USING TRAFFIC SIGNAL CONTROL TO BETTER SERVE PEDESTRIANS AND LIMIT SPEEDING ON URBAN ARTERIALS:
ADAPTIVE WALK INTERVALS AND SPEEDING OPPORTUNITIES

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Abstract

Safe and efficient multimodal transportation system is a priority for many agencies around the world. Traffic signal, in practice, are commonly applied to regulate conflicts, increase capacity, and decrease vehicular delay, with little or no attention given to speed control and pedestrian delay. The dissertation investigates two broad subjects, using traffic signal to improve pedestrian service and to control speed on urban roads. These subjects are presented in the dissertation in three separate research.

First research (chapter 2), introduces a novel concept of an adaptive walk interval. The idea is to use information from the recently passed signal cycles to predict the length of next parent phase, and then to set the walk interval length to fit within that parent phase with only a small chance of forcing the parent phase to run longer. Two methods of predicting the length of the parent phase are proposed, ratio estimation and stratification. The methods were tested in simulation with coordinated-actuated and fully actuated control, with and without pedestrian recall, and with and without permissive windows. The results show that with almost no impact on vehicular delay and without requiring any expensive equipment, adaptive walk intervals can significantly decrease pedestrian delay.

Second research (chapter 3), explores the ways that traffic signal coordination creates – or limits – speeding opportunities on bidirectional arterials. Two measures of speeding opportunity are proposed. The first measure is number of unconstrained vehicles, meaning all vehicles arriving at a stopline on green and with no vehicle less than 5 s ahead of them. The second measure is the number of speeders in a traffic microsimulation in which 20 percent of the vehicles have been assigned a desired speed in the “speeding” range. Theoretical analysis,
confirmed by two case studies, show how speeding opportunities are related to cycle length, specified progression speed (as in input to signal timing software), intersection spacing, degree of saturation, and recall settings. The key role of clusters of intersections with near-simultaneous greens, a byproduct of bi-directional coordination with short intersection spacing, is studied. It is shown that clusters with many intersections create a strong speeding incentive. Also, by reducing the cycle length and progression speed, the cluster size can be reduced. The impact of changes in progression speed tends to be stepped, meaning that they make a difference only when the change in progression speed is enough to alter cluster size. A case study also shows that dividing an arterial into smaller “coordination zones”, with each zone having its own cycle length, can substantially reduce speeding opportunities with little or no increase in vehicular delay, mainly due to lowering cycle length.

Third research (chapter 3), explores empirical data of traffic signal control and traffic flow on excessive speeding at signalized intersections in urban arterials. A logit regression is applied to estimate the risk of a vehicle being an excessive speeder passing through a signalized intersection. Drivers are classified into Speeder and non-Speeder groups. The experienced traffic flow condition and signal status by each driver are investigated. The modelling results revealed that the risk of speeding increases with elapsed green time and doubles with long headway (>5). The risk of excessive speeding can be limited by using traffic signal control to reduce the number of vehicles pass with long headway after green start.
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Chapter 1 : Introduction

In practice traffic signal timing is applied mainly to regulate conflicts, increase capacity, and decrease vehicular delay, with little or no attention given to speed control and pedestrian delay. This research sets out to develop new concepts, operation strategies, and models to better serve pedestrians and limit speeding on urban arterials.

1.1 Walk Interval and pedestrian delay

For urban streets to be livable, streets should be optimized not only for vehicles but also for pedestrians. The traditional approach to urban signal timing has been to focus on minimizing vehicular delay. Provision for pedestrian have focused only on meeting the minimum safety requirements, often resulting in short walk intervals, which results in long waiting time at crosswalks. These long wait times have a direct impact on both safety and walkability (Koh et al., 2014; Brosseau et al., 2013).

Pedestrian phases in most cases run concurrently with a parallel vehicular phase which can be called the parent phase. When the length of the vehicular phase is fixed, the ped phase time can be set equal to the vehicular phase. However, if the vehicular phase that runs concurrently with a pedestrian phase is actuated, the length of its green is not known; it could run anywhere between a (known) minimum and maximum green, depending on when a gap is detected. In such a case, walk intervals are generally kept to a minimum length to prevent the pedestrian phase from constraining the ending of green. This common practice imposes unnecessary delay on pedestrians, when the vehicular phase has an amount of traffic that nearly always holds the green beyond its
minimum. There is a need for traffic signal control logic that will better serve pedestrian while still achieving the traditional goals of safety and vehicular capacity.

### 1.2 Speeding and Signalized Urban Arterials

In nations around the world speeding is a major driver of fatal crashes. In 2015 in the US alone, 9,557 lives were lost in speeding-related crashes (NCSA, 2016). Under the banner of Vision Zero, many USA cities are increasingly focusing on speed control in an attempt to improve traffic safety. In the USA, 83% of speeding-related fatalities occurred on roads other than freeways that is arterials, collectors, and local roads (NCSA, 2016). On urban roads, speeding is particularly dangerous due to the prevalence of vulnerable pedestrians and cyclists.

Effectively controlling speeds on urban arterials poses unique challenges. The traditional methods for speed control, including signage, enforcement and physical measures, have limitations in the urban context. In the absence of speed cameras or other robust enforcement, drivers tend to ignore posted speed limits, choosing a speed at which they feel safe based on road geometry and other environmental factors (Martens et al., 1997). Traditional enforcement is ineffective unless it is constantly in effect, which is prohibitively expensive, while automated enforcement using speed cameras can be effective, they are controversial and not permitted by law in many states. Although physical measures such as speed humps and chicanes are effective for controlling speed, these measures are unsuitable for arterials for several reasons including emergency vehicle response, bus service, and limited space. Furthermore, arterial, reduction in

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1 NHTSA classifies a crash as a speeding-related in the following cases: (a) when any driver who is involved in the crash is charged with speeding-related offense, (b) if police officer states that a contributor to the accident was any of the following: driving too fast, exceeding the posted speed limit, or racing. Speed has been identified as an influencer on increasing crash severity and risk of involvement in a crash (1).
speed below a certain threshold will result in diversion of traffic to local streets, which is undesirable.

In fact, some believe that traffic signals contribute to the speeding problem by giving drivers an incentive to beat the red light when they see a signal that has been green for a while, or has just turned yellow (Persaud et al., 1997). On phases without actuated control, the length of a green interval doesn’t rely on detecting gaps between traffic; the phase will run to its scheduled end even when little or no traffic is using the approach. This creates the possibility of drivers approaching an intersection facing at full speed, a green light and an empty road ahead of them—an obvious opportunity for speeding.

Traffic signal coordination is a very common strategy that is mostly applied along urban arterials. On one-way arterials with close intersection spacing, traffic signals timed to offer a green wave can control speed by eliminating the incentive to go any faster than the progression speed. However, on two-way arterials, it is impossible to offer green waves in both directions when intersection spacing is short. Instead, typical coordination plans turn groups of intersections—often as many as six green at the same time, giving drivers a clear opportunity to speed.
1.3 Research objectives

This research aims to advance the operation of traffic signal control to better serve pedestrians and limit speeding on urban arterials through focusing on operation strategies, and specifically in the following objectives.

1. Develop a novel adaptive signal control method that gives pedestrians longer WALK intervals, and therefore less delay, without impacting operation efficiency (without sacrificing service to motor traffic). The study also discusses and tests the impact of different operation strategies on pedestrian delay at different demand levels. This objective is addressed in chapter 2.

2. Explore the ways in which traffic signal coordination creates – or limits – speeding opportunities on bidirectional arterials. The study also introduces two methods to measure speeding opportunities. Also, discuss the differences between controlling speed in one way and two-way coordination. The research presents methods to limit speeding opportunity without impacting operating efficiency. This objective is addressed in chapter 3.

3. Explore the effect of traffic signal control and traffic flow on the risk of excessive speeding. Also, test the hypothesis that drivers with nominal speeding opportunity are more likely to speed. Nominal speeding opportunity is defined as in the case when at a stopline, drivers are approaching and the signal is green and they are not constrained by a vehicle ahead of them. This objective is addressed in chapter 4.
1.4 Dissertation structure

This dissertation consists of three separate but related research. Chapter 2, first research, develops a novel adaptive signal control of walk interval. Current common operation strategies are to keep walk intervals to minimum length when the pedestrian phase run concurrently with actuated vehicular phase in order to not constraining the ending of vehicular green. This chapter proposed the concept of an adaptive walk interval whose length is based on the predicted length of the concurrent phase’s green. Two methods for predicting the green time needed by a vehicular phase are introduced and tested for different cases. One of the methods is appropriate to be used for intersection approaches with random arrival process, and the other one is for a metered arrival process. The chapter explores different signal operation strategies such as applying full and no permissive period on pedestrian delay as well as the adaptive walk interval. The finding shows that adaptive walk can reduce pedestrian delay dramatically with no impact on vehicular delay.

Chapter 3, second research, explores using traffic signal control to improve speed control on two-way urban arterials. Previous research has recognized that on one-way arterials traffic signals can be timed to offer a green wave that controls speed by eliminating the incentive to go any faster than the progression speed. The chapter discusses that even though on two-way arterials it is impossible to provide green waves that positively control speed as easily as on one-way roads, traffic signal timing still plays a vital role in creating or limiting speeding opportunities. Two measures of speeding opportunity are introduced and tested on microsimulation environment. A validation field test of the proposed measures of speeding opportunity is conducted. The research finding shows that speed opportunity can be limited without significant impact on vehicular delay. A novel finding regarding changes in progression
speed and its effect is presented; a change in progression speed can have no effect at all on limiting speeding opportunities unless they lead to a decrease in cluster size.

Chapter 4, third research, investigates excessive speeding behavior and traffic signal timing. It is a data-based study were cameras used to extract the traffic flow variables such as headway and speed for each vehicle per lane, and traffic signal heads status per seconds. Previous research focused on how the length of yellow and red intervals influence traffic safety, but the length of green interval and its impact on traffic safety or excessive speeding is not found in literature. Excessive speeding in the research is defined as the fastest drivers' speed observed (relative to mean speed) and based on road environment. The chapter presents a case study on an intersection on an arterial in the city of Boston. The conducted exploratory data analysis does not relate excessive speeding to a specific time of the day. The results of logit regression model relate green interval's length, headway, and experience discharge rate to excessive speeding.

Chapter 5 provides the concluding remarks of the dissertation and suggestion for future work.
Chapter 2

Adaptive Walk Interval
Chapter 2 : Adaptive Walk Interval

This part of the dissertation is published. This chapter re-presents the following publications:


Declaration of Contribution

The authors Prof. Peter Furth and Ahmed Halawani confirm contribution to the paper as follows: study conception and design: Furth and Halawani; data collection, processing and classification: Halawani; analysis and interpretation of results: Furth and Halawani; draft manuscript preparation: Furth and Halawani.

2.1 Abstract

If the vehicular phase concurrent with a pedestrian phase is running under fully actuated control or is a non-coordinated phase under coordinated-actuated control, the length of its green is not known when the phase begins; it could run anywhere between a (known) minimum and maximum green, depending on when a gap is detected. In such a case, walk intervals are generally kept to a minimum length to prevent the pedestrian phase from constraining the ending of green that can lead to situations in which the pedestrian phase has cleared yet the concurrent phase continues its green for 20 more seconds. Proposed is the concept of an adaptive walk interval whose length is based on the predicted length of the concurrent phase’s green, which in turn is calculated cycle by cycle with data from recently past cycles. That way, during heavy traffic periods in which the vehicular phase usually goes well beyond its minimum, a longer-
than-minimum walk interval can be provided with very little impact on signal operation or vehicular delay. Two methods, ratio estimation and stratification, are proposed and tested for predicting the green time needed by a vehicular phase; both methods use as data only traffic signal timing data from the last several cycles. Simulation tests with coordinated-actuated and fully actuated control, with and without pedestrian recall, and with and without permissive windows show that adaptive walk intervals can markedly reduce pedestrian delay with almost no impact on vehicular delay.
2.2 Introduction

Concerns for equity, livability, and promoting sustainable modes of transportation have enhanced attention toward reducing pedestrian delay at signalized intersections. While practice has long focused on minimizing vehicular delay, provisions for pedestrians have traditionally focused only on meeting the minimum safety requirements, often resulting in long pedestrian delay. Those safety requirements, laid out in the Manual on Uniform Traffic Control Devices (MUTCD, 2009), as well as the Traffic Signal Timing Manual (Koonce et al., 2008), consist mainly of a policy minimum length for the walk interval $W_o$ (7 s is recommended for $W_o$ although 4 s may be used in exceptional cases) and sufficient pedestrian clearance time, the crossing length divided by a low-percentile pedestrian speed, usually taken to be 3.5 ft/s.

Recent research on reducing pedestrian delay includes attention given to two-stage pedestrian crossings (Furth and Wang, 2015; Wang and Tian, 2010), longer permissive intervals for actuated pedestrian phases (Kothuri et al., 2013), overlaps involving leading pedestrian intervals that can reduce cycle length requirements (Furth et al., 2012), and exploring the use of fully actuated versus coordinated control at different levels of vehicular and pedestrian demand (Kothuri et al., 2015). Because pedestrians tend to arrive without a coordinated pattern (except at multistage crossings), reducing pedestrian delay is mainly a matter of shortening the signal cycle or lengthening the walk interval, or both (Vallyon and Turner, 2011). This paper focuses on the second idea: lengthening the walk interval for a given cycle.
2.3 Maximum, Minimum, And Adaptive Walk Intervals

Pretimed signals can afford the longest possible walk intervals for a given set of splits, which makes them popular in heavily pedestrianized cities like New York and San Francisco. For a given pedestrian clearance (PedClear) time and given green interval (G) and change interval (YAR) on the concurrent vehicular phase, the walk interval (W) can be maximized by simply giving it the entire split minus the needed clearance:

\[ W_{\text{max}} = G + YAR - PedClear \]  

With coordinated-actuated control, walk intervals that run concurrently with the coordinated phase can likewise easily be maximized. The coordinated phase may begin earlier than its nominal start time in the cycle, but it always ends at its scheduled time. By choosing the controller setting “rest in walk,” walk intervals concurrent with the coordinated phase will run to their maximum length (Koonce et al., 2008) as given by Equation 2-1. In this case, G is the coordinated phase’s green time in a particular cycle, which may be longer than its nominal green if it begins early.

However, for crossings concurrent with a vehicular phase whose ending time is not fixed, but rather is demand responsive—which includes the non-coordinated phases at intersections with coordinated actuated control as well as all phases at intersections with fully actuated control—the walk interval must be set without knowing in advance the length of the green interval. Standard practice is to limit the walk interval to a minimum length so that the vehicular phase can respond immediately when a gap is detected, subject only to minimum green, because the phase is constrained from switching until any concurrent pedestrian phase has cleared. While pedestrian clearance may time concurrently with the vehicular change interval, PedClear is
typically greater than YAR, and so the goal is to have the pedestrian phase almost cleared—that is, having only YAR seconds left until it is fully cleared—when the concurrent vehicular green reaches its minimum green ($G_{\text{min}}$).

The minimum length walk interval is given by

$$W_{\text{min}} = \max(G_{\text{min}} + YAR - \text{PedClear}, \ W_o)$$

(2-2)

$W_{\text{min}}$ is the longest walk interval that will not constrain vehicle operations and will therefore not affect vehicular delay, subject to the Manual on Uniform Traffic Control Devices’ policy minimum. While $W_{\text{min}}$ often equals $W_o$, there are many pedestrian phases for which they are not equal. For example, if $G_{\text{min}} = 20$ s, YAR = 5 s, and PedClear = 13 s, then $W_{\text{min}} = 12$ s. Setting the walk interval to $W_o$ instead of $W_{\text{min}}$ creates additional delay for pedestrians with no benefit to vehicular traffic.

Unfortunately, $W_o$ is often used in such cases because of the industry focus on meeting only pedestrian minimum needs, rather than on giving them the best possible service.

Even if the concurrent walk interval is set to $W_{\text{min}}$, the pedestrian signal can still display (solid) “Don’t Walk” while the concurrent vehicular phase remains green, sometimes for a considerable time, leaving waiting pedestrians wondering why they are told to wait while the concurrent traffic phase still has a green. Figure 2-1 illustrates how $W_{\text{min}}$ relates to the minimum and maximum green times for a concurrent vehicular phase with demand-responsive end-of-green. As shown in the figure, $W_{\text{min}}$ is chosen so that pedestrian clearance (c) will end simultaneously with the change interval if the green runs for its minimum value (b). But if the green runs longer (a), there can be a long time while the vehicular signal is green yet the parallel pedestrian signal says “Don’t Walk.”
Short walk intervals increase pedestrian delay, which in turn increases the likelihood of noncompliance (1). This can become a safety problem because pedestrians crossing with the green but without a walk signal will get no advance indication of when the signal is about to change until the onset of yellow. Therefore, they may be caught in the middle of the intersection when a conflicting movement is released.

If it could be predicted that a demand-responsive phase would run longer than its minimum green, then its walk interval could be lengthened correspondingly, reducing pedestrian delay without constraining vehicular operation. This is what will be called an adaptive walk interval—one whose length varies from cycle to cycle between $W_{\text{min}}$ and $W_{\text{max}}$ based on a prediction of how long the concurrent vehicular phase is expected to run. Adaptive walk intervals will be short when traffic is light and the vehicular phase is expected to gap out later. At an extreme, an adaptive walk interval will have length $W_{\text{max}}$ when the vehicular phase is expected to max-out. Adaptive walk intervals represent a middle ground between the long walk intervals offered by pre-timed control and the short walk intervals offered by actuated control.

The objective of this research is to develop a method for setting walk interval lengths on a cycle-by-cycle basis based on predicted lengths of the concurrent vehicular green. Because predictions are imperfect, inevitably, vehicular intervals will sometimes gap out sooner than predicted. So lengthening the walk interval beyond $W_{\text{min}}$ will sometimes force the phases to end later than they otherwise would, increasing vehicular delay to traffic whose red interval is thus lengthened. That creates a trade-off: adaptive walk intervals should be able to lower pedestrian delay, but at the cost of some increase in vehicular delay.

The hypothesis is that substantial reductions in pedestrian delay are possible with very little increase in vehicular delay. This is based on the idea that in many situations, vehicular
phases routinely run considerably longer than their minimum green, offering an opportunity to increase walk intervals with almost no impact on traffic operations.

Figure 2-1: Minimum walk interval in relation to minimum and maximum green of a concurrent vehicular phase.
2.4 Pedestrian Delay and Permissive Windows

Research on pedestrian behavior has established that pedestrians tend to start walking not only during the walk interval but also for a short period at the start of “Don’t Walk.” The Highway Capacity Manual suggests 4 s as this default extra pedestrian green (HCM, 2010). Therefore, effective pedestrian green ($g_{ped}$) and effective pedestrian red ($r_{ped}$) are given by

$$g_{ped} = W + 4, \quad r_{ped} = C - g_{ped} = C - (W + 4) \quad (2-3)$$

where $W$ is length of the walk interval and $C$ is cycle length.

If the pedestrian phase is on recall, average pedestrian delay ($d_{ped}$) can readily be calculated based on the assumption that pedestrians arrive randomly and uniformly over the cycle:

$$d_{ped} = \frac{r_{ped} r_{ped}}{c} \quad (2-4)$$

In Equation 2-4, the first fraction represents the fraction of pedestrians arriving on effective red, and the second fraction is the average wait of those who arrive on effective red. If the pedestrian phase is not on recall, Equation 2-4 is still a good approximation if pedestrian demand is great enough that a call for the pedestrian phase is made during almost every red interval.

However, when pedestrian demand is low enough that there is almost always a full cycle or more between pedestrian calls, pedestrians no longer have a significant chance of arriving during an active walk interval, and so pedestrian delay rises to

$$d_{ped} = \frac{C - L_{window}}{2} \frac{C - L_{window}}{c} \quad (2-5)$$
where \( L_{\text{window}} \) is the length of the permissive window, the part of the concurrent vehicular phase’s green during which a pedestrian call will be accepted and will trigger an instantaneous walk interval (Kothuri et al., 2013). Where and when vehicular demand is so low that there is no vehicular call by the time a phase is scheduled to begin, the permissive window can begin earlier than the concurrent green, in which case a pedestrian call will trigger a phase change.

In practice, signals are often programmed with no permissive window, and so Equation 2-5, giving pedestrian delay when pedestrian demand is low and signals are not on recall, degenerates to the following:

\[
d_{\text{ped}} = \frac{C}{2}
\]

(2-6)

For intermediate levels of pedestrian demand, average pedestrian delay will lie between the extremes of Equation 2-4 and either Equation 2-5 or 2-6.

Where signals are demand responsive, Equations 4 to 6 are approximate, since cycle length varies from cycle to cycle, and, in the case of a full permissive window, a vehicular phase may gap out before the permissive window has expired, effectively closing the window. Therefore, with actuated control, pedestrian delay is more accurately estimated by simulation or from detector data, as in Kothuri et al. (2013).

Kothuri et al. (2013) studied how permissive windows, which the city of Portland, Oregon, has begun to apply systematically, reduce pedestrian delay. Two options exist for permissive windows. One option, minimum permissive windows, does not constrain signal operations at all; they allow pedestrian calls that, if fulfilled, would still allow the phase to end as early as the vehicular timing would allow. The length of the minimum permissive window is

\[
L_{\text{window}}^{\text{min}} = \max(W_{\text{min}} - W_0, 0)
\]

(2-7)
The minimum permissive window will be nonzero only where the vehicular minimum green is long enough that \( W_{\min} \) exceeds \( W_0 \). Where a minimum permissive interval is applied, the walk interval is ended by a force-off at \( W_{\min} \) \( s \) following the onset of vehicular green.

Another option is to use a full permissive window, which means accepting pedestrian calls up until the time at which a fresh pedestrian phase would force the vehicular phase’s green to reach \( G_{\max} \). The associated walk interval is usually set to \( W_0 \), or it can run until the force-off time just discussed, if that is later than \( W_0 \). Full permissive windows constrain signal timing, though not beyond the limit that would be allowed for serving vehicular demand. Using field tests that measured the difference in time between a pedestrian call and the start of walk, Kothuri et al. (2013) found that using full permissive windows significantly reduced pedestrian delay. However, their study did not look at impacts on vehicular traffic.

### 2.5 Proposed Approach

If knowing in advance that the coming vehicular phase would need a certain amount of green time \( G' \) before it gapped out or maxed out, one would set the walk interval to the length given by Equation 2-8, which is the same as Equation 2-1, except that \( G' \) substitutes for \( G_{\max} \):\[
W_{\text{adapt}} = \max(G' + YAR - Ped\text{Clear}, \ W_o)
\] (2-8)

That way, it would be the longest it could without constraining signal control. Of course, one does not know in advance what the needed green will be, but it can be predicted. Letting \( G' \) then be the predicted green time needed by the concurrent vehicular movement, the adaptive walk interval length is given by Equation 2-8.
Because of uncertainty in the prediction, a downward-biased estimate of needed green time was chosen, such that the actual needed green is lower than the prediction only about 30% to 35% of the time. Using a mean or 50th percentile prediction in Equation 2-8 would force the phase to run longer than it otherwise would about half the time. In contrast, a 30th percentile green time represents a rather short green time, and any green periods forced to run longer will be during cycles with a lot of slack time. So they will not affect capacity and should have little impact on delay. Using a low estimate is also appropriate for testing the hypothesis that substantial delay reduction for pedestrians can be achieved with little effect on vehicular traffic.

A main element of this problem is predicting the needed green. This prediction must be made at the moment the concurrent vehicular phase is about to begin. This is a common problem in adaptive signal control. It can be done using additional detectors that count cars, to estimate queue lengths, as in Cesme and Furth (2014). For this problem, the constraint that no additional detectors should be required is accepted; rather, predictions should be based only on information from traffic signal timing.
2.6 Coordinated-Actuated Control: Site 1

Two intersections were used to develop and test adaptive walk interval logic. One uses coordinated-actuated control; the other was modeled with fully actuated control.

The junction of Columbus Avenue–Heath Street–Centre Street in Boston, which has coordinated-actuated control, was analyzed for the p.m. peak hour (4:45 to 5:45 p.m.). Columbus Avenue, the coordinated arterial, runs north–south and has a six-lane cross section. It is coordinated with two nearby intersections to the south and one to the north. Heath and Centre Streets are the east side and west side legs of the junction, respectively, and each has one lane per direction. They share a single phase in the signal cycle with a concurrent pedestrian phase. Neither has a nearby traffic signal affecting its arrivals.

The pedestrian phase considered is the one concurrent with the Centre–Heath phase, which has a nominal split of 56 s in a cycle of 120 s, and being the non-coordinated phase, it is subject to gap-out. (The other crossing, concurrent with Columbus Avenue’s through movements, does not need adaptive walk intervals because its concurrent phase ends at a preset time.) $G_{\text{min}} = 8$ s. The current walk interval, consistent with Equation 2-2, has length $W_{\text{min}} = W_0 = 7$ s. The crossing length, 72 ft, requires a pedestrian clearance of 21 s.

The latest count found 14 pedestrians using that pedestrian phase during the p.m. peak hour. However, being in a densely populated neighborhood and close to a Metro station, that small volume could be because pedestrians are avoiding this crossing owing to the long wait time, currently 54 s using Equation 2-4 (Highway Capacity Manual formula) and 60 s using the more appropriate low-demand formula, Equation 2-6. In cycles in which the vehicular phase runs to its maximum—and vehicular demand during the p.m. peak is such that Centre–Heath maxes
out in 43% of the cycles—the walk interval could have length $W_{\text{max}} = 35$ s without affecting signal operations. Because of the long wait and the apparent inconsistency of a “Don’t Walk” showing while the concurrent vehicular phase remains green, often for more than 20 s, pedestrian compliance is low, and that is a safety concern considering the six-lane cross section.

## 2.7 Predicting Length of Vehicular Phase

One method that could be used to predict the green time needed by the Centre–Heath phase is to measure the needed green times from the last several cycles and to use an average, possibly weighted. Such an estimate will reflect the general level of traffic, which tends to rise and fall in a pattern as the peak comes and goes. Needed green is measured as the time until gap-out or, if there is no gap-out, max-out. Green time that occurs after gap-out waiting for a walk interval to clear is excluded. However, an additional and valuable piece of information is available when a green interval is about to begin, which is precisely the moment at which an adaptive walk interval length must be determined: the length of the red period that just ended. For a given level of demand, a long red period will usually lead to a greater need for green. As is well-known from traffic theory, for a given red period ($R$), the time until the queue clears under uniform, deterministic arrivals and deterministic departures is given by

$$G_{\text{needed}} = R \frac{v}{s-v} = R\theta$$

(2.9)

where

$v = \text{approach volume},$

$s = \text{saturation flow rate},$ and

$\theta = v/(s - v).$
By rearranging, the ratio $v/(s - v)$ becomes

$$
\theta = \frac{G_{\text{needed}}}{R} \quad (2-10)
$$

Because of random issues that affect real traffic flow, the ratio of $G_{\text{needed}}$ to $R$ in a single cycle will be an unreliable estimate of $\theta$; however, an estimate made from several recent observations will offer a more robust estimate. There was use of a ratio estimator estimated from a paired sample from the last five cycles to estimate $\theta$ and its coefficient of variation (CV) (Cochran, 2007). A cycle consists of a red interval and the following green interval. Using $G$ as shorthand for $G_{\text{needed}}$, the ratio estimator and the estimate of its squared CV are

$$
\hat{\theta} = \frac{\hat{G}}{\hat{R}} \quad (2-11)
$$

$$
cv^{2}_{\theta} = cv^{2}_{G} + cv^{2}_{R} - 2r_{GR}cv_{G}cv_{R} \quad (2-12)
$$

where $r_{GR}$ is the correlation coefficient between $G$ and $R$, which, like $CV_{G}$ and $CV_{R}$, can be estimated from the paired sample.

As stated earlier, the predicted needed green should not be the mean, but rather a low percentile such as 30th or 35th. Because ratio estimators have an approximately normal distribution and because $-0.5$ is the 30th percentile value of the standard normal distribution, the proposed predicted value of needed green for the concurrent phase, as a function of the length of its red period that just ended, is

$$
G' = R\hat{\theta}(1 - 0.5cv_{\theta}) \quad (2-13)
$$

The adaptive walk interval length $W_{\text{adapt}}$ is then calculated with Equation 2-8.
2.8 Experimental Design

The corridor was modeled in PTV VISSIM, including the subject intersection and the three other intersections along Columbus Avenue mentioned earlier, using p.m. peak (4:45 to 5:45 p.m.) traffic volumes and signal timings. The simulation was run for a period of 7 h after a 15-min warm-up period. Because none of the approaches in the network is oversaturated and queues therefore rarely spill over from cycle to cycle, a single long run is essentially equivalent to multiple short runs. Seven control alternatives were tested, shown in Table 2-1. The first is the current control scheme.

Table 2-1: Control Alternatives

<table>
<thead>
<tr>
<th>Alternative</th>
<th>Ped Call Type</th>
<th>walk Interval Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>No Per _Min</td>
<td>No recall, no permissive window</td>
<td>( W_{\text{min}} )</td>
</tr>
<tr>
<td>No Per _Adapt</td>
<td>No recall, no permissive window</td>
<td>( W_{\text{adapt}} )</td>
</tr>
<tr>
<td>Full Per _Min</td>
<td>No recall, full permissive window</td>
<td>( W_{\text{min}} )</td>
</tr>
<tr>
<td>Full Per _Adapt</td>
<td>No recall, full permissive window</td>
<td>( W_{\text{adapt}} )</td>
</tr>
<tr>
<td>Recall _Min</td>
<td>Ped recall</td>
<td>( W_{\text{min}} )</td>
</tr>
<tr>
<td>Recall _Adapt</td>
<td>Ped recall</td>
<td>( W_{\text{adapt}} )</td>
</tr>
<tr>
<td>Max Recall</td>
<td>Pretimed, ped recall</td>
<td>( W_{\text{max}} )</td>
</tr>
</tbody>
</table>

Three levels of pedestrian demand were modeled as well:

- **Low**: 0.5 peds/cycle served by pedestrian phase,
- **Medium**: 2 peds/cycle served by pedestrian phase, and
- **High**: 5 peds/cycle served by pedestrian phase.
With low pedestrian demand, there is little chance that a pedestrian will arrive on green unless the walk phase is on recall, and so permissive windows become important. With high pedestrian demand, there is a substantial chance that a pedestrian will arriving during a walk interval, and that chance is far greater in options offering a longer walk interval ($W_{\text{adapt}}$ or $W_{\text{max}}$). At the same time, permissive windows matter little because there will almost always be a registered pedestrian call when the signal turns green.

### 2.9 Results for Site 1

Under current control, average pedestrian delay to pedestrians crossing Columbus Avenue, measured using PTV VISSIM, is 62, 55, and 54 s respectively, for the low-, medium-, and high-demand cases. Pedestrian delay on the treated crossing is shown in Figure 2-2 as a change from the current control alternative. Of course, max recall yields the greatest improvement. However, the three alternatives with adaptive walk intervals yield substantial delay reductions for all levels of demand. For example, with medium demand, the delay reduction is 10 s from just adding the adaptive walk interval; adding permissive windows or recall as well makes that reduction rise to 14 s.
Permissive windows make a substantial impact only under low demand. At high levels of demand, almost no delay reduction occurs with either adding a full permissive window or putting the pedestrian phase on recall. Rather, the big gains when pedestrian demand is higher come with longer walk intervals, which are offered by adaptive and max recall options.

Change in delay to vehicles is less than 1 s in every alternative, and that is effectively undetectable. Surprisingly, even the most aggressive pedestrian treatment, max recall for pedestrians— which is essentially equivalent to making the Centre–Heath phase pre-timed— yields a small net decrease in vehicular delay. That treatment increases delay to Columbus Avenue traffic by only 2 s because arterial traffic is platooned by other nearby signals, and the
time taken from the arterial comes from outside its through band; at the same time, it reduces delay to the cross street by 7 to 9 s.

This result does not seem generalizable, however. The adaptive walk alternatives lengthen the east–west green time by 1 to 2 s at the expense of the arterial phase, indicating that the proposed adaptive control is successful in avoiding placing a substantial constraint on signal operations. In contrast, the maximum recall option shifts 7 to 8 s from the arterial phase to the cross street.

For pedestrians on other crosswalks, change in delay is within 1 s for every alternative except for max recall, which increases delay on the heavily used north–south crosswalks by almost 10 s. This trade-off is a reason to favor adaptive walk intervals over max recall.
2.10 Fully Actuated Control: Site 2

The second test site is the junction of Longwood Avenue (east–west) and Saint Paul Street (north–south) in Brookline, Massachusetts. It has heavy pedestrian traffic going to and from a very large employment area, the Longwood Medical Area, about ½ mi to the east, as well as several nearby colleges and rapid transit stops. The site was studied during the p.m. peak hour, when it sees about 375 pedestrian crossings per hour, of which 285 are along the east–west street. Both streets have one through lane per direction; in addition the southbound approach has a left-turn lane. The critical sum is 800 vehicles per hour. This intersection currently operates with two-phase, pretimed control on a 90-s cycle with 20-s walk intervals, yielding an average pedestrian delay of 24 s. The 90-s cycle allows there to be coordination for Saint Paul Street, which intersects with Beacon Street, a major arterial that has arterial coordination on a 90-s cycle, 800 ft to the north. However, the long cycle imposes a lot of delay on pedestrians, and that seems out of place at a junction of two narrow streets with only two phases. Casual observation shows that pedestrian noncompliance is high. There is no compelling reason to force the signal to coordinate with the Beacon–Saint Paul intersection, because of the following:

1. Intersection spacing enables good progression in only one direction.

2. Substantial turning flows onto and off of Saint Paul Street limit the benefit of progression even for the coordinated movement.

3. Spacing between these intersections is long enough to store queues without causing spillback or starvation.

Applying fully actuated control at this intersection will allow much shorter cycles, and with them, considerably less pedestrian delay. And because this research tests a method that
applies where signals are actuated, it was modeled as having fully actuated operation. There remain two simple phases, with minimum green set at 10 s, which will result in snappy signal operation when demand is light. The same set of control alternatives and demand levels used at the other test site were applied here as well. Pedestrian phase treatments, whether adaptive walk, recall, or permissive windows, were applied to all crosswalks.

Modeling was done in PTV VISSIM, and the model included the Beacon–Saint Paul junction that sends a platoon to the subject intersection every 90 s. No nearby signalized intersections are present in the other directions.

### 2.11 Predicting Vehicular Phase Length

The eastbound, westbound, and northbound approaches have no nearby upstream traffic signals, and so their phase length prediction was done as described earlier, using the length of the just-ended red period along with an estimate of the ratio of needed green to red made from records of the last five cycles.

For the southbound approach, however, the needed green time varies not so much with the length of the red period as with whether a platoon released from the coordinated intersection at Beacon Street is served. Because Beacon Street operates with a 90 s-cycle while the subject intersection is fully actuated (and, it turns out, tends to cycle every 50 s or so), platoons from Beacon Street can arrive at any time in the cycle, or not at all. Therefore, cycles at the subject intersection were divided into three strata, with cycles beginning at the start of southbound red:

- **Early platoon.** Cycles in which the head of the platoon arrives between the start of red and the end of minimum green. Cycles in this group tended to need a long green phase. Platoon
arrival is defined as the start of green at the upstream intersection for the phase discharging the platoon plus a fixed travel time of 18 s; therefore, no detector information is needed to determine platoon arrival time.

• **No platoon.** Cycles in which the head of a platoon does not arrive. Cycles in this group tend to need a short green phase.

• **Late platoon.** Cycles in which the head of the platoon arrives after minimum green has expired. This is the smallest stratum. Cycles in this group exhibit highly variable green times because sometimes the platoon stays together well enough to hold the green until all of it has passed. But at other times, platoon dispersion leads to gap-out after only a leading part of the platoon has been served.

Data from past cycles were collected and stratified into those three groups, with data from the last seven cycles in each stratum retained in memory. When a green period for Saint Paul Street is about to start, it is known with near certainty what stratum that cycle belongs to. If a cycle was in either the early platoon or no platoon stratum, \( G' \) was set equal to the third smallest of the last seven green periods for its stratum. That roughly represents a 33rd percentile value, since two cycles had shorter needed greens and four had longer. For the stratum late platoon, predictions were made with data from the stratum early platoon because the late platoon stratum had a smaller sample size and was more unpredictable. On the basis of those predictions of needed green, the adaptive walk interval was calculated with Equation 2-8.
2.12 Results for Site 2

Average pedestrian delay and average vehicular delay are shown in Figure 2-3 as a change against a base. The base is fully actuated control with no recall, and no permissive window, and 7 s-walk intervals, for which pedestrian delay was 18, 19, and 21 s for the low-, medium-, and high-demand scenarios, respectively. Average vehicular delay for the same scenarios was 24, 22, and 22 s.

As at Site 1, permissive windows were found to be very effective for low demand, but it becomes almost inconsequential with high demand. Adding adaptive walk phases without permissive windows or recall reduces pedestrian delay by 2.3 to 3.5 s, depending on pedestrian demand, or 12% to 17%. With recall, a setting that seems reasonable for a pedestrian-intensive location like this, switching from minimum to adaptive walk intervals reduces average pedestrian delay by 2.7 to 4 s, depending on whether pedestrian demand is light or heavy.
Figure 2-3. Change in delay (s) to cars and pedestrians relative to the base alternative under low, medium, and high pedestrian demand for seven control alternatives at site 2.

For vehicular traffic, most control alternatives increase delay by less than 1 s, and none by more than 2 s, with one exception. That exception is max recall, which for a case like this means providing pretimed control. For that alternative, traffic delay rose by 8 to 10 s, depending on demand, because it forces the cycle to 70 s, while average cycle length hovered in the range of 48 to 56 s for the other alternatives. It is especially instructive to compare pedestrian recall with adaptive phases against pedestrian recall with maximum pedestrian phases.

With adaptive phases, one gains 66% to 72% of the pedestrian delay reduction (depending on pedestrian demand), but at only 2% to 20% of the delay increase imposed on vehicular traffic. That trade-off confirms that adaptive walk phase logic manages to secure substantial benefit for pedestrians with little impact on vehicular traffic.
2.13 Conclusions

Adaptive walk interval logic can substantially reduce pedestrian delay with little or no impact to vehicular traffic at pedestrian crossings that run concurrently with traffic-responsive vehicular phases, as confirmed by tests involving both coordinated-actuated and fully actuated control. Calculations use only information that can be detected by traffic signal control equipment without a need for additional detector. While the control logic for determining adaptive walk intervals must be special purpose programmed, no change is needed to the operation of the local controllers once they are given an updated walk interval length. Full permissive intervals also help reduce pedestrian delay with nearly no impact to vehicular traffic, but substantial gains appear only where pedestrian demand is below two pedestrians per cycle.
Chapter 3

Traffic Signal Control to Limit Speeding
Opportunities on Bidirectional Urban Arterials
Chapter 3: Traffic Signal Control to Limit Speeding Opportunities on Bidirectional Urban Arterials

This part of the dissertation is published. This chapter re-presents the following publications:


Author Contribution Statement

- The main contributor to the paper are Prof. Peter Furth and Ahmed Halawani by contributing to *study conception and design, data collection and analysis and interpretation of results*.
- Jin Li and Weimin Hu contributed to the data collection and Burak Cesme contributed to results interpretation.
- All authors contributed to manuscript preparation of the paper.

3.1 Abstract

While controlling speed on urban arterials is important for safety, conventional traffic calming techniques cannot usually be applied on arterials, and many jurisdictions prohibit automated speed enforcement. Moreover, unlike unidirectional arterials, bidirectional arterials with short intersection spacing are not amenable to green waves that can remove the incentive to speed. This research explores the ways that traffic signal coordination creates – or limits – speeding opportunities on bidirectional arterials. Two measures of speeding opportunity are proposed: number of unconstrained vehicles, meaning vehicles arriving at a stopline on green and with no vehicle less than 5 s ahead of them, and number of speeders in a traffic microsimulation in which 20 percent of the vehicles have been assigned a desired speed in the
“speeding” range. Theoretical analysis, confirmed by two case studies, show how speeding opportunities are related to degree of saturation, cycle length, specified progression speed (as in input to signal timing software), intersection spacing, and recall settings. The important role of clusters of intersections with near-simultaneous greens, a byproduct of bi-directional coordination with short intersection spacing, is examined. Clusters with many intersections are shown to create a strong speeding incentive, and cluster size can be reduced by lowering the cycle length and the progression speed. Case studies show that it is sometimes possible to substantially reduce speeding opportunities with little or no increase in vehicular delay by lowering cycle length, lowering progression speed, dividing an arterial into smaller “coordination zones” with each zone having its own cycle length, and by abandoning coordination altogether.

3.2 Introduction

In an effort to improve traffic safety and livability, many cities, often under the banner of Vision Zero (Vision Zero Network 2018), are paying increasing attention to speed control. According to NHTSA, around 28% of the traffic fatalities in the U.S between 2005 and 2014 were speeding related (NCSA,2016). Speeding on multimode roads (arterials, collectors, and locals) account for 83% of speeding related fatalities (NCSA,2016). On urban roads, speeding is particularly dangerous due to the prevalence of vulnerable pedestrians and cyclists. In addition, by discouraging walking and cycling, speeding reduces livability and contributes to auto dependence with its negative effects on public health, congestion, energy resources, and climate.

The default method of speed control is setting and enforcing speed limits. Recently, for example, New York, Boston, and several other cities lowered their default speed limits from 30
to 25 mph. However, enforcement on urban roads is very difficult to accomplish by conventional methods. Speed cameras offer an effective solution if widely deployed, but they are politically controversial and are forbidden in many states. Without intense enforcement, drivers tend to ignore speed limits, choosing a speed at which they feel safe based on road geometry and other factors of the road environment such as intersection frequency (Debnath et al., 2017).

Road geometry can be very effective at controlling speed, and is the basis for traffic calming devices such as speed humps, chicanes, and neighborhood traffic circles (Ewing et al., 1999). However, these methods are unsuitable for arterials for several reasons including emergency vehicle response, bus service, and a desire to offer attractive speeds in order to discourage travel on local streets. And while the concept of “design speed” can be used to control speed on curvy roads, there is no such effect on most urban arterials because they have little curvature.

Where traffic volumes can be carried with a single lane per direction plus turning lanes, road diets have been highly effective at reducing speed (Knapp et al., 2014), because with a single lane per direction, would-be speeders become impeded by vehicles ahead of them. But how to control speed on multilane arterials?

It has long been recognized that on one-way arterials with close intersection spacing, traffic signals timed to offer a green wave can control speed by eliminating the incentive to go any faster than the progression speed. However, on two-way arterials, it is impossible to offer green waves in both directions when intersection spacing is short (Urbanik et al., 2015), as it often is on urban arterials. In fact, some believe that traffic signals contribute to the speeding problem by giving drivers an incentive to beat the red light when they see a signal that has been green for a while, or has just turned yellow (Persaud et al., 1997). In practice, traffic signal
Timing is applied mainly to regulate conflicts, increase capacity, and decrease delay, with little or no attention given to speed control.

### 3.3 Research Objective and Measures of Speeding

#### Opportunity

While it may be impossible to provide green waves that positively control speed as easily on two-way arterials as on one-way roads, traffic signal timing may nevertheless play an important role in creating or limiting speeding opportunities. The objective of this research is to see how traffic signal timing on two-way urban arterials with short intersection spacing affects speeding opportunities, and to explore ways in which signal timing can reduce speeding opportunities without substantially increasing delay.

To speed, drivers must have both the desire and the opportunity. Absent constraining geometry or a strong threat of legal enforcement, it is natural for a substantial fraction of drivers to have the desire to speed, and so we focus on speeding opportunities. Stoplines are chosen as points of speed measurement, because intersections have the greatest potential for conflict with other road users (Stutts et al., 1996; Choi E, 2010; Bhesania, 1991). At a stopline, approaching drivers have an opportunity to speed if the signal is green and they are not impeded by a vehicle ahead of them.

Two measures of speeding opportunity are proposed:

- **Number (or fraction) of unconstrained vehicles.** A vehicle is considered unconstrained if it arrives at the stopline while the signal is green and its headway with respect to the vehicle ahead of it in the same lane is greater than 5 s. This quantity can be measured both in the field and using traffic simulation.

- **Number (or fraction) of speeding vehicles.** For traffic microsimulation, this quantity is heavily influenced by the use “desired speed” setting chosen by the user. To standardize this measure for microsimulation analysis, we propose assigning to 20% of the vehicles a desired speed within the range considered to be “speeding,” and assigning to the remainder
a desired speed not considered “speeding.” In microsimulation, if the vehicles assigned a high desired speed have the opportunity to speed, they will.

The first measure does not account for speeding opportunities that arise when the vehicle ahead, though less than 5 s away, is speeding, nor does it account for speeders who may be decelerating as they close in on a slower vehicle. The second measure does not have these weaknesses, but suffers from having an arbitrary fraction of vehicles desiring to speed, and therefore cannot be expected to give a measurement that corresponds directly to a field measurement unless calibrated to match the fraction of motorists desiring to speed (a task we did not attempt).
3.4 Speeding Incentives and Opportunities with Two-Way Coordination

On a two-way arterial, ideal intersection spacing is when travel time between adjacent intersections equals half the cycle length, in which case two-way coordination can provide the same green waves as are possible with one-way coordination, and therefore they can provide a means of positive speed control. Offsets follow the “half-cycle alternate” pattern, with each intersection offset half a cycle from its neighbor, as shown in Figure 3-1 a. Small adjustments to offsets can also be made to favor one direction over another.) Since travel time is segment length divided by progression speed, the progression speed for ideal two-way coordination is

$$v_{\text{Progression}} = \frac{S}{C/2}$$ (3-1)

where $v_{\text{Progression}}$ = progression speed, $S$ = segment length or intersection spacing, and $C$ = cycle length. Using lead-lag phasing, deviations from ideal spacing equal to half the split of a left turn phase can also produce ideal bidirectional green waves (Urbanik et al., 2015), also shown in Figure 3-1 a.

With ideal spacing and a progression speed at or below the target speed, there is no incentive or opportunity for the platoon leader to speed. In fact, it may be better to set the progression speed a bit lower than the target speed. Denney et al. (2009) have shown how “holes” in the platoon form as vehicles turn off. A slightly depressed progression speed will slow the platoon leaders down enough to fill these holes, resulting in better capacity utilization; if progression speed is not depressed, vehicles following a hole may speed up to fill them.
Unfortunately, the conditions for ideal spacing cannot be met on most urban arterials. For example, for $C = 90$ s and $v_{Progression} = 40$ ft/s or 27 mph, ideal segment length is 1800 ft, far greater than signal spacing on many urban arterials. If $S$ were actually 600 ft, then, holding $C$ at 90 s, $v_{Progression}$ for ideal two-way progression would be 9 mph, a speed that is impractically low.
a. Ideal two-way coordination, with lead-lag phasing at 2a. Yellow represents the main street’s left turn phases.

b. Distorted progression envelope due to non-ideal intersection spacing

c. Bidirectional coordination with clusters of three intersections

Figure 3-1 Coordination diagrams for a bidirectional arterial
3.5 Degree of Saturation

When the degree of saturation is high, the platoon will fill almost the entire green period, and so nearly all vehicles will be constrained from speeding. With a low degree of saturation, the green interval will continue well beyond the time needed to clear the platoon, and vehicles approaching during that little-used part of the green will not be constrained.

In conventional practice, low degree of saturation at many intersections is common, even during peak periods. One reason is the requirement of a common cycle length, typically set to meet the needs of an arterial’s busiest or most complicated intersection. Intersections with less cross street traffic or fewer phases end up with a cycle length far longer than they need, with long periods of unsaturated green that create speeding opportunities. Using smaller coordination zones can help diminish this phenomenon.

3.6 Non-Ideal Intersection Spacing

Where signal spacing is not ideal, as is the case for most urban arterials, optimal offsets still follow half-cycle synchronization, meaning every intersection’s offset is either 0 or $C/2$, with offsets measured from the center of green averaged over the two directions (Morgan and Little, 1964). However, progression envelopes become distorted, with high progression speed on some segments and low progression speed on others, as illustrated in Figure 3-1b. Segments with high speed progression offer obvious speeding opportunities.
3.7 Unequal Green Intervals

Progression envelopes can likewise be distorted, creating associated speeding opportunities, when green intervals at successive intersections are unequal in length. Unequal green intervals are common; intersections where the cross street demands are light typically give longer green periods to the arterial street. With coordinated-actuated control, arterial green intervals are random as slack time not needed by cross-street and left-turn phases is used to extend the arterial phases. Severe inequality can result where pedestrian phases, concurrent with the cross street phase but requiring far more time, are pushbutton-actuated.

3.8 Short Segments and Intersection Clusters

On many urban arterials, intersection spacing is far too short to apply the ideal two-way progression paradigm, as discussed earlier. The standard solution, applied implicitly by signal timing software, is to cluster intersections together, with simultaneous green within each cluster, such that the travel time between adjacent clusters, measured between cluster centers, roughly equals $C/2$. Returning to the original example, if $C = 90$ s and $v_{Progression} = 40$ ft/s, it was shown that ideal intersection spacing is 1800 ft. If intersection spacing is actually 600 ft, then by forming clusters of three intersections, cluster spacing can be 1800 ft, with two-way coordination as illustrated in Figure 1c.

If $n = \text{cluster size (i.e., number of intersections in a cluster)}$, then cluster size for a given cycle length, progression speed, and intersection spacing is given by

$$n = \frac{v_{Progression} \cdot C/2}{s}$$  

(3-2)
Common traffic signal timing software does not formally identify clusters, but clusters are apparent in their solutions. (Because of left turn treatments and offset adjustments to favor one direction over another, clusters with simultaneous green are not always obvious; to spot them, analysts should compare the middle of green, taking an average between the two directions.) Clusters of 5 or more intersections are not unusual; for example, if $C = 120$, $S = 600$ ft, and $v_{Progression} = 50$ ft/s, $n$ will equal 5.

In practice, equation 3-2 will be rounded to an integer, and because clusters must be made up of an integer number of intersections. The necessity of rounding means that small changes in $v_{Progression}$ may leave the optimal (rounded) cluster size unchanged, which in turn is likely to leave other signal timing parameters unchanged, because offsets are based on clusters and splits are largely independent of both clustering and progression speed. For example, if $C = 90$ s and $s = 600$ ft, changing $v_{Progression}$ from 30 mph (44 ft/s) to 25 mph (36.7 ft/s) when applying signal timing software is likely to leave the signal timing plan unchanged, since the ideal cluster size for the two cases (3.3 and 2.75) both round to 3.

The simultaneous green offered within a cluster (Figure 3-1c) creates obvious speeding opportunities, especially with large cluster size. Within a cluster, drivers may see several green lights ahead of them, giving them an incentive to go as fast as possible knowing that those green lights may not last long. (Lead-lag phasing can be used to smooth the transition between clusters, reducing – but not eliminating – this effect.)
3.9 Hypotheses

Based on the preceding analysis of the nature of two-way arterial coordination, the following hypotheses can be advanced:

**H1:** Speeding opportunities tend to be greater with lower degrees of saturation, which involve longer periods of unsaturated green.

**H2:** Large clusters of intersections with simultaneous green create many speeding opportunities, and arise when a combination of long cycle length and high progression speed make intersection spacing short relative to ideal.

**H3:** Changes in progression speed that are too small to change rounded cluster size are likely to have little or no effect on optimal signal timing parameters and performance measures such as delay and speeding opportunities.

**H4:** Shortening cycle lengths is particularly effective at limiting speeding opportunities because it both lowers the size of intersection clusters with simultaneous green and increases degree of saturation.

**H5:** Compared to conventional arterial coordination, it may be possible to substantially reduce speeding opportunities with little or no increase in vehicular or pedestrian delay.
3.10 Study Site 1: Massachusetts Avenue

To test the effects of signal timing parameters on speeding opportunities, two corridors in Boston, Massachusetts were studied; both are sketched in Figure 3-2.

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**Figure 3-2 Study Corridors.**

a. Massachusetts Avenue

b. Melnea Cass Boulevard
The first is a 0.9 mile stretch of Massachusetts Avenue (Mass. Ave.), a 4-lane arterial, between St. Botolph Street and Melnea Cass Blvd. This stretch involves 9 traffic signals, with intersection spacing averaging 660 ft, with signal timing at the two extreme intersections held constant as a boundary condition. At the time of the study, the traffic signals ran coordinated-actuated with a common cycle of 120 s, except at the Southwest Corridor pedestrian crossing, 300 ft south of St. Botoloph Street, where the cycle length was 60 s. As a rule, the intersections have arterial left turn phases.

A simulation model of the corridor was constructed using VISSIM, using its RBC module for signal control. The period studied was the a.m. peak hour, using traffic volume data supplied by the City of Boston in which the busiest northbound and southbound segments carry 1,317 and 877 vehicles/h, respectively. Each simulation run includes a 15 minute warm up period, and reported results are averages of five simulation runs. Measures of performance included average network delay (average delay to all vehicles), corridor delay (average delay to vehicles running the full length of the corridor, averaged between the two directions), number of unconstrained arrivals summed over all of the Mass. Ave. stoplines, and average cycle length as a proxy for average pedestrian delay.

Three alternatives were evaluated: coordinated-actuated control (the scheme currently operated, with timings re-optimized), fully actuated control, and a “zonal coordination” plan. In the zonal coordination alternative, the SW Corridor pedestrian crossing ran fully actuated (with pedestrian recall), and the remaining 6 interior intersections were grouped into three zones of two intersections each, with coordinated-actuated control within each zone, but no coordination between them. Timing plans for the three zones were determined using Synchro, with small manual adjustments, which resulted in cycle lengths of 65 s for the middle zone and 80 s for the
other two zones. Segments between zones were at least 750 ft long, long enough to avoid harmful queue interactions that might stem from lack of coordination.

Key results are shown in Table 3-1. Coordinated-actuated control, with its long signal cycle and long green periods, has the most speeding opportunities, while also offering the lowest corridor delay. Fully actuated control has the fewest speeding opportunities, but has the longest vehicular delays. The zonal coordination plan has an intermediate number of speeding opportunities, and while its corridor delay is greater than the unzoned coordination plan, it lowers delay so much for crossing traffic and turning traffic that it achieves the lowest average delay for all vehicles.

These results support both hypotheses H4 and H5. The two alternatives with substantially shorter cycle lengths allow substantially fewer speeding opportunities. And by placing less emphasis on corridor delay, one alternative (zonal coordination) was found that reduces speeding opportunities by 37% while simultaneously lowering average vehicular delay; another (full actuation) was found that reduces speeding opportunities by 65% while increasing average vehicular delay by only 11%, or 6 s per vehicle (albeit while increasing average corridor delay substantially).

Table 3-1  Results for Three Signal Timing Plans (Massachusetts Avenue)

<table>
<thead>
<tr>
<th></th>
<th>Average cycle length (s)</th>
<th>Corridor delay (s)</th>
<th>Average network delay (s)</th>
<th>Unconstrained arrivals per hr</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coordinated-Actuated</td>
<td>113</td>
<td>65</td>
<td>55</td>
<td>2,283</td>
</tr>
<tr>
<td>Actuated</td>
<td>78</td>
<td>135</td>
<td>61</td>
<td>798</td>
</tr>
<tr>
<td>Zonal Coordinated</td>
<td>76</td>
<td>110</td>
<td>48</td>
<td>1,431</td>
</tr>
</tbody>
</table>
3.10.1 Field Test Confirmation

A field test was done to compare to confirm the ability of the simulation software to model vehicular movements in a way that accurately represents unconstrained arrivals. The southbound approach to the intersection at Tremont Street was observed from 7:30 to 8:30 a.m. on a weekday, videoing the approach and the replaying the video to manually count vehicles and classify them as unconstrained or not. The total number of arriving vehicles was about 10% greater in the field study than in the simulation (807 versus 729).

The fraction of arrivals classified as unconstrained was 21.7% in the field study, versus 21.8% in the simulation study. This result helps provide some confirmation for the validity of the simulation model for measuring unconstrained arrivals.

3.11 Study Site 2: Melnea Cass Boulevard

The second study site is a 0.87 mile stretch of Melnea Cass Boulevard (MCB), a 4-lane arterial, between Tremont Street and Massachusetts Avenue. In the simulation model, control at the extreme intersections was left unchanged in order to provide consistent boundary conditions, leaving 6 interior intersections, as shown in Figure 3-2b. Intersection spacing averages 600 ft.

Currently, the traffic signals run coordinated-actuated with a common cycle of 120 s in the p.m. peak, with a cluster of five intersections (Kerr Way through Albany Street) whose green is essentially simultaneous. This large cluster means that drivers can see green signals for several intersections ahead of them, giving them a strong incentive to speed to try to get through as many intersections as possible before the green ends. Speeding is a common complaint.
The current layout has left turn lanes on MCB for only some of the intersections, and has protected plus permitted left turns throughout. In all of the alternatives studied including the base case, the missing left turn lanes were added and left turns from MCB are protected only. These changes were made to permit a comparison for operation in the near future when the street is rebuilt with a full set of left turn lanes and with protected lefts.

The corridor was modeled using VISSIM, using its RBC module for signal control. Two different volumes were assigned to the network: pm peak hour, using traffic volumes obtained from the City of Boston, and off peak, defined as 50 percent of pm peak volumes. Each simulation run covers 60 minutes following a 15 minute warm up period, and results are averages from five simulation runs. 80 percent of the vehicles were assigned to a class whose desired speed varies from 28 mph to 32 mph, and 20 percent to a class with desired speed between 38 mph to 42 mph. Performance measures included average vehicular delay, corridor travel time, unconstrained arrivals at all of MCB’s interior stoplines, and number of speeders at all of MCB’s stoplines. Delay is measured compared to desired speed, which had the same distribution in all alternatives. Speeders were defined as vehicles with speed exceeding 35 mph.

All of the control alternatives tested use coordinated-actuated control, with timing parameters determined using Synchro. The minimum split of through movements is sufficient for concurrent pedestrian crossings. An initial set of tests with and without lead-lag phasing allowed showed that lead-lag phasing resulted in significantly less delay, with little difference in speeding opportunities, and so all of the control alternatives allow lead-lag phasing.
3.11.1 Cycle Length, Progression Speed, and Cluster Size
As discussed earlier, two-way coordination creates a relationship between progression speed $v_{Progression}$, cycle length $C$, and cluster size $n$, whose unrounded value is given by equation 3-2. For MCB with its average intersection spacing of 600 ft, Table 3-2 shows the values of unrounded $n$ that correspond to different choices of $v_{Progression}$ and $C$; shading is used to indicate combinations expected to result in the same rounded value of $n$, assuming a bias in rounding in which fractional values above 0.4 are rounded up.

Table 3-2 Implied cluster size as a function of cycle length and progression speed when intersection spacing is 600 ft. Shading indicates cells with a common rounded cluster size

<table>
<thead>
<tr>
<th>Progression Speed (mph)</th>
<th>70</th>
<th>80</th>
<th>100</th>
<th>120</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>1.3</td>
<td>1.5</td>
<td>1.8</td>
<td>2.2</td>
</tr>
<tr>
<td>20</td>
<td>1.7</td>
<td>2.0</td>
<td>2.4</td>
<td>2.9</td>
</tr>
<tr>
<td>25</td>
<td>2.1</td>
<td>2.4</td>
<td>3.1</td>
<td>3.7</td>
</tr>
<tr>
<td>30</td>
<td>2.6</td>
<td>2.9</td>
<td>3.7</td>
<td>4.4</td>
</tr>
<tr>
<td>35</td>
<td>3.0</td>
<td>3.4</td>
<td>4.3</td>
<td>5.1</td>
</tr>
</tbody>
</table>

3.11.2 Impact of Volume, Cycle Length, Progression Speed, and Degree of Recall
Table 3-3 shows the performance measures for two sets of volumes (peak and off-peak), a variety of cycle lengths between 70 and 120 s, and a variety of progression speeds between 15 and 35 mph. Corridor travel time was measured but is not reported for conciseness because it was so strongly correlated with network delay (correlation coefficient 0.99 off-peak, 0.96 peak).

An important methodological finding is that the two measures of speeding opportunity proposed are strongly correlated, with correlation coefficient 0.97 off-peak and 0.98 peak. The
The overall average ratio of speeders to unconstrained vehicles is 0.23 off-peak and 0.19 in the peak, close to the specified fraction (0.20) of drivers whose desired speed lies in the “speeding” range. This finding confirms that unconstrained vehicles is a good measure of speeding opportunity; it also suggests that either of the proposed measures of speeding opportunity can be used with similar fidelity.

Table 3-3 Timing plan performance for different cycle lengths and progression speeds.

<table>
<thead>
<tr>
<th>Cycle Length (s)</th>
<th>Progression Speed (mph)</th>
<th>Ideal cluster size</th>
<th>Pedestrian delay (s)</th>
<th>Off-peak</th>
<th>Peak</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>% Unconstrained Vehicles</td>
<td>% Speeders</td>
</tr>
<tr>
<td>70</td>
<td>15</td>
<td>1.3</td>
<td>28</td>
<td>57</td>
<td>12%</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>1.7</td>
<td>28</td>
<td>33</td>
<td>16%</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>2.1</td>
<td>28</td>
<td>32</td>
<td>16%</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>2.6</td>
<td>28</td>
<td>31</td>
<td>16%</td>
</tr>
<tr>
<td></td>
<td>35</td>
<td>3.0</td>
<td>28</td>
<td>33</td>
<td>16%</td>
</tr>
<tr>
<td>80</td>
<td>15</td>
<td>1.5</td>
<td>33</td>
<td>54</td>
<td>16%</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>2.0</td>
<td>33</td>
<td>44</td>
<td>17%</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>2.4</td>
<td>33</td>
<td>36</td>
<td>22%</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>2.9</td>
<td>33</td>
<td>30</td>
<td>24%</td>
</tr>
<tr>
<td></td>
<td>35</td>
<td>3.4</td>
<td>33</td>
<td>41</td>
<td>24%</td>
</tr>
<tr>
<td>100</td>
<td>15</td>
<td>1.8</td>
<td>43</td>
<td>46</td>
<td>23%</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>2.4</td>
<td>43</td>
<td>38</td>
<td>23%</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>3.1</td>
<td>43</td>
<td>37</td>
<td>25%</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>3.7</td>
<td>43</td>
<td>33</td>
<td>28%</td>
</tr>
<tr>
<td></td>
<td>35</td>
<td>4.3</td>
<td>43</td>
<td>35</td>
<td>32%</td>
</tr>
<tr>
<td>120</td>
<td>15</td>
<td>2.2</td>
<td>53</td>
<td>62</td>
<td>23%</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>2.9</td>
<td>53</td>
<td>48</td>
<td>25%</td>
</tr>
<tr>
<td></td>
<td>25</td>
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<td>53</td>
<td>35</td>
<td>35%</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>4.4</td>
<td>53</td>
<td>35</td>
<td>36%</td>
</tr>
<tr>
<td></td>
<td>35</td>
<td>5.1</td>
<td>53</td>
<td>35</td>
<td>36%</td>
</tr>
</tbody>
</table>
Looking at differences in performance, one obvious result is that speeding opportunities are far greater with off-peak volumes than peak volumes, confirming hypothesis H1.

Another is that speeding opportunities tend to increase with cycle length, illustrated in Figure 3-3 and confirming hypothesis H4. Capacity analysis shows that the minimum common cycle that provides sufficient capacity and satisfied pedestrian minima is 70 s for off-peak and 80 s for peak volumes; increasing cycle length beyond that minimum, especially with the low degree of saturation prevalent in the off-peak, substantially increases speeding opportunity.

![Figure 3-3 Unconstrained vehicles as a function of cycle length, specified progression speed, and traffic volume.](image-url)
The impact of changes in progression speed tends to be stepped, as illustrated in Figure 4 for the case of off-peak volumes and $C = 70$ s. In most cases, it makes a difference only when the change in progression speed is enough to alter cluster size, confirming hypothesis H3.

Figure 3-4 also shows how three recall options – no pedestrian recall (pedestrians use a pushbutton), pedestrian recall (our default option), and maximum recall (same as pretimed operation) – affect speeding opportunities. Weaker recall settings randomly start arterial phases early, creating additional speeding opportunities that are evident in the results. However, stronger recall settings tend to give more unsaturated green time to cross streets, which may create speeding opportunities there, and so it would be unwise to conclude anything about what are the best recall settings for speed control without studying speeding on the cross streets as well as the arterial.

![Figure 3-4 Unconstrained vehicles versus progression speed (off-peak volumes, cycle length = 70 s).](image)
With regard to hypothesis H2, Table 3-3 shows a clear trend of speeding opportunities increasing with cluster size. For the off-peak, the case with the smallest values of $n$ has only 12% unconstrained vehicles, versus 36% for the case with the largest $n$. This confirms the hypothesis that large clusters of intersections with roughly simultaneous green create speeding opportunities.

Table 3-3 also shows the vehicular delay, pedestrian delay, and corridor travel time for each signal timing alternative. Corridor travel time is so strongly correlated with vehicular delay in this case study that it is not further discussed. Pedestrian delay is likewise strongly correlated with cycle length. Note that vehicular delay was measured using VISSIM and is based on desired speed, which follows the same distribution with an average value of 32 mph in every alternative, regardless of progression speed, which is a parameter used only to select signal timing settings.

Because average desired speed is 32 mph, one might expect that the least delay occurs when signal timing has been optimized for a speed of 30 or 35 mph. Indeed, progression at 30 mph gives consistently low-delay results; in contrast, progression at 35 mph clearly tends toward greater delay. This confirms the beneficial effect mentioned earlier of a progression speed slightly below average speed. Solutions with a progression speed of 15 mph have large average delay, but most of those with progression speeds of 20 and 25 mph perform well with respect to vehicular delay.

Figure 3-5 illustrates the tradeoff between average vehicle delay and unconstrained vehicles for all the solutions in Table 3-3. One can see that for both off-peak and peak, Synchro’s recommended timing plan based on default parameters performs well in terms of vehicular delay, but has considerably more speeding opportunities. Off-peak, reducing progression speed from 30 to 25 mph cuts speeding opportunities by 27% while increasing vehicular delay by only 5%
(and leaving pedestrian delay unaffected); in the peak, reducing the cycle length from 100 to 80 s cuts speeding opportunities by 28% without any change in vehicular delay (and with a 23% decrease in pedestrian delay).

Figure 3-5 Delay vs. speeding opportunities for timing plans with different cycle length and progression speed. Solid markers indicate timing plans recommended by Synchro.
3.12 Conclusion and Further Research

An increasing number of cities recognize that controlling vehicle speeds is vital for improving safety and livability. This research has explored the potential for doing so using traffic signal settings on multilane, two-way urban arterials with close intersection spacing.

Number of unconstrained vehicles, measured at stoplines, was found to be a valid measure of speeding opportunities. A field measurement confirmed that this measure could be reliably measured by traffic microsimulation, and simulation study confirms that it is strongly correlated with the number of speeding vehicles in a setting in which 20% of all vehicles are specified as having a desired speed in the “speeding” range.

Both case studies found that compared to conventional arterial coordination, it was possible to substantially reduce speeding opportunities with little or no increase in vehicular delay. Case study experiments confirm that with standard arterial coordination, speeding opportunities increase with longer cycles, lower degree of saturation, and closer intersection spacing. Speeding opportunities are related to cluster size, that is, the number of consecutive intersections with simultaneous offsets, which is inversely proportional to effective intersection spacing (travel time between neighboring intersections measured in number of cycles). Shortening cycle length is particularly effective at limiting speeding opportunities because it both increases effective intersection spacing and reduces periods of unsaturated green; it also has the side benefit of lowering pedestrian delay.

Lowering progression speed – an input to signal timing software – can also reduce speeding opportunities, because it increases effective intersection spacing. However, small changes in progression speed can have no effect at all if they don’t lead to a decrease in cluster
size. Recall and minimum green parameters that make the length of the arterial through phases less variable also help reduce speeding opportunities.

Abandoning coordination altogether has the strongest effect on reducing speeding opportunities, but the delays and queue interactions that can lead to may be unacceptable. Short of that, breaking an arterial into small coordination zones, with coordination within each zone but each zone free to have its own cycle length, was found in one case study to sharply reduce speeding opportunities without increasing network delay. Adaptive control methods that achieve a degree of coordination without imposing a common cycle, such as self-organizing control (Cesme and Furth, 2014), may also be effective at limiting speeding opportunities while maintaining a good level of service; that question is left for further research.
Chapter 4
Excessive Speeding and Traffic Signal Timing
Chapter 4 : Excessive Speeding and Traffic Signal Timing

Beginning with the premise that there is a subset of drivers that if conditions allow, will speed. On a road without geometric restrictions or enforcement what could restrict them is either a vehicle in front of them or a red light. The case in which a driver approaching an intersection stopline faces a green signal and is not constrained by a vehicle less than 5 seconds ahead of them, can be defined as “nominal speed opportunity”.

Both traffic flow and traffic signal control parameters (cycle length, offset, etc.) affect how often speeding opportunities occur. The main objectives of this research are to: (i) Explore the effect of traffic signal control and traffic flow on the risk of speeding (ii) Test the hypothesis that drivers with "nominal speeding opportunity" are more likely to speed.

4.1 Overview

In the USA, almost 28% of all fatal crashes are speeding related (NHTSA 2014). The economic cost of speeding-related crashes alone is estimated to be $40.4 billion per year (NHTSA 2008). NHTSA classifies a crash as a speeding-related in the following cases: (a) when any driver who is involved in the crash is charged with speeding-related offense. (b) if a police officer states that a contributor to the accident was any of the following: driving too fast, exceeding the posted speed limit, or racing.

What about posted speed limit? A number of studies indicate that posted speed limit is not the sole factor that drivers base their driving speed choice on. Most drivers base their travel speed on what feels comfortable and safe given the road environment. On multilane arterials, that “feels safe” speed is often well in excess of the speed limit.
Speeding impacts road safety as well as urban livability. When people perceive that a road is dangerous, some will avoid walking across the road, limiting their mobility and limiting urban life in the area. In urban roads, speed reductions tend to improve the condition for walking and cycling, which can reduce per capita emissions, improve public health, and improve local accessibility (Mackett and Brown 2011).

Traffic control can affect how often there are speeding opportunities. Thus, traffic engineers’ choice of cycles length, offsets, and other parameters influence the frequency of speeding opportunity occurrence (see Chapter 3/Research 2). If a signal follows “actuated control” logic, green will ideally be terminated as soon as the queue discharge ends. This operating principle limits speeding opportunities, because there is essentially no opportunity to speed as a queue discharges. However, on urban arterial, fully actuated control creates excessive delay, and frustration for drivers, who may be forced to stop at every signal; that is why coordinated control is preferred. Coordinated signal control requires a common cycle length, which is chosen to meet the needs of the busiest intersection. Intersections with less demand could end up with a cycle length far longer than needed and therefore long periods of unsaturated green, which create speeding opportunities.
4.2 Research Objective and Data

The objective of this research is to explore the effect of traffic signal control and traffic flow on the risk of excessive speeding. For the purpose of this study, the studied intersection must be multi-lane urban arterial and the intersection should have a coordinated -actuated control because that is the most common urban roads form where speeding frequently occurs.

To study the relationship between traffic signal timing and drivers’ excessive speeding, it is important to understand the traffic condition around each driver and the status of the traffic signal observed by the driver. Additionally, specific information about each vehicle need to be collected such as speed and position.

This part of the dissertation is based on two data sources. The primary data source is video cameras recording the study area. The secondary data source is the signal timing plan and geometry layout which were provided by the city of Boston.

Primary input data needed:

- Vehicle information which includes the following: Vehicle’s speed, position and the lane the vehicle is on.
- The down and upstream link status.
- Signal head statues

The primary data can be collected in different methods. However, the chosen method for is video cameras. Five different cameras were installed to continuously and concurrently record vehicle speeds and position, the traffic signal status observed by each driver at all time, and to continuously observe the statue of the intersection, the downstream link, and the upstream link
(blocked or not). Even though the process of collecting the data was simple, processing and synchronizing the videos to obtain the information was complicated and had multiple analysis steps which is discussed in section 4.4.3.

The collected data is interpreted using a software developed using OPENCV C++ for the which provided the ability to obtain the vehicles position at each time steps (60 FPS). The methods of process the videos are discuss in section 4.4.3.

Finally, the study will include both an exploratory data analysis and development of a statistical model.
4.3 Literature Synthesis

This section briefly highlights and discusses some of the literature findings that are related to this research. This chapter is organized into four sections, section 4.3.1 discusses the relation between traffic signal timing and road safety, section 4.3.2 focuses on speeding and safety, and section 4.3.3 focuses on factors that influence speeding behavior. Section 4.3.4 discuss the relationship between traffic signal control operation strategies and speeding opportunity.

4.3.1 Traffic Signal Timing and traffic Safety

The basic objective of traffic signals can be, and in fact must be, serving both operational efficiency and safety at an intersection for both vehicles and pedestrians. A poorly designed signal-timing plan may make an intersection less efficient, less safe, or both. Traffic signals switch between different phases, each serving one or several traffic movements. Each phase consists of three intervals which are green, yellow and optionally red clearance or all-red.

Based on many studies, the length of yellow interval is a significant factor that influence red-light running (e.g. Bonneson & Son, 2003; Bonneson & Zimmerman, 2004;Parsonson, 1984). For examples, Parsonson (1984) found that Red-Light Running occurrence increased as yellow times went from 3s to 5s.

All-red is a short interval in a traffic signal phase, immediately following the yellow phase, in which the phase that’s in control has a red indication, and conflicting phases are still forced to remain red. The all-red phase length provides time for vehicles entering the intersection during yellow time to clear safely. Too short all-red intervals might lead to severe angle collisions if vehicles that entered on yellow are still in the intersection when a conflicting phase turns green and drivers move.
Green interval duration is usually determined with the intent of maximizing efficiency by minimizing the number of stops, delays or both. A longer green interval doesn’t necessarily mean better service to traffic and in most cases it means longer delay to pedestrians waiting to cross the street with the green light. Also, increasing the cycle length doesn’t always lead to an increase in throughput (Denney et al., 2009).

Many studies have explored different methods to determine optimal length of green based on its impact on delay, number of stops, and throughput. On the other hand, the length of green interval and its relation to traffic safety isn’t found in literature.

4.3.2 Speed and Safety

According to National Highway Traffic Safety Administration, 28% of the traffic fatalities in the U.S between 2005 and 2014 were speeding related and 83% of speeding-related fatal crashes occurred on multimode roads (arterial, collector and local).

Speed is highly correlated with severity of a crash. Based on basic physics law. A vehicle's kinetic energy in a collision increases by the square of the speed. Many studies investigated pedestrian fatality risk as a function of car impact speed (e.g. Rosén and Sander, 2009). Tefft (2013) found that the average probability of fatal injury reaches 26% at impact speed of 25 mph, 50% at 30 mph and 60% at 35 mph.

It has also been found that speed increases the risk of being involved in a crash (Aarts and Van Schagen, 2006). The speed of a moving vehicle is directly related to the distance needed to stop. Also, speed reduces the maneuverability due to the fact that faster reaction time is needed to react to sudden changes on the road when speed increases. Many studies investigated the impact of speed on the risk of involving in a crash.
Elvik et al. (2004) found that road environment, vehicle-related factors, and driver’s behavior have impact on the road safety. However, speed was a constant factor that is always found to have impact on road safety across different contexts.

Finch et al. (1994) examined a wide range of before and after studies (after a speed limit change). These studies were conducted in a number of countries. They draw a conclusion that every increase of 1 mph in the average traffic speed can result in 5% increase in the number of crashes.

Liu and Popoff (1997) studied the relationship between travel speed and collision involvement based on data that was collected in Saskatchewan province in Canada. The results of the study found that up to 60 percent of human error that contributes to collisions may be speed-related in Saskatchewan. Their analysis indicates that casualties will be reduced by about 7% for every 1 km/hr reduction in average travel speed.

At the same time, a number of studies show that greater speed dispersion is associated with increased crash rate (Aarts & van Schagen, 2006; Liu and Popoff, 1997; Garber and Gadiraju, 1989).

Finally, Taylor et al. (2000) concluded that reducing the speed of the fastest drivers (relative to the mean speed) is more likely to improve road safety more than reducing the mean speed, particularly on urban roads. However, other studies have shown that reduction of roadway mean speed has a significant impact on reducing both the severity and frequency of crashes.
4.3.3 Speeding Behavior

As any human action, speeding is a complex behavior. Driver’s speed choice is not affected by a sole factor. Numerous studies have been conducted to identify factors that influence driver’s speed choice. It has been found that demographic, roadway characteristics, posted speed, vehicle characteristics and traffic condition all influence driver speed choice.

Demographic factors and their impact on driving speed have been studied intensively. Age, gender, socioeconomic and education level are examples of demographic factors that have been studied and found to influence driver’s speed choice. For example, driver age was found to be an influential factor of speeding (speed falls with age) (Wasielewski, 1984). It was also shown that, vehicle characteristics such as power of engine, level of comfort and vehicle age influence the choice of driving fast (Hirsh, 1986; Wasielewski, 1984).

Garber & Gadiraju (1989) found that drivers tend to drive at increasing speeds as the roadway geometric characteristics improve, regardless of the posted speed limit. Some researchers suggest that drivers drive as fast as they feel is safe and comfortable. Drivers’ choice of safe speed is influenced by the environment and geometry of the road, weather conditions and number of lanes (Wilmot & Khanal, 1999; Rakauskas et al., 2007; Martens et al., 1997).

Enforcement does not have long lasting effect on driver’s speed choice. Driver’s speed choice is influenced by what he/she find as an appropriate safe speed which is related to road design more than posted speed limits (Martens et al., 1997).
4.3.4 Traffic Signal Control

Most traffic signals phases operate in either pretimed, actuated or coordinated mode. Pre-timed phases don’t rely on any mechanism of vehicles detection; their interval are fixed. Actuated phases rely on detectors to extend the green after the minimum green has timed, with the green interval continuing until either phase gap-out or max-out whichever comes first. Coordinated phases don’t rely on detectors and their phase time has a fixed end point in the cycle. The coordinated phases in actuated coordinated control have no fixed start point, which means their green interval length is not fixed.

The length of green interval in pre-timed control is determined based on historical data. Usually, there are between 2 to 4 timing plans for an intersection. The cycle length of each time plan is determined based on the busiest hour in the plan, which means the intersection will end up with long unsaturated green due to long cycle (most of the time). The signal remains green even though the queue has discharged or even if there is no traffic. Long unsaturated green leads to long gaps in traffic flow between vehicles.

Fully actuated control is more efficient than pre-timed control. The phase usually is not in recall, there has to be demand in order to start the green interval. The length of green interval of actuated control extends only if there is demand (after the minimum green time out). The likelihood of having long gap between vehicles with fully actuated control is lower than pre-timed control.

In the coordinated actuated control, the cycle length is fixed. The cycle length is determined based on historical demand on the arterial and mainly the busiest intersection. Ideally, all the non-coordinated phases will be actuated and their green end by gap-out or max-
out. Slack (as result of gapping out) goes toward the green of the coordinated phase. In that case, most of the coordinated phase in the arterial will end up with long green time. Long green leads to long unsaturated period, which means long headway between vehicles in that period. Coordinated actuated control is the most common form of signal control on urban arterials.

4.3.5 Summary

Speed increases both the severity of a crash and the risk of being involved in a crash. A road’s average speed and speed variance are both positively correlated with crash frequency. A number of studies suggests that variance has more impact. Speed variance increases as the number of excessive speeder increases.

How the length of yellow and all red interval influence traffic safety is well documented by previous research. However, the length of green intervals and its impact on traffic safety or excessive speeding is not found in literature. Neither could we find anything in the literature on how choice of control logic – pre-timed, fully actuated, or coordinated actuated – influences speeding or traffic safety.
4.4 Data Collection and Image Processing

This research “Excessive Speeding and Traffic Signal Timing” is based on two data sources. The primary data source is video cameras recording the study area. The secondary data sources are the signal timing plan and geometry layout which were provided by the city of Boston.

4.4.1 Site Description

The location chosen for this research is the intersection of Huntington Avenue and Parker Street in Boston, MA (Figure 4-1) which has coordinated-actuated control. Huntington Avenue, the coordinated arterial, runs roughly east-west and has a four-lane cross section, with a light rail running in a dedicated median. The intersections proximity to many colleges and the Museum of

Figure 4-1 Intersection of Huntington Avenue and Parker Street.

Figure 4-2 Circled Area Shows Huntington Avenue, Boston, MA , and star shows the study intersection. Map Data Google 2018
Fine Arts accounts for heavy pedestrian traffic along and across the road. Parking is prohibited on Huntington Avenue in the vicinity.

![Signal timing plan diagram](image)

**Figure 4-3** Signal timing plan diagram of the intersection of Huntington Avenue and Parker Street.

The intersection has coordinated-actuated control. Coordinated traffic control always has pre-timed cycle length. In coordinated–actuated control, coordinated phases (phase 2 and 6) have a fixed ending time in the cycle. Non-coordinated phases can have detectors and end either by gapping or maxing-out. Any slack time due to gapping-out goes to the coordinated phases, allowing them to begin their green early. Also, non-coordinated phases might be skipped (phase 5) if no call is present.

The arterial has three operating plans at different periods (Table 4-1).

<table>
<thead>
<tr>
<th>Plan #</th>
<th>Cycle length</th>
<th>Hours of Operation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>90</td>
<td>All other times</td>
</tr>
<tr>
<td>2</td>
<td>120</td>
<td>Monday-Friday – 6:00 AM -10:00 AM</td>
</tr>
<tr>
<td>3</td>
<td>100</td>
<td>Monday-Friday – 3:00 PM -7:00 PM</td>
</tr>
</tbody>
</table>
The observed vehicular phase is phase #6 (eastbound) which is a coordinated phase. The length of phase 6 is unknown; it has a fixed end time but a variable start time. The green of Phase 6 is followed by 3s of yellow then 2s of red before the following phase starts. Pedestrian clearance time is 19s for the pedestrian phase that run concurrently with phase #6 (Figure 4-3).

4.4.2 Data Collection

For the data collection for this research, five video cameras were used. The cameras were positioned at a building at the corner of the intersection of Huntington Avenue and Parker Street in Boston, MA. The purpose of each camera is described as the following:

- Camera 1 is positioned to capture vehicles’ speed and vehicles’ position before and just after crossing the stopline.
- Camera 2 is facing the intersection and captures the activity with the intersection.
- Camera 3 is facing the downstream link in order to identify the link states [blocked or clear].
- Camera 4&5 are focused on the signal light bulbs in order to trace the signal states.

Figure 4-4 The Area Filmed by Camera 1,2 And 3
4.4.3 Image Processing

The detection algorithms were implemented in C++ using OpenCV. OpenCV is an open-source computer vision and machine learning library.

Moving Vehicles detection was done using background subtraction and foreground motion estimation, which is a widely used technique for detecting moving vehicles such as in (Rad et al., 2005; Wan et al., 2014). Many algorithms have been examined which are Gaussian Mixture-based Foreground Segmentation Algorithms and the K-nearest neighbors (KNN) background subtraction algorithm. The used algorithm is (KNN) K-Nearest Neighbors background subtraction which is described in details in (Zivkovic and van der Heijden, 2006). KNN was chosen based on local experiments by the researcher and the result of previous studies by other researchers such as Trnovszký et al. (2017). Shadows of moving objects are detected and identified (marked in gray color) then eliminated. Lane change is also detected and recorded. Also, vehicles that come to full stop are identified. Image processing was used to extract primary information. Then, additional processing was done on the extracted data using R.

Also, the pedestrian signal wasn’t easy to read since the indication of walk and don’t walk has the same location. Image classification methods were used. A Machine learning process was developed to teach the code to distinguish the shape since the color and its brightness wasn’t a stable factor due to the open environment of the experiment. Traffic signal status detection was less complex. Simply, it was based on the location of brightness within the signal head.
Semi-automated method was used to identify the initial queued vehicles which involved manual validation work at the start of each cycle. Pedestrian illegal crossings during vehicular green time are captured (running red light). Cameras were calibrated using measured distance between road marks in the field. The models were trained on selective clips (from original film) that represent all the brightness condition and level of crowdedness.
Figure 4-5 Original Views of Cameras, Motion Detection and Edges Detection.
The first and most important step is synchronizing the videos. Since not all cameras have the same FPS (Frame per second), the synchronization process was based on concurrently observing the same event on all cameras in addition to FPS. Cameras (2, 3, 4 and 5) had a master clock that has the real time printed on each frame. However, the frame rate for the videos wasn’t consistent. Thus, optical character recognition based on artificial neural networks was
used to extract the real time of each frame (figure 2.6). However, Camera #1 is filmed at constant 60 fps and that is the camera used to extract speed. Information captured by image processing was coded into many variables as described below.

1. Speed

Cameras were calibrated using measured distance between road marks in the field.

Vehicles’ speed is measured between two fixed points that are 29 ft apart downstream of the stopline. The main source video (video) was filmed with 60 fps which is the one used to extract vehicles speed (camera #1). The resolution of this method is based on number of frames per second, trap length and vehicles speed. In this study, speed is measured with precision, $\pm \frac{\text{Speed}^2}{\text{Trap dist} \times \text{FPS}}$, around $\pm 1$ mph (average precision of speed between 25 mph to 35 mph is 1.1 mph)

\[
\text{Measured Speed} = \frac{\text{Trap length } \pm \text{ Travel Dist per a frame}}{\text{Travel time}} = \frac{\text{Trap length } \pm \frac{\text{Speed}}{\text{FPS}}}{\text{Travel time}}
\]

\[
= \frac{\text{Trap length } \pm \frac{\text{Speed}}{\text{FPS}}}{\text{Speed}} = \text{Speed } \pm \frac{\text{Speed}}{\text{FPS}}
\]

\[
\text{Precision} = \pm \frac{\text{Speed}}{\text{Trap.length}} = \pm \frac{\text{Speed}^2}{\text{Trap length } \times \text{FPS}}
\]
2. Headway(H)

Headway is measured within a cycle and within a lane as the following:

\[
Headway_{ijl} = \begin{cases} 
X_{ijl} - G_j, & i = 1 \\
X_{ijl} - X_{i-1,jl}, & i \geq 2 
\end{cases}
\]

(Equation 2-1)

Where j is cycle ID, l is lane ID, G_j is the green start time for cycle j, and X_{ijl} is the time when vehicle i at lane l at cycle j reaches the stopline (Figure 4-8). Vehicles are numbered sequentially (i=1,2,3,...etc). Thus, for the first vehicle in a cycle to pass the stopline, its headway is the time from start of green until reaches the stopline.

![Figure 4-8 Notations](image)

**Corrections:**

- If vehicle 1 is part of the initial queue which was stopped at the start of green, then its Headway is set to zero.
• If pedestrians cross illegally during the vehicle green time, the headway of approaching vehicles is modified to equal the time difference between (the time the vehicle reaches the stopline) and (the time the pedestrian clears the crosswalk).

3. **Green Timer (GT):** the time since the green interval starts (starts from zero each time a new green interval starts).

4. **Initial queue (IQ):** A dummy variable for the vehicles that were part of the initial queue (vehicles was stopped at green start) **1 queued, 0 otherwise.**

5. **Next lane Headway (NLH):** Headway between the vehicle and the most immediately preceding vehicle in the other lane.

6. **Downstream Link Status (DLS):** is a dummy variable, 1 if the lane is blocked and 0 otherwise. The downstream link blocked if stopped vehicles inhibit the flow of traffic near the subject intersection. Stopped vehicles could be a long queue at the following red light, or making curbside pickup/dropoff.

7. **Upstream Link Status (ULS):** 1 if any kind of blocking to the discharge flow happened, such as a car completely stops for picking up.

8. **Turning vehicles:** is a dummy variable that indicate whether a vehicle turn at the subject intersection, including U-turns.

9. **Night:** is a dummy variable for night time (1 night, 0 daylight). In our case performed [Nov-16-2018] study, daylight is defined to be the period between 6A.M -5P.M.

10. **Day Periods:** These are three dummy variables that represent different levels of traffic flow and, which are AM Peak, Mid-day, PM peak, and off peak. The selection of AM peak period and PM peak were chosen based on the existing signal timing plan (Table 4-2). The, Mid-day period was introduced /chosen because traffic volume observed then was significantly
different from night time traffic. The reminding time of the day that was not included in previous groups is called off-peak.

Table 4-2 Day Periods Classification

<table>
<thead>
<tr>
<th>Day Periods</th>
<th>Hours</th>
</tr>
</thead>
<tbody>
<tr>
<td>off peak</td>
<td>7 PM to 6AM</td>
</tr>
<tr>
<td>Mid-day</td>
<td>10AM-3PM</td>
</tr>
<tr>
<td>AM Peak</td>
<td>6:00 AM -10:00 AM</td>
</tr>
<tr>
<td>PM peak</td>
<td>3:00 PM -7:00 PM</td>
</tr>
</tbody>
</table>

11. Experienced Average Discharge Rate (EADR)

Drivers only experience or observe the part of the green that is ahead of them. Therefore, we coded a variable that captures the level of congestion experienced by each driver individually which is called experienced average discharge rate(EADR), define as is the number of vehicles that cross the stop line ahead of the subject driver during the current green interval, divided by how long the signal has been green when the subject vehicles cross the stop line:

\[ EADR = \frac{i - 1}{TOGi} \]

Where, \( i \) is the ordinal number of the crossing vehicle in a green interval and \( TOGi \) is the elapsed green time when vehicle \( i \) crosses the stop line.
4.5 Exploratory Data Analysis

In this section the relation between speeding and traffic-related variables is explored. The data was collected from Nov-16-2018 (9 AM) to Nov/17/2018 (9 AM). Activity from 3 A.M to 6 A.M is excluded because the signal was in yellow flashing mode. The observed speed distribution is shown in Figure 4-9 and numerical summary of the figure is shown in Table 4-3. Speed was measured immediately after the stop line. The study excluded data from blocked cycles, even if the blocking was for one lane. There are two cases where a cycle is defined as blocked cycle. First, whenever a car completely stops at any lane when it is supposed to be moving. Second, is when downstream queue spillback blocks the discharge flow. The used sample size in the research is 6,882 Vehicles on 668 cycles.

The speed limit in the road is 25 MPH. The data shows that 52% of drivers exceeded the posted speed limit. 85 percentile speed is 32 MPH. Excessive speeding of $\geq 35$MPH represents 93\textsuperscript{th} percentile of the observed speeds.
Table 4-3  Numerical Summary of Figure 4-9

<table>
<thead>
<tr>
<th>Value</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Mean</strong>, (mph)</td>
<td>23.5</td>
</tr>
<tr>
<td><strong>Standard Deviation (SD)</strong>, (mph)</td>
<td>8.48</td>
</tr>
<tr>
<td><strong>Median</strong>, (mph)</td>
<td>25</td>
</tr>
<tr>
<td><strong>85 Percentile</strong>, (mph)</td>
<td>32</td>
</tr>
<tr>
<td><strong>25 MPH (Posted speed)</strong></td>
<td>0.51</td>
</tr>
<tr>
<td><strong>30 MPH</strong></td>
<td>0.79</td>
</tr>
<tr>
<td><strong>34.99 MPH</strong></td>
<td>0.93</td>
</tr>
</tbody>
</table>

The thick left tail of the density plot in Figure 4-9 is mainly due to vehicles that are part of initial queue, i.e., vehicles stopped behind the stopline at the start of the green phase. Figure 4-10 shows a two-density plot of initial queued vehicles and the remaining traffic. The data shows that initial queued vehicles are never excessive speeders (as expected).

Figure 4-10  Density Plots of Vehicles Speed (both initial queues and the rest of the traffic)
Table 4-4  Numerical Summary of Figure 2-10

<table>
<thead>
<tr>
<th></th>
<th>(Initial queue)</th>
<th>(Otherwise)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Mean</strong> (mph)</td>
<td>13</td>
<td>27.2</td>
</tr>
<tr>
<td><strong>Standard Deviation (SD)</strong>, (mph)</td>
<td>4.76</td>
<td>5.99</td>
</tr>
<tr>
<td><strong>Median</strong>, (mph)</td>
<td>13</td>
<td>28</td>
</tr>
<tr>
<td><strong>85 Percentile</strong>, (mph)</td>
<td>18</td>
<td>33</td>
</tr>
<tr>
<td><strong>Percentile</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Posted speed (25 MPH)</td>
<td>0.99</td>
<td>0.33</td>
</tr>
<tr>
<td>30 MPH</td>
<td>1</td>
<td>0.72</td>
</tr>
<tr>
<td>34.99 MPH</td>
<td>1</td>
<td>0.90</td>
</tr>
</tbody>
</table>

Excessive Speeding (>=35mph) occurs at all times of the day (Figure 4-12 and 4-13).

However, the likelihood of occurrence varies at different times of the day. The risk of speeding is the highest in the night time but the number of speeders is the highest in AM peak. The correlation between hourly volume and the percentage of excessive speeder is - 0.77.

Interestingly, the correlation between the number of speeders and hourly volume is found to be 0.58.

Figure 4-12 hourly volume and the percentages of hourly observed excessive speeder (speed >=35mph).

Figure 4-11 Speed Distribution for Night and Daylight (6A.M -5P.M)
Figure 4-13 Speed Distribution per Hour of Day

Figure 4-14 Number of Excessive Speeder per Hour.

Figure 4-15 Number of Observed Excessive Speeders per Night and Daylight Time per Day
Green interval length for the observed phase is not fixed. The average length is 40 second with standard deviation equal to 9.1. The proportion of speeders increases with elapsed time since green starts (Figure 4-16). However, this is a biased estimation considering the effect of queued vehicles on early green time of a phase.

With the initial queue excluded, as time passes after the green interval starts, the data shows a gradual increase in the proportion of excessive speeders, and at the same time there is a gradual decrease in the proportion of vehicles driving at or below the speed limit (Figure 2-17).

Figure 4-16 Speed distribution over green time including proportion of initial queue
There is a well-known belief that excessive speeding occurs during the yellow period more than other periods. 6% of excessive speeding occurs during yellow period in this case study (Figure 4-19). Of the 195 vehicles crossing at yellow time, 13% were excessive speeders
When a pedestrian phase runs concurrently with a vehicular phase, the green interval is at least equal to pedestrian minimum (WALK + Ped Clearance - Yellow). The minimum pedestrian green interval is equal to 23 sec for the pedestrian phase that run concurrently with the observed vehicular phase. However, the green length on average is 40 sec with standard deviation equal to 9s. Thus, all time after 23 sec is not a safety requirement.

Based on the observed data, 56% of excessive speeding happened after the minimum pedestrian green interval (Table 4-5), even though only 34.6% of all vehicles passed after the pedestrian minimum green. Including the initial queue, the odds of being an excessive speeder when passing the stopline after versus during the ped minimum green is 2.1:1.

Table 4-5 The percentage of observed excessive speeders after and during the minimum pedestrian green interval (23s)

<table>
<thead>
<tr>
<th></th>
<th>Otherwise</th>
<th>Time &gt;23</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Sp &gt;=35</strong></td>
<td>44% (182Veh)</td>
<td>56% (233Veh)</td>
<td>415 (100%)</td>
</tr>
</tbody>
</table>
Generally, headway has a positive relationship with speeding; however, this relation is not constant or linear. Figure 2-20 shows that higher speed is associated with higher headway, which is expected. 62% of observed excessive speeders experienced a headway of 5 seconds or higher. 83% of vehicles traveling within the speed limit experienced headway less than 5s.

![Cumulative Density Plot of Headway for Different Speed Groups.](image)

Headway is an essential variable influencing car following behavior and it is used to discriminate between unconstrained and constrained vehicles (Poe et al., 1996; Fitzpatrick et al., 2003; Pesti and Brewer, 2006; Hashim, 2011). In this research, a threshold of 5 seconds is adopted to discriminate between the two types of vehicles and also. 7% of constrained cars were excessive speeders while 16% of unconstrained cars were excessive speeders (Table 4-6). Thus, having a long Headway (>5s) more than doubles the risk of excessive speeding, compared to having a short headway(≤5s). To conclude, headway clearly influences excessive speeding.
behavior. The results show that speeding is associated with large headway, although large headway doesn’t always lead to speeding, and speeding can also occur with shorter headway.

Table 4-6 Unconstrained Vs Constrained Vehicles (Excluding Initial Queue)

<table>
<thead>
<tr>
<th>Percentile</th>
<th>H &gt; 5 (Unconstrained)</th>
<th>Otherwise (constrained)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean, (mph)</td>
<td>28</td>
<td>26</td>
</tr>
<tr>
<td>Standard Deviation, (mph)</td>
<td>6.49</td>
<td>5.58</td>
</tr>
<tr>
<td>Median, (mph)</td>
<td>29</td>
<td>27</td>
</tr>
<tr>
<td>85 Percentile, (mph)</td>
<td>35</td>
<td>32</td>
</tr>
<tr>
<td>Percentile</td>
<td>Posted speed (25 MPH)</td>
<td>0.27</td>
</tr>
<tr>
<td></td>
<td>30 MPH</td>
<td>0.62</td>
</tr>
<tr>
<td></td>
<td>34.99 MPH</td>
<td>0.84</td>
</tr>
</tbody>
</table>

Table 4-7 The Percentage of Constrained and Unconstrained Speeders.

<table>
<thead>
<tr>
<th>&gt;=35 mph</th>
<th>Unconstrained</th>
<th>Constrained</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>252 (61%)</td>
<td>163 (39%)</td>
<td>415 (100%)</td>
</tr>
</tbody>
</table>
The relationship between the level of congestion and speeding is expected to be negative based on basic traffic flow theory. Level of congestion indicators are usually aggregated by time such as, hourly volume and time-of-day periods (e.g. peak hour, mid-day and off peak). A Measure such as VPGS (Vehicle per green second) is an aggregate measure per cycle, proportional to volume/capacity ratio or degree of saturation in a given cycle. However, this is still an aggregate measure; drivers only experience or observe the part of the green that is ahead of them. Therefore, we coded a variable that captures the level of congestion experienced by each driver individually which is called experienced average discharge rate (EADR) which is defined in section 4.4.3. As expected, the relation between EADR and excessive speeding is found to be negative (Figure 4-21).

![Figure 4-21 Cumulative Density Plot of EADR for Different Speed Groups.](image)
4.6 Statistical Model

The detailed output of image processing analysis includes many variables that will be included in the process of developing the model are shown in the table below.

<table>
<thead>
<tr>
<th>Variables</th>
<th>Units</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Headway (H)</td>
<td>s</td>
<td>Continuous</td>
</tr>
<tr>
<td>Next lane Headway (NLH)</td>
<td>s</td>
<td>Continuous</td>
</tr>
<tr>
<td>Hourly Volume (HV)</td>
<td>Veh/h</td>
<td>Continuous</td>
</tr>
<tr>
<td>Day/night (DN)</td>
<td>-</td>
<td>Binary</td>
</tr>
<tr>
<td>Day Period (DP)</td>
<td>-</td>
<td>Categorical</td>
</tr>
<tr>
<td>Yellow Dummy (YT) (1 if cross stopline during yellow)</td>
<td>-</td>
<td>Binary</td>
</tr>
<tr>
<td>Green interval Length (GT)</td>
<td>s</td>
<td>Continuous</td>
</tr>
<tr>
<td>Cycle Length (CL)</td>
<td>s</td>
<td>Continuous</td>
</tr>
<tr>
<td>Green Timer (GTT) (time since green start)</td>
<td>s</td>
<td>Continuous</td>
</tr>
<tr>
<td>Number of turning vehicles per cycle (NOTVC)</td>
<td>Veh/cycle</td>
<td>Continuous</td>
</tr>
<tr>
<td>Number of vehicles per cycle (NOVC)</td>
<td>Veh/cycle</td>
<td>Continuous</td>
</tr>
<tr>
<td>Initial queue (1 if vehicle belongs to initial queue)</td>
<td>-</td>
<td>Binary</td>
</tr>
<tr>
<td>NOVC / GT</td>
<td>Veh/s</td>
<td>Continuous</td>
</tr>
<tr>
<td>EADR</td>
<td>Veh/s</td>
<td>Continuous</td>
</tr>
</tbody>
</table>

HV is constant for every hour. GT, NOTVC and NOVC vary from cycle to cycle, applies to the cycle in which the car passed the stopline.

4.6.1 Logit Regression

The model form that will be used for the evaluation of the explanatory variables and prediction for this study is the logit regression. Logit regression, also called logistic regression, is used to describe the relationship between a dichotomous response variable and a set of explanatory variables. The explanatory variables can be either continuous, discrete (etc. dummy variables), or a mixture of both.
Logit regression is used to investigate excessive speeding as a function of signal timing and traffic flow interaction. In our case, the response variable is ES (Excessive Speeding) which can only take on two values, ES= 1 for SPEED >=35 mph, and ES = 0 for otherwise. The logit function, a non-linear function, is used to model the probability of excessive speeding.

\[ P(ES = 1|X) = \frac{e^{U(x)}}{1 + e^{U(x)}} \]

U(x) represents the utility function of the excessive speeding decision given by

\[ U(X) = \beta_0 + \beta_1 x_1 + ... + \beta_i x_i = \beta X \]

Where
\[
\begin{align*}
\beta &= \{\beta_0, \beta_1, \ldots \ldots, \} \text{ = a vector of regression coefficients} \\
X &= \{1, X_1, X_2, \ldots \ldots \} \text{ = a vector of explanatory variables.}
\end{align*}
\]

With a logit model, the odds of an event occurring can be expressed as in equation 4-1.

Maximum likelihood is the method used to fit the logit model. Also, logit regression can be seen as a method to model the log odds of event as linear function.

\[ Odds(ES = 1|X) = \left[ \frac{P}{1-P} \right] = e^{\beta X} \quad (4-1) \]

\[ log \left[ \frac{P}{1-P} \right] = \beta X \quad (4-2) \]

The data that will be used to build and assess the model, will not include vehicles that are either part of initial queues or turning vehicles (Figure 4-22). As the previous descriptive analysis shows, the probability that a vehicle will be going at excessive speed is zero given the vehicles are part of initial queue or are turning vehicles (Section 4.5). However, variables such as the number of turning vehicles per cycle and length of initial queue were part of the model.
development process. The total number of the data feeds to the model are 4104 events with 415 excessive speeding events.

A dataset with a binary response variable is considered imbalanced if one of the binary events is much less frequent than the other. As in the case of the data used in this analysis, roughly 10% of the vehicles are excessive speeders. In case of imbalanced class in a dataset, an overall accuracy of logit model can be misleading because a naïve model of classification, which classify everything to the more common response, will have high accuracy. The Naïve model of classification has accuracy of 90%. In such case, the classification evaluation should evaluate the accuracy based on each class, which is known as sensitivity and specificity. **Sensitivity** of a model is the probability of predicting an event when the event is occurring (true positive rate). **Specificity** is the probability that the model predicts an event as a non-occur when an event is a non-occur (true negative rate).
Many different models were tested; four models are selected and shown below in the tables (4-9, 4-10, 4-11 and 4-12)

**Table 4-9 Model A**

<table>
<thead>
<tr>
<th>Variable</th>
<th>Estimate</th>
<th>SE</th>
<th>Z-Value</th>
<th>P-Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intercept</td>
<td>-2.69</td>
<td>0.091</td>
<td>-29.365</td>
<td>&lt;2e-16</td>
</tr>
<tr>
<td>Headway(s)</td>
<td>0.068</td>
<td>0.006</td>
<td>10.535</td>
<td>&lt;2e-16</td>
</tr>
<tr>
<td>NextLaneHeadway(s)</td>
<td>0.006</td>
<td>0.008</td>
<td>0.764</td>
<td>0.445</td>
</tr>
<tr>
<td>Yellow Dummy</td>
<td>0.299</td>
<td>0.230</td>
<td>1.299</td>
<td>0.194</td>
</tr>
<tr>
<td>Turning Veh (Veh/Cycle)</td>
<td>-0.118</td>
<td>0.099</td>
<td>-1.188</td>
<td>0.235</td>
</tr>
</tbody>
</table>

| BIC                       | 2615.249 |
| Nagelkerke R²             | 0.055    |
| \( LL(\beta) \)           | -1286.826|
| \( LL(\theta) \)          | -1344.219|

**Table 4-10 Model B**

<table>
<thead>
<tr>
<th>Variable</th>
<th>Estimate</th>
<th>SE</th>
<th>Z-Value</th>
<th>P-Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intercept</td>
<td>-1.86</td>
<td>0.169</td>
<td>-11.02</td>
<td>&lt;2e-16</td>
</tr>
<tr>
<td>Headway (s)</td>
<td>0.038</td>
<td>0.007</td>
<td>5.21</td>
<td>1.85e-07</td>
</tr>
<tr>
<td>GT (s)</td>
<td>0.012</td>
<td>0.004</td>
<td>2.78</td>
<td>0.005</td>
</tr>
<tr>
<td>EADR (Veh/s)</td>
<td>-3.109</td>
<td>0.388</td>
<td>-7.99</td>
<td>1.26e-15</td>
</tr>
</tbody>
</table>

| BIC                       | 2542.48  |
| Nagelkerke R²             | 0.088    |
| \( LL(\beta) \)          | -1254.603|
| \( LL(\theta) \)         | -1344.219|
Table 4-11 Model C

<table>
<thead>
<tr>
<th>Variable</th>
<th>Estimate</th>
<th>SE</th>
<th>Z-Value</th>
<th>P-Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intercept</td>
<td>-2.078</td>
<td>0.176</td>
<td>-11.8</td>
<td>&lt;2e-16</td>
</tr>
<tr>
<td>(Headway &gt;5) Dummy</td>
<td>0.788</td>
<td>0.113</td>
<td>6.9</td>
<td>3.13e-12</td>
</tr>
<tr>
<td>GT (s)</td>
<td>0.016</td>
<td>0.004</td>
<td>3.9</td>
<td>8.66e-05</td>
</tr>
<tr>
<td>EADR (Veh/s)</td>
<td>-3.132</td>
<td>0.375</td>
<td>-8.3</td>
<td>&lt;2e-16</td>
</tr>
<tr>
<td>BIC</td>
<td>2519.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nagelkerke R²</td>
<td>0.099</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\[ LL(\beta) = -1243.2 \]
\[ LL(0) = -1344.219 \]

Table 4-12 Model D

<table>
<thead>
<tr>
<th>Variable</th>
<th>Estimate</th>
<th>SE</th>
<th>Z-Value</th>
<th>P-Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intercept</td>
<td>-2.70</td>
<td>0.080</td>
<td>-33.46</td>
<td>&lt;2e-16</td>
</tr>
<tr>
<td>(Headway &gt;5) Dummy</td>
<td>1.10</td>
<td>0.106</td>
<td>10.41</td>
<td>&lt;2e-16</td>
</tr>
<tr>
<td>BIC</td>
<td>2593.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nagelkerke R²</td>
<td>0.055</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\[ LL(\beta) = -1288.4 \]
\[ LL(0) = -1344.219 \]

The models are built on 3689 controls (Non-Excessive Speeder) and 415 cases (ES). Model A relates excessive speed probability to headway, headway to vehicles in next lane, yellow and number of turning vehicles. Model B relates the excessive speeding probability to the headway, green timer and experienced average discharge rate. Model C is like model B, except that headway is treated as a dummy variable (greater than 5 s or not). Finally, Model D relates excessive speeding probability to only the headway more than 5 sec (as dummy variable), which we have defined in other research (see Ch.3) as a nominal speeding opportunity.

A P-value greater than 0.05 indicates that the null hypothesis of no relationship between the predictor and a variable cannot be rejected. The results of Model A indicate that Headway is
the only variable that is statistically significant. The results of Model B, C and D indicate that all used variables are statistically significant.

Bayesian information criterion (BIC), also called the Schwarz Bayesian Criterion, is an evaluation criterion that measures the trade-off between model fit and complexity of the model. BIC penalizes a model by the number of parameters included. A model with lower BIC values is preferred. In the conducted experiment, Model C has the lowest BIC. Also, the highest Nagelkerke's R-square is for model C. Nagelkerke's R-square (Pseudo R-Squared) has a range from 0 to 1, where the higher the number the better the model.

### 4.6.1.1 The Model

Based on the various metrics reported above, which are Nagelkerke R-squared, the Bayesian Information Criterion (BIC) and log-likelihood, Model C is the best model in term of describing and fitting the data and therefore model C is the chosen one.

**Table 4-13 coefficient, odds ratio and mean value for Model C explanatory variables**

<table>
<thead>
<tr>
<th>Model B</th>
<th>coefficient</th>
<th>odds ratio</th>
<th>Mean (std)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Headway &gt;5) Dummy</td>
<td>0.788</td>
<td>2.198</td>
<td>0.36 (0.48)</td>
</tr>
<tr>
<td>GT (S)</td>
<td>0.016</td>
<td>1.016</td>
<td>23.33 (11.58)</td>
</tr>
<tr>
<td>EADR (Veh/S)</td>
<td>-3.132</td>
<td>0.043</td>
<td>0.33 (0.17)</td>
</tr>
</tbody>
</table>

Also, Headway mean is 06.00 and standard deviation is 06.12.

The results of model C show that for every one-unit increase in green timer(GT), we expect 0.016 increases in the log-odds of excessive speeding, holding all other independent variables constant. The coefficient for the Headway dummy variable is equal to 0.788, which is
equal to the expected change in log odds when headway goes from under 5s to over 5s, holding all other independent variables constant.

The odds ratio of a variable can be calculated by exponentiating its coefficient. Model C indicates that just having headway more than 5 sec increases the odd of being excessive speeder by 119%. Also, an increase in green timer (GT) variable by 10s would increase the odds by 16%. Finally, if EADR increases by 0.33 (e.g. from the mean value to twice the mean), that would decrease the odds of being excessive speeder by 1.4%.

4.6.1.1.1 MODEL PREDICTION

As mentioned before the chosen model is model C (Table 4-11,4-13). Figure 2-23 shows the receiver operating characteristics (ROC) curve of the model on the entire data set. The ROC curve is a popular measure to evaluate and visualize the performance of a classifier. The ROC curve plots the sensitivity and specificity. Each point on the curve represents the sensitivity and specificity for a specific decision threshold (cutoff value).
The area under the ROC curve (AUC) shows how well the model classifies each of the binary decision classes, and can be used to compare the performance between models. AUC of model C is 0.707. AUC = 1 means a perfect model and 0.5 means a model that is no better than a random model prediction.

![Figure 4-23 ROC curve of Model C](image)

A logit model output is probability; prediction is done by choosing a threshold value to classify the model output. The cutoff probability or “threshold value” of 0.5, which minimizes two types of errors, is common, but it is not suitable for an imbalanced dataset. For imbalanced data, there is a rule of thumb that the optimal cutoff value will be in the range of the proportion of positive class (ES=1) in the training data set. In this analysis, the goal is to minimize both type 1 and type 2 error, i.e. maximizing both sensitivity and specificity. The optimal cutoff probability is found to be 0.1. The prediction of the model is shown in table 4-14.
Table 4-14 Classification Result of Model C.

<table>
<thead>
<tr>
<th>Predicted</th>
<th>Sp&gt;=35 mph</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No</td>
</tr>
<tr>
<td>Observed</td>
<td>Sp&gt;=35 mph</td>
</tr>
<tr>
<td></td>
<td>Yes</td>
</tr>
</tbody>
</table>

4.6.1.1.2 MODEL VALIDATION

Model validation is important to guard against overfitting the dataset. Also, it is used to test the performance of the model on different datasets to determine if it can be generalized. Ideally, the validation process needs a different dataset than the one the model was calibrated on. In case of limited data availability, cross validation is used. Cross validation uses different data sets that are systematically chosen from the original data set to estimate and validate the model, thus providing a measure of a model performance and stability. The cross validation method that is applied is K-fold cross validation. In K-fold cross validation method, the data set is partitioned into k folds. Fold size will be nearly equal. The validation process is performed K times. Each time, k-1 subsets are used for model calibration and the remaining subset is used for model validation. The sampling process is random per class (Stratified Random Sample), meaning each class is partitioned into k subsets, in order to insure that each fold proportionally has the same representation as the original dataset.

The results of the application of K-Fold cross validation (with k=5) method are summarized in table 2-14 and table 4-15, for Model C.
The variability of coefficient of the variables is measured across all K-models to measure the model stability (Table 4-15). The table shows the minimum, maximum, standard deviation and coefficient of variance for each variable across all K-models. The largest CV was for green timer, which is 17%. The results indicate that the result doesn’t vary a lot from the original full model, which can be used as indication of stability and robustness.

Table 4-15 Results from the K-Fold Cross Validation for the Coefficient of each variable

<table>
<thead>
<tr>
<th>Variables</th>
<th>Min</th>
<th>Max</th>
<th>Mean</th>
<th>SD</th>
<th>CV</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \beta_0 ) (Constant)</td>
<td>-2.270</td>
<td>-1.943</td>
<td>-2.078</td>
<td>0.127</td>
<td>-0.06</td>
</tr>
<tr>
<td>( \beta_1 ) (Headway D)</td>
<td>0.734</td>
<td>0.874</td>
<td>0.789</td>
<td>0.064</td>
<td>0.08</td>
</tr>
<tr>
<td>( \beta_2 ) (GT)</td>
<td>0.013</td>
<td>0.020</td>
<td>0.016</td>
<td>0.002</td>
<td>0.17</td>
</tr>
<tr>
<td>( \beta_3 ) (EADR)</td>
<td>-3.502</td>
<td>-2.895</td>
<td>-3.136</td>
<td>0.248</td>
<td>-0.07</td>
</tr>
</tbody>
</table>

Figure 4-24 ROC curves of K-fold models with red line showing ROC curve of the original C model with entire data set.
The mean AUC for the K models is 0.70 with standard deviation of 0.034. The models don’t vary much from the original full model.

### Table 4-16 AUC results of the K-fold cross validation

<table>
<thead>
<tr>
<th>Variables</th>
<th>Min</th>
<th>Max</th>
<th>Mean</th>
<th>STD</th>
<th>CV</th>
</tr>
</thead>
<tbody>
<tr>
<td>$AUC$</td>
<td>0.666</td>
<td>0.745</td>
<td>0.705</td>
<td>0.034</td>
<td>0.04</td>
</tr>
</tbody>
</table>
4.6.1.2 Reduced Model

As mention before, there is a number of research that define headway of 5 secs as threshold value of non-constrained vehicles (Poe et al., 1996; Fitzpatrick et al., 2003; Pesti and Brewer, 2006; Hashim, 2011). Thus, the adopted definition of “nominal speed opportunity” is the situation when a vehicle is approaching the stopline when the signal is green and its headway with respect to the vehicle ahead of it in the same lane is greater than 5 s. In this section, a reduced model that has only a dummy variable of Headway >5 or not is tested.

![Cumulative Speed Distribution](image)

*Figure 4-25 The Cumulative Speed Distribution of Constrained and Non-Constrained Vehicles*

Figure 4-25 show The speed distribution of constrained and non-constrained vehicles (excluding queued and turning vehicles). The mean speed difference between constrained and non-constrained vehicles is 2.2 mph. Statistical test was conducted to test the null hypothesis that the mean speed of constrained and non-constrained vehicles is the same. The test showed that the
difference is highly significant (at alpha=5% level of significant) between the two groups (t-statistic=12.15, P-value < 2.2e-16)

The analysis shows that “nominal speeding opportunity” is a classifier that is better than naïve model. However, the performance is a fair classifier. It has an accuracy of classifying true positive and true negative equal to 61% and 66% respectively.

![CDF distribution of vehicles with speed >=35 mph against the rest of vehicles.](image)

**Table 4-17 classification result of nominal speeding opportunity**

<table>
<thead>
<tr>
<th>Observed</th>
<th>Predicted Sp&gt;=35 mph</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No</td>
<td>Yes</td>
<td>Total</td>
<td></td>
</tr>
<tr>
<td>Sp&gt;=35 mph</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No</td>
<td>66%</td>
<td>34</td>
<td>100%</td>
<td></td>
</tr>
<tr>
<td>Yes</td>
<td>39</td>
<td>61%</td>
<td>100%</td>
<td></td>
</tr>
</tbody>
</table>

The results of model D, which is one variable model, can be interpreted as being a driver with headway more than 5 sec will increases the odd of being speeder by 200 % (odd ratio =3).
The AUC of the model using whole data set is 0.63. However, to validate the model and test its stability, K-fold cross validation test was conducted. The model is quite stable based on comparing the AUC values of the five models.

Table 4-18 AUC results of the K-fold cross validation

<table>
<thead>
<tr>
<th>Variables</th>
<th>Min</th>
<th>Max</th>
<th>Mean</th>
<th>STD</th>
<th>CV</th>
</tr>
</thead>
<tbody>
<tr>
<td>AUC</td>
<td>0.60</td>
<td>0.66</td>
<td>0.63</td>
<td>0.023</td>
<td>0.04</td>
</tr>
</tbody>
</table>

The results indicated that drivers with nominal speeding opportunity are more than twice as likely to be excessive speeders. Moreover, 61% of the excessive speeders are drivers with nominal speeding opportunity (Table 4-7).
4.7 Discussion and Conclusions

Excessive speeding has great impact on road safety. It is responsible for high proportion of fatality and severe injuries that results from road crashes. This research has explored the influence of traffic signals on excessive speeding behavior on urban arterials.

The case study was conducted at an intersection on Huntington Avenue, Boston. Cameras were used to capture traffic activity and the signal status. Logit regression model was developed to explore traffic signal control and traffic flow on excessive speeding.

The model relates the excessive speeding probability to the headway, elapsed time and experienced average discharge rate. The main conclusions that can be drawn from this analysis of the relationship between excessive speed and both signal timing and traffic flow can be summarized as follow:

- A major findings of the descriptive analysis is that excessive speeding behavior is not restricted to any time of day.
- Excessive speeding never occurs during initial queue discharge time.
- Excessive speeding increases with elapsed green time.
- Headway is a significant variable that impacts excessive speeding. In our case, drovers with headway < 5s were more than twice as likely to be excessive speeder than drivers with a headway <= 5s.
- Low experienced average discharge rate (EADR) also leads to more speeding.
• (Headway > 5s) is a good predictor of excessive speeding risk, which confirms its use in the earlier research on speeding opportunities (Chapter 3).

• Then, based on the observed data, 56% of speeding happened after the minimum pedestrian green interval, even though only 34.6% of all vehicles passed after the minimum green. Shorter cycle length could limit high percentage of excessive speeders.

• Using traffic signal control to reduce the number of vehicles pass with long headway after green starts, will limit excessive speeding risk.
Chapter 5 : Conclusions and Future Work

5.1 Conclusions

This dissertation studied and highlighted the importance of traffic signal timing plan on traffic safety and urban livability. Three researches were conducted and each one had its own conclusion (see sections 2.13, 3.12 and 4.7).

Two major conclusions are drawn from all three studies. First, pedestrian and vehicular delay aren’t mutually competing, a smart signal operation strategy can reduce both delays. Second, signals control could limit speeding opportunities and improve traffic safety and urban livability with current technology but with different operation plans.

In this section, we provide key major findings of all three studies:

1. Adaptive walk interval logic can substantially reduce pedestrian delay with little or no impact to vehicular traffic.

2. The proposed adaptive control uses only information that can be detected by traffic signal control equipment without a need for additional special detectors.

3. In two-way arterial, lowering progression speed will have no/low impact on reducing speed opportunity unless cluster size was reduced.

4. Green interval length dispersion has positive relationship with excessive speeding. Recall and minimum green parameters can reduce Green interval length dispersion.
5. Zonal coordination, which is based on grouping arterial intersections into zones and the intersections in each zone are coordinated, shows a significant reduction on speeding opportunity without increasing network delay.

6. Traffic signal timing impacts traffic safety. Previous researches illustrated the impact of yellow and all red interval length on safety. However, this research shows that green interval length also affects safety by influencing excessive speeding.

7. The likelihood of having excessive speeders increase as the length of green interval increases.

8. Headway does influence excessive speeding behavior. In our case, more than 62% of vehicles that were define as excessive speeder had headway equal to 5 sec or higher.

5.2 Further work

Some recommendations for futures research:

1. Using the traffic signal control to limit the speeding opportunity was tested in a simulation environment. For future evaluation scenarios, the proposed methodology might be applied and its impact can be tested in real-world arterials.

2. In the second research, the focus was on the actuated coordinated control and its impact on creating/limiting speeding opportunity. For future evaluation scenarios, the impact of decentralized traffic signal controls, which provide a degrees of coordination between signals without the restriction of a common cycle (e.g. self-organizing control), on limiting speeding opportunity might be tested.
3. Comparison between the proposed adaptive walk interval and a demand responsive walk interval might be conducted.
REFERENCES


M. Brosseau, S. Zangenehpour, N. Saunier, L. Miranda-Moreno The impact of waiting time and other factors on dangerous pedestrian crossings and violations at signalized intersections: A case study in Montreal Transportation Research Part F: Traffic Psychology and Behaviour, 21 (2013), pp. 159-172


