit signal incompliance due to aggravating impatience. Each variable has "So-so", "Tolerable", and "Frustrating" fuzzy sets.

*Output Variable*

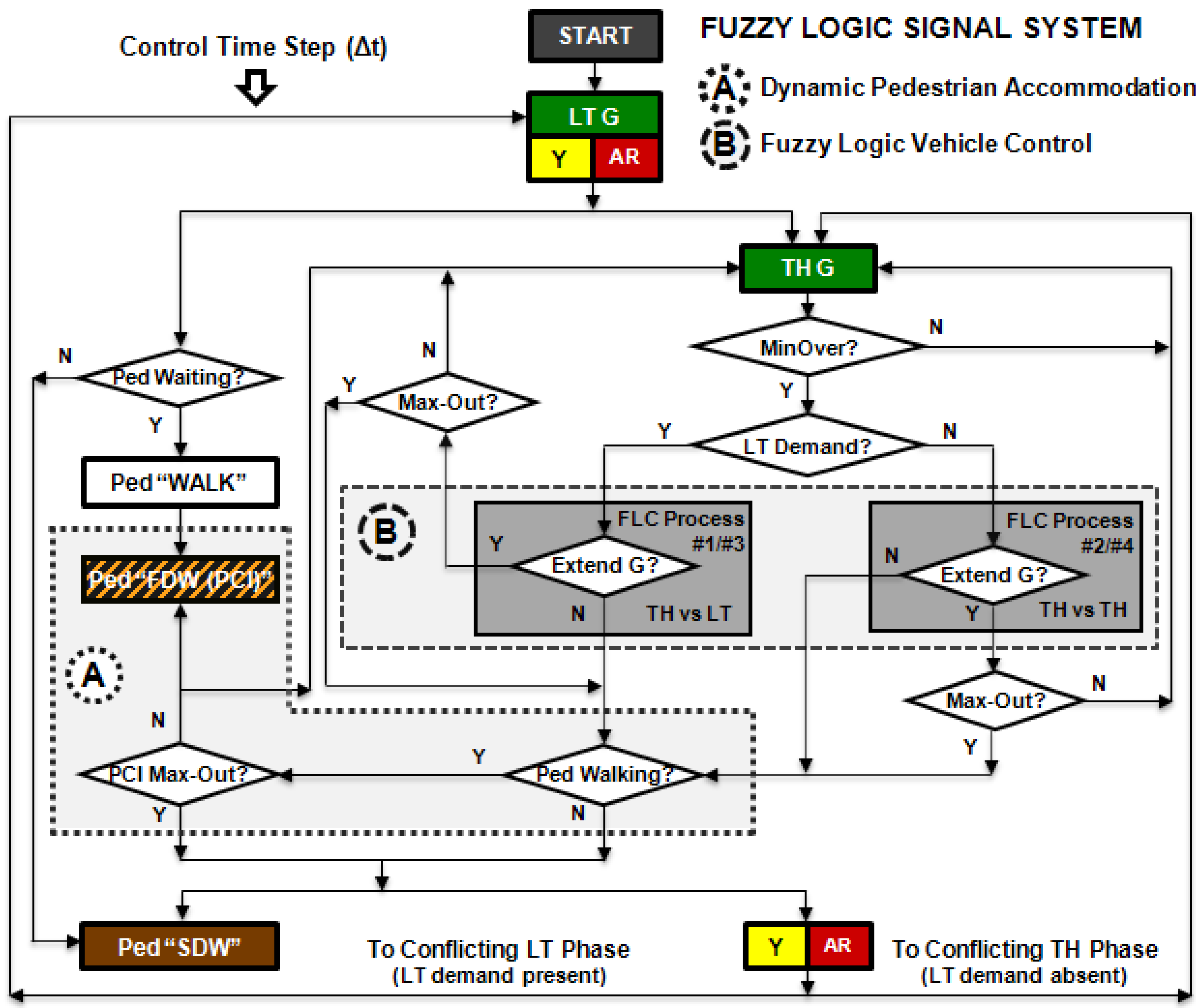
$w_{\Phi 2}(T+\Delta t)$ and $w_{\Phi 4}(T+\Delta t)$ represent the control action taken by the FLC process on Φ2, Φ4 at time point (*T*+*Δt*): "Termination", and "Extension".

**Control Logic**

The signal system has four FLC processes: Process #1 (Φ2 vs. Φ3), Process #2 (Φ2 vs. Φ4), Process #3 (Φ4 vs. Φ1), and Process #4 (Φ4 vs. Φ2). At time steps, one of four FLC processes implemetnes the decision-making for "Extension" or "Termination". FIGURE 2.*b* shows how the system operates.



(a)




(b)

**FIGURE 2 FLC signal system: *(a)* phasing scheme diagram and *(b)* control logic flowchart.**

*Constant "Φ1 to Φ2" Transition*

To start with Φ1. At time point ($T+\Delta t$) when "Φ1 Green-Over" state occurs, Φ2 starts.

*Selective Φ2 Control Process*

If pedestrians exist, "WALK" starts and $G^{Min}_{\Phi2}$ is satisfied to dissipate queuing vehicles. At time point ($T+\Delta t$) when the "Φ2 Min-Over" state occurs, the system activates one FLC process via identifying whether CASE A or CASE B is true.

**CASE A: "Present Φ3 Demand"** In this case, the system activates Process #1 which operates with $x^{c}_{\Phi2}(T+\Delta t)$, $y^{c}_{\Phi2}(T+\Delta t)$ and $z^{n}_{\Phi3}(T+\Delta t)$. At the end of $\Delta t$, Process #1 takes the action on the current green via $w_{\Phi2}(T+\Delta t)$:

(i). If the current green extends, the parallel "PCI" persists concurrently. When green extensions continue until the "Φ2 Max-out" state occurs, the vehicle-based FLC process stops and the system identifies whether Condition I or Condition II is true.

(ii). If the current green ends, the system identifies whether Condition I or Condition II is true.

**CASE B: "Absent Φ3 Demand"** In this case, the system activates Process #2 which operates with $x^c_{\Phi 2}(T+\Delta t)$, $y^c_{\Phi 2}(T+\Delta t)$ and $z^n_{\Phi 4}(T+\Delta t)$. At the end of *Δt*, Process #2 takes the action on the current green:

(i). If the current green extends, the parallel "PCI" continues equally. When the "Φ2 Max-out" state occurs, the fuzzy process stops and the system identifies the trueness in Condition I or Condition II.

(ii). If the current green terminates, the system identifies Condition I or Condition II.

**Condition I: "Vacant Crosswalks"** If sensors discover pedestrians vacate crosswalks, $w_{\Phi 2}(T+\Delta t)$ takes "Termination" and next appropriate phase is activated: for CASE A, Φ3 starts; for CASE B, Φ4 starts.

**Condition II: "Occupied Crosswalks"** If sensors recognize pedestrians occupy crosswalks, the "PCI" and the parallel vehicle green persist together. Hence, slower pedestrians are *fully* protected by the "PCI". When all pedestrians disappear from crosswalks, the system terminates Φ2 to activate the next phase: for CASE A, Φ3 starts; for CASE B, Φ4 starts. When "PCI" extensions continue until the "Φ2 PCI Max-Out" state occurs, Φ2 unconditionally ceases and the next appropriate phase starts.

To consummate a cycle, the logic in Φ4 reiterates in Φ2, after the "Φ3 to Φ4 transition" (if applicable) finishes. In summary, the input variables are involved in a FLC process as follows:

Process #1 – $x^c_{\Phi 2}(T+\Delta t)$, $y^c_{\Phi 2}(T+\Delta t)$, $z^n_{\Phi 3}(T+\Delta t)$;
Process #2 – $x^c_{\Phi 2}(T+\Delta t)$, $y^c_{\Phi 2}(T+\Delta t)$, $z^n_{\Phi 4}(T+\Delta t)$;
Process #3 – $x^c_{\Phi 4}(T+\Delta t)$, $y^c_{\Phi 4}(T+\Delta t)$, $z^n_{\Phi 1}(T+\Delta t)$;
Process #4 – $x^c_{\Phi 4}(T+\Delta t)$, $y^c_{\Phi 4}(T+\Delta t)$, $z^n_{\Phi 2}(T+\Delta t)$.

**FLC Configuration**

The fuzzifier includes membership functions which transform input variables into fuzzy values processable for the inference engine. Trapezoid membership function was utilized to simplify the problem, which is mathematically defined in terms of max{min[(*x-s*)/(*t-s*),*1*,(*v-x*)/(*v-u*),*0*]} and four breakpoints (*s,t,u,v*) (*2*).

As an "intelligent brain", the inference engine contains "IF…AND…THEN…" rules which linguistically describe operational conditions for current and next phases. TABLE 1 Section A shows the generic format of a rule base. Respectively, the statements after "IF" and "THEN" are termed "*premise*" and "*consequence*". "AND" is termed "*operator*", and all operators interconnect premises to establish a rule base. The inference engine reaches a conclusion through identifying the similarity between an input (*a*,*b*,*c*) and some premises ($A_1,B_1,C_1;\ldots;\ A_i,B_i,C_i;\ldots;\ A_n,B_n,C_n$). An input can trigger multiple rules because the input and the premises in triggered rules are represented by fuzzy sets and fuzzy relationships. Hence, all consequences from different rules are strictly valid and then they are aggregated for an output space consisting of fuzzy control actions. To be defuzzified for a final decision, the output space is a compromise among these conclusions from all triggerred rules. Essentially, all inference rules are indirectly associated with pursuing operational and safety goals for intersection users: (i) Pedestrians are accommodated safely and timely; (ii) Vehicles are served timely to avoid signal noncompliances, and a platoon is dissipated entirely to prevent rear-end collisions (*23*). TABLE 1 Section B exhibits the rule base for the FLC Process #1. Synoptically, this system was desgined to manage the intersection to satisfy safety and operational needs for multimodal travelers. FLC can be flexible, robust, and adaptive in tackling dynamic intersection operations since membership functions implicitly span a vast variety of possibilities. Mamdani method was used for the aggregation process, which was based on Zadeh's work on fuzzy algorithms for complex systems and decision processes (*28,29*). This method was among the first control systems built using fuzzy set theory, which was proposed as an effort to control a steam engine and boiler by synthesizing linguistic rules from experienced human operators.

The defuzzifier transforms the output space into a final decision. Several techniques have been developed to produce a final crisp output. Traditional methods include Maximum Criterion, Mean of Maximum, and Center of Gravity each of which is effective for distinct problems (*30*). Because of the "binary" characteristic in traffic signal control, Maximum Criterion method is the reasonable choice.

**TABLE 1 Fuzzy Logic Inference Engine for the Intersection FLC Signal System**

**Section A: Generic Format of Fuzzy Logic Rules**

| Fuzzy Rules | Premises (Crisp Inputs: $X = a$, $Y = b$, $Z = c$) | Consequences |
|---|---|---|
| Rule 1: | IF {$x$ is "$A_1$" } AND {$y$ is "$B_1$"} AND {$z$ is "$C_1$"} | THEN {"E[1]" or "T[2]"} |
| | … … … … … … | |
| Rule i: | IF {$x$ is "$A_i$"} AND {$y$ is "$B_i$"} AND {$z$ is "$C_i$"} | THEN {"E" or "T"} |
| | … … … … … … | |
| Rule n: | IF {$x$ is "$A_n$"} AND {$y$ is "$B_n$"} AND {$z$ is "$C_n$"} | THEN {"E" or "T"} |
| Crisp Output: | {"E" or "T"} | |
| Where: | $x$, $y$, $z$ = Input (state) variables related to traffic conditions; | |
| | $a$, $b$, $c$ = Values of input variables; | |
| | $A_i$, $B_i$, $C_i$ = Natural language expressions (fuzzy sets) for traffic conditions, i=1, …, n. | |

**Section B: Inference Rule Base in the FLC Process #1 (Φ2 vs. Φ3)**

| Discharge Headway {$y^c_{\Phi 2}(T+\Delta t)$} | | Approach Flow Level {$x^c_{\Phi 2}(T+\Delta t)$} | | |
|---|---|---|---|---|
| "Short" | | "Sparse" | "Moderate" | "Intense" |
| Queue Length {$z^n_{\Phi 3}(T+\Delta t)$} | "So-so" | T | E | E |
| | "Tolerable" | T | T | E |
| | "Frustrating" | T | T | T |
| **Discharge Headway {$y^c_{\Phi 2}(T+\Delta t)$}** | | **Approach Flow Level {$x^c_{\Phi 2}(T+\Delta t)$}** | | |
| "Long" | | "Sparse" | "Moderate" | "Intense" |
| Queue Length {$z^n_{\Phi 3}(T+\Delta t)$} | "So-so" | T | E | E |
| | "Tolerable" | T | T | E |
| | "Frustrating" | T | T | T |

NOTE: [1] Terminate current vehicle green; [2] Extend current vehicle green.

## GA Optimization

The methods for establishing inference engines and defuzzifiers are normative in FLC applications, while those for formulating databases supported by membership functions are mostly subjective and prone to incur biases. GA is powerful in resolving combinatorial optimization problems (*31*). A GA optimizer was developed in C++ to tune forty parameters of membership functions in four FLC processes. FIGURE 3 depicts the optimization framework for the FLC signal system.

### *Algorithmic Logic*

GA search is underpinned by the principle of evolution and survival of the fittest, which borrows the terminology from natural genetics (*32*). The optimization procedure is iterative by "generations". In each generation, the procedure maintains a population of *chromosomes* each of which links genes in linear succession. Each chromosome represents a candidate solution. Initially, the procedure starts with a randomly generated population. Each solution is evaluated quantitatively for its "fitness for survival". Then, a new population is formed by selecting "more fit" solutions, in which some solutions undergo alterations through crossover and mutation operators to yield new solutions. The process of evaluation, selection, and alteration iterates for generations and "converges" to a near-optimum solution (*33*).

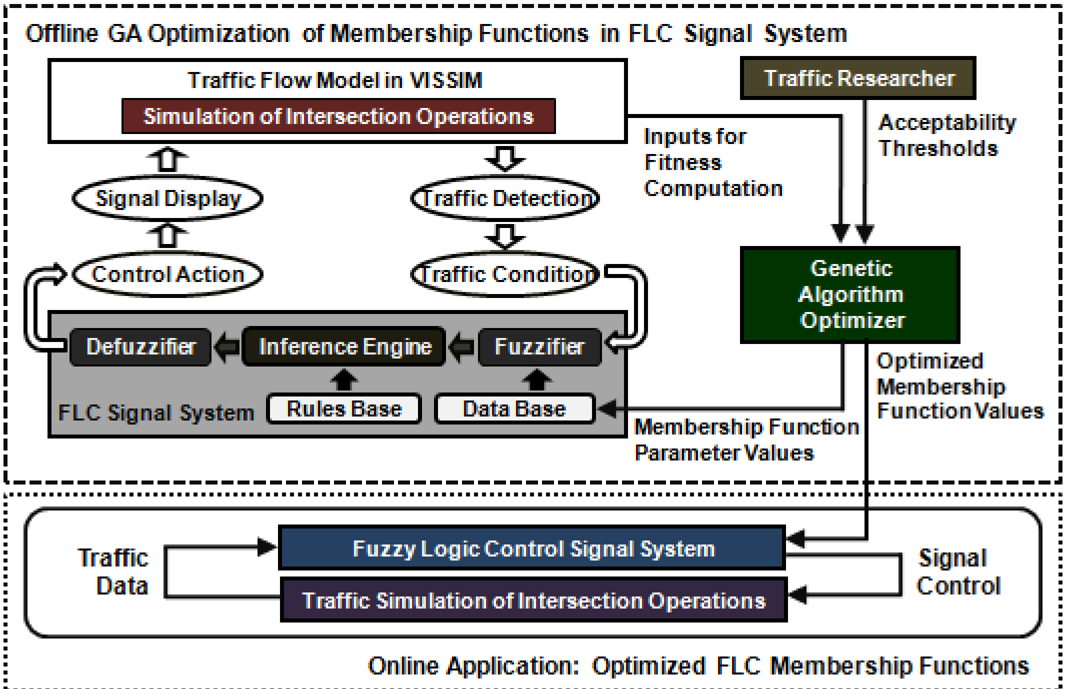



**FIGURE 3 Optimization framework for the FLC signal system.**

*Implementation Design*
GA needs a genetic solution representation, an initial population, a "fitness" evaluation function, a "more fit" selection procedure, the genetic operators which alter the composition of solutions, and the control parameters regarding the population size and the probabilities of adopting genetic operators.

**Representation Scheme** Given the focus on the FLC system, a generic scheme was utilized to transform a solution into a binary string. Each individual parameter bounded between the minimum and maximum values is treated as a gene whose binary string was fixed at 6 in length. In this case, an encoding procedure transformed the decimal values of parameters into integers which are represented by binary strings, as Equation (3) shows (*34*). To convert the binary strings back to their decimal values, the decoding schema was employed as shown in Equation (4):

$$X_i = X_i^{Min} + d_i\left(X_i^{Max} - X_i^{Min}\right)/\left(2^L - 1\right) \quad (3)$$
$$d_i = \left[\left(2^L - 1\right)\left(X_i - X_i^{Min}\right)\right]/\left(X_i^{Max} - X_i^{Min}\right) \quad (4)$$

Where: $X_i$ – The transformed value of the *i*-th parameter, *i* = 1, 2, …, 40;
$X_i^{Min}$ – Minimum value as the lower bound for the *i*-th parameter;
$X_i^{Max}$ – Maximum value as the upper bound for the *i*-th parameter;
$L$ – The length of the binary string for a candidate solution (i.e., chromosome);
$d_i$ – The decimal value of the parameter.

**Initial Population** The initial population size was set to 30 candidate solutions each of which included 40 parametes to be optimized. The initial values of 40 parameters was randomly determined within their corresponding bounds.

**Evaluation and Selection** The solution evaluation included running the new signal system with forty parameters encoded into a binary string and determining the fitness value in terms of the inverse of the overall Measure-of-Effectiveness (MOE) defined as the weighted average of three MOEs: 0.30×(average delay per pedestrian)+0.30×(average total delay per vehicle)+0.40×(average number of stops per vehicle). Roulette wheel procedure selected the "more fit" solutions from each population. The probability of being selected was directly proportionate to the fitness values, then two solutions were randomly chosen using these probabilities to create new solutions in the next generation.

**Genetic Operators** Mutation operator randomly selected a gene and replaced it with a random number selected between that gene's bounds. Such genes were within their new permissible ranges. If any gene was not, a new random number (selected from the new bounds) replaced it. We just randomly chosen one digit of each parameter and mutated 1 to 0 and 0 to 1. Crossover operator combined the features of two parent solutions to form two new solutions by switching corresponding segments of the parents.

**Control Parameters** Identifying appropriate values for control parameters is more an art than a science (*33*). These values were determined after initial experiments: population size, 30; crossover probability, 0.6; mutation probability, 0.05. The number of generations was limited to 60 due to high computational requirements.

## SYSTEMS EVALUATION

Practical limitations make it difficult to evaluate the new system easily in real-world context, so an in-lab platform should be invited as a surrogate way in which the system can be implemented precisely and evaluated quantifiably. Today traffic microsimulation is essential in transportation research due to its cost-effectiveness, unobtrusiveness, risk-free nature, and high-speed computation. VISSIM, a simulation tool, is widely applied due to its powerful traveler-modeling capability, accurate multimodal detection, flexible control logic via vehicle-actuated programming, seamless interface for object-oriented programming, and processable input/output files (*35*).

### Comparison Strategy

In VISSIM environment, the new system was compared with the standard "Dual-Ring 8-Phase" vehicle-actuated controller (VAC) vastly deployed and conventionally cited as "NEMA (National Electrical Manufacturers Association) control"(*5*). Approximately 15% of pedestrian population walk more slowly at a speed less than 3.5 fps (*36*). Therefore, the mean walking speed was conservatively set to 3.0 fps. To reflect previous findings, a researcher-customized speed distribution was modeled with maximum and minimum walking speeds set to 8.0 fps and 1.0 fps. The new dynamic "PCI" function provides *full* signal protection for *all* pedestrians some of whom walk at the lowest speed ($S_p$=1.0 fps). To maintain a uniform degree of signal protection in one comparison case, the static "PCI" in the VAC was timed using 1.0 fps as $S_p$ to guarantee adequate "PCI" duration – this small $S_p$ follows the philosophy to which the current countermeasure aforementioned resorts. The MUTCD "PCI" timing standard makes it necessary to implement another comparison case which adopts 3.5 fps as $S_p$ (*14*). For both VAC-based cases, $G_{min}$ rises to $G_{MIN}$=PCI+WALK when pedestrians arrive and the in-between period ($G_{max}$–$G_{min}$) determines the new maximum green ($G_{MAX}$) (FIGURE 4).

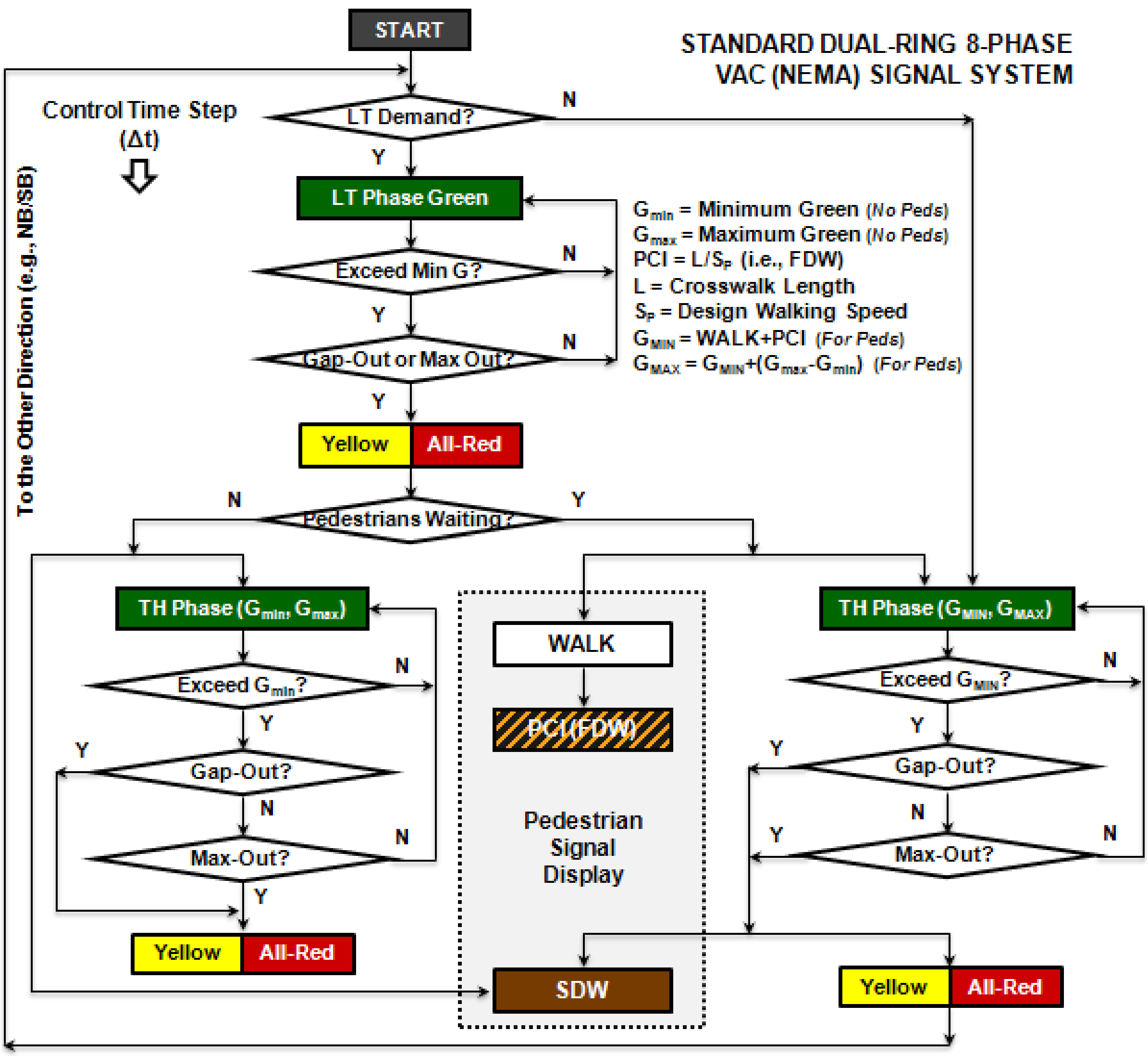



**FIGURE 4 Control logic flowchart of standard dual-ring 8-phase VAC (NEMA) system.**

### Test Intersection

This research utilized Federal Highway Administration's Next Generation Simulation (NGSIM) project data and summary report for an urban intersection under VAC control (*37*). This intersection has four typical multi-lane approaches which receive TH, RT, and LT vehicle movements. A pedestrian crosswalk lies downstream each of four STOP lines. FIGURE 5 geometrically delineates the test intersection and the detector deployments for the FLC signal system.

### Study Scenarios

Two vehicle flow levels of mixed traffic composition were examined: "Existing Flows" and "High Demand". The observed traffic volumes at the test intersection represented "Existing Flows" condition. To explore additional scenarios, the observed volumes were augmented at a fixed rate (40%) to create "High Demand" condition which approaches the maximum intersection capacity. Two pedestrian flow levels were investigated: "Moderate" – 50 pedestrians per hour per 2-way crosswalk (pphpc); "Crowded" – 150 pphpc. Hence, four operational conditions were modeled and combined with three comparion cases to yield twelve study scenarios. The analysis period spanned one hour.

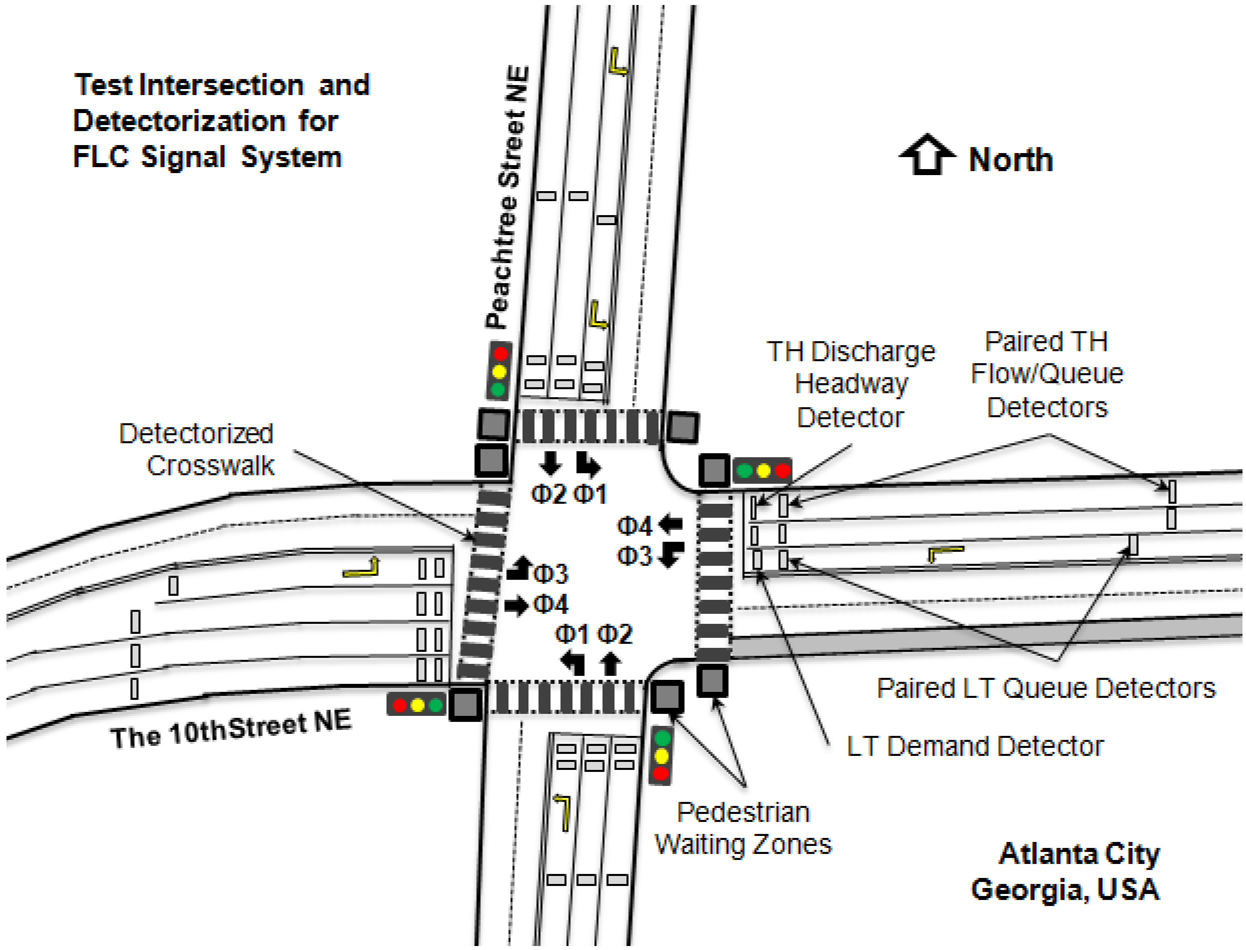



**FIGURE 5 Test Intersection Illustration and Approach Detectorization.**

**Timing Settings**

Most NGSIM's signal timing data were applied to the VAC control in "Existing Flows" condition. For "High Demand" condition, "Existing Flows" VAC green timings were proportionally enlarged to meet enhanced traffic demands. For the FLC system, its TH green timings maintained consistent with the VAC's counterparts, while its LT greens equaled the values higher than the averages of VAC's LT greens. TABLE 2 exhibits the timing settings for three comparison cases.

**TABLE 2 Basic Signal Timing Settings for Three Comparison Cases**

**SECTION 1: Standard Vehicle-Actuated Contoller (Dual-Ring & 8-Phase NEMA Signal Control System)**

| Basic Signal Timing (s) (*Static* FDW (PCI)) | | "Existing Flows" Traffic Condition | | | | | | | | "High Demand" Traffic Condition | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | LT Phase | | | | TH (RT) Phase | | | | LT Phase | | | | TH (RT) Phase | | | |
| | | I | III | V | VII | II | IV | VI | VIII | I | III | V | VII | II | IV | VI | VIII |
| Green Interval (No Pedestrians) | $G_{min}$ | 5.0 | 8.0 | 8.0 | 5.0 | 15.0 | 12.0 | 15.0 | 16.0 | 6.0 | 10.0 | 10.0 | 6.0 | 19.0 | 15.0 | 19.0 | 20.0 |
| | $G_{max}$ | 10.0 | 15.0 | 10.0 | 15.0 | 45.0 | 30.0 | 45.0 | 30.0 | 13.0 | 19.0 | 13.0 | 19.0 | 57.0 | 38.0 | 57.0 | 38.0 |
| Green Interval (With Pedestrians) | ${}^{a}G_{MIN}$ | / | / | / | / | 61.0 | 74.0 | 61.0 | 74.0 | / | / | / | / | 61.0 | 74.0 | 61.0 | 74.0 |
| | ${}^{a}G_{MAX}$ | / | / | / | / | 91.0 | 92.0 | 91.0 | 88.0 | / | / | / | / | 99.0 | 97.0 | 99.0 | 92.0 |
| | ${}^{b}G_{MIN}$ | / | / | / | / | 23.0 | 26.0 | 23.0 | 26.0 | / | / | / | / | 23.0 | 26.0 | 23.0 | 26.0 |
| | ${}^{b}G_{MAX}$ | / | / | / | / | 53.0 | 44.0 | 53.0 | 40.0 | / | / | / | / | 61.0 | 49.0 | 61.0 | 44.0 |
| Intergreen Interval | Yellow | 4.0 | 4.0 | 4.0 | 4.0 | 4.0 | 4.0 | 4.0 | 4.0 | 4.0 | 4.0 | 4.0 | 4.0 | 4.0 | 4.0 | 4.0 | 4.0 |
| | All-Red | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| Pedestrian Interval | WALK | / | / | / | / | 7.0 | 7.0 | 7.0 | 7.0 | / | / | / | / | 7.0 | 7.0 | 7.0 | 7.0 |
| | ${}^{a}$PCI | / | / | / | / | 54.0 | 67.0 | 54.0 | 67.0 | / | / | / | / | 54.0 | 67.0 | 54.0 | 67.0 |
| | ${}^{b}$PCI | / | / | / | / | 16.0 | 19.0 | 16.0 | 19.0 | / | / | / | / | 16.0 | 19.0 | 16.0 | 19.0 |

**SECTION 2: Fuzzy Logic Control Signal System**

| Basic Signal Timing (s) (*Dynamic* FDW (PCI)) | | "Existing Flows" Traffic Condition | | | | "High Demand" Traffic Condition | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | LT Phase | | TH (RT) Phase | | LT Phase | | TH (RT) Phase | |
| | | Φ1 | Φ3 | Φ2 | Φ4 | Φ1 | Φ3 | Φ2 | Φ4 |
| Green Interval | $G_{LT}$ | 13.0 | 10.0 | / | / | 16.0 | 13.0 | / | / |
| | Min $G_{TH}$ | / | / | 12.0 | 15.0 | / | / | 15.0 | 19.0 |
| | Max $G_{TH}$ | / | / | 30.0 | 45.0 | / | / | 38.0 | 57.0 |
| Intergreen Interval | Yellow | 4.0 | 4.0 | 4.0 | 4.0 | 4.0 | 4.0 | 4.0 | 4.0 |
| | All-Red | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| Pedestrian Interval | WALK | / | / | 7.0 | 7.0 | / | / | 7.0 | 7.0 |

NOTE: ${}^{a}$ When design walking speed $S_p$ = 1.0 fps for the PCI (FDW) timing, ${}^{b}$ When design walking speed $S_p$ = 3.5 fps for the PCI (FDW) timing

VAC System: Phase I & VI – WB (westbound), Phase III & VIII – NB (northbound), Phase II & V – EB (eastbound), Phase IV & VII – SB (southbound)

$G_{max}$&$G_{MAX}$ – Maximum green length, $G_{min}$&$G_{MIN}$ – Minimum green length, $G_{MIN}$=WALK+PCI, $G_{MAX}=G_{MIN}+(G_{max}-G_{min})$, PCI=L/$S_p$, L=Crosswalk length

FLC System: Φ1 & Φ2 (NB-SB), Φ3 & Φ4 (EB-WB), $G_{LT}$ – Fixed LT green length, $G_{TH}$ – TH green length

**Performance Measurement**

The HCM prescribes Level-of-Service criteria for vehicles and pedestrians at signalized intersections using average control delay (including deceleration, queue moving time, stopped time, and acceleration) and average pedestrian delay (*10*). Traffic simulation is often employed as a standard approach to address operational analysis issues that cannot be effectively resolved using the HCM-based or other analytical procedures (*38*). VISSIM tracks individual traveler interactions to compute delay for each traveler traversing a facility, which measures average total delay as the difference in travel time at a lower speed compared with that at the free-flow speed (*35*). Therefore, total delay includes HCM-related control delay and delays resultant from other conditions (e.g., congestion, car-following) (*39*). By definition, a vehicle is queued if its speed drops below 5.0 km/h and remains under 10 km/h (*35*). VISSIM reports average and maximum queues observed during the analysis period, using the current queue length measured upstream every time step. Number of stops denotes the total number of all occasions when a vehicle enters the queue condition (*35*). VISSIM reports average stops per vehicle captured during the 1-h period. It is widely agreed that average number of stops is related to the occurrence frequency of rear-end collisions besides its association with delays.

**Simulation Data**

Transportation studies commonly use simulation tools to evaluate intersection performance by averaging results from multiple runs for varied operational conditions (*40*). Accordingly, six replications, with unique random seeds on disparate magnitude levels, were implemented for VAC-related study scenarios to accommodate the stochastic variations from underlying random models. Each replication lasted 3,600 simulation seconds. During run time, an external program, as an automation client in seamless dialogue with the simulation server, periodically captured, aggregated, computed, and exported the data. With all random seeds used as a pool, the GA optimizer contains a module which randomizes the assignment of random seeds for individual runs for the sake of computational efficiency and statistical correlation.

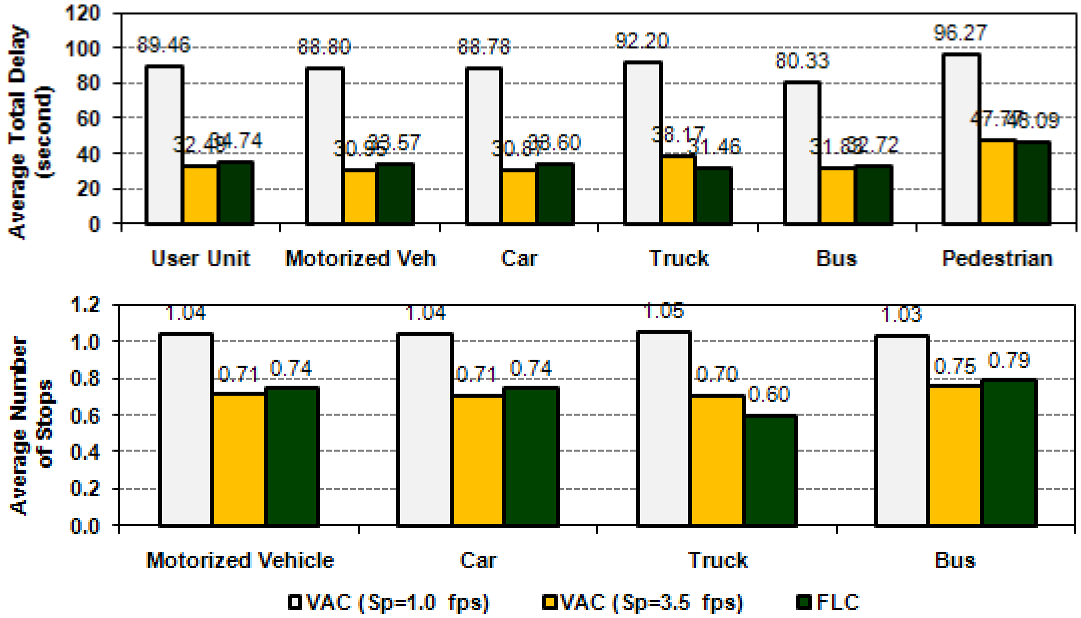
## RESEARCH RESULTS

FIGURE 6 to FIGURE 7 present results for “Moderate” and “Crowded” pedestrians, which also exhibt the effect of “Existing Flows” and “High Demand” conditions on comparision cases. VAC results are reported using means of six replications and FLC results using those optimized by GA. The MOEs for a motorized vehicle denote the weighted averages based on cars, trucks, and buses involved in a study scenario; while those for a user unit account for all motorized vehicles and pedestrians. The proportion of trucks and buses in traffic composition is low.

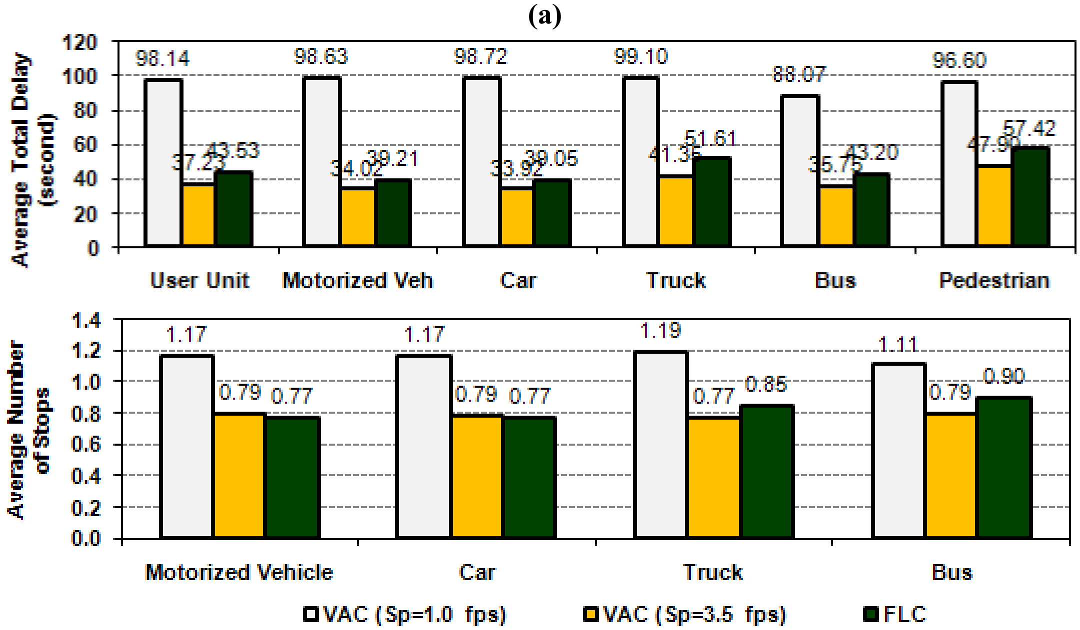
**“Existing Flows” Condition**

FIGURE 6*a* exhibits, with “Moderate” pedestrians, the VAC “1.0-fps” case generates much higher average total delay for each travel mode than the FLC system. For instance, the FLC system tremendously reduces the average total delay per user unit from 89.46 s to 34.74 s by 61.17%, per motorized vehicle from 88.80 s to 33.57 s by 62.20%, and per pedestrian from 96.27 s to 46.09 s by 52.12%. Simultaneously, average total delays generated by the VAC “3.5-fps” case are close to those by the FLC system. For example, average total delays are 32.49 s and 34.74 s per user unit, 30.95 s and 33.57 s per motorized vehicle, 47.77 s and 46.09 s per pedestrian, respectively for the “3.5-fps” case and the FLC system. FIGURE 6*a* also demonstrates that, in contrast to the VAC “1.0-fps” case, the FLC system sharply decreases average number of stops for vehicles. For example, it diminishes the average stops per motorized vehicle (or car) from 1.04 to 0.74 by 28.25%, per truck from 1.05 to 0.60 by 42.86%, and per bus from 1.03 to 0.79 by 23.30%. Additionally, average number of stops generated by the VAC “3.5-fps” case closely approximate the counterparts by the FLC case. For example, average number of stops are 0.71 s and 0.74 s per motorized vehicle (or car) resepectively for the “3.5-fps” case and the FLC case.

FIGURE 6*b* shows, with “Crowded” pedestrians, these MOEs are universially enhanced from their counterparts in FIGURE 6*a*, which reveals the operational impact of pedestrian flows. For instance, average total delay per car produced by the VAC “3.5-fps” case varies from 30.87 s to 33.92 s, and average stops per bus generated by the FLC case ascends from 0.79 to 0.90. FIGURE 6*b* reveals the FLC system significantly lessens average total delay per user unit from 98.14 s to 43.53 s, per car from 98.72 s to 39.05 s, per truck from 99.10 s to 51.61 s, per bus from 88.07 s to 43.20 s, and per pedestrian from 96.60 s to 57.42 s. The FLC case also largely decreases average number of stops per motorized vehicle (or car) from 1.17 to 0.77, per truck from 1.19 to 0.85, per bus from 1.11 to 0.90. Similar to FIGURE 6*a*, these MOEs from the VAC “3.5-fps” case are fairly close to those from the FLC case.




(a)



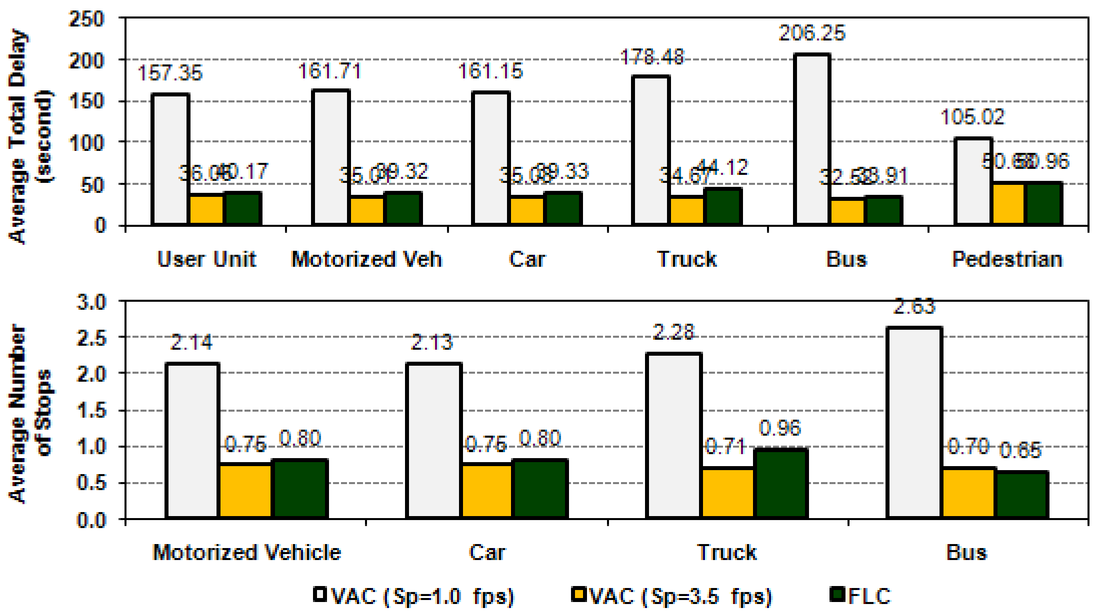

(b)

**FIGURE 6 Average total delay and number of stops under “Existing Flows” condition: (*a*) “Moderate” (50 pphpc) pedestrian flow level and (*b*) “Crowded” (150 pphpc) pedestrian flow level.**

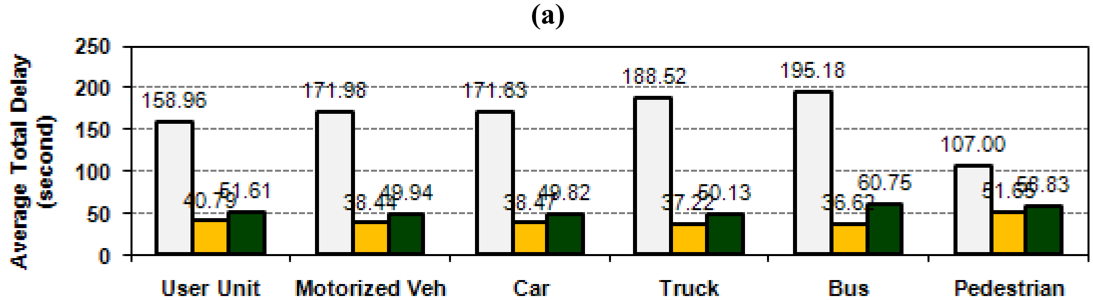
## "High Demand" Condition

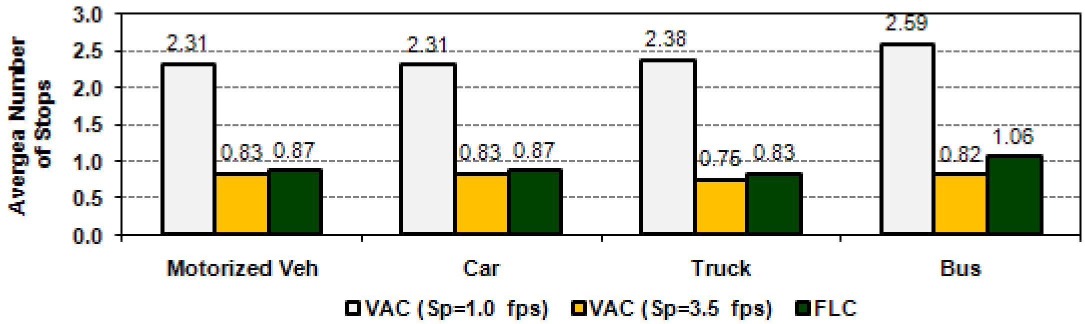
FIGURE 7*a* illustrates, compared with the VAC "1.0-fps" case, the FLC system substantially lowers average total delay per user unit from 157.35 s to 40.17 s, per mororized vehicle from 161.71 s to 39.32 s, and per pedestrian from 105.02 s to 50.96 s. It should be noted average total delays generated by the VAC "3.5-fps" case are close to those by the FLC case. Respectively, average total delays are 36.06 s and 40.17 s per user unit, 35.01 s and 39.32 s per motorized vehicle, 50.68 s and 50.96 s per pedestrian for the "3.5-fps" case and the FLC system. FIGURE 7*a* displays the FLC case enormously decreases average number of stops per motorized vehicle from 2.14 to 0.80, per car from 2.13 to 0.80, per truck from 2.28 to 0.96, per bus from 2.63 to 0.65. Average number of stops created by the VAC "3.5-fps" case roughly approximate those by the FLC case, which are 0.75 s and 0.80 s per motorized vehicle (or car).

FIGURE 7*b* demonstrates, with "Crowded" pedestrians, most MOEs for each mode are enlarged than the counterparts in FIGURE 7*a*. For instance, average total delay per car by the VAC '1.0-fps" case changes from 161.15 s to 171.63 s, and average number of stops per bus by the FLC system rises from 0.65 to 1.06. compared with the "1.0-fps" case, the FLC considerably lessens average total delay per user unit from 158.96 s to 51.61 s, per motorized vehicle from 171.98 s to 49.94 s, per pedestrian from 107.00 s to 58.83 s. The differences in average total delays by the VAC "3.5-fps" case and the FLC case are relatively not considerable: 40.79 s and 51.61 s per user unit, 51.65 s and 58.83 s per pedestrian, respectively for the "3.5-fps" case and the FLC system. The FLC system substantially alters average number of stops per motorized vehicle (or car) from 2.31 to 0.87, in comparison with the "1.0-fps" case. The differences in average number of stops by the "3.5-fps" case and the FLC case are small for motorized vehicles (0.83 vs. 0.87).




(a)






**(b)**

**FIGURE 7 Average total delay and number of stops under "High Demand" condition: (*a*) "Moderate" (50 pphpc) pedestrian flow level and (*b*) "Crowded" (150 pphpc) pedestrian flow level.**

## CONCLUDING REMARKS

A FLC-based system was developed for an isolated intersection in urban context. During each phase, the dynamic "PCI" offers pedestrians the crossing time in real-time need; operational, safety, and human factors were incorporated into the decision-making process for vehicle green control. In simulation setting, the performance of the new signal system optimized by GA was evaluated against the standard "Dual-Ring 8-Phase" VAC which adopted two design walking speeds.

The results from the VAC "1.0-fps" case indicate, although all pedestrians are protected by adequate PCI duration, the intersection operations have actually been *breakdown*. Therefore, the current countermeasure, which *simplistically* lowers the design speed to guarantee crossing safety, was operationally deficient. The results from the VAC "3.5-fps" case are close to or a little lower than those from the FLC case. However, the "3.5-fps" standard cannot offer crossing protection for pedestrians walking slowlier than this standard. Furthermore, the VAC omits multifaceted vehicle needs in control logic, which governs the green by rigid "unit-extension" rule and never "thinks" about intersection-wide situations ongoing in current and next phases to improve safety and operations. In contrast, the FLC system provides full pedestrian protection via dynamic "PCI" and satisfies vehicle needs (e.g., safer platoon dissipation, shorter queuing), realizing a reasonable compromise among competing objectives in intersection control.

The new system relinquishes a fixed design speed as a "PCI" timing input, which would close the debate on "the most appropriate design pedestrian". Importantly, this research first addressed the issue of how to holistically integrate all intersection users into a systematic signalization improvement by means of an innovative signal system which is fully capable of *dynamically* accommodating pedestrians and *intelligently* serving vehicles. The application of this system to a transportation network is particularly important to an urban area where multimodal travelers are busily transported. Given there are reportedly over 325,000 signalized intersections in North America, the potential impact of the intellectual merit herein could be significant from perspectives of multimodal safety, operational efficiency, and quality of life for the public.

## FUTURE RESEARCH

Hardware-in-the-loop simulation can further evaluate the system transplanted onto a real-world controller prior to field tests (*41,42*). A key issue in ultimate applications is the accuracy and reliability of pedestrian sensors used for the dynamic PCI, so a large number of experiments are necessary to appraise how the deployable signal system performs in safety, efficiency, reliability, and sensitivity. The offline optimized parameters can be applied in terms of traffic responsive plan selection: using time-of-day schedules or observed multimodal traffic situations, the system triggers the appropriate timing plans which are configured with a set of optimized parameters to procure the intended benefits under varied scenarios. This research was focused on an isolated intersection in "free" operations, and it is worthwhile to further explore how the system plays its part in coordinated signalized corridors.

## ACKNOWLEDGMENTS

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