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16. Abstract

Speeding on arterial roads can be reduced by timing traffic signals to minimize the number of speeding opportunities – occasions when a vehicle arrives at an intersection on a stale green with no vehicle ahead for at least five seconds. The “Safe Waves” approach to traffic signal timing aims to minimize speeding opportunities while still providing good two-way progression using short cycles, short coordination zones, pedestrian recall where pedestrian demand is moderate and undersized phases where it’s low, and offsets that prevent high-speed progression. A test on a suburban arterial found that with Safe Waves signal timing, the number of vehicles exceeding the speed limit fell by 75%, while arterial travel time increased by only 2 seconds per intersection. A simulation-based case study in another suburban corridor found that Safe Waves signal timing reduced speeding opportunities by more than 50%, while network delay per vehicle rose 2 s in the p.m. peak and 21 s in the a.m. peak. A web app, the *Safe Waves Analysis Tool* (SWAT), was developed to enable designers to visualize and count the speeding opportunities afforded by a proposed arterial signal timing plan.

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SAFE WAVES: SIGNAL TIMING GUIDE, ANALYSIS TOOL, AND CASE STUDIES

FINAL REPORT

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Disclaimer

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Executive Summary

This study of Safe Waves: Signal Timing Guide, Analysis Tool, and Case Studies was undertaken as part of the Massachusetts Department of Transportation (MassDOT) Research Program. This program is funded with Federal Highway Administration (FHWA) State Planning and Research (SPR) funds. Through this program, applied research is conducted on topics of importance to the Commonwealth of Massachusetts transportation agencies.

As more cities adopt Vision Zero principles, they feel a stronger need to reduce speeding on arterial roads. Unfortunately, physical traffic calming techniques involving horizontal and vertical deflection, while highly effective for speed control on local streets and streets with one lane per direction, cannot generally be applied on multilane arterials. This raises the question, can traffic signals be effectively used for speed management on multilane arterials? This study examines an approach to speed management based on timing traffic signals in a way that minimizes opportunities for speeding.

Furth and Halawani pioneered the concept of the relationship between arterial traffic signal timing and speeding opportunities (1). They defined "speeding opportunity" as the event of a vehicle arriving at an intersection on stale green and with a gap of at least 5 seconds to the vehicle ahead. They found that speeding opportunities measured in the field correlated closely with speeding opportunities measured using PTV VISSIM simulation. In a simulation environment, they tested different signal control plans on two urban arterials and found that some reduced speeding opportunities by 50% or more with little or no impact on average delay. A follow-up study of another urban arterial found that signal timing could reduce speeding opportunities by up to 50% with little or no increase in traffic delay and a nearly 70% reduction in pedestrian delay compared to traditional arterial signal timing (2).

These findings gave rise to a new approach to traffic signal timing called the Safe Waves approach: minimizing the number of speeding opportunities while still providing reasonably good arterial progression. has been called the Safe Waves approach to traffic signal timing. The techniques it uses include short cycles, short coordination zones, low progression speed, pedestrian recall, and fully actuated signal control.

The present study was initiated with three objectives that aim at advancing arterial speed management using signal timing.

The first objective was to develop a guideline for Safe Waves traffic signal timing. That guideline, which forms Chapter 2 of this report, explains the signal timing and coordination principles that relate to speeding opportunities, and offers a step-by-step procedure for timing traffic signals to minimize speeding opportunities. While past studies of Safe Waves signal timing on urban arterials with moderate or high pedestrian demand have identified the value of having pedestrian phases on recall (1), this study adds new guidance for intersections with low pedestrian demand. The technique that applies in this situation is for the pedestrian crossing to be on demand (i.e., not on recall), and to undersize the phase that serves the pedestrian movement. With this technique, also called the "oversized ped" technique, pedestrian service will force a phase to run beyond its allotted time in the cycle, triggering a recovery period in which queues that may have built up are served and the cycle gets back in sync with the background cycle. Both of the case studies done as part of this project showed that with undersized phases, cycle length could be substantially shorter, leading to fewer speeding opportunities and better coordination, and that the disruption caused by pedestrian service created no significant deterioration in arterial performance.

A second objective was to develop software that could evaluate the number of speeding opportunities afforded by a proposed arterial traffic signal timing plan. Until now, the only ways to measure speeding opportunities have been either to count them in the field – which can be done for an existing timing plan, but not for a proposed plan – or using microsimulation, which is too labor-intensive to be a routine part of developing traffic signal timing plans. A new app, the Safe Waves Analysis Tool (SWAT), was developed and has been implemented as a web-app, meaning users can run it without having to download an executable program.

SWAT's input is a Microsoft Excel file that users prepare containing information on road geometry, signal timing, and traffic flow. Preparing these inputs generally requires that users perform, in parallel, signal timing analysis using software such as Synchro to get saturation flow rates and actuated effective green times.

SWAT generates progression diagrams that enable users to visualize speeding opportunities as well as progression. It also counts and reports speeding opportunities by intersection as well as overall, and measures arterial delay. That will enable designers to test and refine proposed arterial signal timing plans as they search for one few speeding opportunities as well as low vehicle and pedestrian delay.

SWAT is based on a deterministic simulation with platoon dispersion that tracks individual vehicles along the arterial in one direction at a time. At entry points to the network, entry headways are uniform, equal to the inverse of the saturation flow rate. Where saturation flow rate varies within the signal cycle, SWAT applies the appropriate saturation flow rate for each interval; for example, for vehicles turning right onto the arterial from a side street, one arrival rate applies during the side street's green and another, assuming right turn on red, during its red interval. At exit points, vehicles are removed as uniformly as possible given the arrival pattern. Vehicles advance along the arterial following a platoon dispersion model adapted from Bonneson, Pratt, and Vandehey in which vehicles at the head of a platoon tend to go faster and those at the tail slower (3). Platoon dispersion affects downstream headways, and can therefore be an important factor in the creation of speeding opportunities.

SWAT accounts for midblock entries and exits, which affect speeding opportunities, because a vehicle entering midblock may become a speeding opportunity if it enters during a period of unsaturated green, and vehicles exiting create "holes" in the platoon that can become headways long enough to create a speeding opportunity. At intersections, it accounts for the "holes" created in the platoon when vehicles turn off; it also accounts for the finite length of exclusive turn lanes and the turn lane spillback they can cause. By comparison, Synchro, the most popular intersection analysis app used for signal timing in Massachusetts, lacks all of these elements of functionality in its progression diagrams and queue modeling.

The study's third objective was to conduct field test case studies in two corridors. The main purpose of the field test was to determine how speeding behavior changed when signals were retimed to reduce the number of speeding opportunities. Previous studies have all relied on simulation, which can measure number of speeding opportunities, but only a field test can measure actual chosen speeds. While the original plans were to conduct field tests in two corridors, it turned out that the signal control equipment in one corridor was incapable of running coordinated timing at several intersections, and so a field test was conducted only in the Route 114 corridor.

The stretch of Route 114 studied is in Danvers, MA, with six signalized intersections between (and including) Brooksy Village Drive and a southbound onramp to I-95 south. Route 114 has two thru lanes per direction and exclusive left turn lanes at every intersection where left turns are allowed. Average daily traffic is 26,158 vehicles per day. The corridor sees very little pedestrian activity, with arterial crossings at only two intersections, where there is demand for 4 or fewer crossings per hour.

Proposed signal timing plans for three weekday periods of the day were developed following the Safe Waves approach, with the following notable features. At the corridor's busiest intersection, an undersized phase associated with a low-demand pedestrian crossing was used, and the other intersection with a pedestrian crossing uses a concurrent crossing instead of (the existing) exclusive pedestrian phase. These changes allowed for the corridor as a whole to have cycle lengths of 66 s (AM peak) and 84 s (PM peak), in comparison with the existing cycle lengths of 120 s and 95 s. Offsets were chosen to promote progression at a speed at or below a target speed of 30 mph (chosen because the 85th percentile speed in the corridor is between 30 and 34 mph). It is the nature of two-way coordination that there are only a few discrete possibilities for progression speed for a given cycle length and intersection spacing; where the choice was, for example, between offsets that allowed for progression at 40 mph and progression at 20 mph, the offsets corresponding to 20 mph were chosen.

Analysis using SWAT indicated that the proposed signal timing plan would reduce speeding opportunities by more than 50% in the a.m. peak and midday, and by 29% in the p.m. peak, which is more congested and therefore has fewer speeding opportunities.

To test how this change in signal timing would affect actual speeds, speed data was collected using radar counters at four of the intersections, for both arterial directions. Overall, the number of vehicles exceeding the speed limit (40 mph) fell by 79%. Similar reductions were seen in the number of vehicles exceeding 45 mph (74%) and exceeding 35 mph (78%).

The change in travel time due to the new timing plan was measured using travel time data supplied by INRIX. For the day as a whole, arterial travel time increased on average by less than 2 s per intersection, with no increase observed in the p.m. peak period and increases of about 3 s per intersection in the a.m. and midday periods. These results are consistent with the change in arterial travel time predicted by SWAT.

While pedestrian demand in the corridor is low, reducing pedestrian delay is still valuable as a safety measure, because pedestrians are more likely to comply with signals when delay is reasonably short, and crossing a busy 4-lane arterial against the signal is risky. Averaging over the intersections with pedestrian crossings and over the three periods, delay for pedestrians crossing the arterial fell from 56 to 37 s, a reduction of 19 s.

In the second corridor studied, a simulation-based study was conducted because it was not possible to conduct a field study. The study area is State Route 16, Revere Beach Parkway, from the intersection with Lewis Street to the intersection with Washington Avenue, with nine signalized intersections in Everett and Chelsea, MA. The arterial is divided and has three through lanes per direction and exclusive left turn lanes wherever left turns are allowed. Average daily traffic is 50,331 vehicles/day. It has some intersections with moderate pedestrian demand, while pedestrian demand is very low (4 or less calls per hour) at several other intersections.

The Safe Waves signal timing plan that was developed that features undersized phases at two of the intersections with very low pedestrian demand; this technique lowered the necessary cycle length at one of these intersections from 180 to 90 s in the p.m. peak, and from 140 to 84 s in the a.m. peak. At the other intersection, the undersized ped technique likewise allowed a substantially lower cycle length. The result was that cycle length in the busiest part of the corridor could be lowered from 150 s to 90 s in the p.m. peak, and from 110 s to 84 s in the a.m. peak.

At the remaining intersections with pedestrian crossings, pedestrian service was put on recall, either because of moderate pedestrian demand or because the pedestrian recall would not require a longer cycle, but would lessen speeding opportunities. At the Washington Avenue intersection, where there is a wide median and therefore pedestrians have to make a two-stage crossing, crossing phases were coordinated using the left-turn overlap technique described in Lao and Furth, which lowered average pedestrian delay by 67%, from 140 s to 47 s (4).

From observing current operations and checking the signal control equipment, it was found that only one intersection in the corridor (at Spring Street) is running according to planned signal timing; at others, there were issues such as broken detectors and lack of coordination. Hence, two base cases were modeled: the actual existing operation, and the planned operation, following the timing plan that is on file, in which there are two coordination zones but also two intersections running free, one at the western edge of the study section and one in the middle.

It turns out that while intersection-level results can vary substantially between the two base cases, overall results vary little; that is, in terms of overall performance measures such as speeding opportunities, delay, and travel time, there is almost no difference between the existing operation and the currently planned operation.

Simulation analysis found that with the Safe Waves timing plan, compared to either of the base cases, the number of speeding opportunities corridor-wide fell by more than 50% in both the a.m. and p.m. period. In the p.m. period, the increase in average vehicle delay network-wide was less than 1.4 s per intersection, while in the a.m. peak it was 4.2 s per intersection. As mentioned before, average pedestrian delay for crossing the arterial at Washington Street fell from 140 s to 47 s; at the other intersections (averaging overall), average pedestrian delay for crossing the arterial fell from 64 s to 36 s, a reduction of 28 s.

In conclusion, Safe Waves signal timing was found to be effective at managing speed at little "cost" in terms of added vehicle delay, and with substantial reductions in pedestrian delay. In the field test, the number of speeding vehicles fell by about 75%, while average arterial delay per intersection increased by only 2 s, and average pedestrian delay fell by 18.5 s. The Safe Waves timing plan induced this reduction in speeding behavior by reducing the number of speeding opportunities by more than 50% in the a.m. and midday periods, and by 29% in the more congested p.m. peak period. In a simulation study in another corridor, Save Waves signal timing was found to reduce speeding opportunities by more than 50%, while vehicle delay increased by 2.7 s per intersection and average pedestrian delay fell by 95 s at the intersection with two-stage crossings and by 27.5 s, on average, at the other intersections.

An app was successfully developed for counting and visualizing the speeding opportunities associated with any proposed arterial signal timing plan. This can support a policy, if jurisdictions wish to impose it, of requiring that traffic signal timing analysis include speeding opportunities as a performance measure, and of incentivizing designers to find solutions with fewer speeding opportunities.

A guidance document was created to help designers create signal timing plans following the Safe Waves approach – that is, with few speeding opportunities, yet still providing reasonably good two-way progression. Together with the SWAT app, it is hoped that more and more signal timing plans that effectively manage arterial speeds and thereby improve safety will be developed and implemented.

The most pressing future research needs are for more field studies to explore how Safe Waves signal timing affects speeding opportunities and speeding behavior, and for additional development of the Safe Waves Analysis Tool, adding features that make it easier to use. Another area for future research is seeing whether Artificial Intelligence methods or adaptive signal control can create or result in Safe Waves signal timing plans more easily. Finally, further research is needed on the undersized phase technique to provide guidance on needed cycle length and needed splits, including the amount of slack needed, to ensure that queues quickly dissipate and that offsets quickly return to sync.

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1 Introduction

More and more, government agencies and citizens alike recognize that managing speed is critical to road safety and is a responsibility of the road owner as well as individual drivers. On local streets, agencies have found success in applying physical traffic calming measures that impose vertical or horizontal deflection on vehicles, such as speed humps and neighborhood traffic circles. However, multilane arterial roads are generally not amenable to that kind of physical treatment, because vertical deflection is too jarring for public transportation and interferes with emergency routes, and because on multilane roads, any horizontal deflection sharp enough to control speed introduces a high risk of sideswipe collision. Speed cameras have proven effective at curbing speeding, but many jurisdictions, including the Commonwealth of Massachusetts, do not permit their use, and in many other jurisdictions, their use is severely limited by law or policy.

Consequently, speed management on arterial roads remains a difficult challenge for road safety with few tools to address it. Multilane arterials are especially critical for speed management because they permit passing, making it particularly easy to speed during non-congested periods. It has long been recognized that traffic signals can help control speed. However, making signals turn red in an arbitrary, uncoordinated way is annoying and can engender rear-end collisions.

Recently, Furth et al. proposed a new approach to traffic signal timing that can be called the Safe Waves approach: timing signals to provide coordination but, at the same time, to limit the number of speeding opportunities the timing plan creates (1). In parallel, they introduced a definition for speeding opportunity: the event in which a vehicle arrives at an intersection on a stale green and with no vehicle ahead of it for 5 seconds. This definition makes speeding opportunities quantifiable: they can be readily observed and counted in the field or in microsimulation.

Furth et al.'s seminal research showed that with conventional arterial signal timing, which feature long coordination zones, long signal cycles, high progression speeds, and coordinated-actuated control in which slack time goes to the arterial thru phase, there can be a lot of speeding opportunities – hundreds or even thousands per hour on an arterial. They also showed that by timing traffic signals differently, with shorter coordination zones, shorter cycles, lower progression speeds, and pedestrian recall, the number of speeding opportunities could be drastically reduced while still providing good progression for through traffic, with little or no deterioration in performance in terms of average vehicle delay. This alternative approach to signal timing is called the *Safe Waves* approach.

1.1 Project Objectives and Roadmap

This project has three objectives.

The first purpose is to develop a guide for timing traffic signals following the Safe Waves approach. That guide, provided as **Chapter 2**, also explains the concept of speeding opportunities, and explains the factors that create or inhibit speeding opportunities in a signalized corridor. It also summarizes studies that have tested the effectiveness of Safe Waves signal timing.

The second purpose is to develop an app to evaluate arterial signal timing plans in terms of speeding opportunities using standard signal timing data. Until now, while speeding opportunities can be counted in the field, there has been no way to evaluate a proposed signal timing plan in terms of speeding opportunities short of developing a full-scale microsimulation. The newly developed app, named the Safe Waves Analysis Tool or SWAT, is freely available and runs on a server at Northeastern University. It is described in **Chapter 3**, and **Chapter 4** is its User Manual.

As inputs, SWAT uses timing and flow information that is reported by commonly used signal timing apps such as Synchro. For output, it provides both a numerical evaluation (i.e., how many speeding opportunities per hour does this timing plan create) and a visualization – high-fidelity time-space progression diagrams in which speeding opportunities are clearly identified, which can be valuable for refining the timing plan. With an app like this, designers can apply their creativity to crafting signal timing plans that minimize speeding opportunities while still performing acceptably in terms of traditional metrics (capacity, delay, travel time, queues, and so forth).

The third purpose is to conduct field studies of the Safe Waves approach and test its effectiveness. Two corridors were selected. One is a 6-intersection stretch of Route 114 in Danvers, an undivided 4-lane road with a speed limit of 40 mph. The other is a 9-intersection stretch of Rt 16 in Everett and Chelsea, also called Revere Beach Parkway, a 6-lane divided road with speed limit 35 mph.

Chapters 5 and 6 describe how the Safe Waves approach was applied to the case study corridors. Their development involves some innovative techniques including peak hour factors based on the busiest 30 minutes (instead of the busiest 15 minutes, which is the industry norm), using recall for frequently called pedestrian phases, using undersized, on-demand phases for long but infrequently called pedestrian crossings (colloquially called the "oversized ped" technique), and providing progression for pedestrians through multistage crossings.

Chapter 7 has case study results. To confirm the validity of the SWAT model, SWAT estimates of number expected speeding opportunities were compared to counts of speeding opportunities measured in the field and, where field measurement wasn't possible, to number speeding opportunities measured using microsimulation.

A second set of results compares existing signal timing to Safe Waves signal timing in terms of speeding opportunities, speeding behavior, delay, and arterial travel time. Where possible, comparisons were made using field data; where constrained by limitations of the signal control equipment, some comparisons were made using microsimulation results.

Chapter 8 contains the conclusions and a description of future research needs.

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2 Guidance for Safe Waves Traffic Signal Timing: Minimizing Speeding Opportunities while Providing Good Arterial Progression

2.1 Introduction: Speeding Opportunities and Why They Matter

The relation of traffic speed to safety is universally recognized, and more and more, road owners are accepting the responsibility for managing speed on public roads as a way of ensuring public health and safety. Using traffic signal timing to minimize speeding opportunities is a way to manage speeds on signalized arterial roads that are not amenable to other valuable means of speed management including physical traffic calming, speed cameras, and road diets that result in one thru lane per direction.

Approaching an intersection, vehicles facing a red signal have to stop; and when the signal is green, drivers generally cannot speed if there is a vehicle directly ahead of them discharging from the queue. However, drivers approaching an intersection on a stale green with no vehicle ahead of them for a considerable distance are unconstrained and therefore able to speed. Furth et al. (2018) proposed a technical definition that makes speeding opportunities quantifiable:

> ***A speeding opportunity is the event of a vehicle arriving at an intersection on a stale green when there is no vehicle ahead of them (in their lane) for 5 seconds.***

Figure 1- A vehicle arriving at stale green (Google StreetView)

With a technical definition like this, speeding opportunities can be counted. At a given intersection approach, there might be hundreds of speeding opportunities per hour, and in a corridor, there might be thousands of speeding opportunities per hour. Granted, not every driver with a speeding opportunity will speed; but for drivers inclined to speed, one way to prevent that is to remove their opportunities to speed.

It would be easy to eliminate most speeding opportunities by making signals red most of the time; however, that would frustrate drivers unacceptably and drastically reduce accessibility; it could divert traffic to local neighborhood streets. Therefore, this guidebook focuses on ways to reduce speeding opportunities while still maintaining good arterial progression, called the *Safe Waves* to traffic signal timing.

Can signal timing substantially reduce the number of speeding opportunities? And can it reduce speeding? A few studies have been conducted, and results are promising. A first set of studies used microsimulation, sometimes with field measurements for confirmation (1, 2); further results are described in later chapters of this report. Results, summarized in the table below, indicate reductions in speeding opportunities of up to 66% with little or no increase in delay to traffic.

Table 1- Study results

Arterial	Period of the Day	Percent change in speeding opportunities	Change in average vehicle delay networkwide (s)	Change in average arterial delay per intersection (s)
Huntington Av, Boston (urban; 7 intersections)	AM peak	-34%	0.1	-
Huntington Av, Boston (urban; 7 intersections)	midday	-51%	5.6	-
Melnea Cass Blvd, Boston (urban; 6 intersections)	midday	-66%	-3.0	-
Melnea Cass Blvd, Boston (urban; 6 intersections)	PM peak	-30%	1.0	-
Massachusetts Av, Boston (urban; 7 intersections)	AM peak	-37%	-7.0	-
Route 16, Everett, MA (suburban; 9 intersections)	AM peak	-59%	21.0	4.2
Route 16, Everett, MA (suburban; 9 intersections)	PM peak	-52%	7.4	1.4
Route 114, Danvers (suburban; 6 intersections)	AM peak	-56%	-	2.1
Route 114, Danvers (suburban; 6 intersections)	midday	-54%	-	2.4
Route 114, Danvers (suburban; 6 intersections)	PM peak	-29%	-	0.1

In 2023, a field test of Safe Waves signal timing was performed, described later in this report. It was on state route 114 in Danvers, MA, a stretch with 6 traffic signals, 40, 36,158 vehicles per day with two thru lanes per direction, and an 85th percentile speed of 33 mph. With Safe Waves signal timing, the number of speeding vehicles on the arterial fell by 75 percent, while the average arterial delay per intersection increased by just under 2 seconds. This 75% reduction in speeding vehicles was found whether "speeding" was defined as vehicles exceeding 35, 40, or 45 mph.

2.2 Principles of Arterial Coordination and How They Relate to Speeding Opportunities

The dominant form of arterial coordination in the U.S. is coordinated-actuated control, which, loosely speaking, features a fixed cycle length and a fixed ending time within the cycle for the arterial thru phase, which is the coordinated phase. Phases for the non-coordinated movements are variable in length, terminating when detectors find that there is no more traffic passing, subject to minimum and maximum green time. The nominal or "programmed" time for non-coordinated phase corresponds to the maximum green time. If non-coordinated phases terminate early, the slack goes to starting the arterial phase early. An intersection's offset is how long after time 0 in a "master clock cycle" the arterial phase at each intersection should end. Offsets are generally chosen to enable arterial traffic to progress in either direction through intersection after intersection at or near a desired speed.

This section describes principles of signal timing with arterial coordination that offer insight into how coordination can inadvertently create speeding opportunities.

2.2.1 Long Cycles Lead to Long Periods of Unsaturated Arterial Green, which Create Speeding Opportunities.

With coordinated-actuated control, if the signal cycle is significantly longer than needed, the arterial phase will have long periods of unsaturated green (green time outside what is used to serve the platoon formed by queuing at this intersection or an upstream intersection). Long periods of unsaturated green time create speeding opportunities. To minimize speeding opportunities, then, there is a clear motivation to keep the cycle length close to what an intersection needs for capacity, and little more.

2.2.2 On-Demand Pedestrian Service Leads to Long Periods of Unsaturated Arterial Green, which Create Speeding Opportunities.

Where the arterial is wide and the side street's traffic volume is low, the side street's programmed split will typically be based on the time needed to serve the concurrent pedestrian crossing, which can be considerably more than what's needed to serve vehicle traffic. If the pedestrian crossing is on demand (i.e., not on recall), then in cycles with no pedestrian call, the side street will terminate early, which in turn will make the coordinated phase begin early. The extra, unneeded green time that the coordinated phase will then get creates speeding opportunities. To minimize speeding opportunities, then, there is a clear motivation to have pedestrian service on recall. (There is an exception, discussed later, that applies when pedestrian demand very low.)

Industry practice generally frowns on putting pedestrian crossings on recall except in downtowns and anywhere else pedestrian demand is high enough that there is pedestrian demand in most cycles. Where pedestrian demand is lower, the general thinking is that it's better to make the pedestrian crossing on demand because doing so reduces vehicle delay without harming pedestrians; after all, the thinking goes, how can skipping pedestrian service hurt pedestrians if they aren't there? However, this reasoning deserves to be questioned in two ways.

- First, the benefit to vehicles is overstated, because the extra green time that the coordinated phase gets from skipping pedestrian service is not needed for capacity, and is usually outside the through band. Often, an early return to green allows queued cars to advance to the next intersection only to be stopped there. At the same time, the shorter green for the side street (which means a longer red) increases delay for side street vehicles. As a result, the net benefit to vehicles is small, and possibly negative.
- Second, having pedestrian service on demand is *not* harmless to pedestrians; in fact, it increases average pedestrian delay by up to 10 seconds, even under the assumption that pedestrians always push the button (5). Granted, under this assumption, there is no increase in delay for pedestrians who arrive during the minor street red; but pedestrians arriving during the early part of the minor street green would have zero delay if pedestrian service were on recall, instead of being delayed by a full cycle when pedestrian service is on demand.

2.2.3 Coordination Demands a Common Cycle Length, which Can Lead to Many Intersections Having Cycles Longer than Needed – Unless a Corridor is Divided into Multiple Coordination Zones, each with a Cycle Length Tailored to its Need.

Each intersection has a minimum cycle length needed for capacity, depending on factors such as cross street traffic, turning demand, and auxiliary lanes. However, within a coordination zone – that is, a series of intersections coordinated with one other – every intersection has to run the same cycle length. In order to ensure sufficient capacity, the intersection within a coordination zone that has the longest needed cycle length, the so-called "critical intersection," will determine the cycle length for that zone.

This can create a large mismatch between the cycle length needed (for capacity) and the cycle length assigned (for coordination). For example, suppose needed cycle length is 70 s at some intersections, 80 s at others, and 110 s at one especially busy intersection. Unless the corridor is broken into different coordination zones, the entire corridor would have to have a cycle length of 110 s, and in keeping with coordinated-actuated control logic, the slack time will go the arterial thru phase. This will create a lot of speeding opportunities at all but the critical intersection.

The mismatch between the cycle length needed for capacity and what's needed for coordination, a corridor can be divided into multiple coordination zones, giving each zone a cycle length tailored to the capacity need of its intersections. A single intersection demanding a particularly long cycle can be a zone by itself.

2.2.4 Coordination Breaks Need Queuing Space for One Cycle's Worth of Arrivals to Prevent Spillback.

On segments that form the boundary between adjacent zones, there is no coordination between the segment's upstream and downstream intersections, and queues can vary widely between cycles. In some cycles, vehicles released at the upstream intersection can arrive at the downstream intersection just after its arterial red began, forming a long queue. To prevent queues from spilling back to the upstream intersection, the minimum segment length for a boundary segment between coordination zones is:

$$L_{seg,break} \geq (f.s.)\ \frac{v}{N}\ \frac{C_{max}}{3600} L_{vehInQ} \qquad (1)$$

where

> $L_{seg,break}$ = length of a segment between two coordination zones, excluding the intersection areas at either end (ft)
>
> *f.s.* = factor of safety (suggested default = 1.2)
>
> v/N = approach volume per lane (veh/h)
>
> C_{max} = the longer of the cycle lengths of the two bordering zones (s)
>
> $L_{vehInQueue}$ = average spacing for vehicles in a queue (default = 25 ft)

The concept of a minimum segment length for a coordination break is not new, and in industry practice, it is usually addressed by informal rules of thumb, such as a minimum length of 1000 ft. However, a rule of thumb like this can be too restrictive. For example, with an hourly volume of 400 vehicles per lane C_{max} = 90 s and the suggested default factor of safety, 300 ft is sufficient length for a coordination break segment to prevent spillback; if hourly volume per lane instead is 700/h and C_{max} =110 s, a zone boundary segment should be at least 640 ft long. These illustrative segment lengths exclude intersection areas.

2.2.5 Two-Way Coordination Requires Half-Cycle Offsets, which Leads to Clusters of Simultaneous Green Intersections

A fundamental principle of two-way coordination is that the symmetry in time-space required to provide a through band in each direction can only be achieved with half-cycle synchronization, i.e., using offsets 0 or $C/2$, where C is the common cycle length for the coordination zone (6). "Offset" in this context is how long after time 0 in a master cycle the *middle of arterial green* occurs, and so an adjustment is needed to convert to an offset based on start or end of arterial green, as offset is typically defined in practice. In addition, small adjustments are needed to account for lead-lag phase sequencing and for lengthening a favored direction's through band at the expense of the other direction's (7). These adjustments can often mask the simple pattern half-cycle synchronization that is ubiquitously applied on bidirectional arterials.

This need for half-cycle synchronization is in sharp contrast to one-way coordination, which has no such limitation. That is, with one-way coordination, the full range of offsets from 0 to *C* may be used.

As a result, where intersection spacing is regular, offsets inevitably follow an alternating pattern (1) with simultaneous green clusters of *n* intersections offset by half a cycle from each other, where *n* can equal 1, 2, 3, …. A "simultaneous green cluster" is a series of intersections with equal offset. The *Traffic Signal Timing Manual* (7) shows examples with *n* = 1, 2, and 3:

- In the *single alternate* pattern, *n* = 1. Each intersection is offset by 0.5C from its neighbors; that is, successive offsets are {0, 0.5*C*, 0, 0.5*C*, …}.
- In the *double alternate* pattern, *n* = 2, and each cluster of two intersections is offset by half a cycle from the neighboring cluster. That is, successive offsets are {0, 0, 0.5*C*, 0.5*C*, 0, 0, 0.5*C*, 0.5*C*, …}.
- In the *triple alternate* pattern, *n* = 3, and each cluster of three intersections is offset by half a cycle from the neighboring cluster. That is, successive offsets are {0, 0, 0, 0.5*C*, 0.5*C*, 0.5*C*, 0, 0, 0, 0.5*C*, 0.5*C*, 0.5*C*, …}.

Figure 2 shows single, double, and quad (*n* = 4) alternate patterns and the through bands they produce.

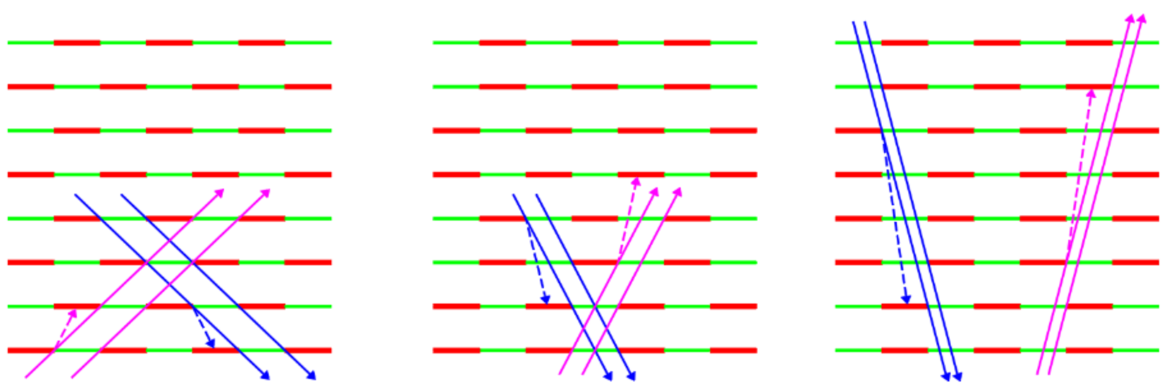

Figure 2- Different alternate offset patterns

The cluster size that's appropriate depends on the signal spacing, cycle length, and chosen progression speed, as explained later. First, however, it's important to consider cluster size affects speeding opportunities.

2.2.6 Large Clusters of Simultaneous-Green Intersections Create Speeding Opportunities and Speeding Incentives

A large cluster of simultaneous-green intersections means that drivers will see a long string of green signals ahead of them, an obvious incentive to speed in order to get through as many lights as possible before the green ends.

If the cluster size, for example, is 4, a speeding driver who leaves intersection 1 will arrive too early at intersection 5 (the next cluster) to get a green, and therefore will not have a speeding opportunity there; however, this driver will have a speeding opportunity at intersections 2, 3, and 4. The longer the cluster size, the more speeding opportunities.

Speeding opportunities are greatest during periods of moderate traffic volume. When traffic volume is near saturation, drivers (e.g., the driver in the previous example approaching intersections 2, 3, and 4) will be inhibited from speeding by queues that will have formed at the intersections they approach. On arterials whose traffic volume is highly directional, congestion during a peak hour can mean few speeding opportunities in the peak direction while there a lot of speeding opportunities in the reverse direction.

2.2.7 Where Signal Spacing is Regular, Progression Speed is Proportional to Cluster Size, and Desired Progression Speeds often Lead to Large Clusters of Intersections with Simultaneous Green

If intersection spacing is regular, how large will the clusters of simultaneous intersections be? This can be readily answered by considering that with half-cycle synchronization, a vehicle can advance without encountering a red signal by following a speed that allows it to pass from one cluster to the next in half a cycle; that speed is called the progression speed. Therefore, cluster size and progression speed are related by the following equation:

$$v_{progression} = \frac{2Dn}{C} \tag{2}$$

where D = distance between neighboring signalized intersections, n = cluster size, and C = cycle length. If D is in miles and C is in hours, the calculated speed will be in mph; if D is in feet and C in seconds, the calculated speed will be in ft/s, which can be converted to mph by multiplying by 3600/5280.

Of the four variables in this relationship, D is usually fixed; designers generally aim for the lowest possible value of C that satisfies capacity needs; and they usually have a desired progression speed; together, those choices determine cluster size n by rearranging equation 2:

$$n_{unrounded} = \frac{C\, v_{progression}}{2D} \tag{3}$$

Here, cluster size is given the name $n_{unrounded}$ because this formula does not produce integer solutions except by chance. For example, with D = 660 ft, C = 120 s, and $v_{progression}$ = 25 mph = 36.67 ft/s, equation 3 yields $n_{unrounded}$ = 3.33.

Because cluster size must be integer, signal timing apps will implicitly round the specified progression speed up or down to a *quantum progression speed*, i.e., a progression speed corresponding to an integer value of n. For this example, quantum progression speeds are shown in Table 2. If a designer specifies a desired progression speed of 25 mph, the timing plan given by a signal timing app will have either a progression speed of 22.5 mph with clusters of 3 simultaneous green intersections, or 30 mph with clusters of 4. Rounding up will create more speeding opportunities.

Table 2- Cluster size vs. progression speed

Cluster size n	$v_{progression}$ (mph)
1	7.5
2	15.0
3	22.5
4	30.0
5	37.5
6	45.0

Table 2 shows the relationship between cluster size and progression speed for C = 120 s and intersection spacing = 1/8 mi = 660 ft.

Where intersection spacing is irregular, cluster size can vary along the arterial. Nevertheless, there will be a pattern in which the average distance from the middle of one cluster to the middle of the next will be roughly the same, corresponding to a progression speed given by:

$$v_{progression} = \frac{2(average\ distance\ between\ neighboring\ clusters)}{C} \quad (4)$$

2.2.8 Large Clusters of Simultaneous-Green Intersections not only Create Speeding Opportunities and Speeding Incentives; They often Result in Mediocre Progression

Single alternate offsets (i.e., n = 1) are ideal in two respects: they minimize speeding opportunities, and they provide the best progression. As one can see in Figure 2, with red signals bordering the front and back of the through band in both directions, there is positive control against speeding at every intersection.

However, in most situations, a cluster size of n = 1 leads to an unacceptably low progression speed. For the example used before, with C = 120 s and D = 1/8 mi (660 ft), progression speed with n = 1 would be only 7.5 mph. For a progression speed of 30 mph, the needed cluster size will be 4, as shown in Figure 2; as discussed earlier, clusters this large create speeding opportunities and an incentive to speed.

At the same time, as cluster size increases, progression quality decreases. Referring back to Figure 2, with single alternate offsets, there is good progression whether traffic volume is heavy or light; regardless of whether a vehicle released from the first intersection is at the start of the platoon or near the end, it can progress in the through band through intersection after intersection. However, with a large cluster size, only vehicles released near the start of green are in the through band; vehicles that are later in the platoon will soon hit a red signal. When volume is high, those vehicles that hit a red signal result in standing queues that must be cleared before the next platoon can advance, which will then push vehicles from the front of the platoon toward the rear, leading to their being stopped soon at a red signal. Thus, large cluster sizes deliver poor performance in two respects: they lead to a lot of speeding opportunities (especially with light to moderate traffic), and they offer only mediocre progression quality (especially with heavy traffic).

2.2.9 Lowering Progression Speed may Result in only a Small Increase in Travel Time

There are clear advantages to keeping cluster size small. Reviewing equation 3, a small cluster size can be achieved by (a) keeping cycle lengths as short as possible, and (b) using low progression speeds. In addition, Denney and Curtis have shown that using progression speeds a little below average speed improves efficiency by allowing platoons to densify (8).

At first glance, the idea of using a low progression speed seems to contradict the goal of providing reasonably good progression. It may seem self-evident that with a lower progression speed, travel time along the arterial will increase. However, as the previous discussion points out, a lower progression speed will lead to a smaller cluster size; this, in turn, will improve the quality of progression, i.e., more vehicles will be able to pass through intersection after intersection without stopping, particularly when traffic volume is moderate to heavy. Instead of going fast through a few intersections, then being stopped at a red light, drivers will advance at a slower speed through a longer series of green lights.

From the principles explained in this section, guidelines for *Safe Waves* signal timing follow.

2.3 Step-by-Step Guide to Safe Waves Arterial Signal Timing

This guide takes as its starting point that designers have acquired the input needed to time each intersection in isolation and determine its necessary cycle length. Inputs include traffic volumes, intersection geometry, factors that affect saturation flow rate, timing parameters including yellow, red clearance, and minimum green times, and design parameters including maximum volume / capacity ratio. For each intersection, phases are chosen, which involves determining whether and where to allow permitted turning conflicts. Next, a phase sequence is selected. From that, a necessary cycle length can be determined – necessary in the sense of providing sufficient capacity.

It is beyond the scope of this document to elaborate on the procedure for determining necessary cycle length. Suffice it to say that every intersection has a minimum cycle length needed for capacity, which varies from intersection to intersection because of differences in traffic volume, number of phases, number of lanes, crossing distances, and so forth; and that an intersection's necessary cycle length can sometimes be lowered further by reconsidering the inputs that determine it.

2.3.1 Minimize Cycle Length, Especially for Intersections that Need a Longer Cycle than Their Neighbors.

Minimizing cycle length is critical to reducing speeding opportunities, because short cycles minimize the amount of unsaturated green and help reduce the size of simultaneous green clusters. Designers should consider whether each intersection's necessary cycle length could be reduced, giving special attention to intersections whose necessary cycle length is significantly greater than its neighbors'.

Ways to reduce that necessary cycle length include:

a. ***Use a peak hour factor based on the busiest 30 or 60 minutes instead of the busiest 15 minutes.*** Industry practice is to base cycle length on traffic volume in the busiest 15 minute period, with the idea that while cycle overflows may be tolerable within a subset of that busiest 15 minute period, by the end of the period, queues should have dissipated. Peak hour factor is the ratio of peak hour volume to the volume in the design period within that peak hour (i.e., the peak 15 minutes), and is always 1.0 or less; the further from 1.0, the longer the needed cycle. Designing instead for the busiest 30 minutes (or 60 minutes) will allow for a shorter cycle, while allowing for temporary congestion during a subset of that 30 (or 60) minute period. For example, suppose a movement's peak hour volume is 2,000 veh/h, with 15-minute counts of {400, 500, 650, and 450}. The peak hour factor based on the busiest 15 minutes is 2000 / (4*650) = 0.77, while the peak hour factor based on the busiest 30 minutes is 2000 / (2*Sum(500, 650)) = 0.87. The peak hour factor based on the busiest 60 minutes is 1.

b. ***Allow a greater volume/capacity (v/c) ratio.*** Designers often aim for a v/c ratio, also called degree of saturation, well below 1.0, such as 0.90, in order to provide slack capacity to deal with temporary demand fluctuations. However, recall that v/c ratio is usually calculated using volumes that apply only in the busiest 15 minutes (or 30, or 60 minutes), which means that at all other times, the intersection will have slack capacity even if the calculated v/c equals 1.0. Therefore, consider allowing a v/c ratio of 0.95, 0.97, or even 1.0.

c. ***Avoid excessive red clearance time.*** While there is no uniform standard in the U.S. for red clearance, many jurisdictions follow a formula that provides enough time for a vehicle that entered the intersection at the last moment of yellow to clear the entire intersection. This is overly conservative, not accounting for the fact that drivers whose movement next gets the green need time to react and advance to the conflict zone with the vehicles whose green just ended (9). Consider reducing red clearance time where it would be safe.

d. ***Count the pedestrian phase end buffer toward needed pedestrian clearance time*** (as explicitly allowed by the MUTCD) by making the Flashing Don't Walk interval (FDW) equal needed pedestrian clearance minus the pedestrian phase end buffer, rather that setting FDW equal to needed pedestrian clearance. For example, consider a crossing with these features:

- minimum Walk = 7 s
- needed pedestrian clearance = 20 s (calculated by dividing crossing length by a walking speed of 3.5 ft/s)
- Flashing Don't Walk (FDW) ends at the onset of the concurrent vehicle phase's yellow, in keeping with agency polity
- the concurrent vehicle phase's yellow plus all red lasts 5 s.

The final two facts mean that the pedestrian phase end buffer will last 5 s.

To meet MUTCD requirements regarding pedestrian safety needs, the minimum split needed is Sum(7, 20) = 27 s. This can be accommodated in the timing plan by subdividing the split thus: {Walk = 7 || FDW = 15 || phase end buffer = 5}.

However, many designers make it their practice set FDW equal to needed pedestrian clearance, which for this example would lead to a needed split of Sum (7 + 20 + 5) = 32 s. (One thing that promotes this practice is that the FDW interval is often called the "pedestrian clearance interval," a misnomer because pedestrians can also clear during the pedestrian phase end buffer.) Typically, this practice increases a cycle's lost time by the length of the minor street's change interval, which increases cycle length both directly, by increasing the minor street split, and indirectly, increasing the needed length of other phases to compensate for their longer red during the minor street phase.

e. ***Where there is a long but infrequently used pedestrian crossing, consider using an undersized phase for the on-demand pedestrian service, a technique known colloquially as "oversized ped".*** For example, suppose a pedestrian movement concurrent with the side street needs a split of 32 s, but the concurrent vehicle movement itself typically needs a split of only 17 s. If pedestrian calls are infrequent – say, once every 4 cycles or less – the side street could be programmed with a split of 22 s, which is 5 s longer than what's needed on average by vehicles, but 10 shorter than what's needed by peds. Whenever there is a ped call, the side street will give peds full service and thus control the cycle for 32 s, forcing it to "borrow" 10 s from the following phases. However, if there is no ped call for the next few cycles, the side street can easily "repay" the time that it borrowed, allowing the other affected phases to recover with little overall impact. Most controllers have built-in logic for handling oversized ped service calls with minimal disruption to traffic.

Both the Route 114 and Route 16 case studies presented later in this document show how undersized phases created large reductions in cycle length at critical intersections.

f. ***Don't limit cycle length to multiples of 10 s.*** In practice, cycle lengths are almost always multiples of 10 s; however, with modern signal timing software and control equipment, there is no reason to be bound to this convention. If necessary cycle length is 84 s, let the cycle length be 84 s, rather than round it up to 90 s. (Designers are reluctant to round down, because rounding down leads to violating the maximum v/c ratio constraint).

2.3.2 Look for Double Cycling Possibilities

If one intersection's necessary cycle length is half of its neighbor's, they can be coordinated by running two cycles of the shorter one within each cycle of the longer one. Identify intersections that might be candidates for double cycling. When looking for similarity between intersections in terms of necessary cycle length, an intersection that double cycles can be considered to have double its necessary cycle length. For example, if one intersection needs a cycle of 110 s and another needs just 52 s, the second can double cycle and be treated as though its necessary cycle length is 104 s, which is quite similar to what the other intersection needs.

However, avoid lengthening a zone's cycle by more than a few seconds to make double cycling possible. For example, suppose a critical intersection needs a cycle length of 100 s while a neighboring intersection needs a cycle length of 60 s. It's probably better to coordinate the pair at 100 s than to raise the cycle length to 120 s so that the shorter one can double cycle.

The same intersection spacing requirement that applies for coordination breaks (equation 1) also applies for double cycling, which can lead to spillback queues if intersections are too close.

2.3.3 Divide the Corridor into Zones within which Necessary Cycle Length Varies Little, with Zone Boundaries at Segments Long Enough for a Cycle's Queue

Dividing a corridor into coordination zones is a creative process that aims to create good progression opportunities without creating many speeding opportunities.

The primary constraint is that intersections that are closely spaced must be in the same coordination zone in order to prevent spillback. Minimum segment length for a coordination break is given by equation 1.

The aim is to create zones in which differences between the various intersections' necessary cycle length is small. A good target is to keep the difference between the maximum and minimum needed cycle length within a zone to 15 seconds or less. However, this is a not a hard rule; it must sometimes be violated because of close intersection spacing. A zone with a single intersection can be appropriate when one intersection stands out with an exceptionally long necessary cycle length; otherwise, zones with at least two or three intersections are preferred in order to provide reasonable progression.

In some corridors, it may be obvious how intersections should be grouped together. Where it is not obvious, designers can create a few alternatives and, in a later step, evaluate them and choose the best.

2.3.4 Choose a Progression Speed, Allow Lead-Lag Phasing, and then Time each Zone's Signals

Recall that a low progression speed helps reduce speeding opportunities by leading to smaller clusters of simultaneous green intersections. Therefore, choose a progression speed 5 or 10 mph lower than the speed limit. A lower progression speed also helps make traffic flow efficient by compressing platoons.

Allow lead-lag phasing to vary from intersection to intersection, choosing the option that creates the best progression. This allows for small deviations from strict half-cycle offsets that benefit progression in both directions. Good progression at a safe speed helps limit speeding opportunities and improves service at the same time.

Single zone intersections will run free, of course. They can be fully actuated; that way, cycle length will automatically adapt to level of demand. Note that with fully actuated control, pedestrian phases can be on recall, which may be appropriate where pedestrian demand is high.

Signal timing is usually done using an app, one zone at a time. In principle, the app should be able find the offsets and lead-lag sequence that yield the best progression; in practice, manual adjustments can often lead to a still better solution.

2.3.5 Check for Large Clusters of Simultaneous Green Intersections and Consider Offset Flips to Reduce Cluster Size

Using the progression diagram produced by the signal timing app, identify the clusters of intersections with simultaneous green within each coordination zone, and pay attention to large clusters, which allow a platoon leader to advance at a high speed through several intersections. Then consider making adjustments to reduce cluster size, either by changing offsets directly or by lowering the progression speed and running the app again. "Changing offsets" in this context means changing offset by half the cycle length – what might be considered "flipping" the offset. Reducing cluster size implies lowering the progression speed.

Consider the example first introduced with equation 3, with intersection spacing = 660 ft, C = 120 s, and desired progression speed = 25 mph = 36.67 ft/s. As explained earlier, the signal timing app may create a solution with clusters of n = 3 or 4 simultaneous green intersections. If the cluster size if 4, one will be able to see that the slope of a line connecting middle of green at intersection 1 to middle of green at intersection 5 (the next cluster) would be a speed of 30 mph (44 ft/s), which exceeds the specified progression speed. In such a case, consider flipping offsets to reduce cluster size to 3.

2.3.6 Run the Save Waves Analysis Tool to Visualize and Quantify Speeding Opportunities

Once a timing plan has been devised, use the Save Waves Analysis Tool (SWAT) to visualize and quantify speeding opportunities. This tool, published by Northeastern University under the sponsorship of the Massachusetts Department of Transportation, can be used free of charge using the URL http://newton.neu.edu:8080/safewaves/.

As input, this tool requires signal timing, volume, and saturation flow rate information taken from reports produced by a signal timing app.

Examine the coordination diagrams produced by SWAT. They are similar in form to those produced by signal timing apps, but are more precise; in addition, they flag speeding opportunities. That allows designers to see the "hot spots" and make further adjustments in a timing plan to diminish or eliminate them. To test adjustments, make the changes in the signal timing app, then use its output to revise the inputs to SWAT.

If several alternative signal timing plans are being considered, evaluate them all and compare them in terms of speeding opportunities versus other performance measures (mainly, average vehicle delay and average pedestrian delay). For reporting on overall performance, it is also helpful to run SWAT on the existing timing plan so that one can compare how the new timing plan compares in terms of speeding opportunities.

2.4 Case Study Highlights

This section presents highlights from three applications of Safe Waves traffic signal timing, which serve as useful examples with practical lessons.

2.4.1 Case Study 1: Huntington Avenue, Boston, MA

This was a simulation-based study, with details in Figure 3 (2). Note that since that study was completed, the corridor has been changed by converting one lane in each direction into an exclusive bus-bike lane.

Huntington Avenue had, at the time of the study (2019), two thru lanes per direction, separated by a median reservation 32 to 40 ft wide hosting light rail tracks and platforms. Average daily traffic was 17,000 vehicles per day, and the speed limit was 25 mph, having been lowered in 2017 from 30 mph. Some of the crossings, especially those at college campuses and at light rail stops, have high pedestrian volumes.

The study area has nine signalized, with intersection spacing varying from 500 to 980 ft. The outer two intersections were held as boundary conditions that were to keep their original signal timing so as not to disrupt their coordination with intersections beyond the study area. Of the inner seven subjects to retiming, two, Intersections 2 and 8, are signalized crossings with no signalized side street movements.

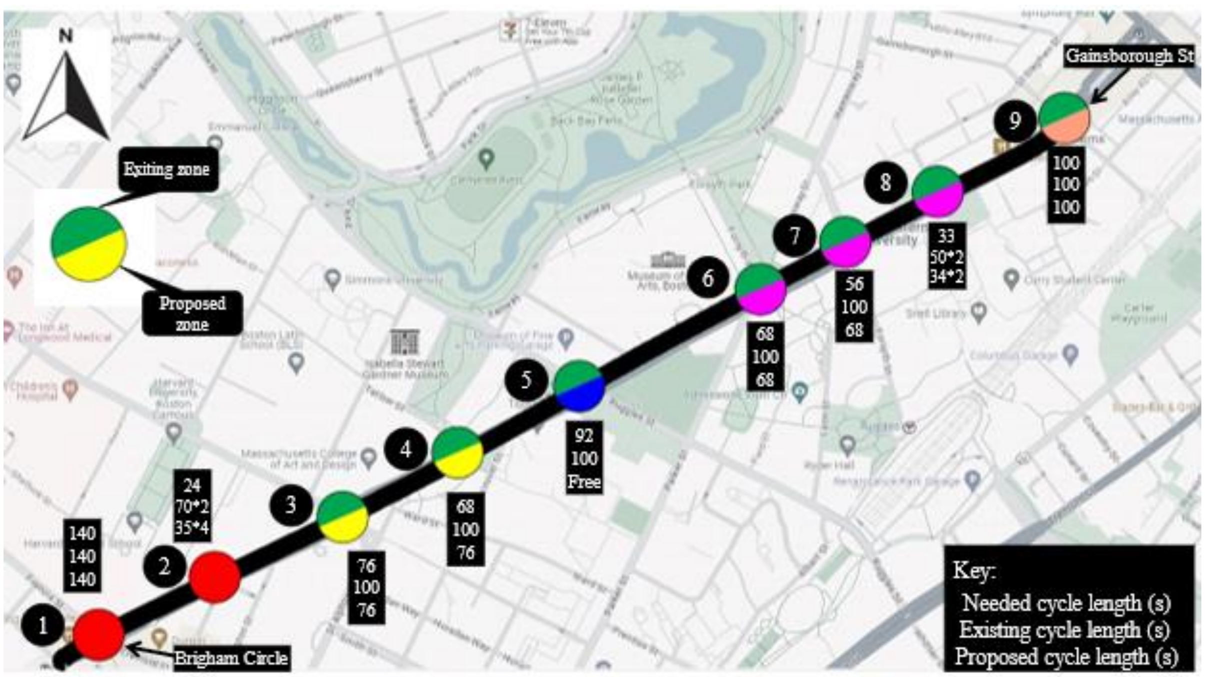

Figure 3- Huntington Ave study area

Figure 3 shows the intersections and, for the a.m. peak, their necessary cycle length based on capacity. Also shown are their existing cycle lengths and coordination zone. There were two coordination zones:

- Intersections 1 and 2, with C = 140 s. Intersection 1 is the critical intersection, with a necessary cycle length of 140 s. Intersection 2, a pedestrian crossing, double cycles (thus C = 70 s). This zone continues to the west with additional intersections before Intersection 1.
- Intersections 3-9, with C = 100 s. (This zone continues to the east with additional intersections beyond Intersection 9.) Intersection 5, with a necessary cycle length of 92 s, is the critical intersection. None of the other intersections needs a cycle longer than 76 s.

In 2019, high speeds were common. The arterial was empty for much of its green time everywhere except intersections 1 and 5, which creates opportunities for speeding; for the same reason, it was common for pedestrians to cross against the light.

2.4.2 Short Coordination Zones, and Better Pedestrian Service at Two-Stage Crossings

The main technique used to reduce speeding opportunities on this street was breaking up the long coordination zone so that at every intersection, the cycle length chosen for coordination would closely match the cycle length needed for capacity.

The first step was determining each intersection's necessary cycle length (C). A large factor in cycle design was a desire to reduce pedestrian delay crossing the arterial. Because of the wide median, all of the arterial crossings are configured as two-stage crossings, and at many intersections, signals were timed such that pedestrians could not cross in a single pass; they would have to walk to the median in one cycle, wait there, and then finish their crossing in the next cycle. This meant unacceptably long delay and high levels of pedestrian non-compliance. For determining necessary cycle length, then, the side streets were given a minimum split that would allow pedestrians with a speed of 4.0 ft/s to clear in a single stage. (The two-stage option is still available for slower pedestrians.) The only exceptions were Intersections 2 and 8, the two signalized pedestrian crossings, which were instead configured to have very short cycles with two-stage crossings that would be coordinated by having each crossing offset by half a cycle from the other; that way, waiting time in the median will be little.

Necessary cycle lengths for the a.m. peak are shown in Figure 3. One can that at the intersections subject to retiming, necessary cycle length is in the range of 56 to 76 s, with exceptions only for the pedestrian crossing intersections (which need a much shorter cycle) and Intersection 5, which needs a cycle longer than 90 s.

The proposed plan divides the arterial into 5 short coordinations, also displayed in Figure 3, as follow:

> **Intersections 1 and 2** remain a coordination zone with C = 140 s. Intersection 2, a pedestrian crossing, uses quad cycling (thus, C = 35 s) with the two crossings staggered in time, i.e., the eastbound roadway crossing runs 17 s before the westbound roadway crossing, so that pedestrians have good progression in both directions.
>
> **Intersections 3 and 4**: C = 76 s.
>
> **Intersection 5** is its own zone. It will run free, but with pedestrian crossings on recall.
>
> **Intersections 6 to 8:** C = 68 s. Intersection 8, also a pedestrian crossing, continues to double cycle (thus, C = 34), with the two crossings staggered so that pedestrians have good progression in both directions.
>
> **Intersection 9** remains coordinated with intersections beyond the edge of the study area with C = 100 s.

With short zones, cycles contain far less slack (difference between the design cycle length and needed cycle length). With the existing timing, there were two intersections with more than 40 s of slack in their cycle, and five with more than 20 s of slack. In the proposed plan, the greatest slack at any intersection is 14 s.

2.4.3 Coordination Within Each Zone

The next step was choosing offsets for coordination within each zone with two or more intersections.

> **Intersections 1 and 2**: As mentioned before, Intersection 2 quad cycles with C = 35 s and with the two roadways offset from one another by 17 s. Offsets between Intersection 1 and 2 were chosen to give ideal one-way progression for eastbound (i.e., offset equals the travel time), because eastbound is the peak direction in the a.m. peak and because quad cycling at intersection 2 makes coordination from intersection 2 to 1 almost meaningless.
>
> **Intersections 3 and 4:** Because they are close, they have simultaneous offset. To promote progression at a safe speed, they use lead-lag phasing, with a leading left leaving the zone (intersection 3 westbound, intersection 4 eastbound) and lagging left entering the zone (intersection 3 eastbound; intersection 4 westbound has no left turn.)
>
> **Intersections 6 to 8**: As described earlier, the two pedestrian crossings at intersection 8 are staggered in time and thus offset from one another by 17 s. Between intersection 7 and intersection 8, intersection 8 eastbound is offset 17 s later than intersection 7, while intersection 8 westbound is offset 17 s earlier than intersection 7; these offsets correspond to a progression speed of 20 mph. Intersections 6 and 7 have simultaneous offset because of their proximity, but again use lead-lag phasing to promote progression at a safe speed, with leading lefts (where lefts are allowed) entering the 6-7 pair and lagging lefts leaving the 6-7 pair.

With these offsets, the longest cluster of simultaneous green intersections is 2, and even within those clusters, green starts are not simultaneous due to lead-lag phasing.

2.4.4 Results

Speeding opportunities, as measured by microsimulation, fell by 33.5% in the a.m. peak, while average network delay remained unchanged. The same process was followed to get a new signal timing plan for the midday; in that period, speeding opportunities fell by 51% while average network rose by about 6 s per vehicle.

2.5 Case Study 2: Rt 114, Danvers, MA

Rt 114 in Danvers has two lanes per direction, 36,158 vehicles per day, and a speed limit of 40 mph (though 85th percentile speed is 33 mph). It has six signalized intersections, two of which have pedestrian crossings. There is very little crossing demand because of the non-intense land use and because the two sides of Rt 114 are connected by a grade-separated rail trail. This example concerns traffic in the a.m. peak, and provides results from a field study in which the Safe Waves signal timing plan was implemented in June, 2023.

Figure 4- Intersection at R114

The study area has six signalized intersections, none of them a major intersection, shown in Figure 4, with spacing varying from 450 to 1000 ft. There are no signalized intersections near either edge of the study area.

2.5.1 Finding Needed Cycle Length, and Minimizing Cycle Length Using an Undersized Phase

Intersection 1, with a freeway on-ramp, alternates control between eastbound thru and westbound left; there is no control for westbound thru. It needs only a short cycle. For intersections 2, 3, 5, and 6, needed cycle length is quite similar, between 54 and 66 s. Intersection 4 stood out as needing a cycle of 76 s, partly because the side street there is busier than the rest, and partly because – unlike most of the intersections – it has an arterial pedestrian crossing.

If timed in the conventional way, intersection 4 would need a cycle length of 110, due to its having long pedestrian crossing (concurrent with the side street) and moderately high left turn volume. And because of the short distance between intersections 2, 3, and 4, they have to be coordinated; so if intersection 4 needs a long cycle, intersections 2 and 3 will also, resulting in slack time that will create speeding opportunities. This created an incentive to shorten the necessary cycle for intersection 4.

Because the ped crossing at intersection 4 is called very infrequently (less than twice per hour), the solution was to design a cycle with an undersized side street phase, meaning the side street's programmed split is not long enough to serve the pedestrian crossing. When the crossing is called and served, the phase will overrun its allotted time, making the subsequent phase(s) start late. The cycle is designed with enough slack time to enable it to recover from this kind of interruption and get back in sync with the cycle clock within two cycles. Using undersized phases (also called 'oversized ped') is a technique recognized in the *Traffic Signal Timing Manual* (7), but methods for determining the needed cycle length are not well established. The method used in this project involved trial and error and modeling the recovery process to find a cycle length that would enable the cycle to get back in sync within two cycles after a ped service. This resulted in a needed cycle length of only 66 s.

2.5.2 Coordination Zones and Offsets

Because intersections 2-6 had needed cycle lengths between 54 and 66 s, they were put into the same coordination zone with C = 66 s. For intersection 1, 1000 ft from intersection 2 and with a needed cycle length of 25, one good option would be to let it run free. However, instead the chosen design was to put it into coordination with the rest of the intersections, double cycling (thus, C = 33 s).

Because intersection 1 controls only eastbound thru, it can have one-way coordination with intersection 2. The problem was then finding offsets for good two-way coordination for intersection 2 to 6. While the speed limit is 40 mph, the 85th percentile speed is between 30 and 34 mph, and so the desired progression speed was chosen to be 30 mph (44 ft/s). With C = 66 s, the half-cycle synchronization pattern that fit was (0, 0, 0.5, 0, 0) – that is, intersections 2 and 3 would be a simultaneous green pair, as would intersections 5 and 6, and those pairs would each be offset by half a cycle from intersection 4. Within the two simultaneous green pairs, lead-lag phasing was used to improve progression, with a leading left used when entering the pair and a lagging left when leaving.

Looking at progression diagrams in Synchro, offsets were slightly adjusted to prevent spillback between intersections 2 and 3, which are separated by only 570 ft.

With these offsets, the longest cluster of simultaneous green intersections is 2, and even within those clusters, green starts are not simultaneous due to lead-lag phasing.

2.5.3 Results

Speeding opportunities, as measured using SWAT, fell by a little more 50%. Speeding, measured in the field, fell by about 75%. Whether speeding was defined as exceeding 45 mph, 40 mph, or 35 mph, the same 75% reduction in speeding was observed.

Average delay on the arterial rose by 2 s per intersection. Because of the reduction in cycle length, average pedestrian delay fell 18 s. Change in delay to side street and left turning traffic was not measured, but it probably decreased because of the reduction in cycle length.

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3 Development of the Safe Waves Analysis Tool

3.1 The Need for an Analysis Tool

Speeding Opportunities (SO's) has been proposed as a performance measure to evaluate a signal timing plan for an arterial in terms of its ability to control speed, and thus contribute to safety. Speeding Opportunities is defined as the number of vehicles arriving at an intersection on a stale green and with no vehicle ahead of it for 5 s. In the field, or in a microsimulation model, this performance measure can be readily measured in the field or from microsimulation. However, field measurement is not practical for evaluating alternative timing plans, and microsimulation is complex and expensive. Hence, there is a need to use a tool to evaluate the number of speeding opportunities along an arterial that demands only the kinds of inputs typically used in signal timing.

3.1.1 Synchro Progression Diagrams as a Source?

Synchro is the most commonly used signal design and analysis app in Massachusetts. To help evaluate timing plans, Synchro can draw progression diagrams – see Figure 5 for an example – which seem to offer a way to visualize and possibly measure the number of speeding opportunities. In this diagram, blue lines represent vehicles traveling from top to bottom, and red lines represent those traveling in the opposite direction.

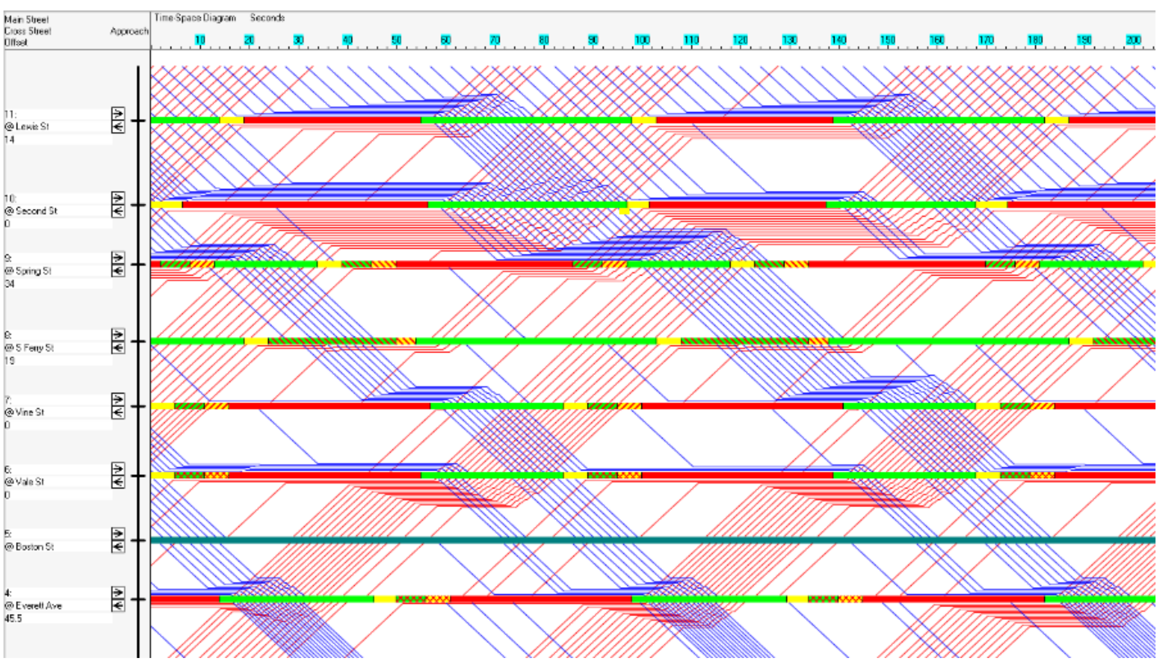

Figure 5- Example Synchro progression diagram

At first glance, this diagram seems to be a suitable tool to measure the number of SO's. However, there are some flaws with the diagram provided by Synchro. The original plan for this research was to use Synchro progression diagrams to measure speeding opportunities, applying image processing to identify and count them. However, Synchro's progression diagrams were found to be an unreliable source for determining the vehicle-gap relationships inherent to speeding opportunities, for two main reasons.

First, with Synchro's progression diagrams, each line may represent a different number of vehicles. This makes it hard or impossible to know whether there is a big gap between two vehicles, and makes it difficult to determine the size of the gap. According to the Synchro manual, the number of vehicles per line varies with the volume of the lane group volume represented as shown in Table 3.

Table 3- Vehicles per line (Synchro's diagram)

Adj Lane Group Volume (vph)	vehicles per line
0 to 899	1
900 to 1799	2
1800 to 2699	3
2700 to 3599	4

Second, after working with several Synchro progression diagrams, it was found that they have frequent inconsistencies and discontinuities between the two sides of an intersection. Therefore, a new tool was developed for evaluating speeding opportunities.

3.2 Safe Waves Analysis Tool (SWAT) Introduction

Safe Waves Analysis Tool (SWAT) is the web-app that was developed for evaluating speeding opportunities. It requires input that can be obtained from Synchro reports, and produces as output progression diagrams that identify and count the number of speeding opportunities at each intersection in each direction along an arterial corridor.

The following sections describe the app's logic. The next chapter is the User Guide.

3.3 Progression Logic

SWAT progresses vehicles along an arterial in a single direction. The app can be run first for one direction of an arterial and then for the other; the two directions do not interact.

It takes information from Synchro reports including the Volumes, Signal Timings and Lane geometries of all the intersections in the study corridor for a single direction.

Figure 6 illustrates the overall flow of the app, broken down into functions numbered from function 1 to function 8.

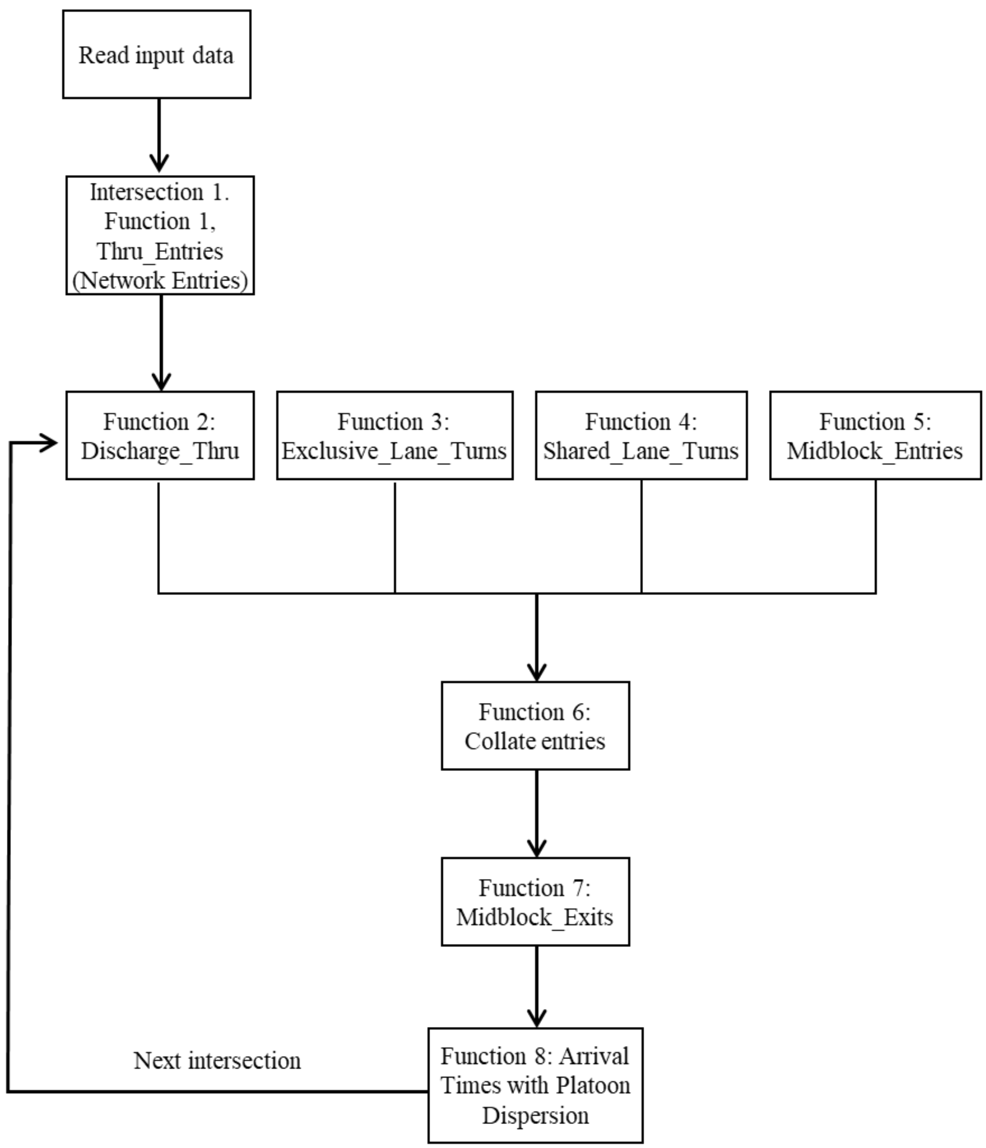

Figure 6- SWAT Logic Flowchart

3.3.1 Master Function (Function 0)

First, the master function reads the input data. Then, it works at the network level, loop over the intersections along the arterial and calling the other 8 functions at each intersection as needed.

3.3.2 Network Entries (Function 1)

Function 1 generates vehicles at the entry point to the arterial by reading the arterial information from the input file, assigning them a projected arrival time at the stop-line of the first intersection. The assignment of headway uses deterministic model. Hence, They have equal headways, with is equal to the inverse of the specified flow rate.

3.3.3 Arterial Queue and Discharge (Function 2)

Function 2 models the arterial queuing at an intersection through to their discharge. It takes as input the vehicles arriving from upstream, including their projected arrival time at the stop-line. At the first intersection, that information is generated in function 1; at subsequent intersections, that information is output from later functions applied to the previous intersection.

Arriving vehicles are assigned a movement direction – Thru, Right turn, Left turn – based on specified turning proportions. The function tracks vehicles as they queue, with turning vehicles queuing in exclusive turn lanes if available, and in the thru lanes otherwise. Vehicles are progressed through to their discharge at the stop line. Vehicles that turn off are then essentially discarded; for thru vehicles, the discharge time is stored for use in later functions.

3.3.4 Receiving from Side Street (Functions 3 and 4)

Functions 3 and 4 generate vehicles that turn from a side street into the arterial in the direction of interest. Function 3 has the logic for turning movements from an exclusive lane, while function 4 has the logic for turning movements from a shared lane. First, the side street leg from which vehicles turn right onto the arterial in the direction of interest is processed, using Function 3 if that side street leg has an exclusive right turn lane and Function 4 otherwise. Then the side street leg from which vehicles turn left onto the arterial in the direction of interest is processed, using Function 3 if that side street leg has an exclusive left turn lane and Function 4 otherwise.

3.3.5 Midblock Generated Trips (Function 5)

Next, Function 5 generates midblock entries. They can represent entries from unsignalized intersections and driveways between two signalized intersections. Midblock entries are generated with uniformly spaced arrival times, and are treated as though they arrive at the same location as vehicles turning from the side street. Their assimilation into arterial traffic, which may involve some queuing delay, is handled in a later function.

Available turning volumes from any unsignalized source between two signalized intersections, can be entered to the input file by user in the provided cells, and the software will automatically calculate the midblock entry by accounting the entered volumes and the difference between upstream and downstream volumes of the segment.

3.3.6 Collating Entries (Function 6)

At this step, there are four lists of entering vehicles: thru vehicles discharged from the previous intersection, turning vehicles from two legs of the side street, and midblock entries. This function collates them into a new list, giving priority first to thru vehicles, then right turns, then left turns, then midblock entries. Each entry is delayed until there is an adequate gap among the higher priority vehicles. In this new list, each vehicle has an assigned time projected to the discharge point of the intersection from which thru and turning vehicles were discharged.

3.3.7 Midblock Exits (Function 7)

In this step, vehicles are removed in accordance with a specified midblock exit volume. Vehicles that were midblock entries are not eligible for midblock exits.

3.3.8 Platoon Dispersion and Arrival Time at the Next Intersection (Function 8)

Function 8 progresses vehicles from the upstream end of a segment, where they were just discharged from an intersection to the next intersection. Rather than translating vehicles with a fixed travel time, this function applies a platoon dispersion model, a continuous version of Robertson's platoon dispersion model as calibrated for U.S. streets by Bonneson et al. (3).

The output of this function is a list of vehicles with their projected arrival time at the next intersection. This list will be used as the input for function 2 at all except the first intersection.

3.4 Progression Diagrams and Speeding Opportunities

The app tracks vehicles from when they are generated until they leave the arterial, either by turning off or continuing beyond the last intersection. This information is then used to draw progression diagrams and to identify which vehicle arrivals at an intersection qualify as a speeding opportunity (arrive on a stale green, no vehicle ahead of them for 5 seconds).

3.4.1 Sample Progression Diagram

Figure 7 is a sample progression diagram produced by SWAT. Time goes from left to right; travel is from top to bottom. Every blue line is a vehicle, and the horizontal lines are intersections with their effective green and red periods for thru traffic indicated. One can see how vehicles queue and discharge; how trajectories sometimes end as vehicles turn off (evident by "holes" in the discharge flow; how trajectories begin at intermediate intersections as vehicles turn on at intersections or midblock.

As a result of the platoon dispersion model, lines of moving vehicles between any pair of intersections are not strictly parallel. They tend to fan out, with the platoon leader going faster and the followers slower. Notice, also, how holes in the discharge flow from an intersection tend to disappear by the time the flow reaches the next intersection; this is also a reflection of the platoon dispersion model, in which a vehicle with a large gap ahead of it tends to go faster and thus fill the gap.

At each intersection, vehicle arrivals that qualify as speeding opportunities are indicated with magenta flags (short vertical lines). A full-height flag represents a full speeding opportunity, while half-height flags represent partial speeding opportunities, that is, vehicles whose probability of being a speeding opportunity is greater than 0 but less than 1. This concept of full and partial speeding opportunities is explained in the next chapter. Figure 7 shows an example of graph output from SWAT.

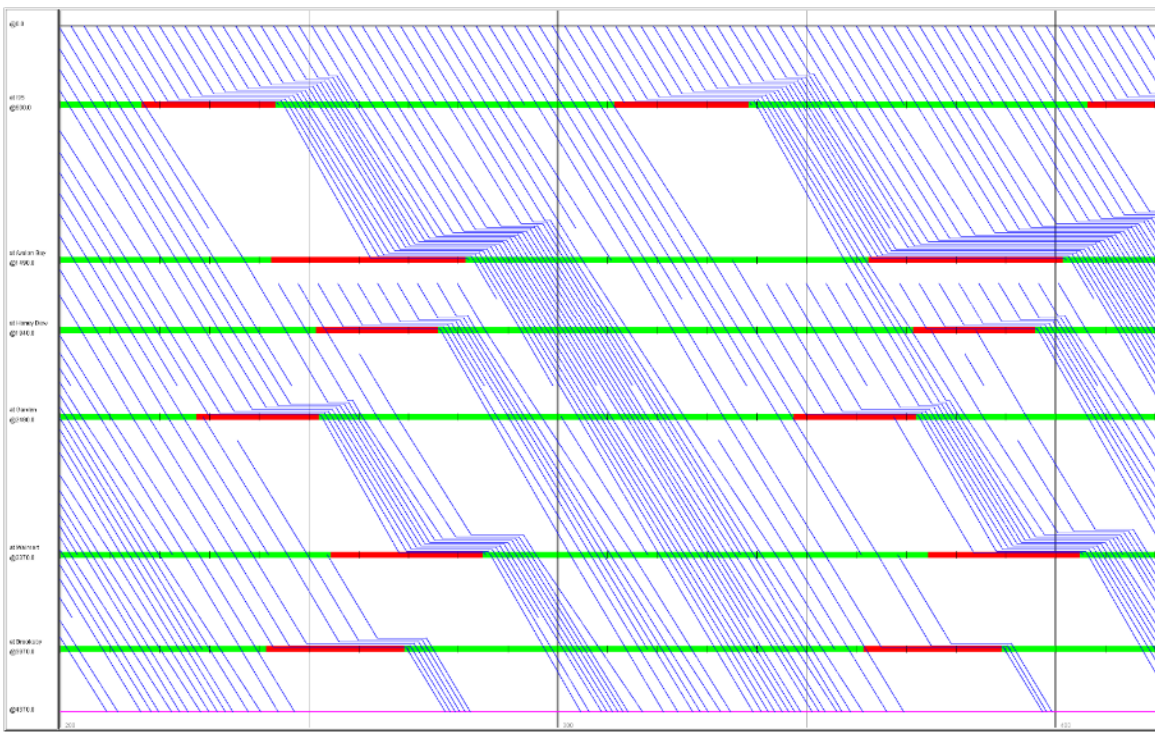

Figure 7- Graph output of SWAT

3.4.2 Sample Output Table

Figure 8 illustrates a table produced by SWAT with the number of speeding opportunities and vehicle delay (for thru vehicles on the arterial) by intersection and overall. It shows the count of thru traffic at each intersection, so that speeding opportunities can also be expressed as a percentage of thru traffic. Corridor travel time is given as the average travel time of all vehicles traveling the full length of the corridor. Users can save a copy as a Microsoft Excel file.

Export Results to Excel File

RESULTS

Route 114, EB, Weekday a.m. peak.

Base case, *

Safe Waves Analysis Tool / 2024-02-13 11:50:32

INTERSECTION	**Speeding Opp's (per hr)**	**Thru Volume (Veh/h)**	**Speeding Opp's (% of thru veh's)**	**Thru Delay (s/veh)**	**Cycle len (s)**	**Corridor Travel Time (s)**	**Corridor Speed (mph)**
1. at I95	336	1223	27.5 %	6.2	95.0		
2. at Avalon Bay	232	1162	20.0 %	9.7	120.0		
3. at Honey Dew	388	1443	26.9 %	2.1	120.0		
4. at Garden	262	1296	20.3 %	3.4	120.0		
5. at Walmart	293	1389	21.1 %	4.4	120.0		
6. at Brooksby	196	1152	17.1 %	2.8	120.0		
TOTAL	1707	7665	22.3 %	28.6		103.5	26.6
AVERAGE per intersection	284	1277	22.3 %	4.8	115.8	17.2	

Figure 8- Speeding opportunities results table

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4 Safe Waves Analysis Tool User Guide

The Speeding Opportunities Analysis Tool (SWAT) web-app can be accessed through this address:

http://newton.neu.edu:8080/massdot/

This app runs on a Northeastern University server and is free.

4.1 Steps to Follow

4.1.1 Download Resources as Needed

The web app's opening page has a list of resources including a User Guide, input file template, and two sample input files (Figure 9). Users can skip this step if they already have an input file template.

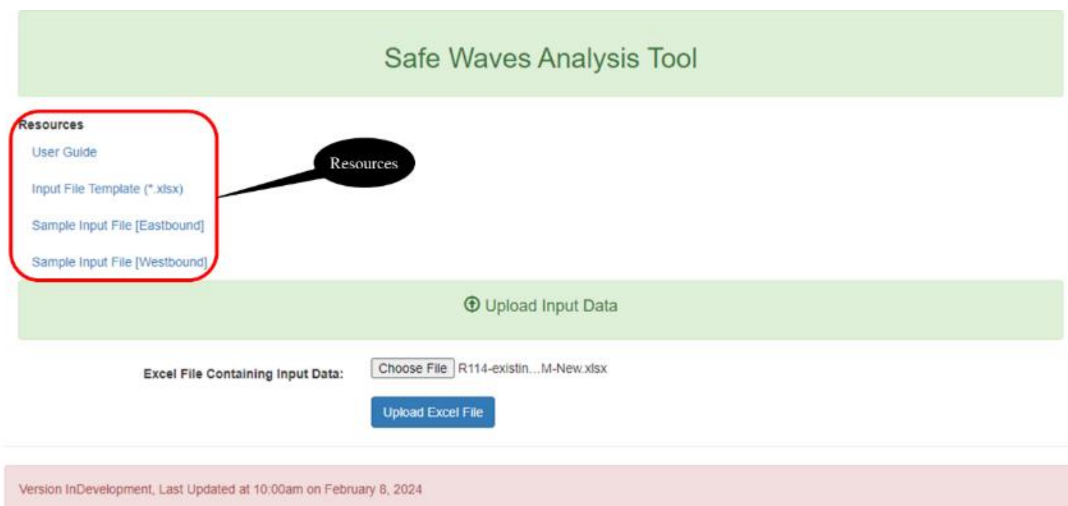


Figure 9- Resources for user

4.1.2 Complete the Input File

The app works for one direction at a time. Complete the input file for a chosen arterial direction. Instructions on completing the input file are provided in a later section.

Note: To analyze both directions of an arterial, two input files are needed. Because intersections must be listed in the order for the specified direction, the input file for one direction will list intersections in the opposite order to the other direction's input file.

4.1.3 Upload the Input File

On the opening page of the web app (Figure 10), users can choose the input file they wish to upload and then and upload it.

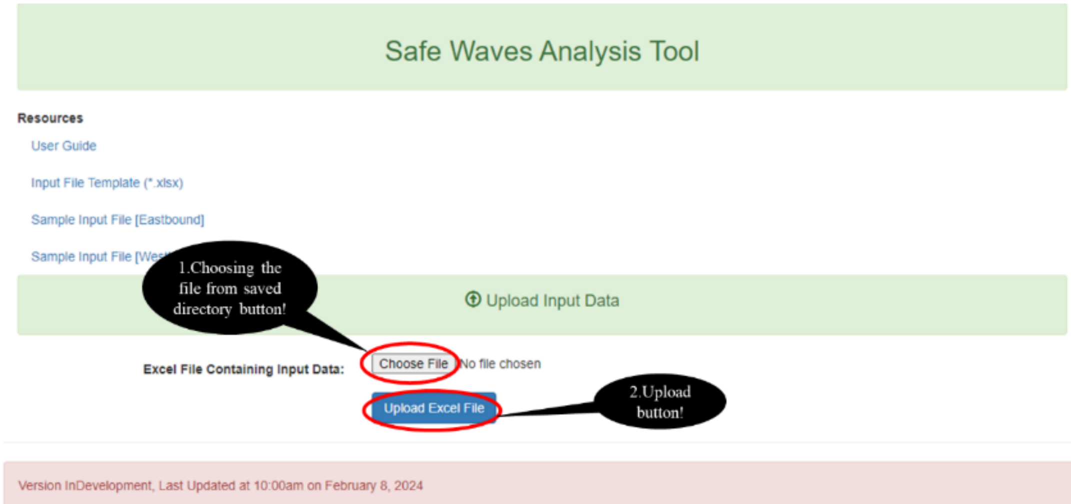


Figure 10- Choosing file and upload

4.1.4 Review Imported Data and Continue

The web-app opens a new page in which it echos the imported information from the input file (Figure 11).

This echo table includes, at the bottom, three rows beyond what's in the input file. The first added row is calculated imbalance in volume for a segment; a negative value means that there are more entries than exits, and a positive the opposite. The next two rows are adjusted midblock entry and exit volumes calculated to correct the imbalance and used in the app. Users can save a copy of the echo table as a Microsoft Excel file in their local machine. If everything seems OK, click <Process Input>.

Process Input File | Export to Excel File

INTERSECTION 1		INTERSECTION 2		INTERSECTION 3		INTERSECTION 4		INTERSECTION 5		INTERSECTION 6	
Intn	3.0	Intn	6.0	Intn	8.0	Intn	12.0	Intn	15.0	Intn	18.0
Intn name	at I95	Intn name	at Avalon Bay	Intn name	at Honey Dew	Intn name	at Garden	Intn name	at Walmart	Intn name	at Brooksby
C	95.0	C	120.0	C	120.0	C	120.0	C	120.0	C	120.0
D_From	500.0	D_From	990.0	D_From	450.0	D_From	550.0	D_From	880.0	D_From	600.0
N_Lanes_Thru	2.0	N_Lanes_Thru	2.0	N_Lanes_Thru	2.0	N_Lanes_Thru	2.0	N_Lanes_Thru	2.0	N_Lanes_Thru	2.0
vRight_Art	191.0	vRight_Art	57.0	vRight_Art	1.0	vRight_Art	0.0	vRight_Art	0.0	vRight_Art	67.0
vThru_Art	1224.0	vThru_Art	1553.0	vThru_Art	1529.0	vThru_Art	1389.0	vThru_Art	1248.0	vThru_Art	1265.0
vLeft_Art	0.0	vLeft_Art	18.0	vLeft_Art	65.0	vLeft_Art	216.0	vLeft_Art	0.0	vLeft_Art	51.0
Sat_Flow_Art	3525.0	Sat_Flow_Art	3539.0	Sat_Flow_Art	3500.0	Sat_Flow_Art	3525.0	Sat_Flow_Art	3536.0	Sat_Flow_Art	3514.0
Offset	27.0	Offset	3.0	Offset	11.0	Offset	108.0	Offset	15.0	Offset	0.0
Change_Interval_Art	6.0	Change_Interval_Art	5.0	Change_Interval_Art	6.0	Change_Interval_Art	5.0	Change_Interval_Art	5.0	Change_Interval_Art	7.0
Eff_G_Art	68.1	Eff_G_Art	80.9	Eff_G_Art	95.5	Eff_G_Art	95.4	Eff_G_Art	89.5	Eff_G_Art	92.2
G_End_Art_Loc	89.0	G_End_Art_Loc	115.0	G_End_Art_Loc	115.0	G_End_Art_Loc	115.0	G_End_Art_Loc	115.0	G_End_Art_Loc	115.0
Len_LT_Lane	0.0	Len_LT_Lane	200.0	Len_LT_Lane	355.0	Len_LT_Lane	480.0	Len_LT_Lane	0.0	Len_LT_Lane	215.0
Len_RT_Lane	0.0	Len_RT_Lane	100.0	Len_RT_Lane	0.0	Len_RT_Lane	0.0	Len_RT_Lane	0.0	Len_RT_Lane	0.0
Leg_Name_RT	at I95	Leg_Name_RT	at Avalon Bay	Leg_Name_RT	at Honey Dew	Leg_Name_RT	at Garden	Leg_Name_RT	at Walmart	Leg_Name_RT	at Brooksby
V_RT	*	V_RT	*	V_RT	25.0	V_RT	*	V_RT	48.0	V_RT	49.0
GreenA_Start_RTX_Loc	*	GreenA_Start_RTX_Loc	*	GreenA_Start_RTX_Loc	0.0	GreenA_Start_RTX_Loc	*	GreenA_Start_RTX_Loc	0.0	GreenA_Start_RTX_Loc	0.0
EffA_RTX	*	EffA_RTX	*	EffA_RTX	7.3	EffA_RTX	*	EffA_RTX	10.7	EffA_RTX	6.2
Sat_Flow_A_RTX	*	Sat_Flow_A_RTX	*	Sat_Flow_A_RTX	1583.0	Sat_Flow_A_RTX	*	Sat_Flow_A_RTX	1583.0	Sat_Flow_A_RTX	1611.0
GreenB_Start_RTX_Loc	*	GreenB_Start_RTX_Loc	*	GreenB_Start_RTX_Loc	*	GreenB_Start_RTX_Loc	*	GreenB_Start_RTX_Loc	*	GreenB_Start_RTX_Loc	12.2
EffB_RTX	*	EffB_RTX	*	EffB_RTX	*	EffB_RTX	*	EffB_RTX	*	EffB_RTX	9.6
Sat_Flow_B_RTX	*	Sat_Flow_B_RTX	*	Sat_Flow_B_RTX	*	Sat_Flow_B_RTX	*	Sat_Flow_B_RTX	*	Sat_Flow_B_RTX	1611.0
Sat_Flow_RTOR_RTX	*	Sat_Flow_RTOR_RTX	*	Sat_Flow_RTOR_RTX	136.0	Sat_Flow_RTOR_RTX	*	Sat_Flow_RTOR_RTX	52.0	Sat_Flow_RTOR_RTX	75.0
V_Lanegroup_RTS	*	V_Lanegroup_RTS	59.0	V_Lanegroup_RTS	*	V_Lanegroup_RTS	2.0	V_Lanegroup_RTS	*	V_Lanegroup_RTS	*
p_RTS	*	p_RTS	0.35	p_RTS	*	p_RTS	0.33	p_RTS	*	p_RTS	*
GreenA_Start_RTS_Loc	*	GreenA_Start_RTS_Loc	0.0	GreenA_Start_RTS_Loc	*	GreenA_Start_RTS_Loc	0.0	GreenA_Start_RTS_Loc	*	GreenA_Start_RTS_Loc	*
EffA_RTS	*	EffA_RTS	18.3	EffA_RTS	*	EffA_RTS	14.6	EffA_RTS	*	EffA_RTS	*
Sat_Flow_A_RTS	*	Sat_Flow_A_RTS	1724.0	Sat_Flow_A_RTS	*	Sat_Flow_A_RTS	1614.0	Sat_Flow_A_RTS	*	Sat_Flow_A_RTS	*
GreenB_Start_RTS_Loc	*	GreenB_Start_RTS_Loc	*	GreenB_Start_RTS_Loc	*	GreenB_Start_RTS_Loc	*	GreenB_Start_RTS_Loc	*	GreenB_Start_RTS_Loc	*
EffB_RTS	*	EffB_RTS	*	EffB_RTS	*	EffB_RTS	*	EffB_RTS	*	EffB_RTS	*
Sat_Flow_B_RTS	*	Sat_Flow_B_RTS	*	Sat_Flow_B_RTS	*	Sat_Flow_B_RTS	*	Sat_Flow_B_RTS	*	Sat_Flow_B_RTS	*
Sat_Flow_RTOR_RTS	0.0	Sat_Flow_RTOR_RTS	38.0	Sat_Flow_RTOR_RTS	0.0	Sat_Flow_RTOR_RTS	4.0	Sat_Flow_RTOR_RTS	0.0	Sat_Flow_RTOR_RTS	0.0
Leg_Name_LT	at I95	Leg_Name_LT	at Avalon Bay	Leg_Name_LT	at Honey Dew	Leg_Name_LT	at Garden	Leg_Name_LT	at Walmart	Leg_Name_LT	at Brooksby
V_LT	*	V_LT	*	V_LT	*	V_LT	*	V_LT	*	V_LT	*
GreenA_Start_LTX_Loc	*	GreenA_Start_LTX_Loc	*	GreenA_Start_LTX_Loc	*	GreenA_Start_LTX_Loc	*	GreenA_Start_LTX_Loc	*	GreenA_Start_LTX_Loc	*
EffA_LTX	*	EffA_LTX	*	EffA_LTX	*	EffA_LTX	*	EffA_LTX	*	EffA_LTX	*
Sat_Flow_A_LTX	*	Sat_Flow_A_LTX	*	Sat_Flow_A_LTX	*	Sat_Flow_A_LTX	*	Sat_Flow_A_LTX	*	Sat_Flow_A_LTX	*
Priority_A_LTX	*	Priority_A_LTX	*	Priority_A_LTX	*	Priority_A_LTX	*	Priority_A_LTX	*	Priority_A_LTX	*
GreenB_Start_LTX_Loc	*	GreenB_Start_LTX_Loc	*	GreenB_Start_LTX_Loc	*	GreenB_Start_LTX_Loc	*	GreenB_Start_LTX_Loc	*	GreenB_Start_LTX_Loc	*
EffB_LTX	*	EffB_LTX	*	EffB_LTX	*	EffB_LTX	*	EffB_LTX	*	EffB_LTX	*
Sat_Flow_B_LTX	*	Sat_Flow_B_LTX	*	Sat_Flow_B_LTX	*	Sat_Flow_B_LTX	*	Sat_Flow_B_LTX	*	Sat_Flow_B_LTX	*
Priority_B_LTX	*	Priority_B_LTX	*	Priority_B_LTX	*	Priority_B_LTX	*	Priority_B_LTX	*	Priority_B_LTX	*
Sat_Flow_LTOR_LTX	0.0	Sat_Flow_LTOR_LTX	0.0	Sat_Flow_LTOR_LTX	0.0	Sat_Flow_LTOR_LTX	0.0	Sat_Flow_LTOR_LTX	0.0	Sat_Flow_LTOR_LTX	0.0
V_Lanegroup_LTS	*	V_Lanegroup_LTS	*	V_Lanegroup_LTS	15.0	V_Lanegroup_LTS	96.0	V_Lanegroup_LTS	*	V_Lanegroup_LTS	14.0
p_LTS	*	p_LTS	*	p_LTS	0.26	p_LTS	0.5	p_LTS	*	p_LTS	0.42
GreenA_Start_LTS_Loc	*	GreenA_Start_LTS_Loc	*	GreenA_Start_LTS_Loc	0.0	GreenA_Start_LTS_Loc	0.0	GreenA_Start_LTS_Loc	*	GreenA_Start_LTS_Loc	0.0
EffA_LTS	*	EffA_LTS	*	EffA_LTS	7.3	EffA_LTS	14.6	EffA_LTS	*	EffA_LTS	6.2
Sat_Flow_A_LTS	*	Sat_Flow_A_LTS	*	Sat_Flow_A_LTS	1397.0	Sat_Flow_A_LTS	1360.0	Sat_Flow_A_LTS	*	Sat_Flow_A_LTS	1726.0
Priority_A_LTS	*	Priority_A_LTS	*	Priority_A_LTS	2.0	Priority_A_LTS	2.0	Priority_A_LTS	*	Priority_A_LTS	2.0
GreenB_Start_LTS_Loc	*	GreenB_Start_LTS_Loc	*	GreenB_Start_LTS_Loc	*	GreenB_Start_LTS_Loc	*	GreenB_Start_LTS_Loc	*	GreenB_Start_LTS_Loc	*
EffB_LTS	*	EffB_LTS	*	EffB_LTS	*	EffB_LTS	*	EffB_LTS	*	EffB_LTS	*
Sat_Flow_B_LTS	*	Sat_Flow_B_LTS	*	Sat_Flow_B_LTS	*	Sat_Flow_B_LTS	*	Sat_Flow_B_LTS	*	Sat_Flow_B_LTS	*
Priority_B_LTS	*	Priority_B_LTS	*	Priority_B_LTS	*	Priority_B_LTS	*	Priority_B_LTS	*	Priority_B_LTS	*
Sat_Flow_LTOR_LTS	0.0	Sat_Flow_LTOR_LTS	0.0	Sat_Flow_LTOR_LTS	0.0	Sat_Flow_LTOR_LTS	0.0	Sat_Flow_LTOR_LTS	0.0	Sat_Flow_LTOR_LTS	0.0
Known_Mid_Entries	0.0	Known_Mid_Entries	298.0	Known_Mid_Entries	0.0	Known_Mid_Entries	0.0	Known_Mid_Entries	0.0	Known_Mid_Entries	0.0
Known_Mid_Exits	0.0	Known_Mid_Exits	53.0	Known_Mid_Exits	0.0	Known_Mid_Exits	0.0	Known_Mid_Exits	0.0	Known_Mid_Exits	0.0
V_Mid_Imbalance	0	V_Mid_Imbalance	-159	V_Mid_Imbalance	-21	V_Mid_Imbalance	-47	V_Mid_Imbalance	189	V_Mid_Imbalance	-87
V_Mid_Enter	0	V_Mid_Enter	457	V_Mid_Enter	21	V_Mid_Enter	47	V_Mid_Enter	0	V_Mid_Enter	87
V_Mid_Exit	0	V_Mid_Exit	53	V_Mid_Exit	0	V_Mid_Exit	0	V_Mid_Exit	189	V_Mid_Exit	0

Figure 11- Echo of table

4.1.5 Choose Parameters for Viewing the Progression Diagram and for Identifying Speeding Opportunities

After generating trajectories, the web-app will then display a page as shown in Figure 12.

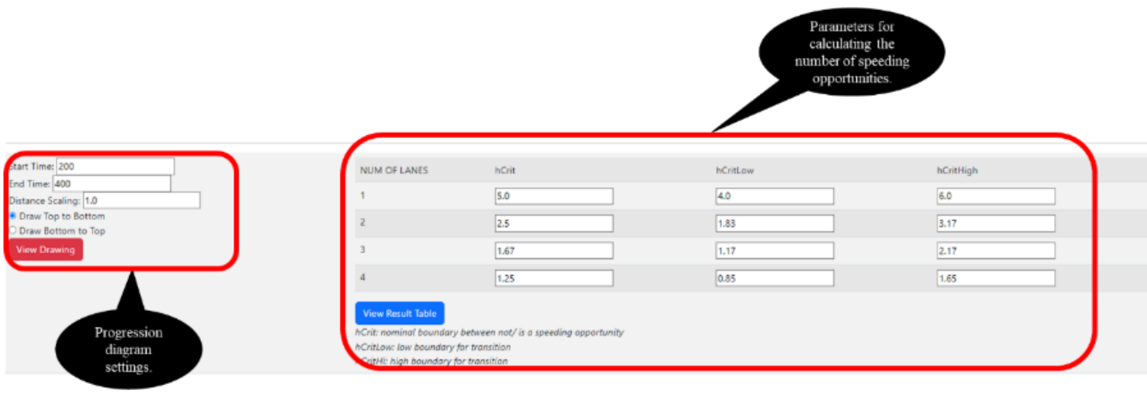


Figure 12- After process page

In the dialogue box on the left, enter preferences for drawing the progression diagram, as follow:

- Start and End Time (s). Minimum start time is 0; Maximum end time is 5000.
- Distance Scaling: The vertical axis is distance, which can be in feet or in meters (whichever is used in the input file). By default, the distance scaling factor is 1.0, which will fit the graph to the screen based on screen resolution. Users can choose a different scaling factor to zoom in or out.
- Choose either <Draw Top to Bottom> or <Draw Bottom to Top> to indicate whether the first intersection (based on the input file) goes at the top or bottom of the progression diagram. Typically, users will choose Top-to-Bottom when analyzing one direction and Bottom-to-Top when analyzing the other direction, so that intersections take the same location in either direction.

On the right, the default parameters used for identifying speeding opportunities will be displayed; users can change them if they wish. For each possible number of lanes N in the thru lane group, there is a critical headway h_{crit} as well as a pair of boundary headways, *hCritLow* to *hCritHigh*. H_{crit} is the nominal threshold for a speeding opportunity. When N=1, the threshold is 5 s; for other values of N, h_{crit} = 5/N. Then, because vehicles are represented by deterministic trajectories when in fact there is some randomness in driver behavior, the probability than any given headway represents a speeding opportunity is modeled as a function that increases linearly from 0 to 1 as the headway increases from *hCritLow* to *hCritHigh* which respectively, whose default values are $h_{crit} – 2/(N+1)$ and $h_{crit} + 2/(N+1)$.

4.1.6 View the Progression Diagram and/or the Speeding Opportunities Summary Table

The web-app page shown in Figure 12 has a button <View Drawing> for viewing the progression diagram, illustrated in Figure 13.

In the progression diagram, every horizontal line represents an intersection. The colors green and red represent the signal state for arterial thru traffic in the direction of interest.

Blue lines represent vehicles, which move to the right (in the positive time direction). That can mean going either up or down, depending on users selected top-to-bottom or the opposite. Blue lines starting just after an intersection are midblock entries or turning movements from a side street. Blue lines that end shortly before an intersection are midblock exits or vehicles that turn into an exclusive turn lane. Blue lines that end at an intersection are vehicles that turn off from a shared turn-thru lane.

Where the vehicle trajectories cross an intersection line, some of them have a magenta flag (short vertical line). A full-height flag means the headway is greater than *hCritHigh* and so the vehicle has a full speeding opportunity. A half-height flag means the headway is between *hCritLow* and *hCritHigh*, which means that vehicle has a probability between 0 and 1 of being a speeding opportunity. While every vehicle with a "partial" speeding opportunity has the same height flag, calculations of speeding opportunities use the actual probability, which can range from 0 to 1.

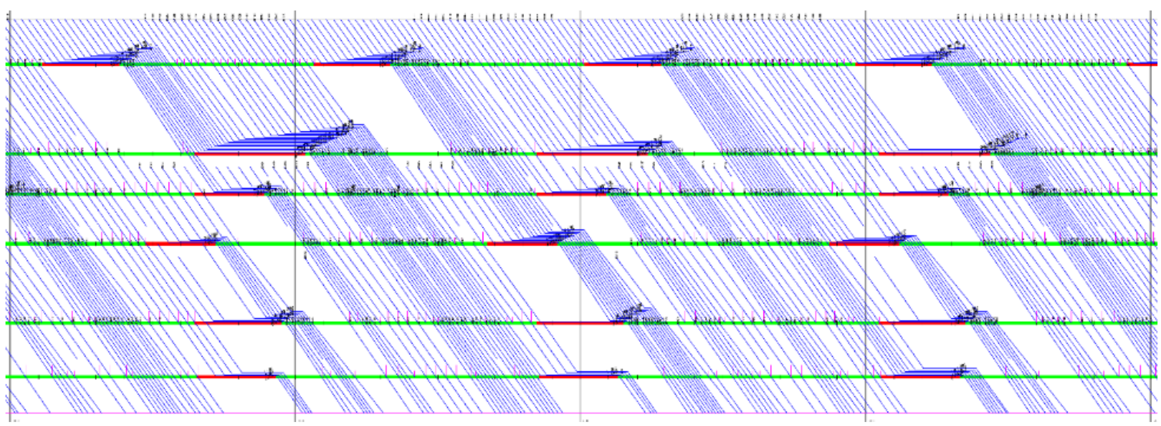


Figure 13- Progression diagram with speeding opportunity flags

4.1.7 View the Speeding Opportunities Table

The page shown in Figure 4 also has the button <View Result Table>. Return to that page (using the <back> button if needed) and press that button to view the speeding opportunities summary table.

Figure 14 shows a sample speeding opportunities table.

Export Results to Excel File

RESULTS

Route 114, EB, Weekday a.m. peak.

Base case, *

Safe Waves Analysis Tool / 2024-02-13 11:50:32

INTERSECTION	Speeding Opp's (per hr)	Thru Volume (Veh/h)	Speeding Opp's (% of thru veh's)	Thru Delay (s/veh)	Cycle len (s)	Corridor Travel Time (s)	Corridor Speed (mph)
1. at I95	336	1223	27.5 %	6.2	95.0		
2. at Avalon Bay	232	1162	20.0 %	9.7	120.0		
3. at Honey Dew	388	1443	26.9 %	2.1	120.0		
4. at Garden	262	1296	20.3 %	3.4	120.0		
5. at Walmart	293	1389	21.1 %	4.4	120.0		
6. at Brooksby	196	1152	17.1 %	2.8	120.0		
TOTAL	1707	7665	22.3 %	28.6		103.5	26.6
AVERAGE per intersection	284	1277	22.3 %	4.8	115.8	17.2	

Figure 14- Speeding opportunities output table

4.2 Conventions for Determining Offsets and Green Start / End Times

It is assumed that users will have already analyzed the corridor using a signal timing and intersection analysis app such as Synchro. (Instructions that follow assume Synchro as the signal timing app.) Transferring timing information from Synchro to the input file requires adhering to conventions described here.

4.2.1 Make Synchro's Offset References Follow the "End of Last Arterial Green" Convention

In the Synchro model, all intersections must use the follow reference point standard for offsets: Reference point = *end of green* for the coordinated phase that *ends last*. For example, in Figure 15, phases 2 and 6 are the coordinated phases, but the reference point (red marker) is NOT the end of both phases 2 and 6, but is end of green for phase 2 only.

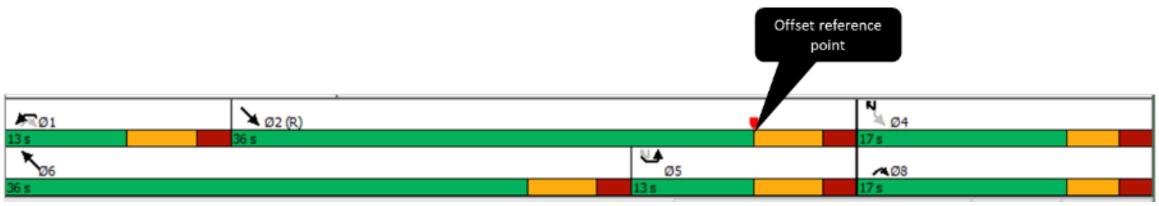


Figure 15- Offset reference

If the Synchro model has intersections has offsets that do not follow this convention, Synchro users can simply change the reference point in Synchro; Synchro will automatically recalculate offsets appropriately.

Many Synchro users follow the practice of designating both coordinated phases as reference phases. If the two arterial phases end simultaneously, as is the case with leading lefts, having both designated as reference phase will not violate SWAT conventions. But at intersections with lagging lefts or lead-lag phasing for the arterial, *the Synchro model will have to adjusted so that only one coordinated phase, the one that ends later, is designated as the reference phase.* This is because in Synchro, where there are two reference phases, Synchro calculates offsets based on the reference phase whose green ends first, which is not consistent with the convention used by SWAT.

4.2.2 Adjusting Synchro Ring Diagrams for Local 0 and Actuated Effective Green Time

Synchro provides a very helpful ring diagram. To use its information in SWAT app, however, two things must be changed. First, in SWAT's convention, local time 0 is defined as the *start of the next phase after the last arterial (coordinated) phase ends*, as illustrated in Figure 16 (a). All timing information entered into SWAT for a given intersection should be calculated based on this local zero point. In Figure 16 (b), the ring diagram has been rearranged so that it begins at local time 0.

Second, for green time, use *actuated effective green* for each phase, not the *programmed green* which is shown in Synchro's diagram. Actuated effective green times can be found in a Synchro report. In Figure 16 (c), actuated effective green times have been superimposed on the ring diagram (orange boxes), along with the length of the programmed change interval (yellow + redClear, blue boxes).

SWAT requires calculating when the actuated effective green starts and ends. In principle, one should be able to determine those start and end time either calculating forward from time 0 or backwards from the end of the cycle. As an example of calculating forwards, in Figure 16 (c), to calculate *start of green* for phase 6, follow the lower ring: it's 10 + 6 = 16; to get phase 6's *end of green*, add 37.2, so it's 16 +37.2 = 53.2.

If none of an intersection's phases are ever skipped (i.e., all phases are on recall), it doesn't matter whether one calculates forwards or backwards.

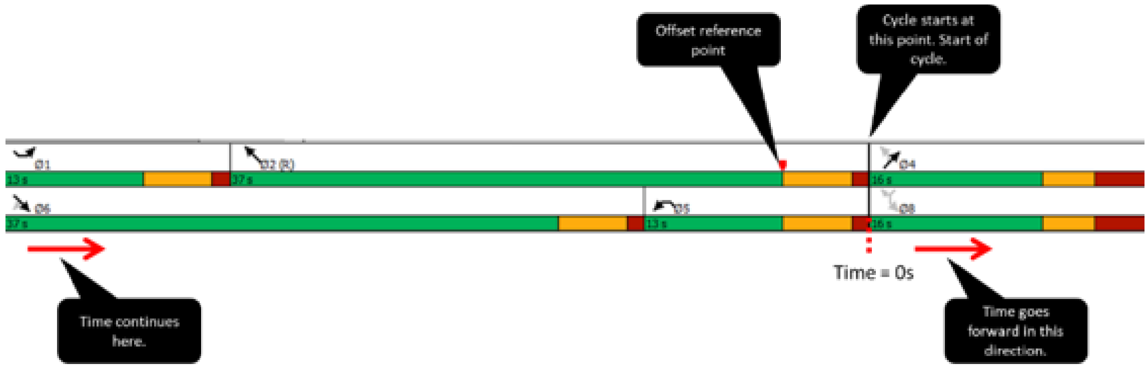
However, if the cycle has phases that are sometimes skipped for lack of a call, then the direction of calculation matters. (That is, one can get different results going forward vs. backwards, and one of them better represents the effective green time.) When should green times be determined by calculating forward, and when by calculating backward?

First determine *which phase in each ring is most likely to be skipped*. Often, the choice is obvious; if it isn't, make an educated guess. (The lower the ratio of actuated effective green to programmed green, the more likely it is that a phase is skipped.)

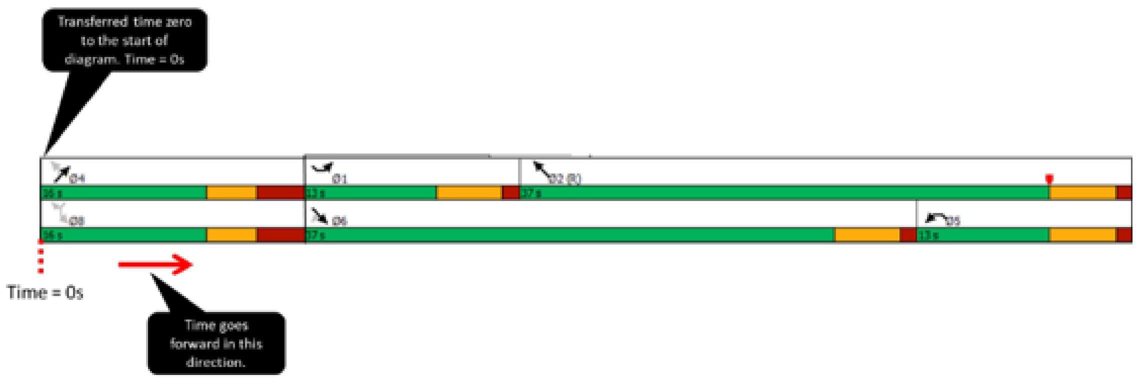
Then use the following rules for whether to calculate forwards or backwards:

- For coordinated phases whose split ends at local time 0: calculate backwards from the end of the cycle.
- For other phases, go forward from the cycle's 0 point up to the end of the phase most likely to be skipped, and go backward from the cycle's 0 point to the start of the phase that follows the phase most likely to be skipped.

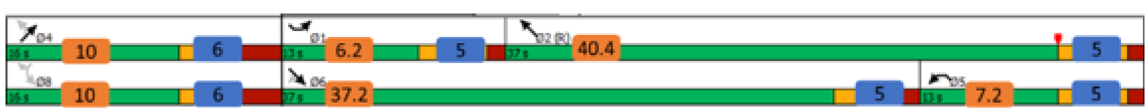
By following this rule, the phase designated as "most likely to be skipped" may get a change interval (yellow + redClear) that is shorter than its programmed change interval. This is appropriate, because the average duration of a change interval for a phase that is sometimes skipped is shorter than its programmed duration.



a) Actual ring diagram in Synchro



b) Rearranged to begin at local time zero



c) With actuated effective green times and change interval times added

Figure 16- Reading the local timing

In Figure 16 (c), suppose phases 01 (upper ring) and 05 (lower ring) are most likely to be skipped. Then green start and ed times for the various phases should be calculated as follows:

- Calculate backward from the end of the cycle for phase 02. It's green end is at (66 – 5) =61, and its green start is 40.4 s before that, or time 20.6.
- Calculate forward from time 0 for all other phases. For example,
    - phase 8's green start at time 0 and ends at 10
    - phase 6's green starts at 10+6 = 16 and ends 37.2 s later, at 53.2

- phase 5’s green starts 5 s later at 58.2 and ends 7.2 s later, at 65.4. Because cycle length is 66 s, that leaves less than 1 s for phase 5’s average change interval during, which suggests that phase 5 is skipped most of the time (when skipped, its change interval duration is 0, of course).

4.3 Completing Input Data

The input file is a Microsoft Excel file. Users can download a shell, as explained earlier, and then fill it with data on the arterial being studied. Each input file applies to a single direction of the arterial.

4.3.1 Units

Time is in seconds.

Distances may be in either meters or ft. Distances shown in the progression diagram will be in the corresponding units.

Speed must be in units of distance per second, i.e., ft/s or m/s.

4.3.2 Colored Cells

By convention, input file cells that are light orange are headers that are not read as data inputs.

Input file cells that are blue are fixed; users must not change them.

4.3.3 Master Tab

The input file is a spreadsheet with two tabs, Master Tab, and Direction 1 input Tab. The input file is only for one direction at a time. For a two-way arterial, users will need to create separate input files for each direction, using the “Direction 1” tab in both cases.

The Master Tab is for general information about the arterial that is being studied. This information includes the name of street, name of the direction, and time of study for report headings.

Also included are corridor-wide parameters: progression speed (speed that vehicles will go when unconstrained) and length per vehicle in the queue (typically 23 ft or 25 ft).

The user can choose the unit of speed which is a drop-down list containing “mph” and “km/h” and it will automatically adjust unit of length for length of vehicles in queue in the Master Tab, distance between intersections and turning pocket lanes in the Direction 1 Tab.

4.3.4 Direction 1 Tab:

In the Direction 1 tab, the first three columns are fixed and include variables names, units, and a short description for each row.

Beginning in the fourth column, there should be one column for every intersection in the study segment. Add columns as necessary.

Starting from the top, the sheet contains 4 main blocks, as shown in the right margin of Figure 17, with data for:

- Block 1: the arterial in the direction of interest
- Block 2: the side street approach from which cars turn right to join the arterial in the direction of interest.
    - This block is subdivided into two subblocks (left margin of Figure 9), one of which is filled while the other remains empty, depending on whether right turns are from an exclusive right turn lane.
- Block 3: the side street approach from which cars turn left to join the arterial in the direction of interest.
    - This block is subdivided into two subblocks (left margin of Figure 9), one of which is filled while the other remains empty, depending on whether left turns are from an exclusive left turn lane.
- Block 4: midblock entries and exits

Note: Any cell which is highlighted blue must not be changed. Some of them contain fixed text, while other contain a formula (so the calculated value may change, but the formula mustn't change). When adding new columns for additional intersections, be careful to copy those formula.

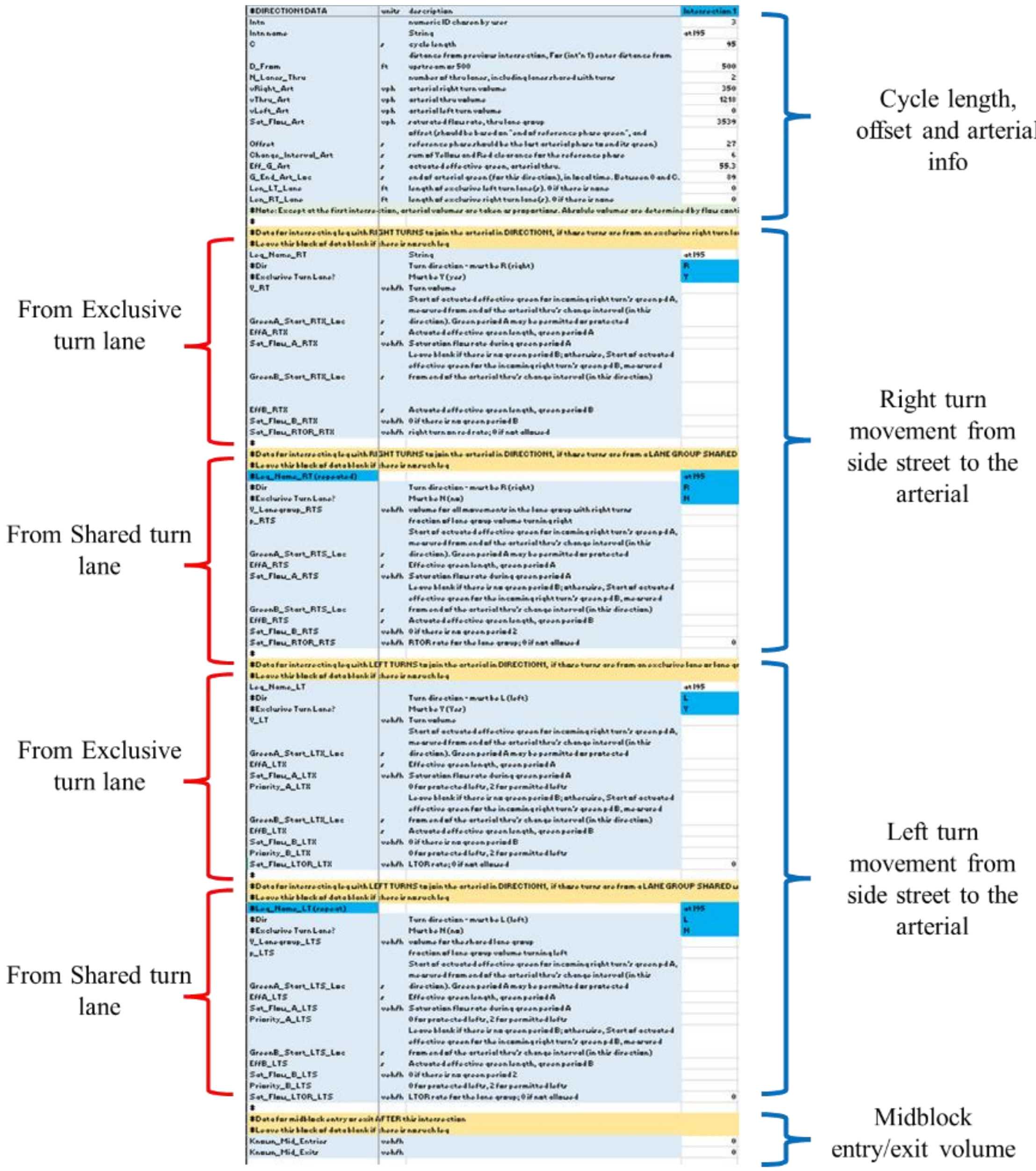


Figure 17- General sections of input file

4.3.5 Info on the Arterial

In this block, enter cycle length, arterial volumes going thru and turning and their saturation flow rates, and arterial phase offsets. (See earlier discussion about how offsets must be based on when the last arterial green ends.) Table 4 shows the definition of each parameter for arterial.

Table 4- Arterial data

Parameter	units	description
Intn	-	numeric ID chosen by user
Intn name	-	Name of the side street
C	s	Cycle length
D_From	ft or m	Distance from previous intersection's stop line to this intersection stop line (Enter 500 for the first intersection)
N_Lanes_Thru	-	Number of thru lanes, including lanes shared with turns for the study direction
vRight_Art	vph	Arterial right turn volume
vThru_Art	vph	Arterial thru volume
vLeft_Art	vph	Arterial left turn volume
Sat_Flow_Art	vph	Saturated flow rate, thru lane group
Offset	s	Offset (should be based on "end of reference phase green", and reference phase should be the last arterial phase to end its green)
Change_Interval_Art	s	Sum of Yellow and Red clearance for the reference phase
Eff_G_Art	s	Actuated effective green length, (direction of interest)
G_End_Art_Loc	s	End of arterial green (for this direction), in local time. Between 0 (end of reference phase green) and C.
Len_LT_Lane	ft or m	length of exclusive left turn lane(s). 0 if there is none
Len_RT_Lane	ft/m	length of exclusive right turn lane(s). 0 if there is none

4.3.6 Info on Right Turns from the Side Street

SWAT assumes there will be at most one leg from which side street right turns will join the arterial in the direction being studied. The right turns block is for data on right turns from *that* approach of the side street. (To be clear: Typically, vehicles from a side street may turn right or left from both legs of the side street; but from only one leg will right turn join the arterial *in the direction of interest*.)

This block consists of two sub-blocks, as shown in Figure 17. One subblock gets completed and the other is left blank, depending on whether the right turns come from an exclusive turn lane or from a lane shared with thru and/or left turning traffic. (Leave both sub-blocks blank if there are no right turns at all joining the arterial in the direction of interest.)

The values entered in these sections are traffic volumes, timing data for the phase(s) that this movement uses, and saturation flow rate(s), as explained in Table 5. Note how inputs differ depending on whether turns come from an exclusive versus shared lane. Note also that data can be entered on two different green periods, one called period A and one called period B; this is explained under the next heading.

Table 5- Right turn from side street

Parameter	Unit	From Shared Lane	From Exclusive Lane(s)	Description
#Dir	-	✓	✓	Turn direction - must be R (right)
#Exclusive Turn Lane?	-	✓	✓	Must be N (no) for Shared Lane group and Y (yes) for exclusive lane(s)
V _RT = right turn volume	veh/h	-	✓	volume for all movements in the lane group with right turns – with an exclusive lane, it's right turn volume only
V_Lanegroup_RTS = volume for right turns and all other movements than belong to the same lane group	veh/h	✓	-	volume for all movements in the lane group with right turns – for right turns from a shared lane, it's right turn volume plus thru and possibly left turn volume
p_RTS	-	✓	-	fraction of lane group volume turning right
GreenA_Start_RTS*_Loc/ GreenA_Start_RTX**_Loc	s	✓	✓	Start of actuated effective green for incoming right turn's green pd A, measured from end of the last arterial thru's change interval (in this direction). Green period A may be permitted or protected
EffA_RTS/ EffA_RTX	s	✓	✓	Effective green length, green period A
Sat_Flow_A_RTS/ Sat_Flow_A_RTX	veh/h	✓	✓	Saturation flow rate during green period A
GreenB_Start_RTS_Loc/ GreenB_Start_RTX_Loc	s	✓	✓	Leave blank if there is no green period B; otherwise, start of actuated effective green for the incoming right turn's green pd B, measured from end of the last arterial thru's change interval (in this direction)
EffB_RTS/ EffB_RTX	s	✓	✓	Actuated effective green length, green period B
Sat_Flow_B_RTS/ Sat_Flow_B_RTX	veh/h	✓	✓	0 if there is no green period 2
Sat_Flow_RTOR_RTS/ Sat_Flow_RTOR_RTX	veh/h	✓	✓	RTOR rate for the lane group; 0 if not allowed

* RTS: right turn from a shared lane group lane(s)
** RTX: right turn from an exclusive lane

4.3.7 Timing periods for side street turning movements

Vehicles turning right from a side street may do so during green intervals with permitted turns, green intervals protected turns, and on red (right turn on red (RTOR) and, for one way streets turning onto one-way streets, left turn on red (LTOR)). Therefore, there is a place for entering data on two different green intervals as well as for turns on red.

The two green intervals are called A and B. Where there are both protected and permitted green intervals for the side street turns, interval A must be the one that comes first in the ring diagram. Thus, depending on which comes first, interval A may be for protected turns or for permitted turns.

If there isn't protected + permitted phasing and is therefore only one green interval, enter data for interval A and leave blank the fields for the second green interval (interval B).

Figure 18.shows the fields for right turns from exclusive turn lane.

	Leg_Name_RT		String
	#Dir		Turn direction - must be R (right)
	#Exclusive Turn Lane?		Must be Y (yes)
	V_RT	veh/h	Turn volume
Section A- first period.	GreenA_Start_RTX_Loc	s	Start of actuated effective green for incoming right turn's green pd A, measured from end of the arterial thru's change interval (in this direction). Green period A may be permitted or protected
	EffA_RTX	s	Actuated effective green length, green period A
	Sat_Flow_A_RTX	veh/h	Saturation flow rate during green period A
Section B- second period.	GreenB_Start_RTX_Loc	s	Leave blank if there is no green period B; otherwise, Start of actuated effective green for the incoming right turn's green pd B, measured from end of the arterial thru's change interval (in this direction)
	EffB_RTX	s	Actuated effective green length, green period B
	Sat_Flow_B_RTX	veh/h	0 if there is no green period B
Sat flow rate for turning on red if permitted!	Sat_Flow_RTOR_RTX	veh/h	right turn on red rate; 0 if not allowed

Figure 18- Turning movements' periods

4.3.8 Left turns from side street to the arterial

As with right turns, there is usually at most one side street approach from which cars turn left to join the arterial in the direction of interest. Like the right turn block, the left turn block supports two green intervals (one protected, one permitted, to be entered in the order in which they occur) and turning on red.

The values entered in these sections are turning movement volume, timing data for the phase(s) that these movement use, and saturation flow rate(s), as explained below in Table 6.

Table 6- Left turn from side street

Parameter	Unit	From Shared Lane	From Exclusive Lane(s)	Description
#Dir	-	✓	✓	Turn direction - must be L (left)
#Exclusive Turn Lane?	-	✓	✓	Must be N (no) for Shared Lane group and Y (yes) for exclusive lane(s)
V_LT = right turn volume	veh/h	-	✓	volume for all movements in the lane group with left turns – with an exclusive lane, it's left turn volume only
V_Lanegroup_LTS = volume for left turns and all other movements than belong to the same lane group	veh/h	✓	-	volume for all movements in the lane group with right turns – in case of left turns from a shared lane case, it's left turn volume plus thru and possibly right turn volume
p_LTS	-	✓	-	fraction of lane group volume turning left
GreenA_Start_LTS_Loc/ GreenA_Start_LTX_Loc	s	✓	✓	Start of actuated effective green for incoming right turn's green pd A, measured from end of the last arterial thru's change interval (in this direction). Green period A may be permitted or protected
EffA_LTS/ EffA_LTX	s	✓	✓	Effective green length, green period A
Sat_Flow_A_LTS/ Sat_Flow_A_LTX	veh/h	✓	✓	Saturation flow rate during green period A
Priority_A_LTS/ Priority_A_LTX	-	✓	✓	0 for protected lefts, 2 for permitted lefts
GreenB_Start_LTS_Loc/ GreenB_Start_LTX_Loc	s	✓	✓	Leave blank if there is no green period B; otherwise, start of actuated effective green for the incoming right turn's green pd B, measured from end of the last arterial thru's change interval (in this direction)
EffB_LTS/ EffB_LTX	s	✓	✓	Actuated effective green length, green period B
Sat_Flow_B_LTS/ Sat_Flow_B_LTX	veh/h	✓	✓	0 if there is no green period 2
Priority_B_LTS/ Priority_B_LTX	-	✓	✓	0 for protected lefts, 2 for permitted lefts
Sat_Flow_LTOR_LTS/ Sat_Flow_LTOR_LTX	veh/h	✓	✓	LTOR rate for the lane group; 0 if not allowed

* LTS: right turn from a shared lane group lane(s)

** LTX: right turn from an exclusive lane

4.3.9 Midblock Entries / Exits:

The last block is for known midblock entries and exits. It means that if there is any unsignalized intersection or parking lot and the entry and exit volume of them are available, user can enter the values in this segment and the app will automatically calculate the difference of upstream and downstream volumes for each segment with accounting for known midblock entries and exits and use those values as midblock entry or exit in the simulation.

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5 Safe Waves Timing Plans for Rt.114, Danvers, MA

5.1 Introduction

The study site (Figure 19) is the 0.66 mile stretch of State Route 114 located in Danvers, where it is Andover Street and is classified as an arterial, between (and including) intersections with Brooksby Village Dr in the east and I-95 in the west.

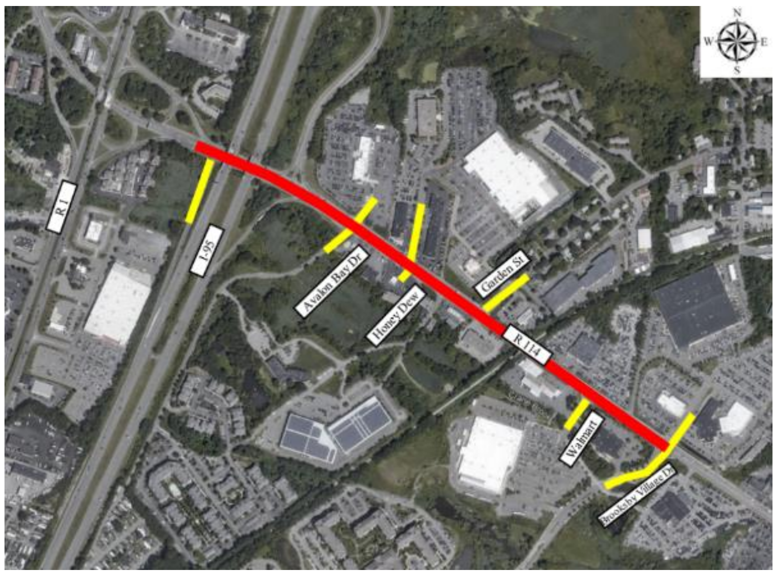


Figure 19- Study segment of Route 114

5.2 Intersections

The study corridor has six signalized and one unsignalized intersection. The five eastern signalized intersections are coordinated with three different plans during weekday daytime, with cycle lengths of 120 seconds in the am peak (6:30-9:30) and midday (9:30-15:30) and 95 seconds in the pm peak (15:30-18:30).

Midway along the corridor, a rail-trail perpendicular to Rt 114 passes overhead. For pedestrians going between areas north and south of Rt 114, this rail trail is the most direct route, and is a reason for the very low pedestrian demand observed at the Rt 114 intersections.

5.3 Overall Geometric Characteristics

Rt. 114 in this segment is undivided and has two lanes per direction. At intersections, wherever left turns are allowed, there are exclusive turn lanes with protected-only phases.

5.4 Speed

The posted speed on this part of Rt. 114 is 40 mph. The speed limit for side streets is 25 mph.

5.5 Traffic Volumes

MassDOT supplied volume data for six weekdays (28-30 of September and 5-7 of October) in 2021 for all of the intersections except the two (one of which is signalized) at I-95. As part of this study, weekday turning volumes were counted at those two intersections.

5.6 Critical Intersections

Three intersections of Brooksby Village Dr, Garden St, and Honey Dew are the critical ones in terms of constraining the cycle length and potentially being capacity bottlenecks. As such, reliable estimates of arterial volumes at these intersections are most important and were given extra attention.

For the three critical intersections, the six days of available counts were analyzed to determine design volume, peak hour factor (PHF) and degree of saturation (X_t). A look at the data showed that at some of the intersections, eastbound and westbound had been confused; They were corrected. Still, some outliers were identified – counts that were inconsistent with counts at the upstream and downstream intersection – and excluded from the data, as described later. The average of the remaining days was used as design volume.

For the non-critical intersections, average volumes were used. Figure 20 represents the hourly volume that are used to design the cycle plan at all intersections in all periods of day.

		Eastbound*			Westbound*			Northbound			Southbound		
		EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
AM peak	R114 @ I95 West		1224	191	140	683	96						
	R114 @ I95 East	57	1167			814	216			298	105		
	R114 @ Avalon Bay	18	1553	57	38	1153	4	93	15	59			
	R114 @ Honey Dew	65	1529	1	16	1160	24	19	5	25	15	2	40
	R114 @ Garden	216	1389	2	0	982	71	2	2	2	96	4	170
	R114 @ Walmart		1248		31	992		73		48			
	R114 @ Brooksby	51	1265	67	68	1041	26	0	0	49	14	2	17
		Eastbound*			Westbound*			Northbound			Southbound		
		EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Midday	R114 @ I95 West		1229	310	256	856	130						
	R114 @ I95 East	82	1058			1073	303			99			258
	R114 @ Avalon Bay	23	1376	60	60	1580	27	69	14	38			
	R114 @ Honey Dew	75	1338	2	22	1466	32	18	3	22	40	2	99
	R114 @ Garden	179	1256	7	0	1246	136	15	11	10	164	7	194
	R114 @ Walmart		1199		96	1310		248		167			
	R114 @ Brooksby	56	1265	54	94	1238	17	1	0	112	40	6	51
		Eastbound*			Westbound*			Northbound			Southbound		
		EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
PM Peak	R114 @ I95 West		1218	350	338	1287	115						
	R114 @ I95 East	102	1194		102	1356	539			129			287
	R114 @ Avalon Bay	32	1437	131	93	1838	29	117	12	44			
	R114 @ Honey Dew	61	1372	1	18	1785	25	26	2	18	34	3	124
	R114 @ Garden	214	1288	6	0	1487	117	17	9	7	152	9	219
	R114 @ Walmart		1211		69	1486		133		233			
	R114 @ Brooksby	50	1340	50	89	1450	16	1	0	118	51	8	60

*: Route 114 (Arterial) is the east-west direction

Figure 20- Hourly turning movement volumes, Rt,114

5.6.1 Volumes at Critical Intersections

Intersections at Brooksby Village Dr, Garden St, and Honey Dew are critical because their timing needs are greatest due to pedestrian crossing and high volumes. There is a crosswalk across Rt 114 at Brooksby Village Dr and at Garden St. The Honey Dew intersection has a high traffic volume (traffic is greatest near I-95) and considerable side street volume associated with Lowe's.

The following figures show the volumes at each intersection for six days at AM peak, Midday, and PM peak. The numbers in red color are the outliers that were excluded based on arterial thru volumes (EBT, WBT) that were inconsistent. They could represent simple errors, or perhaps days in which Route 128 was jammed and some of its traffic diverted to Route 114. If the latter, the signal timing is not meant for such days.

	Dates	AM peak											
R 114 @ Brooksby Village Dr		EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
	9/28/2021	60	1237	63	70	1058	23	0	0	50	11	3	18
	9/29/2021	39	1258	54	68	1036	25	0	0	51	12	1	18
	9/30/2021*	47	1232	60	72	999	34	0	0	56	14	2	21
	10/5/2021	62	1269	83	77	1043	25	0	0	40	13	1	18
	10/6/2021	42	1295	68	56	1025	29	0	0	54	20	2	12
	10/7/2021*	31	1391	56	65	1084	18	0	0	48	15	2	15
	Dates	Midday											
		EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
	9/28/2021	47	1185	47	83	1202	13	3	1	96	35	13	52
	9/29/2021	65	1281	56	70	1278	19	0	0	114	40	6	56
	9/30/2021	56	1343	45	98	1272	20	0	0	109	51	5	54
	10/5/2021	59	1227	62	106	1154	18	0	0	119	40	5	48
	10/6/2021	49	1305	65	85	1230	20	0	0	113	50	3	45
	10/7/2021	58	1247	49	119	1289	14	0	0	118	25	4	53
	Dates	PM peak											
		EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
	9/28/2021	45	1190	31	6	1413	6	1	0	89	40	11	66
	9/29/2021	53	1392	53	103	1458	13	1	0	117	34	9	62
	9/30/2021*	32	1568	59	111	1313	17	3	0	109	44	8	44
	10/5/2021	48	1322	53	121	1416	20	0	0	155	46	6	62
	10/6/2021*	81	1336	42	81	1709	16	0	0	100	81	10	56
	10/7/2021	40	1232	61	113	1390	21	0	0	135	58	5	67

*: Volumes on these dates considered outliers and therefore excluded from the analysis

Figure 21- Volumes at Brooksby Village Dr

	Dates	AM peak											
R 114 @ Garden St		EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
	9/28/2021	230	1377	3	0	998	63	1	0	2	101	1	183
	9/29/2021	222	1390	0	0	987	75	0	0	2	88	2	173
	9/30/2021	204	1368	4	0	993	71	3	3	2	101	7	149
	10/5/2021	207	1448	2	0	945	77	3	2	2	95	3	174
	10/6/2021	217	1361	3	0	987	68	3	4	1	93	5	172
	10/7/2021*	215	1428	6	0	1209	104	1	3	1	109	9	190
	Dates	Midday											
		EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
	9/28/2021	188	1185	7	1	1213	139	9	18	10	163	9	190
	9/29/2021	178	1268	7	0	1315	139	16	13	14	168	6	185
	9/30/2021	204	1272	4	0	1286	117	19	8	8	186	9	182
	10/5/2021	169	1304	7	0	1171	136	15	9	8	156	7	209
	10/6/2021	151	1287	10	0	1226	133	17	8	8	166	2	179
	10/7/2021	185	1222	9	0	1262	151	16	7	11	144	6	220
	Dates	PM peak											
		EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
	9/28/2021	217	1214	7	1	1508	102	16	5	9	117	13	231
	9/29/2021	194	1301	3	0	1550	121	12	13	6	153	10	214
	9/30/2021*	229	1546	4	0	1372	113	21	8	3	147	6	197
	10/5/2021	236	1296	8	0	1529	119	16	7	8	168	9	223
	10/6/2021	190	1170	4	0	1456	140	11	10	7	151	11	191
	10/7/2021	220	1200	7	0	1505	106	27	8	9	173	6	258

*: Volumes on these dates considered outliers and therefore excluded from the analysis

Figure 22- Volumes at Garden St

	Dates	AM peak											
		EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
	9/28/2021	60	1555	4	17	1211	22	19	3	27	10	0	42
	9/29/2021	66	1540	0	22	1154	32	19	7	26	12	0	45
	9/30/2021	66	1484	0	19	1126	21	24	4	32	15	2	38
	10/5/2021	67	1584	2	11	1193	23	17	2	23	19	0	39
	10/6/2021	69	1532	0	19	1134	19	19	9	21	18	4	36
	10/7/2021	59	1479	1	8	1144	28	18	2	20	18	3	40
	Dates	Midday											
		EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
R 114 @ Honey Dew	9/28/2021	73	1278	2	18	1431	33	21	4	18	44	1	100
	9/29/2021	67	1355	1	25	1519	29	16	3	21	41	3	107
	9/30/2021	82	1397	3	17	1500	36	24	4	16	30	2	90
	10/5/2021	81	1340	2	18	1412	36	13	1	22	48	1	114
	10/6/2021	71	1364	3	23	1421	25	15	2	31	32	2	102
	10/7/2021	76	1296	1	33	1512	31	19	2	24	43	5	82
	Dates	PM peak											
		EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
	9/28/2021*	65	1295	2	21	1961	21	17	1	18	38	3	142
	9/29/2021	69	1377	1	16	1744	20	24	2	17	29	2	143
	9/30/2021	56	1447	0	16	1869	25	27	1	14	32	4	121
	10/5/2021	63	1413	0	19	1771	25	31	2	19	34	3	121
	10/6/2021*	56	1396	2	19	1596	40	32	4	18	35	2	96
	10/7/2021	58	1304	1	19	1769	19	24	2	23	34	3	122

*: Volumes on these dates considered outliers and therefore excluded from the analysis

Figure 23- Volumes at Honey Dew

5.7 Peak Hour Factor and Using a 30-Minute Design Period

The Peak Hour Factor (PHF) is the ratio of the peak hour volume to the peak flow in a design period, which is usually a 15-minute period, but can also be a 30-minute or 60-minute period. As shown in the figures below, peak hour factor for the three critical intersections was calculated for 15-minute and 30-minute design periods. (For a 60-minute design period, PHF = 1.) Attention was given only to the arterial through movements (EBT, WBT) because side street and turning movements have low volumes. Several arterial peak hour factors for the 15-minute design period are unusually far from 1.0, which indicates higher than usual peaking within the peak hour. For example, and 15-minute peak hour factor of 0.84 indicates that the peak 15-minute period has a flow 16% greater than the average flow during the peak hour.

Timing traffic signals for a short, sharp peak is inefficient and leads to excessive green time outside of that short peak, which creates more speeding opportunities. Another reason to question 15-minute design periods is that in short periods, boundary effects can strongly distort the count. For example, suppose the cycle length is 2 minutes (120 s), and that the arterial green lasts 1 minute. There are, on average, 7.5 cycles in a 15-minute period. It may be that one 15-minute period has 7 full cycles plus half of a cycle in which the arterial is green, and therefore 8 full phases of arterial traffic. Meanwhile, the next 15-minute period may have 7 full cycles and half a cycle in which the arterial through phases are red, giving it only 7 full phases of arterial traffic. If flow rates in the two periods are identical, the first will have a count of arterial traffic that is 14% greater due to this boundary effect. With longer periods, the distortion due to boundary effects is smaller.

For these two reasons, our design process uses the peak 30-minute period for design, using the average 30-minute peak hour factors shown in the figures below for the arterial through phases. They range from 0.91 to 0.97. While using a 30-minute PHF might lead to a capacity shortfall during the peak 15-minute period, it will be short-lived and the short queues that form during this period will dissipate before the next 15-minute period is over. Drivers may also respond by shifting their departure times a little earlier or later.

For the non-critical intersections, PHF was assumed to be 0.92.

	AM peak											
	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
15 min PHF	0.71	0.84	0.71	0.87	0.95	0.69	-	-	0.68	0.52	0.28	0.62
30 min PHF	0.85	0.91	-	0.91	0.96	-	-	-	0.79	0.84	0.50	0.77
	Midday											
	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
15 min PHF	0.75	0.93	0.77	0.8	0.95	0.53	0.8	0.3	0.85	0.71	0.66	0.73
30 min PHF	0.92	0.97	-	0.88	0.97	-	0.8	0.5	0.92	0.81	0.76	0.86
	PM peak											
	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
15 min PHF	0.76	0.93	0.71	0.72	0.93	0.62	0.3	-	0.81	0.72	0.65	0.79
30 min PHF	0.91	0.97	-	0.86	0.97	-	0.5	-	0.93	0.79	0.74	0.92

Figure 24- PHFs at Brooksby Village Dr

	AM peak											
	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
15 min PHF	0.87	0.85	-	-	0.94	-	0.44	0.46	0.30	0.81	0.50	0.88
30 min PHF	0.91	0.91	-	-	0.97	-	0.63	0.56	0.50	0.90	0.61	0.94
	Midday											
	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
15 min PHF	0.89	0.94	-	0.25	0.95	-	0.64	0.57	0.55	0.85	0.46	0.89
30 min PHF	0.96	0.97	-	0.5	0.97	-	0.81	0.68	0.7	0.95	0.61	0.94
	PM peak											
	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
15 min PHF	0.84	0.94	-	0.25	0.91	-	0.67	0.54	0.6	0.8	0.56	0.87
30 min PHF	0.92	0.97	-	0.5	0.96	-	0.81	0.77	0.77	0.93	0.65	0.94

Figure 25- PHFs at Garden St

	AM peak											
	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
15 min PHF	0.77	0.85	-	0.63	0.95	-	0.70	0.68	0.73	0.55	0.38	0.62
30 min PHF	0.89	0.92	-	0.75	0.97	-	0.88	0.73	0.87	0.76	0.50	0.74
	Midday											
	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
15 min PHF	0.75	0.94	-	0.64	0.94	-	0.72	0.44	0.7	0.76	0.42	0.78
30 min PHF	0.9	0.97	-	0.85	0.97	-	0.82	0.6	0.84	0.91	0.52	0.92
	PM peak											
	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
15 min PHF	0.8	0.93	-	0.69	0.93	-	0.69	0.44	0.69	0.76	0.53	0.73
30 min PHF	0.91	0.96	-	0.8	0.98	-	0.78	0.88	0.81	0.95	0.56	0.85

Figure 26- PHFs at Honey Dew

5.8 Target Degree of Saturation

To account for daily variations in traffic volume and leave room for a little bit of traffic growth, a target degree of saturation (X_{target}) of 0.94 was used for the arterial. This is tantamount to providing 6% slack capacity during the peak hour; during other hours, there will be still more slack capacity.

To confirm that 0.94 covers the variation observed in this corridor, we calculated, for each day of counts, the ratio of average critical arterial movement volume (thru plus right in the critical direction) to the day's volume at the three critical intersections, in the three periods of the day. Out of the 45 cases, all were 0.94 or greater except one (which was 0.93). Therefore, a target degree of saturation should provide sufficient capacity for all but a small number of days.

Informal field observations at the Garden Street intersection indicated that the saturation flow rate on Rt 114 is a bit lower than what Synchro, using HCM equations, determines. Therefore, our signal timing plans cheat a bit in the direction of a degree of saturation lower than 0.94.

5.9 General Principles for a Timing Plan that Reduces Speeding Opportunities

To limit speeding opportunities on a two-way arterial using traffic signal timing, the timing plan follows these principles:

Short cycles – as short as possible while still meeting requirements for pedestrian crossings and vehicular capacity. Where the needed cycle length is especially short (the I-95 intersection), we use double cycling.

Small coordination zones in which all the intersections in a zone need a similar cycle length. Zone breaks can be allowed where block length is long enough to hold a queue containing one cycle worth of cars (600 to 900 ft, depending on traffic volume and cycle length).

Offsets chosen to provide good two-way progression at a moderate speed. The combination of a short cycle and a low progression speed enables two-way progression with small clusters of intersections with near-simultaneous offset, preferably with at most two intersections per cluster, so that a car leaving one intersection at a speed in excess of the progression speed will hit a red light at the next intersection or, if not, at the intersection after it. For visual evaluation of coordination, our progression diagrams are drawn using a progression speed of 30 mph. Actual progression speed depends on relative offsets and intersection spacing.

Left turns are sometimes leading, sometimes lagging, with the position chosen to get the best two-way progression. Arterial lefts are protected-only, which have a very good safety record whether leading or lagging. To avoid driver confusion, lead-lag position does not vary between timing periods.

Pedestrian phases are concurrent unless there is no safe way to provide a concurrent crossing. Currently, the only pedestrian crossings across Rt 114 are at Brooksby Village Dr, which has an exclusive ped phase, and at Garden St., where the crossing is concurrent with the vehicle phase. Concurrent phases are preferred because they allow cycles to be shorter. They also reduce pedestrian delays, and experience tells us that with exclusive pedestrian phasing, pedestrians will still walk concurrently with a vehicle phase rather than wait for an exclusive phase.

Concurrent ped phases can have either *protected* or *permitted* conflicts with turning movements coming from approach of the concurrent thru phase. *"Protected"* in this context means that the conflicting turn runs during a distinct phase. We use the standard that the ped phase should be *protected* against conflicting turn movements with more than 250 veh/h or whose corner geometry allows high speed turns. Otherwise, right- and left-turn conflicts are permitted during the concurrent crossing phase.

Pedestrian service is on recall except where recall would require an exceptionally long cycle and pedestrian demand is small, in which case ped service will be on demand and the concurrent phase will be undersized. In a coordinated plan, if making the ped phase on recall does not increase a coordination zone's needed cycle length or requires only a small increase, ped recall can be valuable for three reasons. One is that skipping ped phases results in large intervals of unnecessary green time for the mainline which create speeding opportunities. Second, ped recall (compared to pushbutton-actuated) reduces average ped delay by as much as 10 s. Third, if ped calls are frequent – every 3 cycles or more – the cycle will have to be long enough to accommodate ped calls every cycle anyway.

However, where ped calls are infrequent and ped recall would require a cycle length that is substantially longer than the other needed cycle lengths within the coordination zone, it's better for the ped phase to be on demand and undersize the concurrent phase in order to enable a shorter cycle. This technique is also called *oversized ped* because, when there's a ped call, ped service will run beyond the programmed split. That, in turn, may force the coordinated phase to begin several seconds later than programmed. The controller has a built-in method (for a Siemens / Yunex controller, the "Shortway" method; for Econolite, the "smoothing" method) to adjust splits over the next few cycles so that phase length reductions are spread equitably and programmed offsets are restored. We calculated the cycle length needed to allow offsets to be restored and queues dissipated over the next two cycles, provided there is no ped call during the restoration period.

In this corridor, the only arterial ped crossings are at Garden Street and Brooksby Village Dr. Both have very low pedestrian demand and long crossings. Therefore, for both intersections, pedestrian service for crossing Rt 114 is on demand with an undersized side street phase.

Ped phases that run with the coordinated phase should all have Rest-in-Walk. The minimum length Walk interval is 7 s.

To keep cycle length low, the pedestrian phase end buffer is considered as ending with the end of red clearance, as allowed in the MUTCD. Ideally, the phase end buffer should begin 3 s earlier if controllers will support that; otherwise, at the start of yellow. Needed pedestrian clearance time equals L/Sp, where L is crossing length and Sp is design crossing speed, taken as 3.5 ft/s. Clearance need should be met by the combination of phase end buffer and FDW, and we determine FDW using the formula FDW = L/Sp – buffer. Our FDW calculations assume that buffer = 3; if, due to controller limitations, the buffer will be longer than 3 s, FDW should be reduced accordingly.

5.10 Timing Periods, Cycle Length, Coordination Zones and Offsets

These plans cover **weekday a.m. peak, midday, and p.m. peak only**. Overnight and weekend signal operations were not studied.

Design of the signal timing plan began with a review of yellow and red clearance times, checking and adjusting them where appropriate for compliance with MassDOT guidelines.

Next, needed cycle length is calculated at three critical intersections, using the volume, PHF, and X_t (target degree of saturation) described earlier. During the design process, calculations were done in Excel, using Synchro's input only for saturation flow rate; once a design was (tentatively) chosen, it was analyzed in full using Synchro.

Next, the coordination zones were determined. Only one block (Walmart to Garden, 820 ft long) is long enough for a zone break. Because needed cycle length on either side of this block was similar, and because of the high proportion of through traffic and small number of pedestrians crossing Rt 114, single coordination zone was chosen for the full set of intersections. The intersection with the I-95 ramp, where one direction of Rt 114 is always green and the other is stopped only for a short left turn phase, will double cycle. Basic cycle length is 66 s for a.m., 70 s for midday and 84 s for p.m.

Finally, phase sequence, splits, and offsets were chosen. In choosing splits, it is tried to be generous with minor movements (i.e., their split was based on a lower target degree of saturation), knowing that they give back the time they don't need. Lead-lag phasing was used to improve progression where intersection spacing was not ideal. Offsets were chosen to provide good two-way progression at a moderate speed, keeping the size of simultaneous offset clusters as small as possible – our aim (accomplished) was no more than 2.

As a starting point, offsets were all 0 or C/2 (where C = cycle length), in order to provide equally good progression in both directions. Adjustments were made first to account for lead-lag and for different arterial green lengths, and then for differences in arterial volume by direction. Small adjustments were also made to ensure that jackrabbits at the head of the platoon going 50 mph would not be able to jump ahead one cycle. Within each coordination zone, one intersection is arbitrarily chosen as the reference intersection, with offset 0. Offsets are referenced to the end of the green for the first coordinated phase.

Figure 27 shows the needed cycle length and the proposed cycle length, offset, and critical volume to capacity (v/c) ratio at each intersection for three periods of weekday.

Intersection	AM Peak					
	Dist to next (ft)	Zone	Needed cycle (s)	Proposed cycle (s)	Offset (End of first coord'd green) (s)	v/c
R114 @ I95 West	990	1	25	33 (double cycle)	0	0.62
R114 @ Avalon Bay	450		53.9	66	5	0.92
R114 @ Honey Dew	550		60	66	5	0.77
R114 @ Garden	880		57.2	66	30	0.80
R114 @ Walmart	600		45.6	66	0	0.80
R114 @ Brooksby			66.2	66	0 (Master Intersection)	0.80
Intersection	Midday					
	Dist to next (ft)	Zone	Needed cycle (s)	Proposed cycle (s)	Offset (End of first coord'd green) (s)	v/c
R114 @ I95 West	990	1	25.1	35 (double cycle)	0	0.71
R114 @ Avalon Bay	450		65.3	70	10	0.78
R114 @ Honey Dew	550		58	70	10	0.73
R114 @ Garden	880		67.2	70	39	0.88
R114 @ Walmart	600		49.3	70	0	0.92
R114 @ Brooksby			62.4	70	0 (Master Intersection)	0.86
Intersection	PM Peak					
	Dist to next (ft)	Zone	Needed cycle (s)	Proposed cycle (s)	Offset (End of first coord'd green) (s)	v/c
R114 @ I95 West	990	1	23.6	42 (double cycle)	0	0.81
R114 @ Avalon Bay	450		75	84	10	0.91
R114 @ Honey Dew	550		67.7	84	10	0.87
R114 @ Garden	880		84.4	84	35	0.91
R114 @ Walmart	600		50.7	84	0	0.90
R114 @ Brooksby			68.5	84	0 (Master Intersection)	0.91

Figure 27- General signal timing at Rt.114

5.11 Intersection – by – Intersection Timing Calculations

The detailed calculation is provided by spreadsheet attached to this document. Following is the proposed cycle information consisting of splits, minimum green, maximum green, extension time, yellow time, red clearance time and operation of split during cycle time for three periods during weekday. All the phase numbering is the same as current plans.

5.11.1 Rt 114 at Brooksby Village

Figure 28 represents the cycle time parameters at intersection of Brooksby Village Dr. All movements are allowed except northbound thru and northbound left. The arterial has exclusive left turn lanes. There are pedestrian crossings on the northern and western sides of the intersection. With the current signal control plan, the northern crossing runs concurrent with the arterial phase while the western crossing (across Route 114) has all ped phase. With the proposed signal control plan, the western crossing will be concurrent with the side street, with an oversized ped phase. A concurrent crossing is safe here because the permitted conflicting turn volume (SBR and NBL, which is prohibited) is only 61 veh/hr in the busiest period (pm peak), amounting to a bit more than one vehicle per cycle. Otherwise, all phases work the same as currently but with different splits. This intersection is the Master intersection at all the periods of day.

Intersection	R 114@ Brooksby Village
Plan	Weekday
Time	6:30-9:30
Offset (s)	0
Cycle length (s)	66
Master Intersection	Yes

Phase	WBL	EB		SB	EBL	WB		NBR
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	18		10	6	18		10
Extension interval (s)	2	2		2	2	2		2
Yellow (s)	4	4		3	4	4		3
Red Clear (s)	2	2		2	2	2		2
Maximum Green (s)	7	30		12	7	30		12
Total split (s)	13	36		17	13	36		17
Lead/Lag	Lead				Lag			
Pedestrian Walk (s)		7 RIW*		7		7 RIW*		7
Ped Clearance (s)		14		16		14		16
Ped phase end buffer (s)		3		3		3		3
Recall (s)		C-Max		Min		C-Max		Min
Overlap	NBR				SBR			

Intersection	R 114@ Brooksby Village
Plan	Weekday
Time	9:30-15:30
Offset (s)	0
Cycle length (s)	70
Master Intersection	Yes

Phase	WBL	EB		SB	EBL	WB		NBR
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	18		10	6	18		10
Extension interval (s)	2	2		2	2	2		2
Yellow (s)	4	4		3	4	4		3
Red Clear (s)	2	2		2	2	2		2
Maximum Green (s)	8	32		13	8	32		13
Total split (s)	14	38		18	14	38		18
Lead/Lag	Lead				Lag			
Pedestrian Walk (s)		7 RIW*		7		7 RIW*		7
Ped Clearance (s)		14		16		14		16
Ped phase end buffer (s)		3		3		3		3
Recall (s)		C-Max		Min		C-Max		Min
Overlap	NBR				SBR			

Intersection	R 114@ Brooksby Village
Plan	Weekday
Time	15:30-18:30
Offset (s)	0
Cycle length (s)	84
Master Intersection	Yes

Phase	WBL	EB		SB	EBL	WB		NBR
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	18		10	6	18		10
Extension interval (s)	2	2		2	2	2		2
Yellow (s)	4	4		3	4	4		3
Red Clear (s)	2	2		2	2	2		2
Maximum Green (s)	10	40		17	10	40		17
Total split (s)	16	46		22	16	46		22
Lead/Lag	Lead				Lag			
Pedestrian Walk (s)		7 RIW*		7		7 RIW*		7
Ped Clearance (s)		14		16		14		16
Ped phase end buffer (s)		3		3		3		3
Recall (s)		C-Max		Min		C-Max		Min
Overlap	NBR				SBR			

Note*: Rest in Walk

Figure 28- Timing plans (Brooksby Village Dr)

5.11.2 Rt 114 at Walmart

Figure 29 represents the cycle time parameters at intersection of Walmart, a T-intersection. There are no pedestrian crossing facilities. An exclusive left turn lane is provided for westbound left movement. The proposed signal control plan is the same as the current plan with new cycle time and split times.

Intersection	R 114@ Walmart
Plan	Weekday
Time	6:30-9:30
Offset (s)	0
Cycle length (s)	66
Master Intersection	NO

Phase	WBL	WB		NB		EB		
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	10		10		10		
Extension interval (s)	2	2		2		2		
Yellow (s)	4	4		3		4		
Red Clear (s)	2	2		2		2		
Maximum Green (s)	8	29		12		43		
Total split (s)	14	35		17		49		
Lead/Lag	Lag							
Pedestrian Walk (s)								
Ped Clearance (s)								
Ped phase end buffer (s)								
Recall (s)		C-Max		Min		C-Max		
Overlap								

Intersection	R 114@ Walmart
Plan	Weekday
Time	9:30-15:30
Offset (s)	0
Cycle length (s)	70
Master Intersection	NO

Phase	WBL	WB		NB		EB		
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	10		10		10		
Extension interval (s)	2	2		2		2		
Yellow (s)	4	4		3		4		
Red Clear (s)	2	2		2		2		
Maximum Green (s)	7	31		15		44		
Total split (s)	13	37		20		50		
Lead/Lag	Lag							
Pedestrian Walk (s)								
Ped Clearance (s)								
Ped phase end buffer (s)								
Recall (s)		C-Max		Min		C-Max		
Overlap								

Intersection	R 114@ Walmart
Plan	Weekday
Time	15:30-18:30
Offset (s)	0
Cycle length (s)	84
Master Intersection	NO

Phase	WBL	WB		NB		EB		
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	10		10		10		
Extension interval (s)	2	2		2		2		
Yellow (s)	4	4		3		4		
Red Clear (s)	2	2		2		2		
Maximum Green (s)	8	38		21		52		
Total split (s)	14	44		26		58		
Lead/Lag	Lag							
Pedestrian Walk (s)								
Ped Clearance (s)								
Ped phase end buffer (s)								
Recall (s)		C-Max		Min		C-Max		
Overlap								

Figure 29- Timing plans (Walmart)

5.11.3 Rt 114 at Garden St

Figure 30 represents the cycle time parameters at intersection of Garden St. All movements are allowed, and the arterial has exclusive left turn lanes in both directions. At this intersection there are pedestrian crossing facilities across the northern leg (runs with Rt 114 WB, with Rest in Walk) and western leg (runs with Garden St, on demand). The Garden Street phase will have "oversized ped," meaning ped service, when called, will extend beyond the programmed split; as explained earlier, the next few cycles will have "shortway" offset corrections to restore offsets to their programmed values. With the proposed signal control plan for this intersection all phases work the same as before with different split length.

Intersection	R 114@ Garden St
Plan	Weekday
Time	6:30-9:30
Offset (s)	30
Cycle length (s)	66
Master Intersection	NO

Phase	WBL	EB		SB	EBL	WB		NB
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	22		10	6	22		10
Extension interval (s)	2	2		2	2	2		2
Yellow (s)	4	4		3	4	4		3
Red Clear (s)	1	1		2	1	1		2
Maximum Green (s)	11	28		12	11	28		12
Total split (s)	16	33		17	16	33		17
Lead/Lag	Lead				Lag			
Pedestrian Walk (s)		7 RIW*		7		7 RIW*		7
Ped Clearance (s)		17		22		17		22
Ped phase end buffer (s)		3		3		3		3
Recall (s)		C-Max		Min		C-Max		Min
Overlap					SBR			

Intersection	R 114@ Garden St
Plan	Weekday
Time	9:30-15:30
Offset (s)	35
Cycle length (s)	70
Master Intersection	NO

Phase	WBL	EB		SB	EBL	WB		NB
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	22		10	6	22		10
Extension interval (s)	2	2		2	2	2		2
Yellow (s)	4	4		3	4	4		3
Red Clear (s)	1	1		2	1	1		2
Maximum Green (s)	9	32		14	9	32		14
Total split (s)	14	37		19	14	37		19
Lead/Lag	Lead				Lag			
Pedestrian Walk (s)		7 RIW*		7		7 RIW*		7
Ped Clearance (s)		17		22		17		22
Ped phase end buffer (s)		3		3		3		3
Recall (s)		C-Max		Min		C-Max		Min
Overlap					SBR			

Intersection	R 114@ Garden St
Plan	Weekday
Time	15:30-18:30
Offset (s)	35
Cycle length (s)	84
Master Intersection	NO

Phase	WBL	EB		SB	EBL	WB		NB
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	22		10	6	22		10
Extension interval (s)	2	2		2	2	2		2
Yellow (s)	4	4		3	4	4		3
Red Clear (s)	1	1		2	1	1		2
Maximum Green (s)	11	46		12	13	44		12
Total split (s)	16	51		17	18	49		17
Lead/Lag	Lead				Lag			
Pedestrian Walk (s)		7 RIW*		7		7 RIW*		7
Ped Clearance (s)		17		22		17		22
Ped phase end buffer (s)		3		3		3		3
Recall (s)		C-Max		Min		C-Max		Min
Overlap					SBR			

Note*: Rest in Walk

Figure 30- Timing plans (Garden St)

5.11.4 Rt 114 at Honey Dew

Figure 31 represents the cycle time parameters at intersection of Honey Dew. All movements are allowed. The arterial has exclusive left turn lanes. There is a pedestrian crossing facility across the northern leg only, served concurrent with the arterial. With the proposed signal control plan, all phases work the same as currently with different splits.

Intersection	R 114@ Honey Dew
Plan	Weekday
Time	6:30-9:30
Offset (s)	5
Cycle length (s)	66
Master Intersection	NO

Phase	WBL	EB		SB	EBL	WB		NB
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	24		10	6	24		10
Extension interval (s)	2	2		2	2	2		2
Yellow (s)	4	4		3	4	4		3
Red Clear (s)	1	1		3	1	1		3
Maximum Green (s)	8	30		12	8	30		12
Total split (s)	13	35		18	13	35		18
Lead/Lag	Lead				Lag			
Pedestrian Walk (s)		7 RIW*				7 RIW*		
Ped Clearance (s)		19				19		
Ped phase end buffer (s)		3				3		
Recall (s)		C-Max		Min		C-Max		Min
Overlap					SBR			

Intersection	R 114@ Honey Dew
Plan	Weekday
Time	9:30-15:30
Offset (s)	5
Cycle length (s)	70
Master Intersection	NO

Phase	WBL	EB		SB	EBL	WB		NB
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	24		10	6	24		10
Extension interval (s)	2	2		2	2	2		2
Yellow (s)	4	4		3	4	4		3
Red Clear (s)	1	1		3	1	1		3
Maximum Green (s)	8	34		12	8	34		12
Total split (s)	13	39		18	13	39		18
Lead/Lag	Lead				Lag			
Pedestrian Walk (s)		7 RIW*				7 RIW*		
Ped Clearance (s)		19				19		
Ped phase end buffer (s)		3				3		
Recall (s)		C-Max		Min		C-Max		Min
Overlap					SBR			

Intersection	R 114@ Honey Dew
Plan	Weekday
Time	15:30-18:30
Offset (s)	10
Cycle length (s)	84
Master Intersection	NO

Phase	WBL	EB		SB	EBL	WB		NB
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	24		10	6	24		10
Extension interval (s)	2	2		2	2	2		2
Yellow (s)	4	4		3	4	4		3
Red Clear (s)	1	1		3	1	1		3
Maximum Green (s)	10	45		13	10	45		13
Total split (s)	15	50		19	15	50		19
Lead/Lag	Lead				Lag			
Pedestrian Walk (s)		7 RIW*				7 RIW*		
Ped Clearance (s)		19				19		
Ped phase end buffer (s)		3				3		
Recall (s)		C-Max		Min		C-Max		Min
Overlap					SBR			

Note*: Rest in Walk

Figure 31- Timing plans (Honey Dew)

5.11.5 Rt 114 at Avalon Bay Dr

Figure 32 represents the cycle time parameters at intersection of Avalon Bay Dr. All movements are allowed. And exclusive left turn lanes provided for left turning cars from arterial. At this intersection there is pedestrian crossing facilities at northern side of the intersection. With current signal control plan the northern crossing runs concurrent with the arterial phase.

Intersection	R 114@Avalon Bay
Plan	Weekday
Time	6:30-9:30
Offset (s)	5
Cycle length (s)	66
Master Intersection	NO

Phase	EBL	WB		NB	WBL	EB		SB
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	20		10	6	20		10
Extension interval (s)	2	2		2	2	2		2
Yellow (s)	4	4		3	4	4		3
Red Clear (s)	1	1		3	1	1		3
Maximum Green (s)	8	32		10	8	32		10
Total split (s)	13	37		16	13	37		16
Lead/Lag	Lead				Lag			
Pedestrian Walk (s)		7 RIW*				7 RIW*		
Ped Clearance (s)		15				15		
Ped phase end buffer (s)		3				3		
Recall (s)		C-Max		Min		C-Max		Min
Overlap								

Intersection	R 114@ Avalon Bay
Plan	Weekday
Time	9:30-15:30
Offset (s)	5
Cycle length (s)	70
Master Intersection	NO

Phase	EBL	WB		NB	WBL	EB		SB
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	20		10	6	20		10
Extension interval (s)	2	2		2	2	2		2
Yellow (s)	4	4		3	4	4		3
Red Clear (s)	1	1		3	1	1		3
Maximum Green (s)	8	34		12	8	34		12
Total split (s)	13	39		18	13	39		18
Lead/Lag	Lead				Lag			
Pedestrian Walk (s)		7 RIW*				7 RIW*		
Ped Clearance (s)		15				15		
Ped phase end buffer (s)		3				3		
Recall (s)		C-Max		Min		C-Max		Min
Overlap								

Intersection	R 114@ Avalon Bay
Plan	Weekday
Time	15:30-18:30
Offset (s)	10
Cycle length (s)	84
Master Intersection	NO

Phase	EBL	WB		NB	WBL	EB		SB
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	19		10	6	19		10
Extension interval (s)	2	2		2	2	2		2
Yellow (s)	4	4		3	4	4		3
Red Clear (s)	1	1		3	1	1		3
Maximum Green (s)	10	45		13	10	45		13
Total split (s)	15	50		19	15	50		19
Lead/Lag	Lead				Lag			
Pedestrian Walk (s)		7 RIW*				7 RIW*		
Ped Clearance (s)		17				17		
Ped phase end buffer (s)								
Recall (s)		C-Max		Min		C-Max		Min
Overlap								

Note*: Rest in Walk

Figure 32- Timing plans (Avalon Bay Dr)

5.11.6 Rt 114 at I-95

Figure 33 represents the cycle time parameters at intersection of I-95. The signal controls only westbound left and eastbound thru; westbound thru in not interrupted by the signal. In the current signal plan, it has a fixed cycle length, but the cycle length is different from the neighboring intersection, and so it is not coordinated. With the proposed signal control plan, it will be coordinated with the neighboring intersection, with half the cycle length, also called double cycling plan.

Intersection	R 114@ I-95
Plan	Weekday
Time	6:30-9:30
Offset (s)	0
Cycle length (s)	33
Master Intersection	No

Phase	WBL	EB						
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	10						
Extension interval (s)	2	2						
Yellow (s)	3	4						
Red Clear (s)	2	1						
Maximum Green (s)	8	15						
Total split (s)	13	20						
Lead/Lag								
Pedestrian Walk (s)								
Ped Clearance (s)								
Ped phase end buffer (s)								
Recall (s)	C-Max							
Overlap								

Intersection	R 114@ I-95
Plan	Weekday
Time	9:30-15:30
Offset (s)	0
Cycle length (s)	35
Master Intersection	No

Phase	WBL	EB						
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	10						
Extension interval (s)	2	2						
Yellow (s)	3	4						
Red Clear (s)	2	1						
Maximum Green (s)	9	16						
Total split (s)	14	21						
Lead/Lag								
Pedestrian Walk (s)								
Ped Clearance (s)								
Ped phase end buffer (s)								
Recall (s)	C-Max							
Overlap								

Intersection	R 114@ I-95
Plan	Weekday
Time	15:30-18:30
Offset (s)	0
Cycle length (s)	42
Master Intersection	No

Phase	WBL	EB						
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	10						
Extension interval (s)	2	2						
Yellow (s)	3	4						
Red Clear (s)	2	1						
Maximum Green (s)	12	20						
Total split (s)	17	25						
Lead/Lag								
Pedestrian Walk (s)								
Ped Clearance (s)								
Ped phase end buffer (s)								
Recall (s)	C-Max							
Overlap								

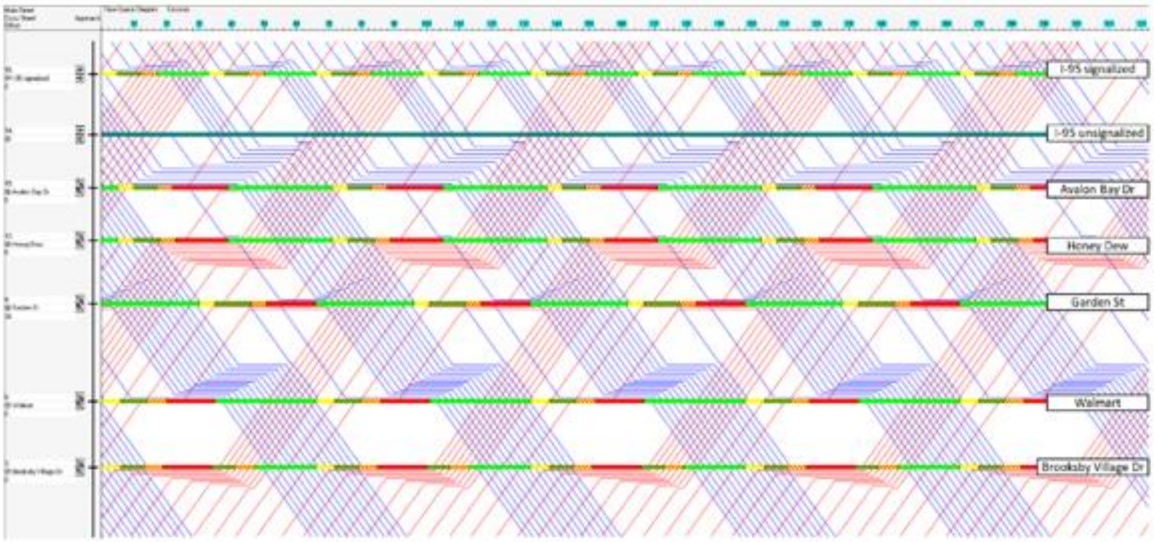
Figure 33- Timing plans (I-95)

5.12 Progression Diagrams

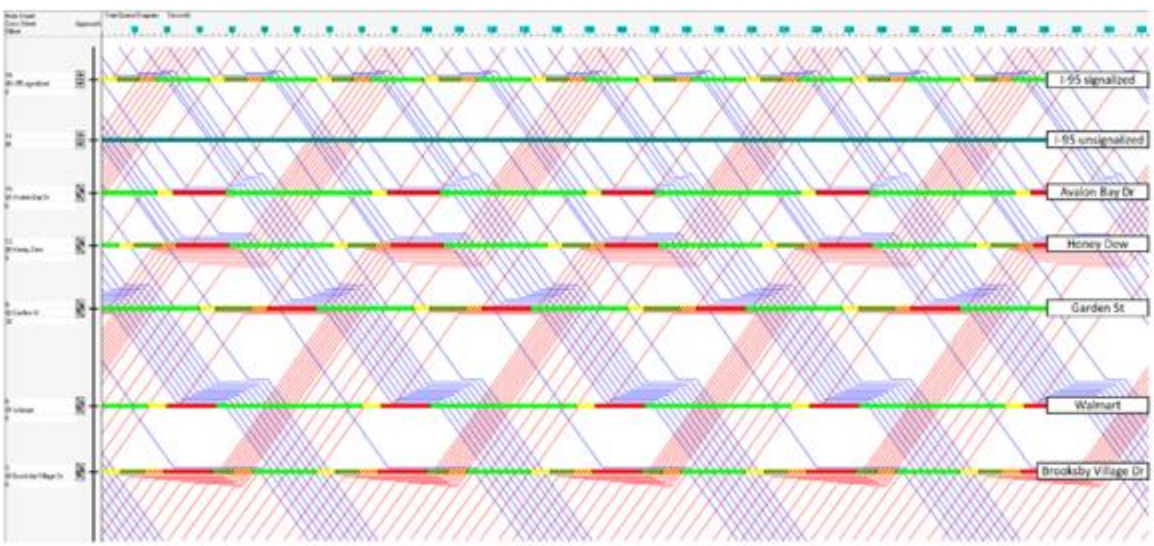
Progression diagrams are shown below provided for the current and proposed signal plan in two conditions: 100% of design volume and 75% of design volume. The 75% case is examined because speeding opportunities tend to be greater when volumes are lower – that is, outside the peak volume period – and there is surplus green time, and we wanted to ensure that the plan would help limit speeding opportunities in these off-peak periods as well as in the design period.

The horizontal axis is time and vertical axis is location of intersections. Blue lines represent eastbound vehicles going “downhill” and red lines representing westbound vehicles going “uphill”.

Speeding opportunities occur when a vehicle arrives at an intersection on a stale green with no vehicles ahead of them. In the proposed plan few such speeding opportunities are apparent. When the head of a platoon arrives at the next intersection, it usually goes no go any faster (steeper up or down) without running into red. With 75% volume, there are more vehicles arriving at intersections with a speeding opportunity, but not many more.

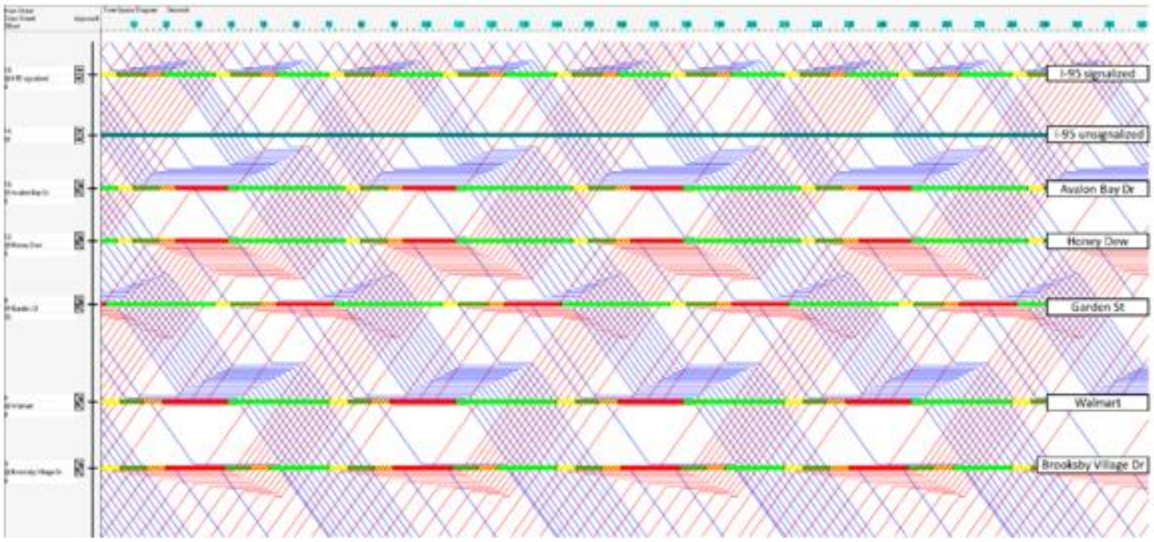


100 % volume

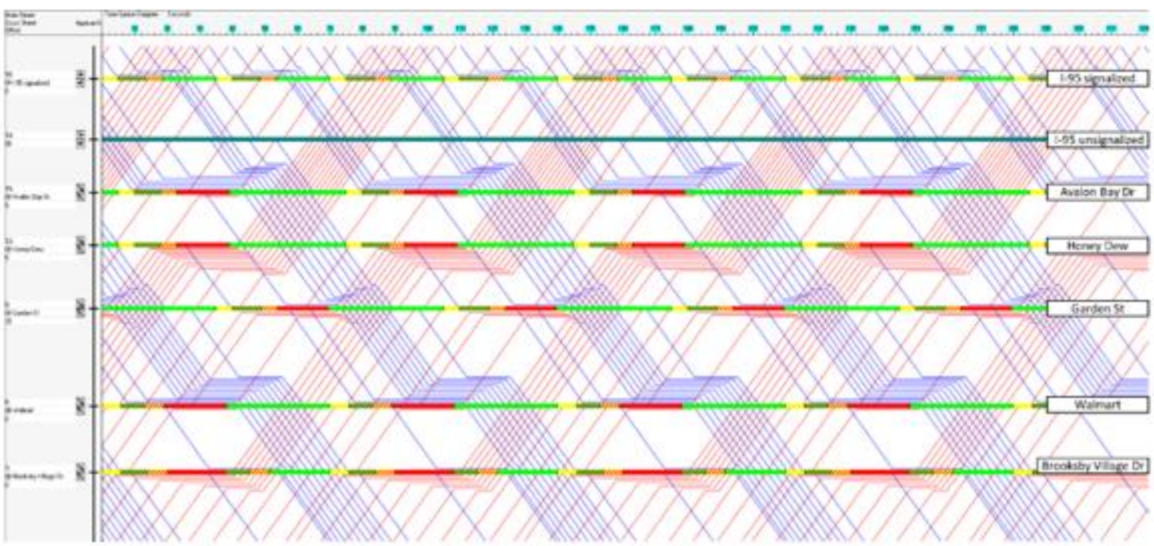


75% volume

Figure 34- Progression diagram (Proposed AM plan)

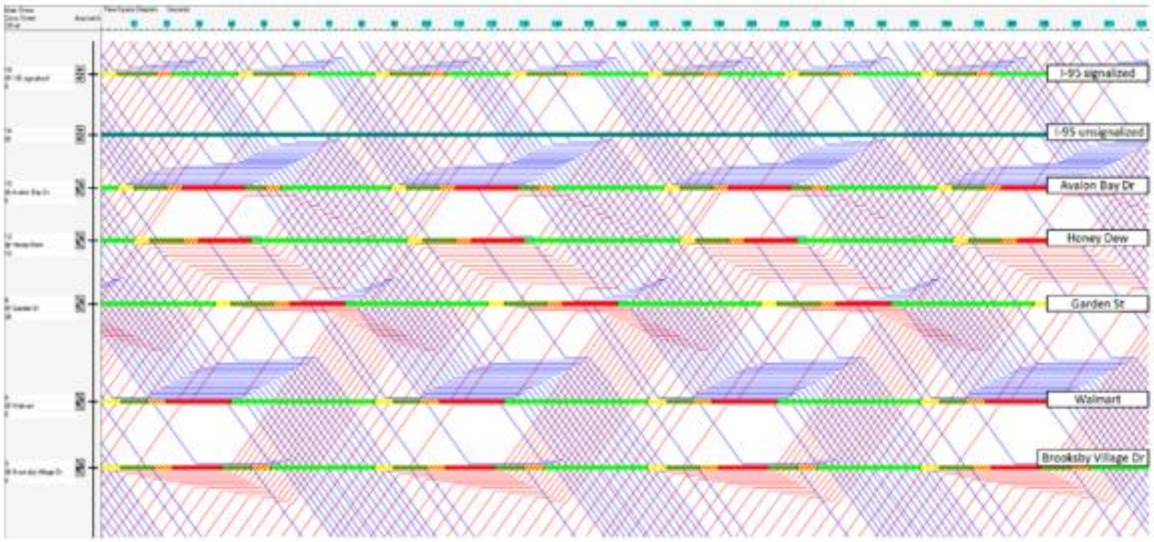


100 % volume

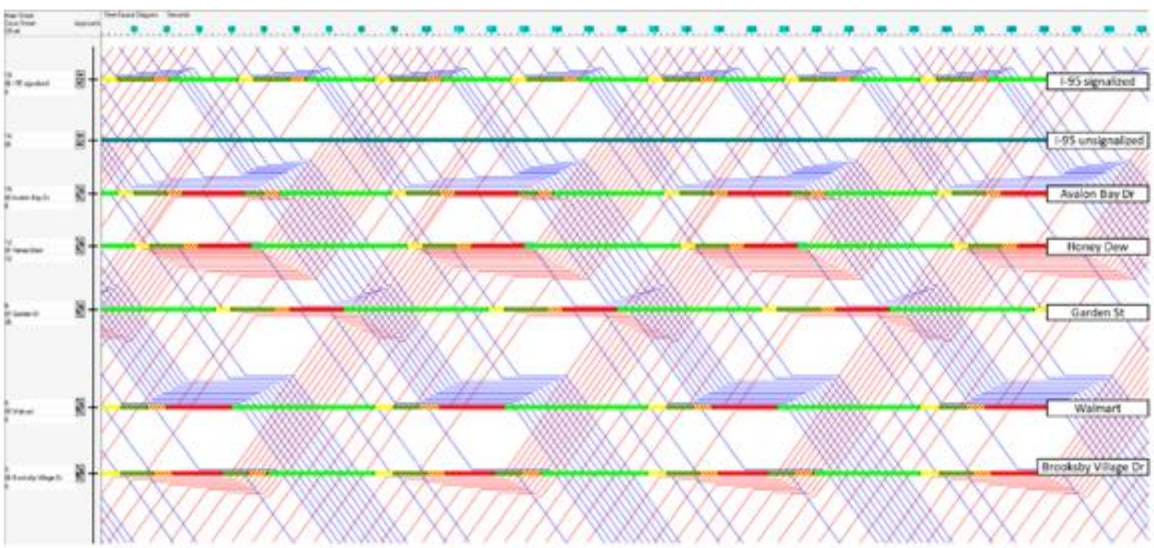


75% volume

Figure 35- Progression diagram (Proposed midday plan)



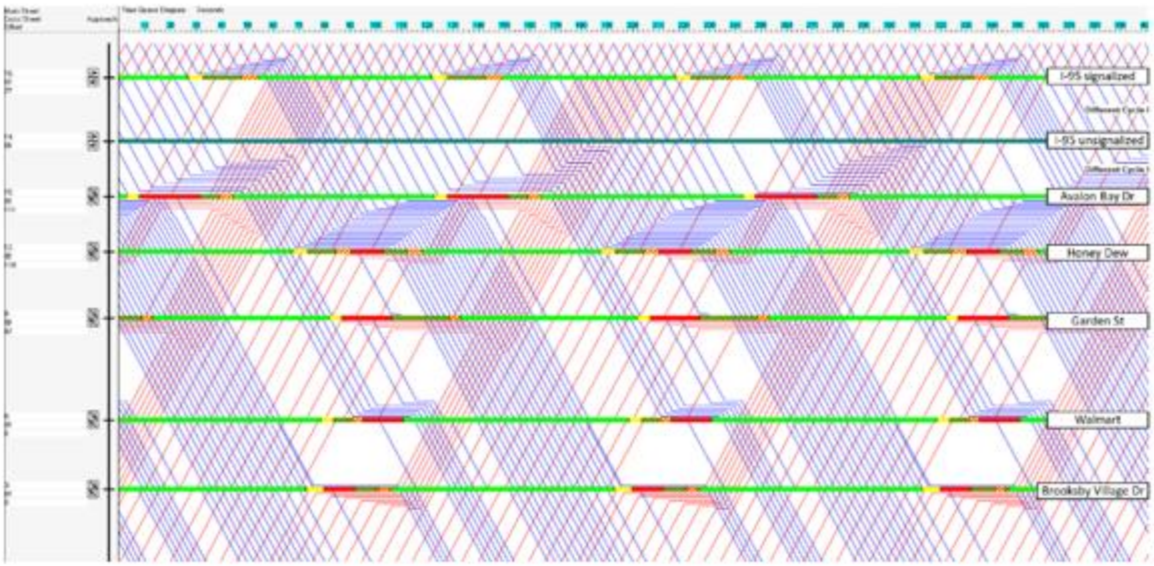
100 % volume



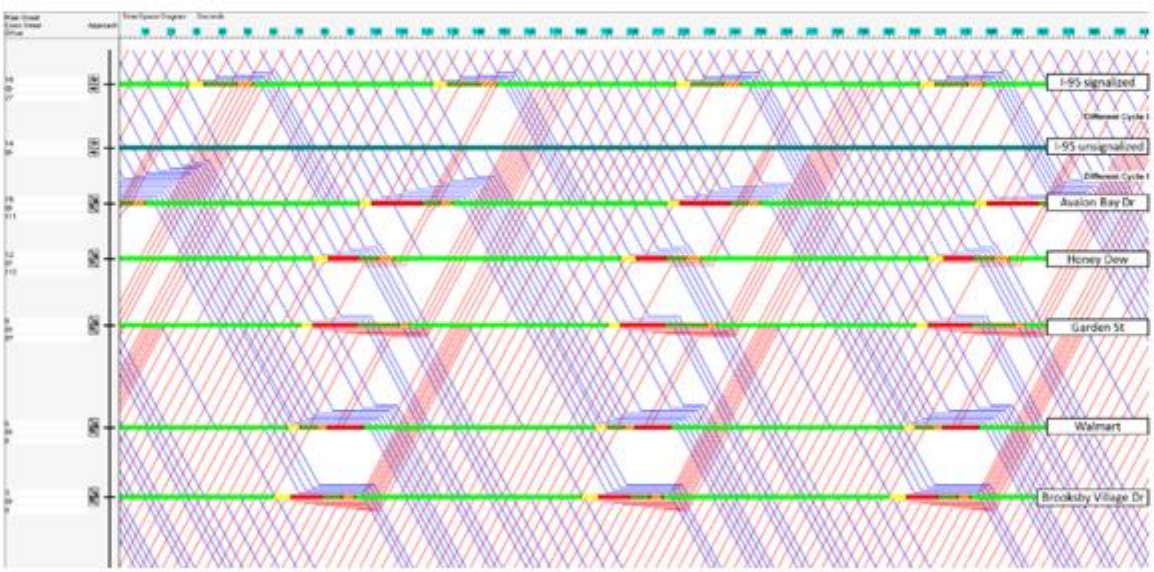
75% volume

Figure 36- Progression diagram (Proposed PM plan)

In the following figures, the *current* signal plan for three periods of weekday in two conditions of 100% and 75% presented. With their longer cycles, there are many more speeding opportunities than in the proposed plan.

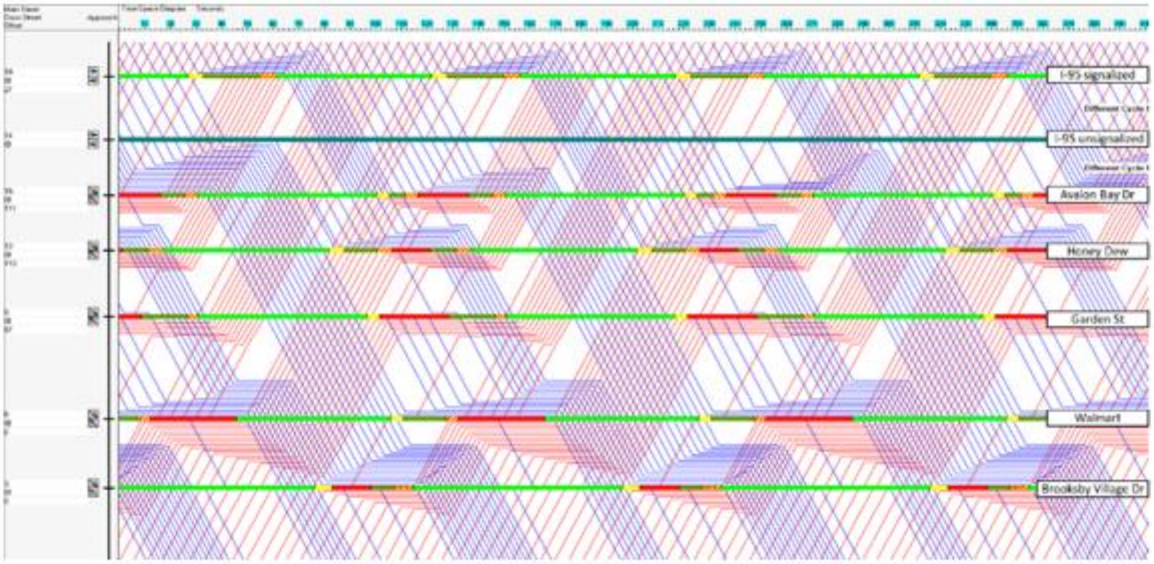


100 % volume

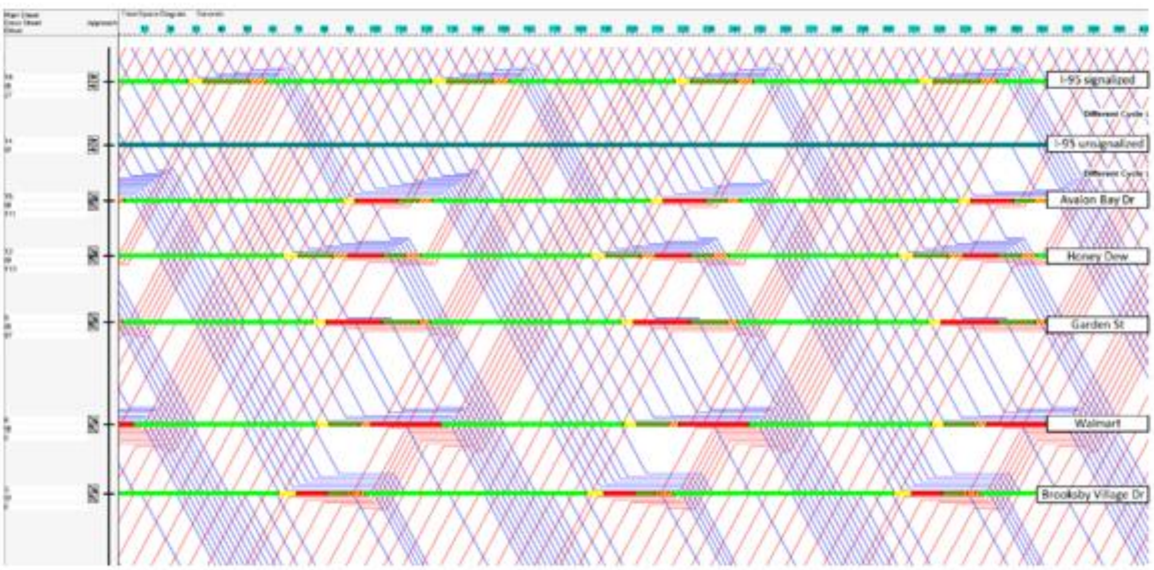


75% volume

Figure 37- Progression diagram (Current AM plan)

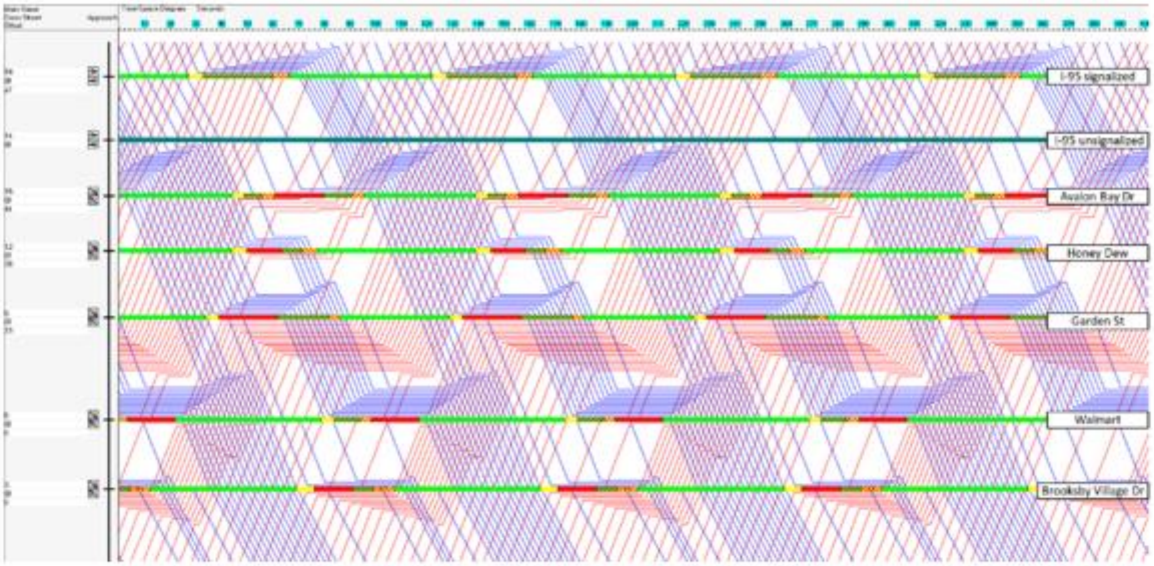


100 % volume

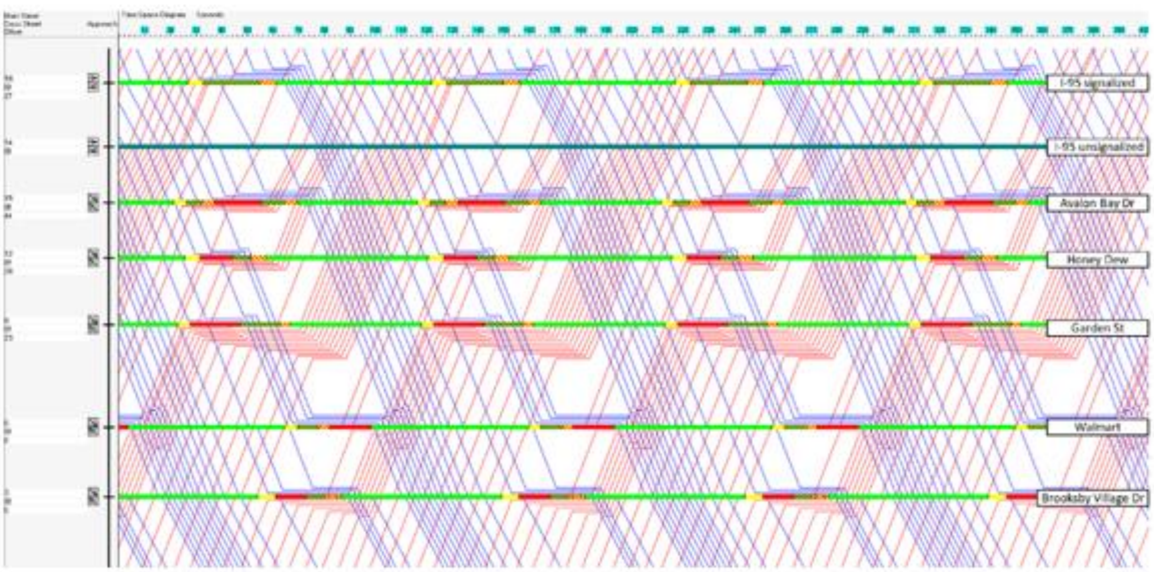


75% volume

Figure 38- Progression diagram (Current midday plan)



100 % volume



75% volume

Figure 39- Progression diagram (Current PM plan)

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6 Safe Waves Timing Plans for Route 16, Everett and Chelsea, MA

6.1 Introduction

The study site is a portion of Route 16 located in Everett and Chelsea, where it is known as Revere Beach Parkway, and is classified as an arterial. The study site extends from the intersection with Lewis St in the west to Washington Ave in the east, a distance of 1.12 miles (Figure 40).

It is a divided highway with three lanes in each direction, plus left turn lanes at every intersection where left turns are permitted. The speed limit is 35 mph on the arterial and 25 mph of the side streets.

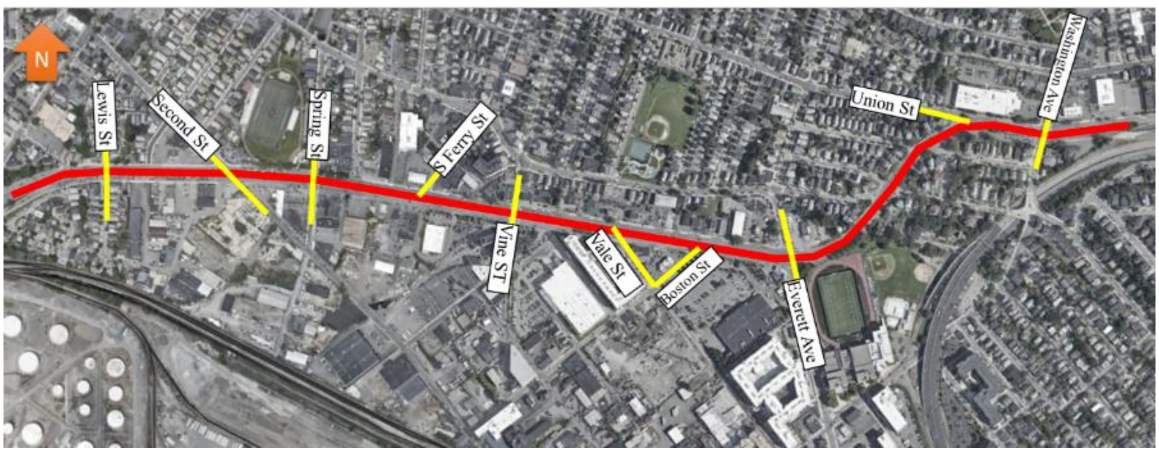


Figure 40- Study segment of Route 16

6.2 Intersections and Current Signal Timing

The study corridor has nine signalized intersections, numbered 1-9, beginning in the west at Lewis Street. There is only one unsignalized intersection where the median is broken, Boston Street (shown in Figure 40), between intersections 6 and 7.

In the current plan, there are two coordination zones in the AM (7:00-10:00 AM) and PM (3:00-7:00 PM) periods. Intersections 2-5 (Second St, Spring St, South Ferry St, and Vine St) are coordinated with cycle lengths of 110 s AM / 150 s PM. Intersections 8 and 9 (Union St and Washington Ave) are coordinated with a 120 s cycle both AM and PM periods. All intersections run free at other times of the day and on weekends. Intersections 1 (Lewis St), 6 (Vale St), and 7 (Everett Ave) run free at all times. Note that “running free” in this corridor does not mean fully actuated, since there are no mainline detectors; during free operations, the arterial through phases run for a fixed phase length, but minor phases have a variable length (between their minimum and maximum green), and exclusive pedestrian phases are skipped unless called. Most of the intersections have exclusive pedestrian phases. Pedestrian demand is very low at some intersections.

6.3 Hourly Volumes

Volume data is from the “Route 16 Priority Corridor Study: Chelsea and Everett, Massachusetts” published by Central Transportation Planning Staff (CTPS) in November 2019. Its data is from turning movement counts conducted Thursday December 6, 2018, at all intersections.

In the morning, the peak hour on Rt 16 is unusually early, running from 6:15 to 7:15 at most intersections, with the busiest 15-minute period 6:15-6:30. Separate data collection was conducted at two intersections to confirm this pattern. It was found the same early pattern of traffic volumes, with total volumes only slightly below those reported by CTPS.

6.4 Peak Hour Movement Volumes

The figures below show the peak hour volumes taken from the CTPS report, which were used to design the cycle plan.

	Eastbound*				Westbound*				Northbound			Southbound		
	EBU	EBL	EBT	EBR	WBU	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
R16 @ Lewis St	-	-	1584	8	-	-	2047	6	23	10	12	19	18	36
R16 @ Second St	-	-	1187	428	-	-	1793	92	178	37	3	28	56	73
R16 @ Spring St	10	55	1139	8	19	13	1678	23	34	18	29	37	42	163
R16 @ South Ferry St	15	126	977	-	-	-	1591	128	-	-	92	-	-	-
R16 @ Vine ST	-	-	970	99	1	38	1486	22	52	53	28	41	170	181
R16 @ Vale St	-	-	924	134	3	5	1425	-	117	-	2	-	-	-
R16 @ Boston St	-	-	914	15	9	132	1433	-	-	-	42	-	-	-
R16 @ Everett Ave	10	63	745	146	2	74	1389	8	126	76	34	71	221	51
R16 @ Union St	-	-	852	-	-	-	1487	182	-	-	-	175		12
R16 @ Washington Ave	-	80	769	178	11	168	1429	33	136	89	19	57	190	104

*: Route 16 (Arterial) is the east-west direction

Figure 41- AM peak design volumes

	Eastbound*				Westbound*				Northbound			Southbound		
	EBU	EBL	EBT	EBR	WBU	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
R16 @ Lewis St			2426	29			2154	5	22	14	9	9	14	27
R16 @ Second St			2027	417			1839	128	268	51	2	45	41	52
R16 @ Spring St	35	114	1871	45	39	44	1742	50	49	50	60	26	35	132
R16 @ South Ferry St	21	330	1645				1808	69			72			
R16 @ Vine ST			1631	86	18	24	1606	139	135	194	29	54	101	136
R16 @ Vale St			1573	159	5		1415		372		8			
R16 @ Boston St			1544	51	4	84	1412		8		220			
R16 @ Everett Ave	49	184	1377	158	8	57	1224	21	185	230	50	55	151	42
R16 @ Union St			1490				1299	224				123		11
R16 @ Washington Ave		218	1185	213	20	135	1264	31	139	234	23	57	133	120

*: Route 16 (Arterial) is the east-west direction

Figure 42- PM peak design volumes

6.5 Peak Hour Factor and Using a 30-Minute Design Period

The Peak Hour Factor (PHF) is the ratio of the peak hour volume to the peak flow in a design period, which is traditionally a 15-minute period, but can also be a 30-minute or 60-minute period. The figures below show PHF calculated for 15-minute and 30-minute design periods for all intersections. (For a 60-minute design period, PHF = 1.) Attention was given only to the arterial through movements (EBTR, WBTR) because at most intersections, side street and turning movements have low volumes.

In the AM peak, several arterial peak hour factors for the 15-minute design period are unusually far from 1.0, which indicates sharp peaking within the peak hour. For example, the 15-minute peak hour factor of 0.87 at Lewis Street westbound indicates that the 15-minute period between 6:30 and 6:45 am has a flow about 13% greater than the average flow during the peak hour. Timing traffic signals for a short, sharp peak is inefficient and leads to excessive green time outside of that short peak, which creates more speeding opportunities. Another reason to question 15-minute design periods is that in a period that short, boundary effects can strongly distort a count. If a cycle length is 2 minutes (120 s), there are, on average, 7.5 cycles in a 15-minute period. It may be that one 15-minute period has 7 full cycles and half of a cycle in which the arterial phase is green, while another 15-minute period has 7 full cycles and half a cycle in which the arterial through phase is red. If flow rates in the two periods are identical, the first will have a count that's 14% higher. With longer periods, distortion due to boundary effects is smaller.

For these reasons, in the design process the peak 30-minute period was used for design. Average 30-minute peak hour factors are shown in the figures below. For peak direction arterial movements, they range from 0.92 to 0.97. While using a 30-minute PHF might lead to a capacity shortfall during the peak 15-minute period, it will be short-lived and the short queues that form during this period will dissipate before the next 15-minute period is over. Drivers may also respond by shifting their departure times a little earlier or later.

		Eastbound				Westbound				Northbound			Southbound		
		EBU	EBL	EBT	EBR	WBU	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
AM peak	R16 @ Lewis St	-	-	0.925	0.5	-	-	0.869	0.875	0.893	0.5	0.417	0.6	0.625	0.813
	R16 @ Second St	-	-	0.905	0.926	-	-	0.87	0.741	0.754	0.775	0.375	0.775	0.632	0.75
	R16 @ Spring St	0.667	0.875	0.836	0.438	0.643	0.6	0.924	0.75	0.645	0.625	0.727	0.6	0.625	0.805
	R16 @ South Ferry St	0.625	0.927	0.847	-	-	-	0.885	0.864	-	-	-	-	-	0.5
	R16 @ Vine ST	-	-	0.910	0.734	0.250	0.636	0.874	0.708	0.697	0.639	0.600	0.733	0.755	0.931
	R16 @ Vale St	-	-	0.861	0.766	0.500	0.750	0.886	-	0.756	-	0.250	-	-	-
	R16 @ Boston St	-	-	0.846	0.750	0.875	0.821	0.849	-	-	-	0.696	-	-	-
	R16 @ Everett Ave	0.563	0.845	0.865	0.875	0.25	0.944	0.934	0.6	0.808	0.639	0.867	0.804	0.964	0.656
	R16 @ Union St	-	-	0.954	-	-	-	0.931	0.862	-	-	-	-	0.682	0.536
	R16 @ Washington Ave	-	0.681	0.840	0.810	0.536	0.850	0.909	0.643	0.899	0.778	0.813	0.726	0.881	0.721
		Eastbound				Westbound				Northbound			Southbound		
		EBU	EBL	EBT	EBR	WBU	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
PM Peak	R16 @ Lewis St	-	-	0.93	0.83	-	-	0.93	0.67	0.78	0.75	0.63	0.63	0.80	0.78
	R16 @ Second St	-	-	0.95	0.87	-	0.25	0.99	0.94	0.91	0.74	0.50	0.75	0.79	0.87
	R16 @ Spring St	0.81	0.90	0.93	0.87	0.65	0.83	0.93	0.85	0.92	0.72	0.79	0.64	0.75	0.83
	R16 @ South Ferry St	0.75	0.96	0.96	-	-	-	0.98	0.91	-	-	-	-	-	0.38
	R16 @ Vine ST	-	-	0.97	0.88	0.85	0.82	0.91	0.74	0.84	0.89	0.60	0.66	0.70	0.90
	R16 @ Vale St	0.50	-	0.96	0.86	0.63	0.00	0.97	-	0.90	-	0.67	-	-	-
	R16 @ Boston St	0.25	0.00	0.94	0.83	0.34	0.83	0.84	-	0.56	-	0.84	-	-	-
	R16 @ Everett Ave	0.80	0.92	0.91	0.89	0.38	0.70	0.80	0.64	0.79	0.93	0.85	0.76	0.83	0.98
	R16 @ Union St	-	-	0.90	-	-	-	0.87	0.86	0.00	-	--	-	0.92	0.63
	R16 @ Washington Ave	-	0.78	0.95	0.91	0.69	0.91	0.93	0.69	0.93	0.79	0.84	0.75	0.87	0.81

Figure 43- 15-min PHFs (Rt.16)

		Eastbound				Westbound				Northbound			Southbound		
		EBU	EBL	EBT	EBR	WBU	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
AM peak	R16 @ Lewis St	-	-	0.93	0.5	-	-	0.93	0.88	0.89	0.57	0.63	0.75	0.68	0.81
	R16 @ Second St	-	-	0.92	0.93	-	-	0.92	0.91	0.96	0.97	0.75	0.91	0.83	0.82
	R16 @ Spring St	0.8	0.95	0.91	0.7	0.9	0.86	0.93	0.86	0.91	0.77	0.89	0.75	0.71	0.93
	R16 @ South Ferry St	0.68	0.96	0.91	-	-	-	0.95	0.89	-	-	-	-	-	-
	R16 @ Vine ST	-	-	0.920	0.760	-	0.700	0.940	0.770	0.880	0.770	0.670	0.810	0.810	0.950
	R16 @ Vale St	-	-	0.920	0.870	1.000	0.750	0.930	-	0.810	-	0.500	-	-	-
	R16 @ Boston St	-	-	0.91	1.0	0.88	0.82	0.93	-	-	-	0.89	-	-	-
	R16 @ Everett Ave	0.82	0.87	0.95	0.93	0.5	0.98	0.97	0.67	0.88	0.79	0.96	0.92	0.97	0.75
	R16 @ Union St	-	-	0.98	-	-	-	1.00	0.93	-	-	-	0.86	-	0.75
	R16 @ Washington Ave	-	0.880	0.930	0.890	0.630	0.950	0.970	0.900	0.910	0.880	0.930	0.800	0.950	0.920
		Eastbound				Westbound				Northbound			Southbound		
		EBU	EBL	EBT	EBR	WBU	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
PM Peak	R16 @ Lewis St	-	-	0.97	0.97	-	-	0.99	0.80	0.78	0.83	0.71	0.58	1.00	0.82
	R16 @ Second St	-	-	0.99	0.96	-	-	0.99	1.00	0.93	0.94	1.00	0.87	0.93	0.96
	R16 @ Spring St	0.98	0.95	0.99	0.87	0.74	0.86	0.98	0.98	1.00	0.74	0.92	0.88	0.88	0.94
	R16 @ South Ferry St	0.81	0.98	0.98	-	-	-	0.99	0.96	-	-	-	-	-	-
	R16 @ Vine ST	-	-	0.98	0.92	0.85	0.88	0.97	0.89	0.89	0.93	0.66	0.80	0.81	1.00
	R16 @ Vale St	-	-	0.99	0.92	0.63	-	1.00	-	0.98	-	0.67	-	-	-
	R16 @ Boston St	-	-	0.94	0.89	0.61	0.94	0.94	-	-	-	0.92	-	-	-
	R16 @ Everett Ave	0.90	0.96	0.92	0.99	0.60	0.80	0.95	1.00	0.93	0.94	0.85	0.94	0.88	0.98
	R16 @ Union St	-	-	0.93	-	-	-	0.93	0.89	-	-	-	0.93	-	0.83
	R16 @ Washington Ave	-	0.89	0.98	0.93	0.79	0.97	0.98	0.78	0.94	0.96	0.88	0.86	0.98	0.99

Figure 44- 30-min peak hour factors (Rt.16)

6.6 Target Degree of Saturation, Lane Utilization Factor, and Slack Capacity

To account for daily variations in traffic volume, a target degree of saturation of 0.92 was used. Additional slack capacity is provided because our design uses the HCM default lane utilization factor which, for a lane group with 3 lanes, is 0.91. Together, this means that if drivers equally utilize all lanes, the design will have 17% slack capacity during the peak half-hour; during other hours, there will be still more slack capacity.

6.7 General Principles for a Timing Plan that Reduces Speeding Opportunities

To limit speeding opportunities on a two-way arterial using traffic signal timing, the timing plan follows these principles:

Short cycles – as short as possible while still meeting requirements for pedestrian crossings and vehicular capacity.

- Where a needed cycle length is especially short (the S. Ferry Street intersection), we considered double cycling (i.e., using a cycle half as long), but ultimately did not apply it.
- At two intersections, @Second Street and @Vale Street, where long pedestrian phases would force the cycle length to be especially high, yet pedestrian demand is so low that most cycles have no pedestrian call, we applied the "**oversized pedestrian phase**" technique, which means giving the ped phase a split that is shorter than the time needed to serve a ped call. This means that in cycles in which there is a ped call, the ped phase will run longer than programmed, which can result in the next coordinated phase beginning later than programmed. The controller will then follow a built-in recovery strategy to get the coordinated phase's offset back to where it belongs by shortening phases until it's recovered. Expected operations at these two intersections with oversize ped phases are analyzed in Appendix A, which shows that negative impacts are minimal.

In the western part of the corridor, **the proposed cycle lengths are 84 s (AM) and 90 s (PM),** far shorter than the current cycle lengths (110 s AM, 150 s PM). In the eastern part, the proposed cycle length is **72 s (AM and PM),** versus the current 120 s (AM and PM).

Short coordination zones in which all the intersections in a zone need a similar cycle length, in order to avoid intersections getting a lot of excess green. Zone breaks can be allowed where block length is long enough to hold a queue containing one cycle worth of cars (usually 600 to 900 ft, depending on traffic volume and cycle length).

In the proposed plan, there is one short coordination zone in the east (intersections 9 and 10, same as in current operations), and one coordination zone in the west that is rather long, with 7 intersections. It is considered breaking the western zone into smaller zones, but because the needed cycle length was very similar throughout the zone, there was no reason to break it up.

Moderately low progression speed. A combination of a short cycle and a low progression speed enables good two-way progression without creating speeding incentives, because it avoids large clusters of intersections with simultaneous green (i.e., nearly the same offset). We developed our progression plans using a progression speed of 30 mph, which is a little below the speed limit of 35 mph. Even where intersections are closely spaced, we managed to avoid clusters of more than two intersections with simultaneous green.

Offsets chosen to provide good progression yet limit speeding opportunities. Offsets provide good progression for vehicles traveling at the target speed, but as much as possible, also make it such that a car leaving one intersection at a speed in excess of the target speed will hit a red light at the next intersection or, if not, at the intersection after it. Flexibility for finding the best offset is gained by allowing left turns to be leading or lagging. To prevent motorist confusion, lead-lag position for a given phase does not change between both timing periods.

Pedestrian phases are *concurrent* wherever it's safe, have leading pedestrian intervals where turning volumes are moderately large and an LPI wouldn't increase the cycle length, and are *on recall* except where that would force the cycle to be longer. Currently, there are exclusive ped phases throughout the corridor except at Vine Street, and all pedestrian phases are pushbutton actuated, meaning they are skipped if not called. In our proposed plan:

- Pedestrian phases are ***concurrent*** at all but two intersections – wherever conflicting permitted turn volume would be less than 250 vehicles/h and the intersection geometry forces turns to be made at low speed.
- ***Leading pedestrian intervals*** are proposed for the two intersections where permitted conflicting right turn volumes from the side street exceed 125 veh/h. They give pedestrians a head start so that they can establish their priority in the crosswalk before a right turning car can reach the conflict point. They are at Vine Street, which currently has a concurrent pedestrian phase without an LPI, and Spring Street, which currently has an exclusive pedestrian phase.
- Ped phases are ***on recall*** at all except the two intersections with oversize ped phases, where ped recall would force the cycle to be long. Ped recall avoids excessive green time for the arterial, which promotes speeding. Ped phases that run with the coordinated phase, of course, have Rest-in-Walk.

Oversize ped phases are used where ped calls are infrequent and ped recall would force the cycle to be longer. At the Vale Street and Second Street intersections, where the skew angle allows high speed turns, crossings are fully protected, with little or no concurrent vehicle movement. Ped phases there are not on recall, because ped recall would force the cycle to be long; and because those ped phases are called infrequently (less than once every 5 cycles), our proposed timing uses the "oversize pedestrian" technique in which the nominal split given to the ped phase is less than what will be used to serve it if called, but more than enough on average.

Pedestrian timing for safety and convenience. Ped phases have a minimum 7 s Walk interval. Pedestrian clearance need is determined for a walking speed of 3.5 ft/s, and is met partly with the FDW interval (called Ped Clearance in the timing tables) and partly with the phase end buffer, which per the MUTCD must be at least 3 s. In our plan the pedestrian phase end buffer is the concurrent vehicle phase's change interval (Yellow + All Red). At most intersections, crossings can be made in a single stage; where a two-stage crossing is needed, signals are timed to give pedestrians good progression through the two half-crossing stages. Two stage crossings are applied only where there is a wide median with signal heads and pushbuttons.

6.8 Timing Periods

These plans are for **weekday a.m. and p.m.** There is currently a midday period (10 am to 3 pm) with no coordination. Because of how the p.m. peak has spread since the onset of the covid pandemic, and because the proposed coordination plans provide good progression in both directions, it is proposed to use the a.m. peak plan from 6 am to 12 noon, and the pm plan from 12 noon am to 7 pm.

Evening, overnight, and weekend signal operations were not studied; they can remain as they are now, or they can follow one of the two plans provided.

6.9 Cycle Length, Coordination Zones, Splits, and Offsets

Design of the signal timing plan began with a review of yellow and red clearance times, checking and adjusting them where appropriate for compliance with MassDOT guidelines. For the most part, recommend yellow times are slightly shorter and red clearance times that are slightly longer than what are used now.

Next, efficient phasing plans (ring diagrams) were determined for each junction, with particular attention to incorporating pedestrian movements concurrently subject to limits on permitted turning conflicts. Then each intersection's needed cycle length were calculated using the volume, PHF, X_t (target degree of saturation), and pedestrian timing constraints described earlier. Design calculations were done in Excel, using Synchro's input only for saturation flow rate; once a design was (tentatively) chosen, it was analyzed in full using Synchro.

Next, the corridor was divided into coordination zones based on needed cycle length (within a coordination zone, there must be a common cycle length), and with the constraint that intersections within 600 ft of one another were required to be in the same coordination zone. Where a coordination break was considered on a segment between 600 and 900 ft long, it was checked to ensure that the traffic volume and cycle length were such that a full cycle of vehicles could be stored between signalized intersections. The short distance between Second Street, which would needs a very long cycle without an oversize ped phase, and Spring Street, which doesn't, was especially constraining; because of the short distance, they must be coordinated to prevent spillback and starvation. The solution to this dilemma was applying an oversize ped phase at the Second Street intersection, which brought its needed cycle length close to that of Spring Street.

The result, shown in **Error! Reference source not found.**, was two coordination zones in both t he morning and afternoon plans. Just as in the current timing plan, the two easternmost intersections (Union, Washington) will be in their own coordination zone; however, improved phasing plan allows a cycle length of 72 s, versus 120 s in the current plan. A western coordination zone will operate with cycles of 84 s (AM) and 90 s (PM), in contrast with current cycle lengths of 110 s (AM) and 150 s (PM).

Given cycle lengths, splits were determined to ensure sufficient capacity for every movement and sufficient time for pedestrian crossings.

Finally, phase sequence (lead vs lag) and offsets were chosen to provide good progression while at the same time limiting speeding opportunities, by having platoons released from the upstream intersection arrive near the start of green, not in the middle of green, which would allow jackrabbits at the head of the platoon to zoom right through. Within each coordination zone, one intersection is arbitrarily chosen as the reference intersection, with offset 0. Offsets are referenced to the end of the green for the first coordinated phase.

Figure 45 shows, for the AM and PM timing plans, the coordination zones and each intersection's needed cycle length, proposed cycle length, offset, and highest volume to capacity (v/c) ratio during the peak 30-minutes. The only v/c ratios greater than 1.0 are for Second Street northbound; however, we believe that for this approach, Synchro greatly underestimates the saturation flow rate. Second Street has a single lane in each direction, and in Synchro's model and calculations, a vehicle waiting to turn left completely blocks the lane, whereas in reality, the great width of the intersection makes it possible for through vehicles to get by when another car is waiting in the intersection to turn left.

<table>
<tr><th rowspan="2">Intersection</th><th colspan="6">AM peak</th></tr>
<tr><th>Dist to next (ft)</th><th>Zone</th><th>Needed cycle (s)</th><th>Proposed cycle (s)</th><th>Offset (End of first coord'd green) (s)</th><th>v/c</th></tr>
<tr><td>R16 @ Lewis St</td><td>730</td><td rowspan="8">1</td><td>74.6</td><td>84</td><td>0 (Master intersection)</td><td>0.79</td></tr>
<tr><td>R16 @ Second St</td><td>500</td><td>76.1</td><td>84</td><td>42</td><td>0.82</td></tr>
<tr><td>R16 @ Spring St</td><td>600</td><td>82.1</td><td>84</td><td>33</td><td>0.89</td></tr>
<tr><td>R16 @ South Ferry St</td><td>575</td><td>14.2</td><td>84</td><td>10</td><td>0.62</td></tr>
<tr><td>R16 @ Vine ST</td><td>600</td><td>67.9</td><td>84</td><td>75</td><td>0.75</td></tr>
<tr><td>R16 @ Vale St</td><td>470</td><td>79.3</td><td>84</td><td>75</td><td>0.88</td></tr>
<tr><td>R16 @ Boston St</td><td>550</td><td colspan="4">unsignalized</td></tr>
<tr><td>R16 @ Everett Ave</td><td>1350</td><td>74.2</td><td>84</td><td>30</td><td>0.7</td></tr>
<tr><td>R16 @ Union St</td><td>430</td><td rowspan="2">2</td><td>16.9</td><td>72</td><td>0</td><td>0.56</td></tr>
<tr><td>R16 @ Washington Ave</td><td></td><td>71.6</td><td>72</td><td>0 (Master intersection)</td><td>0.71</td></tr>
<tr><th rowspan="2">Intersection</th><th colspan="6">PM peak</th></tr>
<tr><th>Dist to next (ft)</th><th>Zone</th><th>Needed cycle (s)</th><th>Proposed cycle (s)</th><th>Offset (End of green) (s)</th><th>v/c</th></tr>
<tr><td>R16 @ Lewis St</td><td>730</td><td rowspan="5">1</td><td>84.9</td><td>90</td><td>0 (Master intersection)</td><td>0.92</td></tr>
<tr><td>R16 @ Second St</td><td>500</td><td>100</td><td>90</td><td>45</td><td>0.86</td></tr>
<tr><td>R16 @ Spring St</td><td>600</td><td>82.6</td><td>90</td><td>40</td><td>0.93</td></tr>
<tr><td>R16 @ South Ferry St</td><td>575</td><td>19.4</td><td>90</td><td>0</td><td>0.77</td></tr>
<tr><td>R16 @ Vine St</td><td>600</td><td>81.5</td><td>90</td><td>92</td><td>0.93</td></tr>
<tr><td>R16 @ Vale St</td><td>470</td><td>2</td><td>100</td><td>90</td><td>0</td><td>0.86</td></tr>
<tr><td>R16 @ Boston St</td><td>550</td><td colspan="5">unsignalized</td></tr>
<tr><td>R16 @ Everett Ave</td><td>1350</td><td>3</td><td>90.7</td><td>90</td><td>30</td><td>0.9</td></tr>
<tr><td>R16 @ Union St</td><td>430</td><td rowspan="2">4</td><td>29.2</td><td>72</td><td>0</td><td>0.56</td></tr>
<tr><td>R16 @ Washington Ave</td><td></td><td>69.8</td><td>72</td><td>0 (Master intersection)</td><td>0.82</td></tr>
</table>

Figure 45- General signal timing at Rt.16

6.10 Timing Plans by Intersection

Following are the proposed signal timing plans at each intersection for weekday AM and PM periods. Where concurrent pedestrian crossings are proposed, we show the volumes of permitted conflicting turning movements. Calculations used to determine these settings are provided in an accompanying spreadsheet.

Phase numbering is unchanged from the current plan. Extension intervals are also all unchanged. (Extension intervals on the arterial are irrelevant because there are no detectors.) Recall settings are as follow:

- Min: Recall, but require no more than minimum green
- Max: Recall and require maximum green
- C-Max: Recall, and run as the coordinated phase until its forceoff. Forceoffs are based on max green. Coordinated phases all have Rest in Walk, so ped phases concurrent with a coordinated phase are all on recall as well.
- Ped: Recall the vehicle phase and the concurrent ped phase.

In addition, all intersections should use ***fixed forceoff*** rather than floating forceoff. This allows slack time to be used more flexibly, and is especially critical at the two inersections with oversize ped phases.

6.10.1 Rt 16 at Washington Ave

The current timing plan has a pushbutton-actuated exclusive pedestrian phase; however, it is only long enough for pedestrians crossing Rt 16 to go halfway, making them wait in the median to finish a cycle later. This leads to unacceptably long delay for pedestrians who comply, making it such that most pedestrians will not comply, which is a safety issue.

Figure 46 shows the preferred coordinated sequence, with leading lefts as in the current plan. In the proposed plan, pedestrian crossings are *concurrent* and are *on recall*. Crossings across Rt 16 are still two-stage, as they are now, but are coordinated with moderately good pedestrian progression so that pedestrians have either no wait or only a moderately short wait in the median. The pedestrian phases are protected from all left turn conflicts and from all right turn conflicts from Washington Ave. The only permitted conflict is right turns from Rt 16, whose volume is less than 250 veh/h, and whose sharp turning angle forces turning vehicles to go slowly (Figure 3-2. We also not that there is little pedestrian demand for this crossing because on either side of the intersection, there are no homes or businesses along the south side of Rt 16.

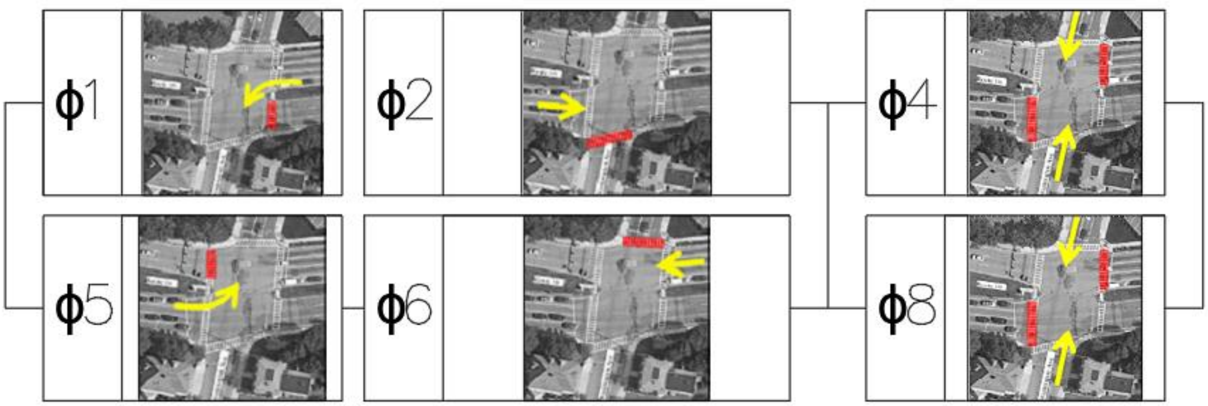


Figure 46-Proposed phase sequence at Washington Ave

Figure 47- Permitted turn volumes (Washington Ave)

The proposed phasing sequence offers excellent coordination for the 2-stage crossing across Rt 16 for pedestrians walking along the left side of the street (that is, northbound pedestrians walking in the western sidewalk, and southbound pedestrians using the eastern sidewalk): they begin in phase 4/8 and continue immediately in phase 1/5. Pedestrians walking on the right side of the street will have to wait in the median while the arterial is served (as they do in the current timing plan) – they will begin in phase 1/5, wait in the middle during phase 2/6, and finish in phase 4/8.

This scheme requires that each of the 8 pedestrian signals for crossing Rt 16 be wired and controlled separately, but is simple in that there are no pedestrian overlaps; that is, each pedestrian crossing times with a single vehicular phase.

Figure 48 shows the timing plan parameters for the intersection with Washington Ave. The left turn phases are on recall because of their concurrent pedestrian phases. Minimum green times satisfy pedestrian clearance needs. This intersection is master in morning and afternoon plan.

Intersection	R16 @ Washington Ave
Plan	Weekday
Time	6:00 am-12:00 noon
Offset (s)	0
Cycle length (s)	72
Master Intersection	Yes

Phase	WBL	EB		SB	EBL	WB		NB
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	10		10	6	10		10
Extension interval (s)	2	2		3	2	2		3
Yellow (s)	3	3.5		3	3	3.5		3
Red Clear (s)	3	1.5		3	3	1.5		3
Maximum Green (s)	13	23		19	13	23		19
Total split (s)	19	28		25	19	28		25
Lead/Lag	Lead				Lead			
Leading pedestrian interval (s)								
Pedestrian Walk (s)	7	7 RIW*		7	7	7 RIW*		7
Ped Clearance (s)	5	12		11	5	12		11
Ped phase end buffer (s)	6	5		6	6	5		6
Recall (s)	Ped	C-Max		Ped	Ped	C-Max		Ped
Overlap								

Intersection	R16 @ Washington Ave
Plan	Weekday
Time	12 noon - 19:00
Offset (s)	0
Cycle length (s)	72
Master Intersection	Yes

Phase	WBL	EB		SB	EBL	WB		NB
	1	2	3	4	5	6	7	8
Minimum Green (s)	12	19		18	12	19		18
Extension interval (s)	2	2		3	2	2		3
Yellow (s)	3	3.5		3	3	3.5		3
Red Clear (s)	3	1.5		3	3	1.5		3
Maximum Green (s)	13	23		19	13	23		19
Total split (s)	19	28		25	19	28		25
Lead/Lag	Lead				Lead			
Leading pedestrian interval (s)								
Pedestrian Walk (s)	7	7 RIW*		7	7	7 RIW*		7
Ped Clearance (s)	5	12		11	5	12		11
Ped phase end buffer (s)	6	5		6	6	5		6
Recall (s)	Ped	C-Max		Ped	Ped	C-Max		Ped
Overlap								

Note*: Rest in Walk

Figure 48- Proposed timing plans (Washington Ave)

6.10.2 Rt 16 at Union St

The proposed phases and sequence are the same as in the current plan. This is a Y-intersection in which the internal left turn (EBL) is not allowed, and there are no signalized pedestrian crossings. (There is no crossing of Rt 16, because of the nearby crossing at Washington Street and because the Y-geometry results in virtually no demand for a crossing there; and the crossing of Union Street is outside the boundaries of the signalized intersection, 80 ft upstream of the stop-line. That crossing could be made safer by adding a small median with a "Crosswalk - Yield to Pedestrians" sign.)

Figure 49 shows the timing plan parameters for the intersection with Union St.

Intersection	R16 @ Union St
Plan	Weekday
Time	6:00 am to 12 noon
Offset (s)	0
Cycle length (s)	72
Master Intersection	

Phase		EB		SB		WB		
	1	2	3	4	5	6	7	8
Minimum Green (s)		10		10		10		
Extension interval (s)		3		2		3		
Yellow (s)		3.5		3		3.5		
Red Clear (s)		1.5		3		1.5		
Maximum Green (s)		38		23		38		
Total split (s)		43		29		43		
Lead/Lag								
Pedestrian Walk (s)								
Pedestrian Clearance (s)								
Ped phase end buffer (s)								
Recall (s)		C-Max		Max		C-Max		
Overlap								

Intersection	R16 @ Union St
Plan	Weekday
Time	12 noon - 19:00
Offset (s)	0
Cycle length (s)	72
Master Intersection	

Phase		EB		SB		WB		
	1	2	3	4	5	6	7	8
Minimum Green (s)		10		10		10		
Extension interval (s)		3		2		3		
Yellow (s)		3.5		3		3.5		
Red Clear (s)		1.5		3		1.5		
Maximum Green (s)		38		27		38		
Total split (s)		43		29		43		
Lead/Lag								
Pedestrian Walk (s)								
Pedestrian Clearance (s)								
Ped phase end buffer (s)								
Recall (s)		C-Max		Max		C-Max		
Overlap								

Figure 49- Proposed timing plans (Union St)

6.10.3 Rt 16 at Everett Ave

The proposed phase sequence at this intersection differs in two ways from the existing plan: first, there is no exclusive pedestrian phase (instead, pedestrians are concurrent), and second, instead of both of Rt 16's left turns leading, both are lagging.

For the proposed concurrent pedestrian phases, Figure 50 shows the conflicting turn volumes that will be permitted. All permitted conflicts (considering both peak hours) have volumes well below 250 veh/h. The maximum conflicting volume, EBR during the p.m. peak, is 158 veh/h; for the proposed cycle length, that's 4.8 right turns per cycle. Mitigating this conflict is the fact that the stopline for EBR is 50 ft distant from the crosswalk, giving pedestrians a head start in space, allowing pedestrians to establish their presence and priority before a turning vehicle arrives without the need for a leading pedestrian interval. And while the deflection angle for this turn is less than 90 degrees, the narrow receiving road (only one lane southbound) forces turns to be made slowly.

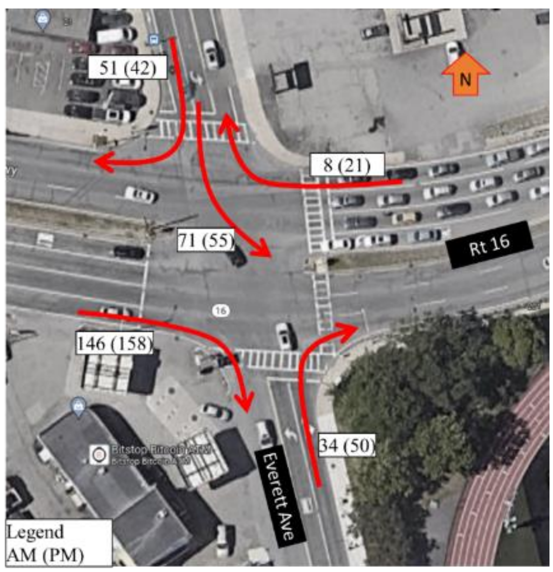


Figure 50-Permitted turning volumes (Everett Ave)

Figure 51 shows the timing plan parameters for the intersection with Everett Ave.

Intersection	R16 @ Everett Ave
Plan	Weekday
Time	6:00 am to 12 noon
Offset (s)	40
Cycle length (s)	84
Master Intersection	

Phase	WBL	EB		SB	EBL	WB		NB
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	19		31	6	19		31.5
Extension interval (s)	2	4		5	2	4		5
Yellow (s)	3	3.5		3	3	3.5		3
Red Clear (s)	2	1.5		3	2	1.5		3
Maximum Green (s)	6	31		31	6	31		31
Total split (s)	11	36		37	11	36		37
Lead/Lag	Lag				Lag			
Leading pedestrian interval (s)								
Pedestrian Walk (s)		7 RIW*		7		7 RIW*		7
Pedestrian Clearance (s)		14		27		14		27
Ped phase end buffer (s)		3		3		3		3.5
Recall (s)		C-Max		Ped		C-Max		Ped
Overlap								

Intersection	R16 @ Everett Ave
Plan	Weekday
Time	12 noon - 19:00
Offset (s)	30
Cycle length (s)	90
Master Intersection	

Phase	WBL	EB		SB	EBL	WB		NB
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	19		31	6	19		31
Extension interval (s)	2	4		5	2	4		5
Yellow (s)	3	3.5		3	3	3.5		3
Red Clear (s)	2	1.5		3	2	1.5		3
Maximum Green (s)	15	28		31	15	28		31
Total split (s)	20	33		37	20	33		37
Lead/Lag	Lag				Lag			
Leading pedestrian interval (s)								
Pedestrian Walk (s)		7 RIW*		7		7 RIW*		7
Pedestrian Clearance (s)		14		27		14		27
Ped phase end buffer (s)		3		3		3		3
Recall (s)		C-Max		Ped		C-Max		Ped
Overlap								

Note*: Rest in Walk

Figure 51- Proposed timing plans (Everett Ave)

6.10.4 Rt 16 at Vale St

The ring diagram shown in Figure 52 shows the proposed phase sequence for the intersection with Vale St. As in the current plan, eastbound left (into a commercial driveway) is not allowed, and pedestrian crossings are on the western and southern sides of the intersection only. Because the skew angle permits high speed turns, pedestrian crossings are fully protected in both the current and proposed plans. However, unlike the current plan, the proposed plan doesn't have an exclusive ped phase; instead, WBL is allowed to time concurrently with peds crossing Rt 16.

Formally, peds crossing Route 16 will have a 2-stage crossing. (There is a wide median, equipped with signal heads and a pushbutton). However, the timing plan is such that pedestrians walking south will be able to cross in a single pass (begin in Phase 9, finish in Phase 4), and a large fraction of pedestrians walking north will also be able to cross in a single stage as well, because the clearance time is long enough for people walking as slow as 2.0 ft/s to finish the half-crossing, and for people walking 4.0 ft/s, if they start in the first 4 s of WALK, to finish the full crossing.

Neither of the ped crossing phases (Phase 9, Phase 8) will be on recall. Both are oversize ped phases, in the sense that their programming split is shorter that the time they will use when called; however, those phases are not called often. As the appendix shows, the impact of these oversize ped phases on operations is negligible because one is oversized by only 2 s, and the other can use the time leftover from Phase 5, the WBL phase that is not called in most cycles (demand is only 5 vehicles/h).

The coordinated phases and Vale Street will be on recall. We also propose that the southern half-crossing in Phase 4 (concurrent with Vale Street) be on recall; that way, when Phase 9 is served, signals for the southern half-crossing can have long WALK interval, with ped clearance taking place during Phase 8.

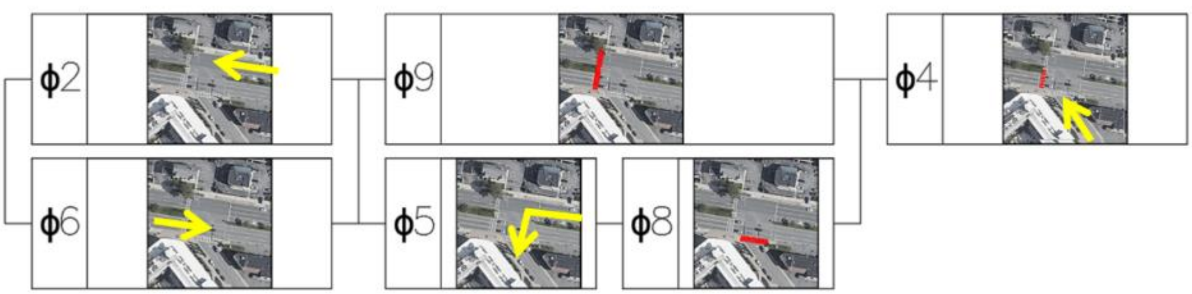


Figure 52- Proposed Ring diagram (Vale St)

Figure 53 shows the timing parameters for the intersection at Vale St.

Intersection	R16 @ Vale St
Plan	Weekday
Time	6:00 am to 12 noon
Offset (s)	74
Cycle length (s)	84
Master Intersection	

Phase		WB		NB/SB(VALE)	WBL	EB		Ped (VALE)	Ped (R16)
	1	2	3	4	5	6	7	8	9
Minimum Green (s)		10		8	6	10			
Extension interval (s)		2		2	2	2			
Yellow (s)		3.5		3	3	3.5			
Red Clear (s)		1.5		3	2	1.5			
Maximum Green (s)		33		12	6	33			
Total split (s)		38		18	11	38		15	26
Lead/Lag									
Leading pedestrian interval (s)									
Pedestrian Walk (s)				7				7	7
Pedestrian Clearance (s)				5				15	18
Ped phase end buffer (s)				6				3	3
Recall (s)		C-Max		Ped		C-Max			
Overlap									

Intersection	R16 @ Vale St
Plan	Weekday
Time	12 noon - 19:00
Offset (s)	9
Cycle length (s)	90
Master Intersection	

Phase		WB		NB/SB(VALE)	WBL	EB		Ped (VALE)	Ped (R16)
	1	2	3	4	5	6	7	8	9
Minimum Green (s)		10		8	6	10			
Extension interval (s)		2		2	2	2			
Yellow (s)		3.5		3	3	3.5			
Red Clear (s)		1.5		3	2	1.5			
Maximum Green (s)		29		24	6	29			
Total split (s)		34		30	11	34		15	26
Lead/Lag									
Leading pedestrian interval (s)									
Pedestrian Walk (s)								7	4
Pedestrian Clearance (s)								15	21
Ped phase end buffer (s)								3	3
Recall (s)		C-Max		Ped		C-Max			
Overlap									

Figure 53- Proposed timing plans (Vale St)

6.10.5 Rt 16 at Vine St

The proposed phase sequence is the same as current except for the introduction of an LPI for Vine Street. As in the current plan, all vehicle movements are allowed except eastbound left, and the pedestrian crossings (western and southern sides of the intersection only) are concurrent with vehicle phases.

However, because right turn volume from Vine Street SB reaches 186 veh/h in the AM peak and 136 veh/h in the PM peak (see Figure 54), we propose a 6-second LPI for Vine Street, making the crossing there “partially protected.” The LPI does not increase the necessary cycle length because the Vine Street split is governed by the pedestrian phase.

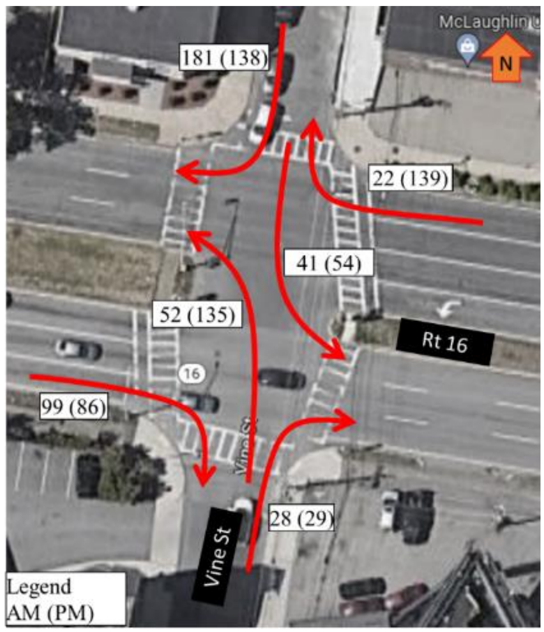


Figure 54- Permitted turning volumes (Vine St)

Figure 55 shows the timing parameters for the intersection with Vine Street.

Intersection	R16 @ Vine ST
Plan	Weekday
Time	6:00 am to 12 noon
Offset (s)	75
Cycle length (s)	84
Master Intersection	

Phase	WBL	EB		NB/SB(VINE)		WB		
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	13		30		13		
Extension interval (s)	2	3		3		3		
Yellow (s)	3	3.5		3		3.5		
Red Clear (s)	2	1.5		3		1.5		
Maximum Green (s)	6	27		29		38		
Total split (s)	11	32		35		43		
Lead/Lag	Lag							
Leading pedestrian interval (s)				6				
Pedestrian Walk (s)		7 RIW*		7		7 RIW*		
Pedestrian Clearance (s)		8		26		8		
Ped phase end buffer (s)		3		3		3		
Recall (s)		C-Max		Ped		C-Max		
Overlap								

Intersection	R16 @ Vine ST
Plan	Weekday
Time	12 noon - 19:00
Offset (s)	85
Cycle length (s)	90
Master Intersection	

Phase	WBL	EB		NB/SB(VINE)		WB		
	1	2	3	4	5	6	7	8
Minimum Green (s)	2	13		30		13		
Extension interval (s)	2	3		3		3		
Yellow (s)	3	3.5		3		3.5		
Red Clear (s)	2	1.5		3		1.5		
Maximum Green (s)	6	32		30		43		
Total split (s)	11	37		42		48		
Lead/Lag	Lag							
Leading pedestrian interval (s)				6				
Pedestrian Walk (s)		7 RIW*		7		7 RIW*		
Pedestrian Clearance (s)		8		26		8		
Ped phase end buffer (s)		3		3		3		
Recall (s)		C-Max		Ped		C-Max		
Overlap								

Note*: Rest in Walk

Figure 55- Proposed timing plans (Vine St)

6.10.6 Rt 16 at South Ferry St

Figure 56 shows the timing plan parameters for the intersection with S outh Ferry St, a T intersection in which the minor street is a one-way street with departing traffic only. As in the existing plan, the signal controls only eastbound left and westbound thru; eastbound thru is not controlled as there are no pedestrian crossings across Rt 16.

We considered double cycling at this intersection (i.e., running it with half the cycle length of its coordination zone). Instead, however, because this signal does not control eastbound thru traffic, we coordinated it as though it's a one-way street westbound, using an offset that constrains traffic released from the upstream intersection to a desired progression speed.

Intersection	R16 @ South Ferry St
Plan	Weekday
Time	6:00 am to 12 noon
Offset (s)	10
Cycle length (s)	84
Master Intersection	

Phase	WB	EBL						
	1	2	3	4	5	6	7	8
Minimum Green (s)	13	6						
Extension interval (s)	5	5						
Yellow (s)	3.5	3						
Red Clear (s)	1.5	1						
Maximum Green (s)	49	26						
Total split (s)	54	30						
Lead/Lag								
Leading pedestrian interval (s)								
Pedestrian Walk (s)	7 RIW*							
Pedestrian Clearance (s)	8							
Ped phase end buffer (s)	3							
Recall (s)	C-Max							
Overlap								

Intersection	R16 @ South Ferry St
Plan	Weekday
Time	12 noon - 19:00
Offset (s)	8
Cycle length (s)	90
Master Intersection	

Phase	WB	EBL						
	1	2	3	4	5	6	7	8
Minimum Green (s)	13	3						
Extension interval (s)	5	5						
Yellow (s)	3.5	3						
Red Clear (s)	1.5	1						
Maximum Green (s)	45	36						
Total split (s)	50	40						
Lead/Lag								
Leading pedestrian interval (s)								
Pedestrian Walk (s)	7 RIW*							
Pedestrian Clearance (s)	8							
Ped phase end buffer (s)	3							
Recall (s)	C-Max							
Overlap								

Note*: Rest in Walk

Figure 56- Proposed timing plans (S Ferry St)

6.10.7 Rt 16 at Spring St

The proposed phasing sequence for this intersection differs in two ways from the existing plan: first, there is no exclusive pedestrian phase (instead, pedestrians are concurrent), and second, instead of both of Rt 16's left turns leading, one leads and one lags.

Figure 57shows the turning movements that will be permitted during pedestrian crossings. All volumes are well below 250 veh/h. For southbound on Spring, where the right turn volume in the a.m. peak is 163 veh/h, a 6 sec leading pedestrian interval is proposed to enable pedestrians crossing Rt 16 to establish their presence and priority before right turning vehicles arrive. Other right turn volumes are 60 veh/h or less.

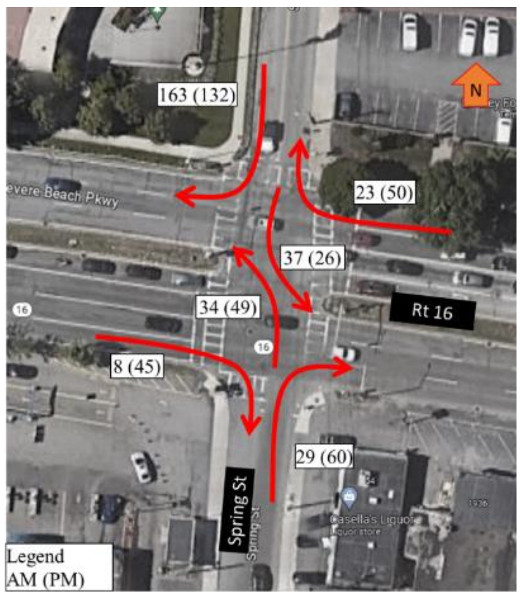


Figure 57- Permitted turning volumes (Spring St)

Figure 58 shows the timing plan parameters for the intersection with Spring St.

Intersection	R16 @ Spring St
Plan	Weekday
Time	6:00 am to 12 noon
Offset (s)	33
Cycle length (s)	84
Master Intersection	

Phase	EBL	WB		NB/SB	WBL	EB		
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	13		30	6	13		
Extension interval (s)	2	7		2	2	7		
Yellow (s)	3	3.5		3	3	3.5		
Red Clear (s)	2	1.5		3	2	1.5		
Maximum Green (s)	6	32		24	6	32		
Total split (s)	11	37		30	11	37		
Lead/Lag	Lag				Lead			
Leading pedestrian interval (s)				6				
Pedestrian Walk (s)		7 RIW*		7		7 RIW*		
Pedestrian Clearance (s)		8		26		8		
Ped phase end buffer (s)		3		3		3		
Recall (s)		C-Max		Ped		C-Max		
Overlap								

Intersection	R16 @ Spring St
Plan	Weekday
Time	12 noon - 19:00
Offset (s)	32
Cycle length (s)	90
Master Intersection	

Phase	EBL	WB		NB/SB	WBL	EB		
	1	2	3	4	5	6	7	8
Minimum Green (s)	6	13		10	6	13		
Extension interval (s)	2	7		2	2	7		
Yellow (s)	3	3.5		3	3	3.5		
Red Clear (s)	2	1.5		3	2	1.5		
Maximum Green (s)	9	35		24	9	35		
Total split (s)	14	40		36	14	40		
Lead/Lag	Lag				Lead			
Leading pedestrian interval (s)				6				
Pedestrian Walk (s)		7 RIW*		7		7 RIW*		
Pedestrian Clearance (s)		8		26		8		
Ped phase end buffer (s)		3		3		3		
Recall (s)		C-Max		Ped		C-Max		
Overlap								

Note*: Rest in Walk

Figure 58- Proposed timing plans (Spring St)

6.10.8 Rt 16 at Second St

This is a skewed intersection with heavy turn volumes that can be made at high speed. No change to the phasing plan is proposed; as in the current plan, there are no left turns allowed from the arterial, and there is an exclusive ped phase.

Figure 59 shows the proposed timing plan parameters. Because of the low pedestrian volume and the long pedestrian phase time, it uses an oversize pedestrian phase, meaning the nominal split given to the ped phase (12 s) is less than what's needed to serve the pedestrian movement (36 s), so that when the pedestrian phase is served, subsequent phases will start late, triggering a recovery process described in Appendix A that will get the intersection back in sync. Minimum green and splits were chosen to ensure that the recovery will be quick and balanced. As shown in Appendix A, the intersection is expected to recover by the end of the cycle following the one with a ped call, with no green deficit (i.e., no overflow queue) on Route 16 and only a small and transient green deficit (overflow queue) on Second Street.

Intersection	R16 @ Second St
Plan	Weekday
Time	6:00 am to 12 noon
Offset (s)	42
Cycle length (s)	84
Master Intersection	

Phase	EB/WB	Ped	NB/SB						
	1	2	3	4	5	6	7	8	9
Minimum Green (s)	38		15						
Extension interval (s)	2		2						
Yellow (s)	3.5		3						
Red Clear (s)	3.5		3						
Maximum Green (s)	38		21						
Total split (s)	45	12	27						
Lead/Lag									
Leading pedestrian interval (s)									
Pedestrian Walk (s)		7							
Pedestrian Clearance (s)		26							
Ped phase end buffer (s)		3							
Recall (s)	C-Max		Min						
Overlap									

Intersection	R16 @ Second St
Plan	Weekday
Time	12 noon - 19:00
Offset (s)	41
Cycle length (s)	90
Master Intersection	

Note*: Rest in Walk

Phase	Ped	EB/WB	NB/SB						
	1	2	3	4	5	6	7	8	9
Minimum Green (s)	43		17						
Extension interval (s)	2		2						
Yellow (s)	3.5		3						
Red Clear (s)	3.5		3						
Maximum Green (s)	43		22						
Total split (s)	50	12	28						
Lead/Lag									
Leading pedestrian interval (s)									
Pedestrian Walk (s)		7							
Pedestrian Clearance (s)		26							
Ped phase end buffer (s)		3							
Recall (s)	C-Max		Min						
Overlap									

Figure 59- Proposed timing plans (Second St)

6.10.9 Rt 16 at Lewis St

As in the current plan, there are only two vehicle phases (one for each street), since no left turns are allowed from arterial. The existing exclusive pedestrian phase has been replaced with concurrent crossings. As shown in , permitted turn conflicts have low volumes. In the current operation, this intersection is not coordinated, but we propose that it be coordinated, with pedestrian crossings on recall in order to avoid excessive arterial green.

Figure 60 shows the proposed timing plan parameters.

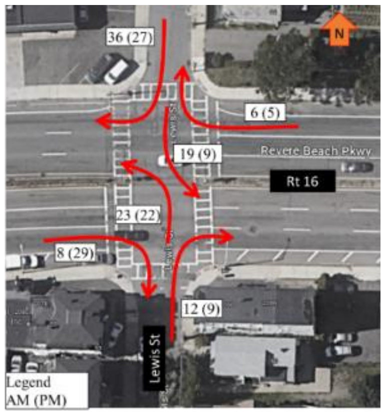


Figure 60- Permitted turning volumes (Lewis St)

Intersection	R16 @ Lewis St
Plan	Weekday
Time	6:00 am to 12 noon
Offset (s)	0
Cycle length (s)	84
Master Intersection	Yes

Phase	EB/WB		NB/SB					
	1	2	3	4	5	6	7	8
Minimum Green (s)	10		10					
Extension interval (s)	5		5					
Yellow (s)	3.5		3					
Red Clear (s)	1.5		3					
Maximum Green (s)	43		30					
Total split (s)	48		36					
Lead/Lag								
Leading pedestrian interval (s)								
Pedestrian Walk (s)	7 RIW*		7					
Pedestrian Clearance (s)	8		26					
Ped phase end buffer (s)	3		3					
Recall (s)	C-Max		Ped					
Overlap								

Intersection	R16 @ Lewis St
Plan	Weekday
Time	12 noon - 19:00
Offset (s)	0
Cycle length (s)	90
Master Intersection	Yes

Phase	EB/WB		NB/SB					
	1	2	3	4	5	6	7	8
Minimum Green (s)	14		20					
Extension interval (s)	5		5					
Yellow (s)	3.5		3					
Red Clear (s)	1.5		3					
Maximum Green (s)	49		30					
Total split (s)	54		36					
Lead/Lag								
Leading pedestrian interval (s)								
Pedestrian Walk (s)	7 RIW*		7					
Pedestrian Clearance (s)	8		26					
Ped phase end buffer (s)	3		3					
Recall (s)	C-Max		Ped					
Overlap								

Note*: Rest in Walk

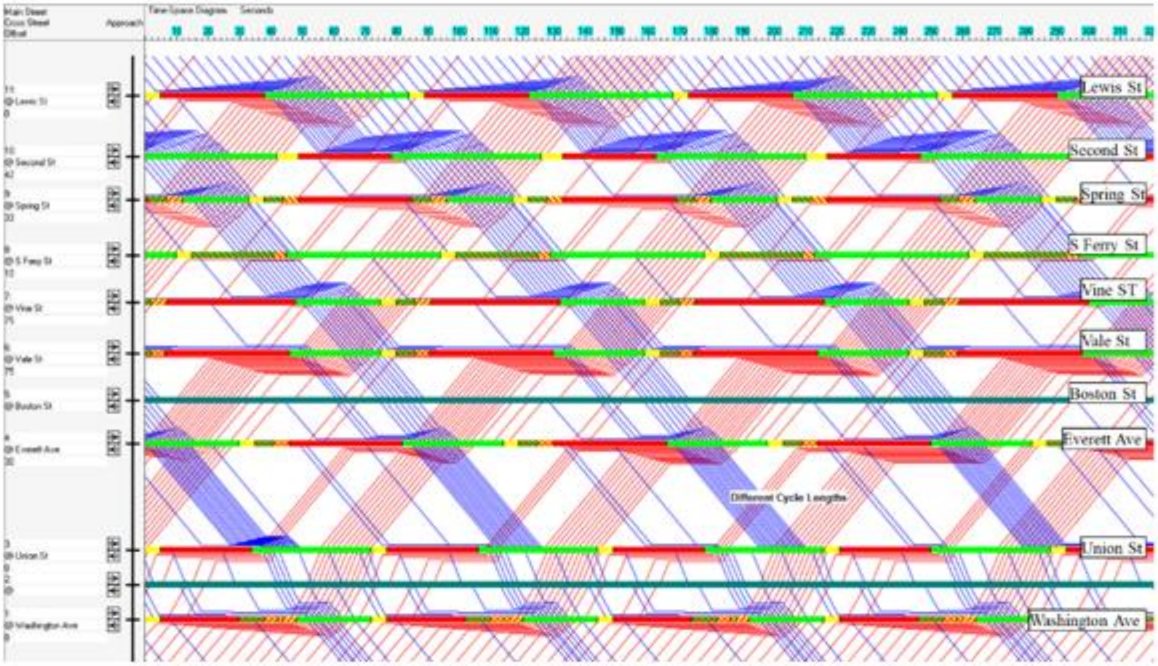
Figure 61- Proposed timing plans (Lewis St)

6.11 Progression Diagrams

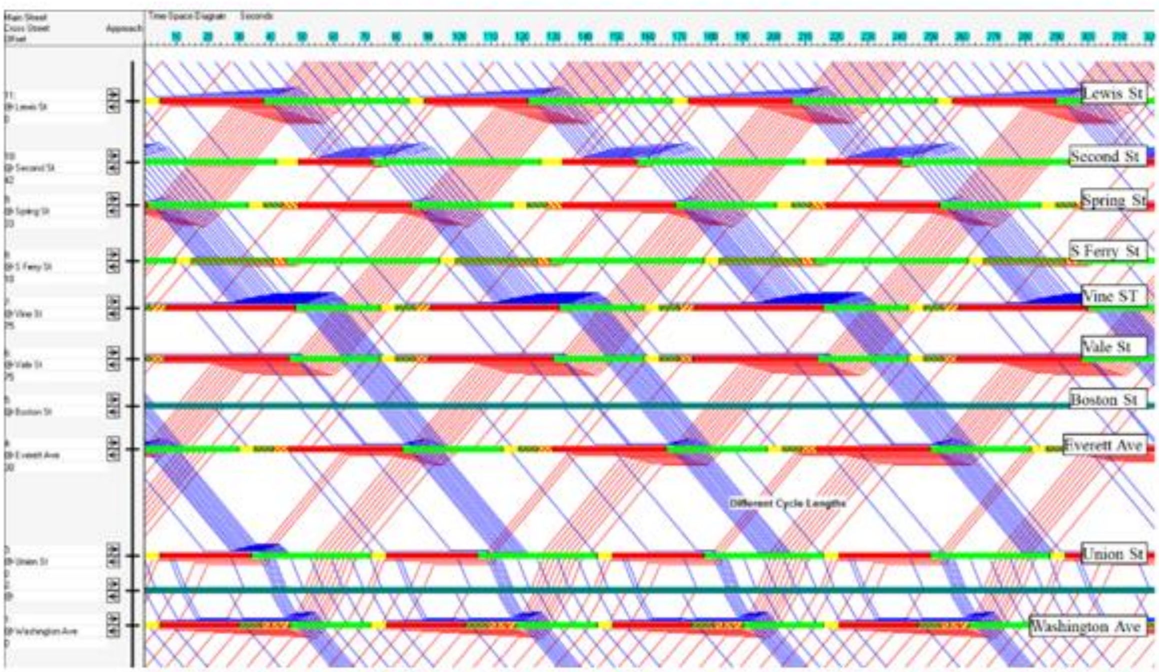
Progression diagrams are shown for both the current and proposed signal plan in two conditions: 100% of design volume and 75% of design volume. The 75% case is examined because speeding opportunities tend to be greater when volumes are lower – that is, outside the peak period – because of surplus green time.

The horizontal axis is time and vertical axis is distance along the arterial, with intersection locations shown. Blue lines going “downhill” represent eastbound vehicles and red lines going “uphill” represent westbound vehicles. The progression speed used in these diagrams is 30 mph. That is a little lower than the speed limit (35 mph) because a lower progression speed helps limit speeding opportunities while still providing good service if vehicles can get a green wave. One can see that progression is rather good for both directions except at coordination zone breaks.

Speeding opportunities occur when vehicles arrive at an intersection on a stale green with no vehicle ahead of them. In the proposed plan, with 100% volume, few such speeding opportunities are apparent in either direction. When the head of a platoon arrives at the next intersection, it usually cannot go faster that the progression speed (steeper up or down) without running into red. With 75% volume, there are more vehicles arriving at intersections with a speeding opportunity, but not many more.

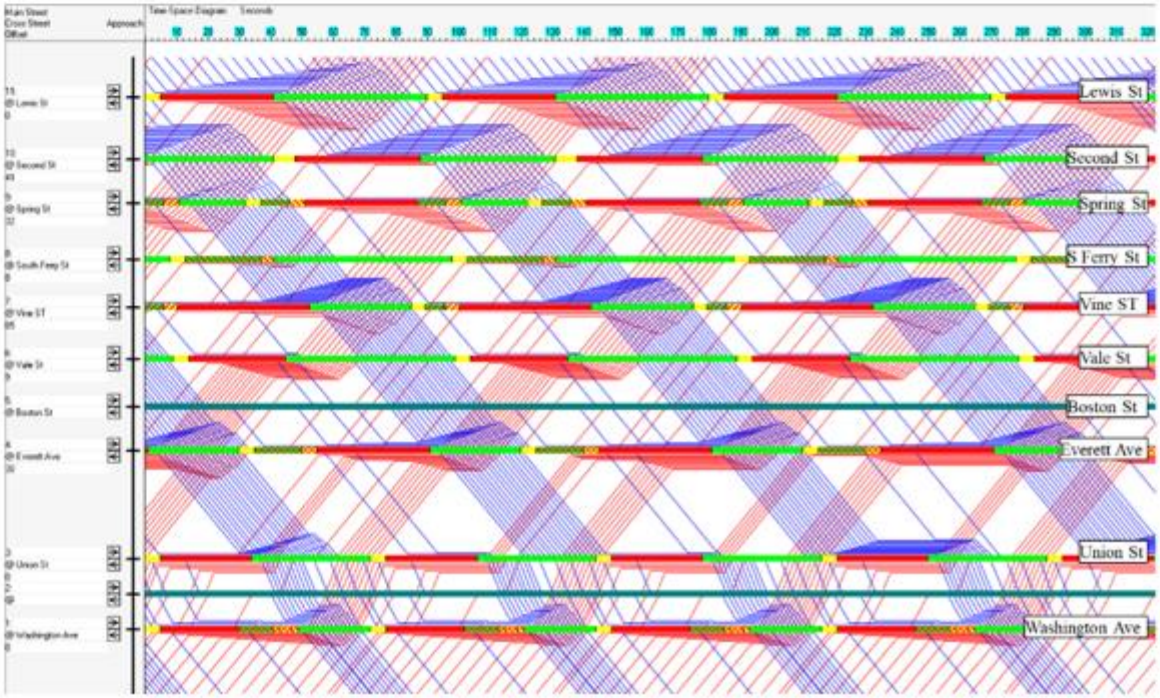


100 % volume

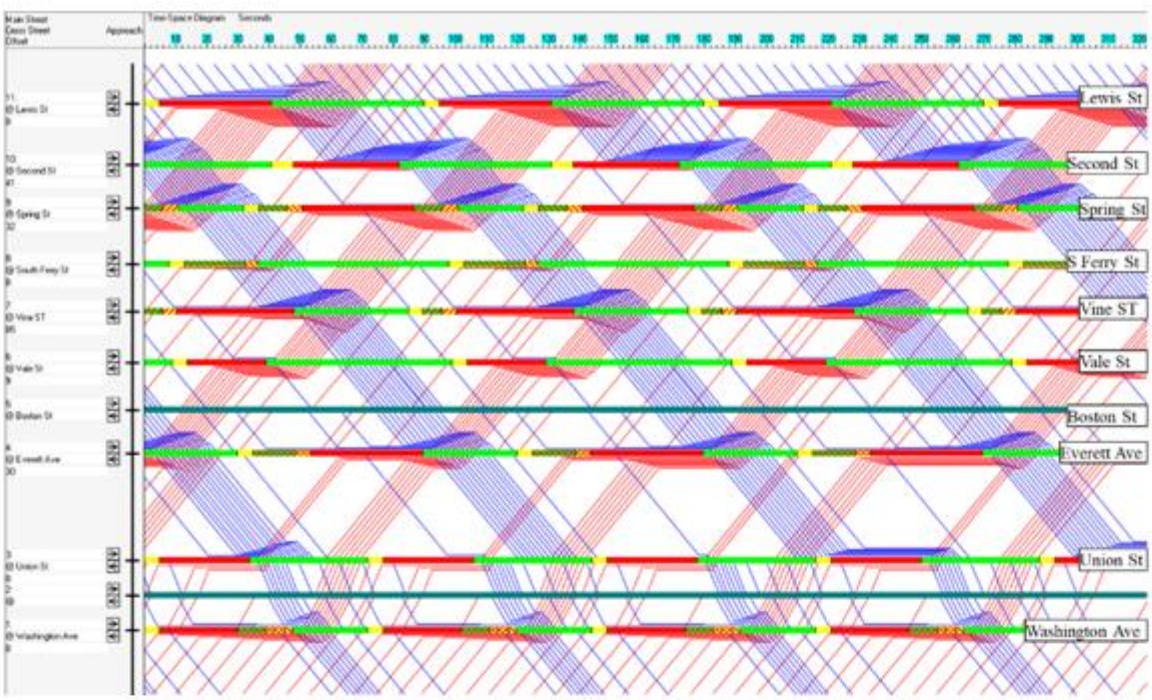


75% volume

Figure 62- Progression diagrams (Proposed AM plan)



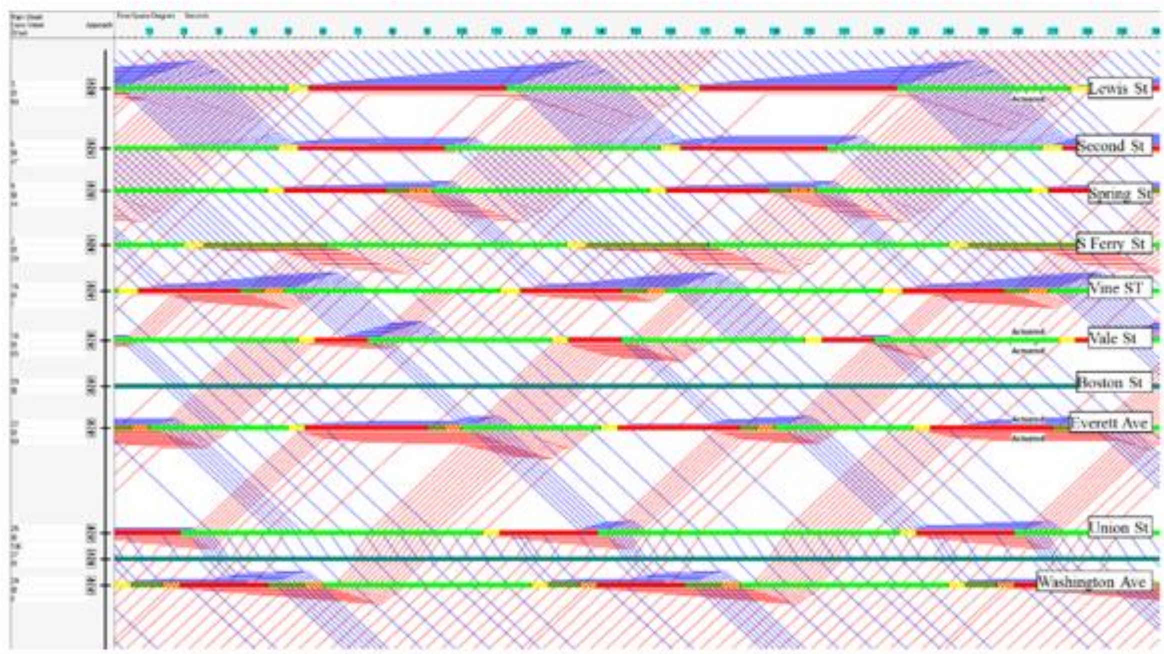
100 % volume



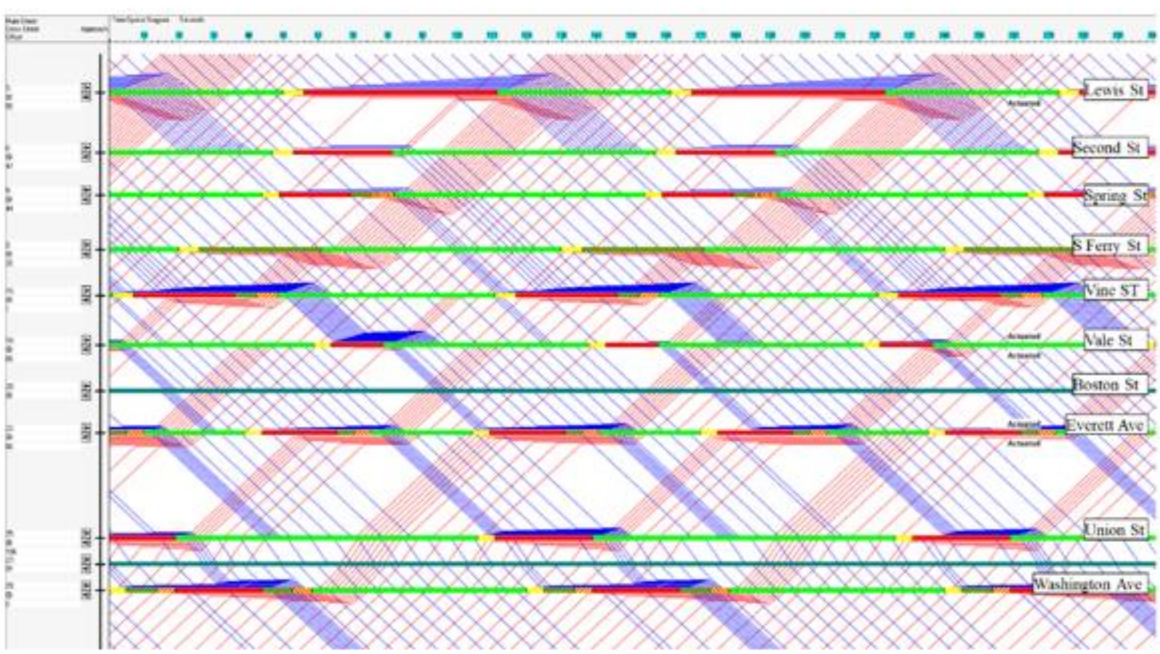
75% volume

Figure 63- Progression diagram (Proposed PM plan)

In the following figures, the *current* signal plan for the a.m. and p.m. period in two conditions of 100% and 75% are presented. With their longer cycles, there are many more speeding opportunities than in the proposed plan.

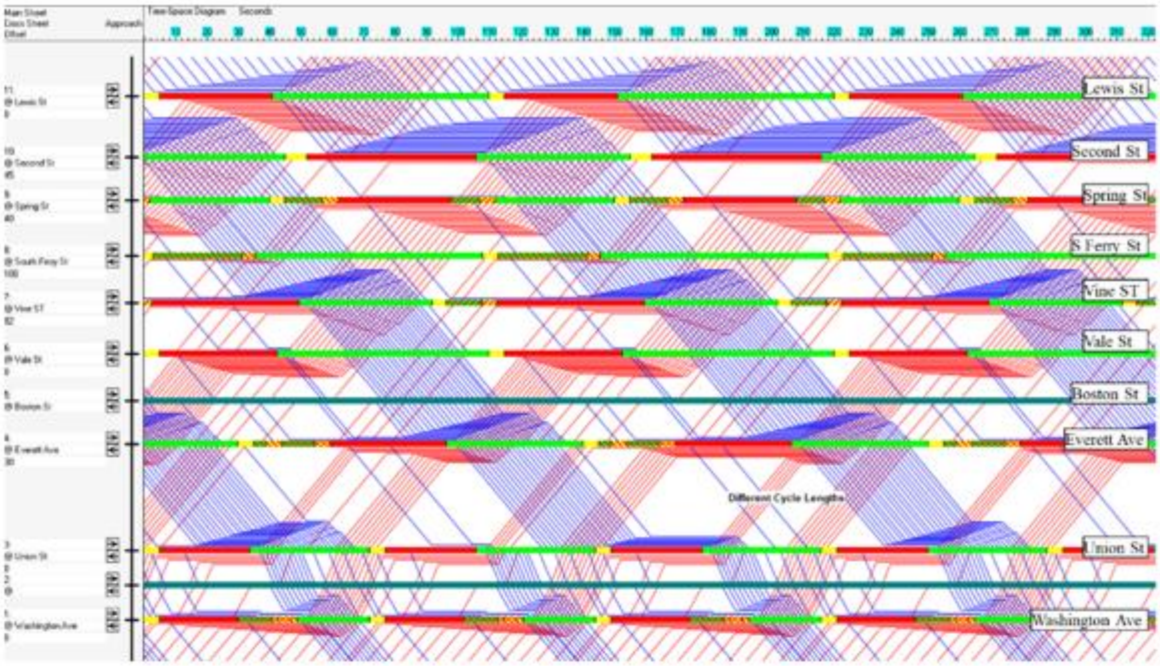


100 % volume

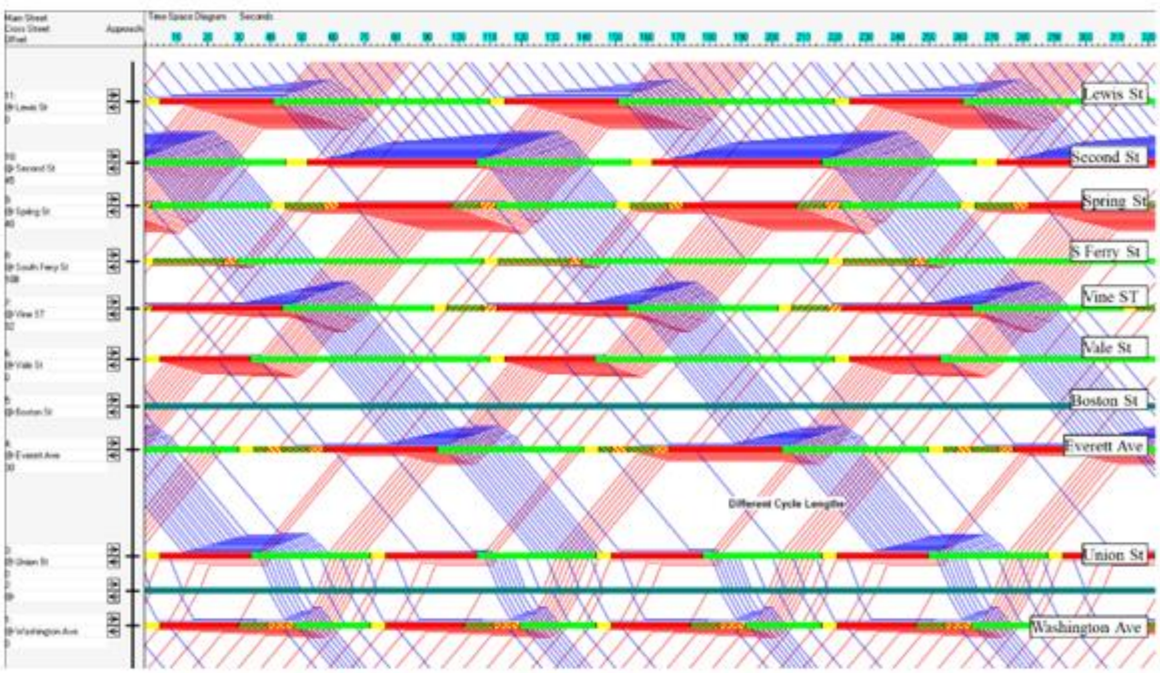


75% volume

Figure 64- Progression diagrams (Current AM plan)



100 % volume



75% volume

Figure 65- Progression diagrams (Current PM plan)

6.12 Oversized Ped Operation at Second Street and Vale Street Intersections

In this section the design and operation of oversized ped phase at two intersections of Second St and Vale St is explained.

6.12.1 Oversized Ped Phases at the Second Street Intersection

In the proposed plan for Rt 16, the Second St intersection has an oversized exclusive pedestrian phase. Its proposed nominal split (the time "set aside" for it in the cycle) is 12 s. When skipped, it will use 0 s, of course, but when called, it will run for 36 s, which will result in the coordinated phase beginning later than its programmed offset, getting out of sync with the background cycle.

Controllers have different methods to recover to the background cycle. The controller at this intersection, a Siemens m60, has several recovery modes described in the SEPAC 5.4 user manual. We propose applying the most common recovery method called "shortway." It recovers as quickly as possible by shortening phases, subject to two constraints: minimum green must always be respected, and the cycle length during recovery won't be forced to be less than 80% of the programmed cycle length. When this 80% limit is constraining, phase time reductions are proportional to programmed splits, subject to minimum green. The routine that supervises recovery is called "Coordinator." Every cycle, Coordinator compares the start time of the coordinated phase with a sync pulse that follows the background cycle. If the measured lag is greater than the programmed offset, it starts the recovery mode operation.

Because the recovery process is affected by programmed splits (because of the proportionality rule) and by minimum greens, a spreadsheet was developed that predicts phase lengths in the cycles that follow an oversized ped phase and used to test various combinations of these parameters. The goal was to find settings that would produce a recovery that is quick and balanced, in the sense of avoiding situations in which there is a long queue on one street while the other street gets more green time than needed.

Figure 66 and Figure 67 illustrate expected splits that will follow an oversized pedestrian call at the Second Street intersection for the a.m. and p.m. plan, respectively. The table shows, for each cycle, the lateness of the coord phase's green start; it also shows each phase's "green deficit", which is the expected green need minus the green time received, where green need is calculated from the phase's demand and capacity (critical movement within that phase), the preceding red time, and any green deficit inherited from the previous cycle. Green deficit can be directly translated into overflow queue – a large green deficit means a large overflow queue, and when the green deficit goes to zero, it means that no more overflow queue is expected. These tables assume that there is no ped call during recovery (a pretty safe assumption). One can see that, in both a.m. and p.m. periods, by the end of one cycle after the cycle with the oversize ped call, the coord phase lateness disappears, as do green deficits for Second Street. Rt 16, it turns out, never has a green deficit in either a.m. or p.m., meaning no overflow queuing on the arterial is expected.

cycle counter	Phase	1 Rt 16 (EB/WB)	2 Ped	3 2nd St (NB/SB)	C (s)	Coord phase lateness (s)
	volumes (veh/h)	1885	-	218	-	-
	Saturation rate (veh/h)	5050	-	1400	-	-
	Min green (s)	38	-	15	-	-
	Change Interval (s)	7	-	6	-	-
	Programmed Split (s)	45	12	27	84	-
0	**Cycle with ped call**	-	-	-	-	-
	Actual split (s)	57	36	21	102	18
	End time (s)	45	81	102	-	-
	Needed g (s)	-	-	17	-	-
	Actual g (s)	-	-	15	-	-
	Green deficit (s)	-	-	2	-	-
1	**1st cycle after ped call**	-	-	-	-	-
	Adjusted split (s)	45	1	16	62	-
	Actual split (s)	45	0	16	61	(5)*
	End time (s)	63	63	79	-	-
	Needed g (s)	34	-	10	-	-
	Actual g (s)	38	-	10	-	-
	Green deficit (s)	-4	-	0	-	-
2	**1st cycle after ped call**	-	-	-	-	-
	Adjusted split (s)	45	12	27	84	-
	Actual split (s)	50	0	21	71	-18
	Needed g (s)	5	-	10	-	-
	Actual g (s)	43	-	15	-	-
	Green deficit (s)	-38	-	-	-	-

* Numbers in parentheses are negative numbers

Figure 66- Shortway recovery for AM plan

cycle counter	Phase	1 Rt 16 (EB/WB)	2 Ped	3 2nd St (NB/SB)	C (s)	Coord phase lateness (s)
	volumes (veh/h)	2056	-	321	-	-
	Saturation rate (veh/h)	5050	-	1400	-	-
	Min green (s)	43	-	17	-	-
	Change Interval (s)	7	-	6	-	-
	Programmed Split (s)	50	12	28	90	-
0	**Cycle with ped call**	-	-	-	-	-
	Actual split (s)	53	36	23	109	19
	End time (s)	50	86	109	-	-
	Needed g (s)	-	-	26	-	-
	Actual g (s)	-	-	17	-	-
	Green deficit (s)	-	-	9	-	-
1	**1st cycle after ped call**	-	-	-	-	-
	Adjusted split (s)	50	6	15	71	-
	Actual split (s)	50	0	21	71	(0)*
	End time (s)	69	69	90	-	-
	Needed g (s)	41	-	15	-	-
	Actual g (s)	43	-	15	-	-
	Green deficit (s)	-2	-	-	-	-
2	**1st cycle after ped call**	-	-	-	-	-
	Adjusted split (s)	52	12	32	96	-
	Actual split (s)	56	0	27	83	-7
	Needed g (s)	12	-	17	-	-
	Actual g (s)	49	-	21	-	-
	Green deficit (s)	-37	-	-	-	-

* Numbers in parentheses are negative numbers

Figure 67- Shortway recovery for PM plan

6.12.2 Oversize Ped Phases at the Vale Street Intersection

At Vale Street, oversize ped phases are expected to have almost no impact on operations. There are two oversize ped phases. One is for crossing Rt 16 – the split is 26 s, while the time needed to serve it if called is 28 s. The difference, only 2 s, is too small to make much impact on operations.

The other oversized ped phase is for crossing Vale Street. The proposed split is 15 s while the time needed to serve the crossing if called is 25 s. However, immediately preceding this phase is WBL with a split of 11 s, and this phase is skipped in most cycles because demand for WBL is only 5 vehicles per hour. So in the vast majority of cycles, if there a ped call for this crossing, it will begin 11 s earlier than programmed, and therefore be able to complete its 25 s service before its own split expires. In the rare situation that there is a call for both WBL and the crossing across Vale Street, that crossing will end 10 s late, forcing the Vale Street phase to begin late. A minimum green for Vale Street was chosen that will ensure that it doesn't get a large green deficit in that first cycle, and that the green deficit disappears in the next cycle, while avoiding a green deficit on Route 16.

7 Case Study, Route 114 (Field Test)

7.1 Introduction

The study site (Figure 68) is the portion of state Route 114 located in Danvers, where it is Andover Street and is classified as an arterial, between (and including) intersections with Brooksby Village Dr in the east and I-95 in the west. This segment is 0.66 miles long. Rt. 114 in this segment is undivided and has two lanes per direction. At intersections, wherever left turns are allowed, there are exclusive turn lanes with protected-only phases. The posted speed on this part of Rt.114 is 40 mph. The speed limit for side streets is 25 mph.

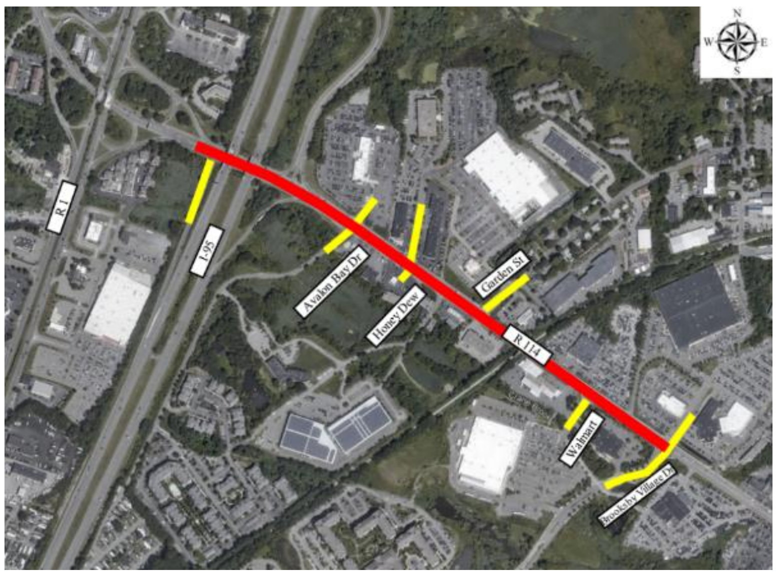


Figure 68- Study segment of Route 114

The study corridor has six signalized and one unsignalized intersection. The five eastern signalized intersections were coordinated with three different plans during weekday daytime, with cycle lengths of 120 seconds in the am peak (6:30-9:30) and midday (9:30-15:30) and 95 seconds in the pm peak (15:30-18:30).

The main objective of this study was to test whether the Safe Waves signal timing approach was effective for speed control. Chapter 5 describes the existing or 'before' signal timing and the Safe Waves signal timing. The 'before' timing prevailed between November 2022 and April 2023. (Prior to Fall 2022, there had been no arterial crossing at Garden Street, and the westbound roadway had a third lane between Honey Dew and the I-95 North on-ramp just beyond Avalon Bay; the crossing at Garden was added and the third lane eliminated as part of another MassDOT safety project that was executed after this project began.) Changes to signal timing for this project were first made on 28th April 2023, but it took several attempts before the final Safe Waves settings were correctly implemented on June 27, 2023.

A secondary objective was to apply and refine the Safe Waves approach to signal timing, drawing lessons that might be used elsewhere. Findings related to this secondary objective are found in Chapter 5.

7.2 Data Collection – INRIX

Data from INRIX, a company that gathers location data from mobile phone apps, was used to measure travel time along the corridor. The 'before' data was for the 3-week period from March 6 to March 24, 2023; the 'after' period was July 3 – July 14, 2023. The 'after' period began a week after the new timing had been implemented, but was limited to two weeks because repaving operations began July 17, 2023, and data from after repaving was complete (November 2023) was not available in time for this study. INRIX data is from weekdays only, and uses 7-9 a.m. as the a.m. peak period, 10 a.m. – 12 noon as the midday period, and 3:45-5:45 pm as the p.m. peak period.

Speed data was also obtained from INRIX. However, it was not used in the study because it is based on the average speed over a segment (intersection-to-intersection) of each vehicle captured, and that average speed typically includes measurements near the intersections where the vehicles are often moving slowly in a queue, depressing the vehicle's average speed. This is believed to be the reason that the fraction of vehicles with high speeds in the INRIX data was systematically lower than what was found with the radar data. (While data aggregating vehicle maximum speeds by segment was requested, INRIX said that was not available.)

7.3 Data Collection – Radar and Camera

Side-mounted radar was used to measure speed, headways, and to count vehicles. It was supplemented by cameras aimed at the traffic signals for getting information on signal state.

Radar and video data were collected on weekdays (Monday to Thursday), with 24 hours collected at each of four intersections, Brooksby Village Dr, Garden St, Honey Dew, and Avalon Bay Dr. 'Before' data collection was conducted in late April, 2024 and 'after' data collection was conducted in early November 2024.

At each intersection, one radar device and one camera were used for each direction. Each radar device can count vehicles in two channels, one for each lane; they were mounted on existing poles, as close as possible to the roadway edge, and oriented so as to capture approaching vehicles (i.e., vehicle fronts) at a 45-degree angle at, or just downstream of, the stop line. Where possible, the radar devices were mounted at the near side of intersection, downstream of the stop line. However, for some intersection approaches, there were no near-side pole beyond the stop line, and so the radar was mounted on a far side pole. Vehicles turning in from the side street were not detected with the near-side mount, but are detected with a far-side mount.

Cameras were aimed at the arterial traffic signal, and thus looking at rear of vehicles (vehicles are moving away from camera), in order to record the state of the signal so that, for measuring speeding opportunities, vehicles passing during the red period, which would be coming from side streets, could be filtered out.

The figures below show how the field devices were mounted at each intersection. Figure 69 is for the intersection with Brooksby Village Drive, Figure 70 for the intersection with Garden Street, Figure 71 for the intersection with the Honey Dew driveway, and Figure 72 for the intersection with Brooksby Village Drive. Radar counter locations and orientation are shown with large polygons and solid arrows in yellow, while camera location and orientation shown with smaller polygons and dashed arrows in red. Of the eight arterial approaches at these four intersections, three had a near-side mount for the radar, capturing vehicles as they cross the stop line; however, a far-side mount, which captures vehicles as they depart the intersection, was used at the following approaches:

- @ Brooksby Village, westbound
- @ Garden, westbound
- @ Honey Dew, westbound
- @ Avalon Bay, eastbound and westbound

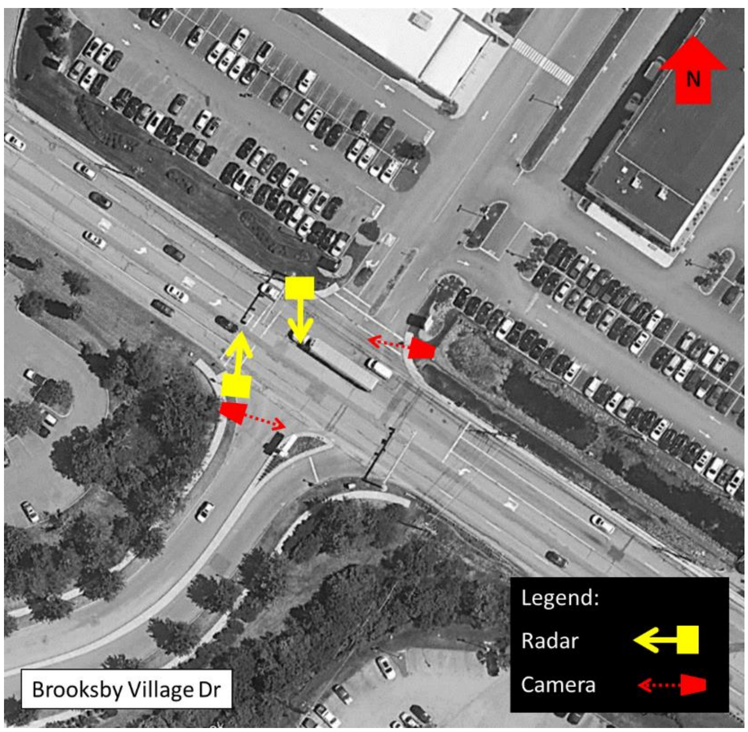


Figure 69- Devices' position at Brooksby Village Dr.

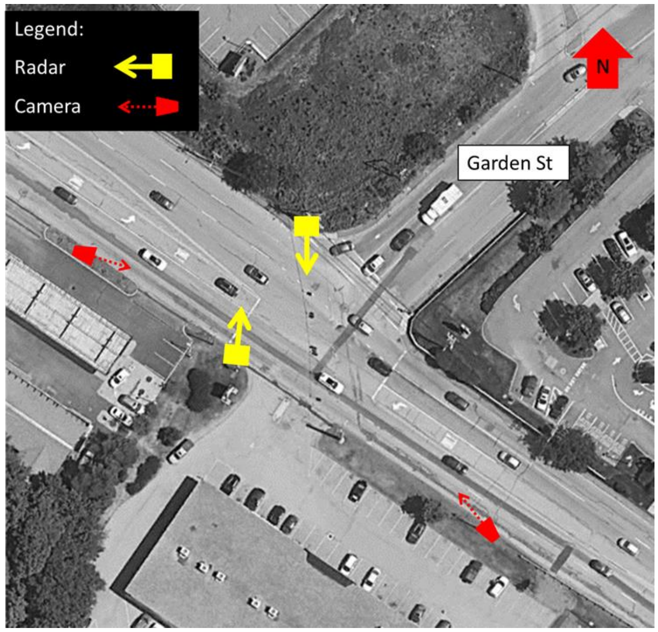


Figure 70- Devices' position at Garden St.

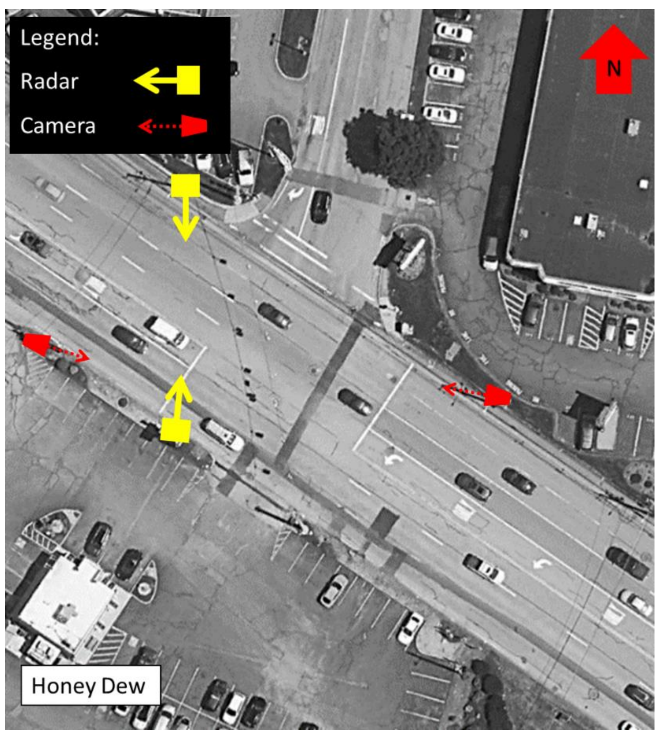


Figure 71- Devices' position at Honew Dew

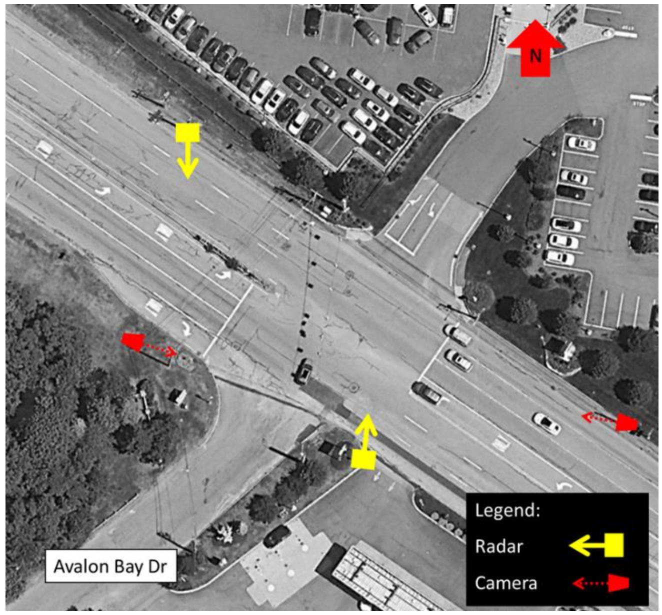


Figure 72- Devices' position at Avalon Bay Dr.

7.4 Data Processing

7.4.1 Extracting Signal State

The purpose of the camera data was to determine signal state. Unfortunately, a signal head detection algorithm couldn't be used, because the video quality was poor, and the algorithm did not work. Instead, the following image processing method was conducted.

Step 1: For each approach, there were two signal heads for thru traffic. The position of all three lenses (green, yellow and red) were determined for each signal head. Both signal heads were used because sometimes one of them was blocked by a truck.

Step 2: A square of 3 by 3 pixels was determined for each lens. The light intensity of those pixels was read every second for each intersection-direction between 6:00am and 7:00pm.

Step 3: Signal state (on or off) was determined for each lens, second by second.

Step 4: Results were manually checked and corrected as needed.

7.4.2 Processing the Radar Data

Radar counters deliver a Microsoft Excel file which consists of a record for each vehicle indicating date, time, speed, and headway for all the recorded vehicles. All the information about both channels (i.e., each lane) is provided in the same file.

When fusing the camera and radar data in order to filter out observations during the red period, a challenge was synchronizing the data streams due to drift. The camera has a built in modem and always is connected to the internet, however, the radar device matches its internal clock to the internet when connected to a computer, but the radar devices were not connected to the internet during the field data collection period, and so their clocks can drift. By manually watching the video and comparing with notable moments in the radar data (a long gap followed by a platoon of closely spaced vehicles), corrections that ranged from 0 to 18 seconds were made to overcome drift.

7.5 Data Challenges and Lessons Learned

The first challenge in this project was discovering that some of the traffic signal controllers had errors in their settings, and so their signals were out of sync or otherwise not working as planned. This issue was discovered when the Safe Waves timing plan was first implemented, when field observations showed the signals were not following the specified offsets. It took several visits by technicians until a consulting engineer hired by MassDOT found and fixed several errors in the controller settings.

Because of this experience, videos from the 'before' period were reviewed to see whether signals were in sync then. It was discovered that the Garden Street intersection had been running free all day rather than operating in coordination as per the signal timing plan.

A second challenge had to with the radar data. The original research plan includes making field counts of the number of opportunities. This required detecting vehicles in the correct lane so that the within-lane headway would be accurate. However, it was found the radar counters often assigned vehicles to the wrong lane, evidenced by count totals that showed far more vehicles using one lane than the other while both the video data and field observations indicated a nearly-equal use of the two thru lanes. At the same time, total vehicle counts (summed over the two lanes) matched the video data well, and so the radar data was deemed valid except for lane assignment and therefore headway. That made it impossible to make field measurements of speeding opportunities. However, radar data was still valuable for measuring vehicle speeds and total vehicle counts.

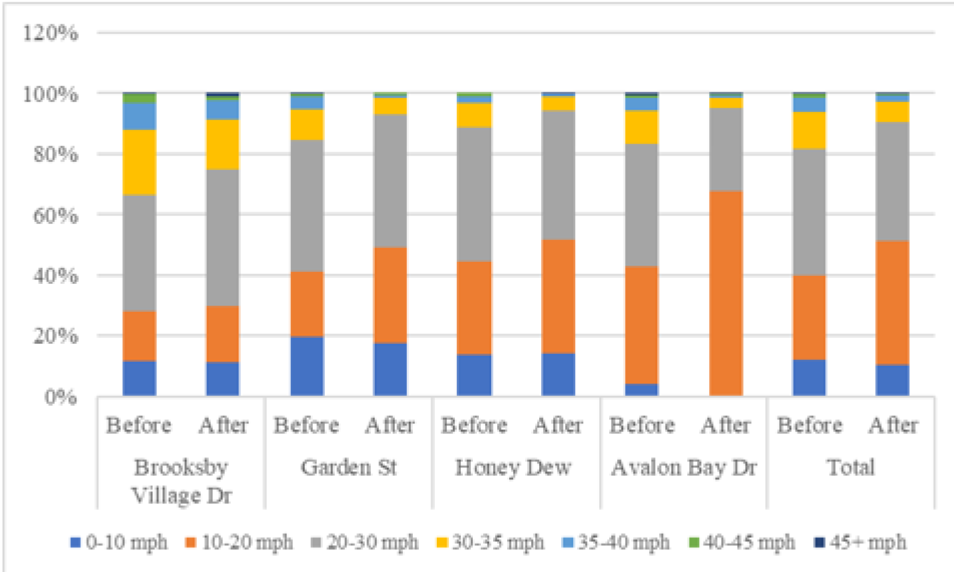
Finally, as mentioned earlier, speed data from INRIX appears to be biased downward because while each vehicle may report its location several times per second, and therefore several times within each intersection-to-intersection segment, those records (which are converted to speed) are first averaged over the segment for each vehicle, and it is those averages that are then aggregated to give speed distribution data. Because the segments are short and vehicles are often advancing slowly due to queuing at the intersections, averaging measurements over a segment in this way is thought to bias speeds downward. Because good speed data was available from the radar measurements, the INRIX speed data was not used.

7.6 Results – Changes in Speed Distribution

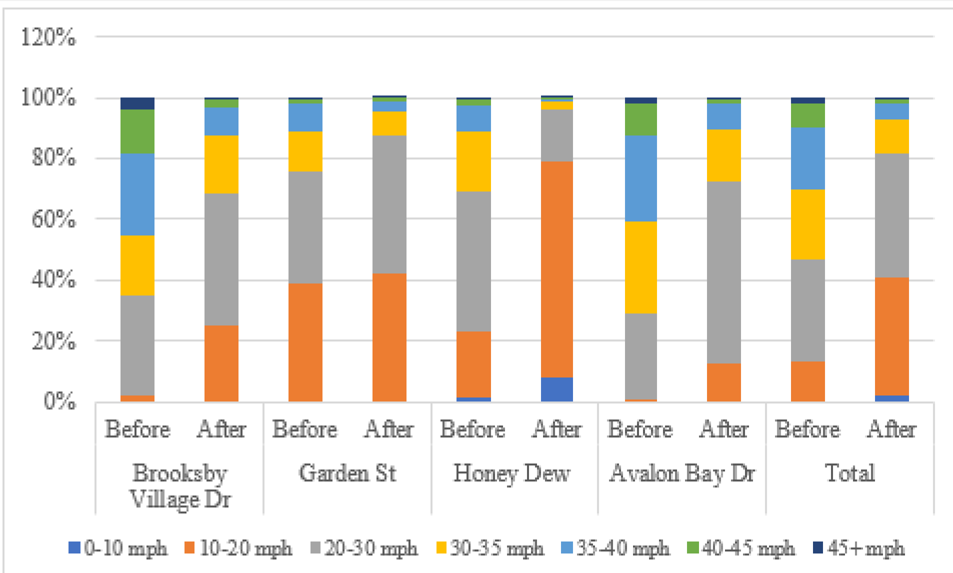
As explained earlier, the speed data for was collected using radar counters at four intersections for each arterial direction. Where the radar device was mounted far-side, the data was filtered by excluding vehicles that passed while the signal was red, excluding the first two seconds of red, when thru traffic might still be clearing.

Figure 73 shows the before vs. after speed distribution at each intersection and overall, by direction, for the a.m. peak (7-9 a.m.) One can see, at each intersection and in both directions, a marked decline in the proportion of vehicles in the higher speed bins. The highest speeds observed (greater than 45 mph) were in the westbound direction, at the Brooksby Village and Avalon Bay intersections; in the 'after' case, those high speeds virtually disappear.

Figure 74 shows before-after speed distributions for midday (10 a.m. - 2 p.m.), and Figure 75 for the p.m. peak (3:45-5:45 p.m.). In addition to a systematic reduction in speed, these graphs also show that, in spite of a posted speed limit of 40 mph, the 85th percentile speed is below 35 mph for all except a few intersection – direction – period combinations.

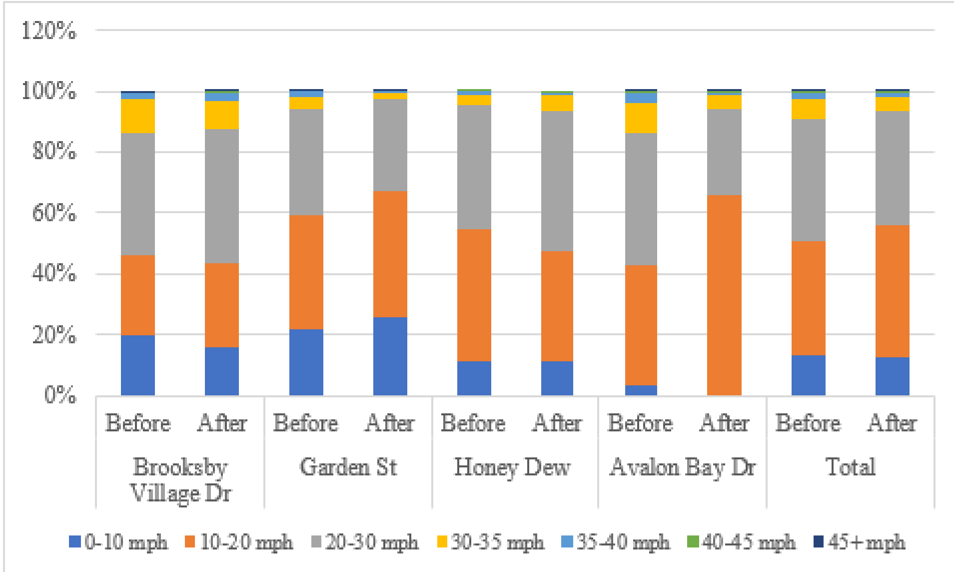


a) Eastbound direction

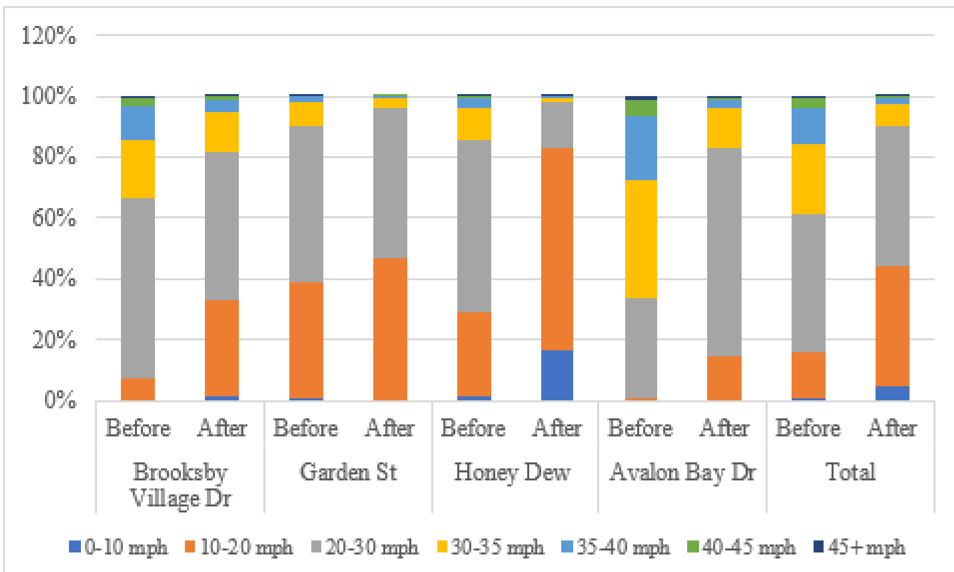


b) Westbound direction

Figure 73- A.m. peak speed distribution

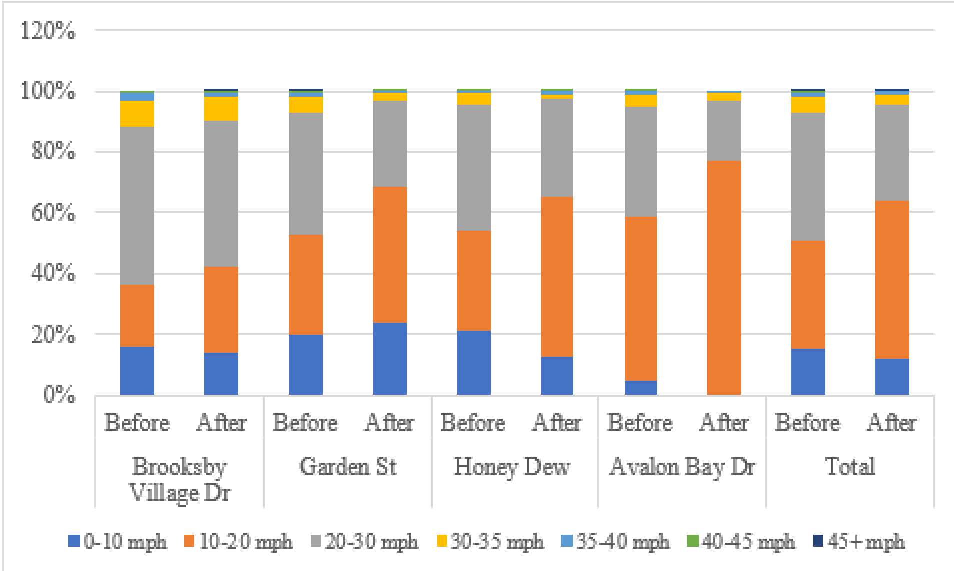


a) Eastbound direction

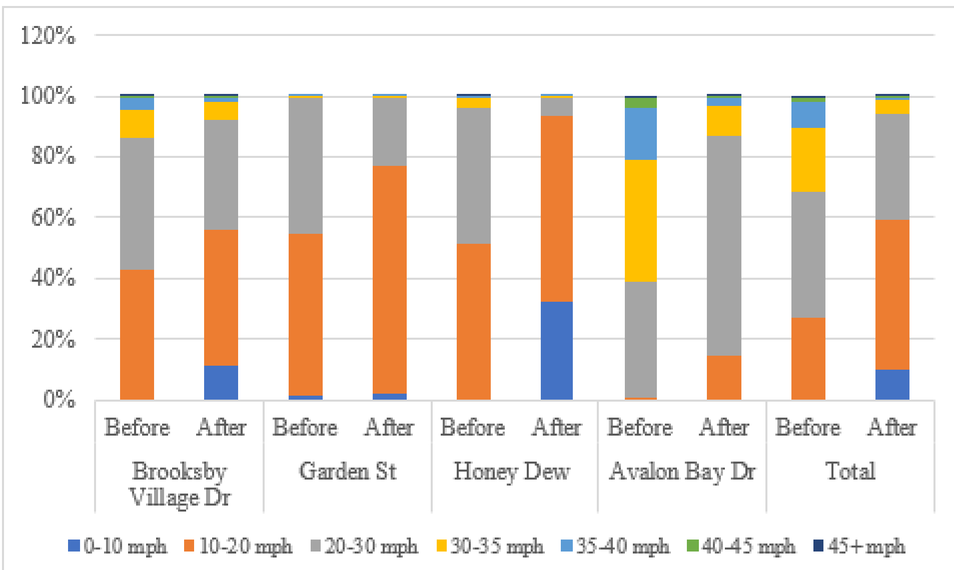


b) Westbound direction

Figure 74- Midday speed distribution



a) Eastbound direction



b) Westbound direction

Figure 75- P.m. peak speed distribution

To better show how speeding behavior changed, Figure 76 shows the proportion of vehicles traveling faster than 35, 40, and 45 mph by period, aggregating over intersections and directions. The same information is presented numerically in Figure 77, with detail by period, and Table 7, which aggregates over the three periods. One can see that over the day, the proportion of vehicles traveling faster than 40 mph, which is the speed limit, fell by 79%; roughly the same reduction is seen for vehicles traveling faster than 35 mph, or faster than 45 mph.

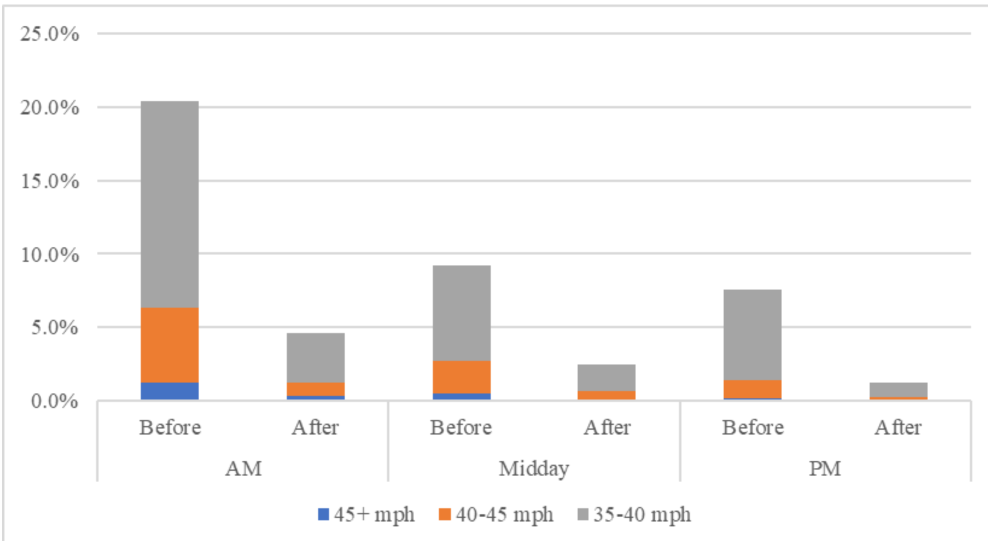


Figure 76- Proportion of vehicles with high speed

Time of day	Alternative	45+ mph	40-45 mph	35-40 mph
AM	Before	1.2%	5.1%	14.1%
	After	0.3%	0.9%	3.3%
Midday	Before	0.5%	2.2%	6.6%
	After	0.1%	0.5%	1.8%
PM	Before	0.2%	1.2%	6.2%
	After	0.0%	0.2%	0.9%

Figure 77- Proportion of vehicles with high speed

Table 7- Reduction in number of speeding

Speed	Reduction
35+ mph	78%
40+ mph	79%
45+ mph	74%

7.7 Results – Changes in Travel Time

Figure 78 compares before-after travel time using INRIX data, measured from between the intersections at Brooksby Village Drive and I-95 ramp, a stretch in which vehicle pass through five intersections. One can see that travel time increased during the AM and Midday periods, while in the PM, when travel time and congestion is the greatest, there was no detectable change in travel time.

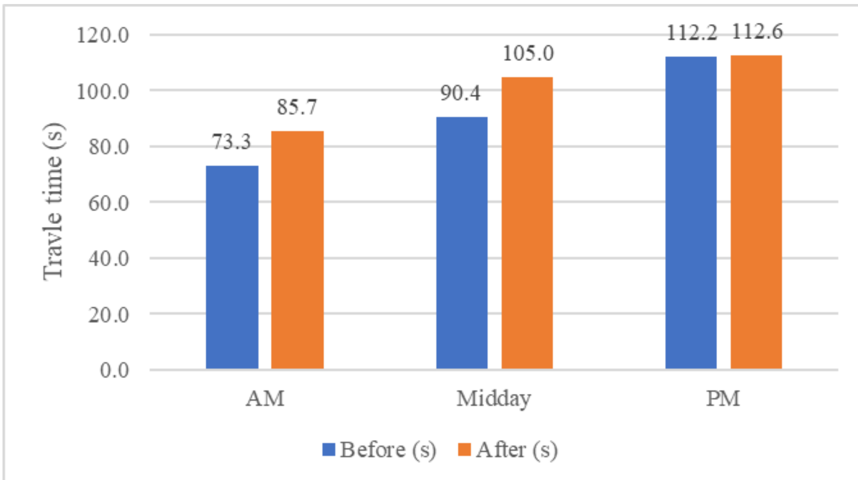


Figure 78- Travel time (Rt.114)

Averaged over the day, the increase in travel time is only 1.8 s per intersection. Figure 79 shows numerical changes in travel time and in average speed, which decreased by 4 mph in the a.m. and midday periods, while there was no detectible change in the p.m. period.

Time of day	Alternative	Travel time (s)	Speed (mph)
AM	Before	73.3	32.4
	After	85.7	27.7
Midday	Before	90.4	26.3
	After	105	22.6
PM	Before	112.2	21.2
	After	112.6	21.1

Figure 79- Travel time and Speed (Rt.114)

7.8 Results – Changes in Speeding Opportunities

As mentioned earlier in this chapter, the radar data proved to be unreliable for measuring number of speeding opportunities because it often assigned vehicles to the wrong lane. Speeding opportunities were therefore measured using estimates from the Safe Waves Analysis Tool (SWAT), aggregating estimates over all six intersections in the corridor. These results are shown in Figure 80.

		Number of SO's	Percentage of SO's	% Change in number of SO's
AM	Before	769	33%	-61%
	After	303	13%	
Midday	Before	459	18%	-54%
	After	211	8%	
PM	Before	353	12%	-28%
	After	258	9%	
Overall	Before	1581	24%	-57%
	After	772	10%	

Figure 80- Changes in speeding opportunities, Rt.114

7.9 Results- Changes in Pedestrian Delay

Two intersections, those at Brooksby Village Dr and at Garden St, have arterial crossings. In both the before and after cases, pedestrian crossings are on demand. Table 8 shows the pedestrian delay by period (it’s the same at both intersections); averaging over all periods, pedestrian delay fell by 18.5 s (33%).

Table 8- Pedestrian delay (s) (Rt.114)

Time of day	Before	After	Change
AM	60.0	33.0	-27.0
Midday	60.0	37.0	-23.0
PM	47.5	42.0	-5.5
Average	55.8	37.3	-18.5

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8 Case Study, Route 16 (Simulation Test)

8.1 Introduction

The study corridor, shown in Figure 81, is 1.12 miles of Route 16 located in Everett and Chelsea, where it is known as Revere Beach Parkway, and is classified as an arterial. From west to east, it starts at the intersection of Lewis St in Everett, MA and continues to the intersection of Washington Ave in Chelsea, MA. The study corridor has nine signalized intersections, numbered 1-9 beginning in the west (at Lewis Street). There is only one unsignalized intersection where the median is broken, Boston Street, between intersections 6 and 7.

Route 16 in this stretch is physically divided and has three thru lanes per direction. Exclusive left turn lanes are provided wherever left turns are allowed. The posted speed for Route 16 in the study segment is 35 mph. The speed limit for the cross streets is 25 mph.

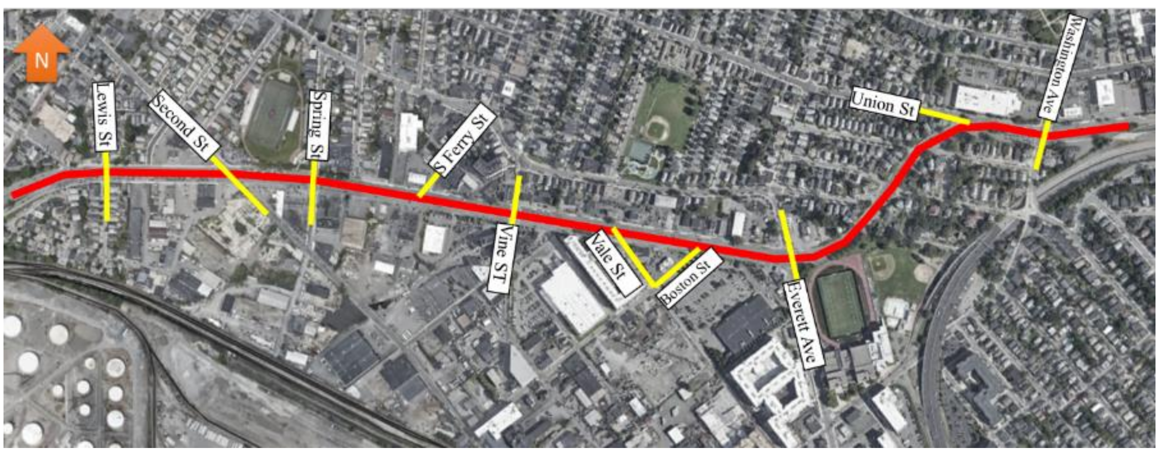


Figure 81- Study segment of Route 16

In the existing timing plan, there are two coordination zones operating in the AM (7:00-10:00 AM) and PM (3:00-7:00 PM) periods only. Intersections 2-5 (Second St, Spring St, South Ferry St, and Vine St) are coordinated with cycle lengths of 110 s AM / 150 s PM. Intersections 9 and 10 (Union St and Washington Ave) are coordinated with a 120 s cycle both AM and PM periods. These intersections run free at other times of the day and on weekends. Intersections 1 (Lewis St), 6 (Vale St), and 8 (Everett Ave) run free at all times. Note that "running free" in this corridor does not mean fully actuated, but semi-actuated, since there are no mainline detectors; the arterial through phases run for a fixed phase length.

In the proposed timing plan, there are also two coordination zones; however, none of the intersections runs free. Intersections 1-7 (Lewis St to Everett Ave) run with a cycle of 84 s in the AM and 90 s in the PM. The second zone consists of intersections 8 and 9 (Union St and Washington Ave) and run with a 72 s cycle both AM and PM periods. Detailed information about the proposed signal timing for Rt.16 is provided in chapter 6.

From several site visits it was determined that the only intersection running with the planned coordinated timing is #3, Spring St. Other intersections were running free (semi-actuated) all day.

Objective of this test was to check the impact of new design on reducing the number of speeding opportunities, using computer simulation.

8.2 Simulation Model (Rt.16)

A simulation model was developed in a microsimulation software named VISSIM. For signal timing an internal module in VISSIM named Ring Barrier Control (RBC) was used to apply the different signal timing plans.

As mentioned earlier, the controllers are not working with their coordination plan. Therefore, three simulation models were developed: existing actual, existing planned, and proposed plan.

Every alternative was run 10 times for one hour after a 15-minute warm-up period. Results are the average of those 10 runs.

8.3 Results - Speeding Opportunities

PTV VISSIM provides a variety of performance measures as output including delay, travel time and number of stops, etc. One of these outputs that can be used to measure number of speeding opportunity is Discharge Rate which will provide discharge time of every vehicle at the stop line (position of signal head) for each travel lane. The output data is stored in series of text files (*.dis) and every file represents the output for one signal head. This data is very helpful to be processed in order to count the number of speeding opportunities at each intersection and direction.

Using Python, the output data was processed and for each signal head the number of speeding opportunities and the number of total vehicles passed the signal head were counted.

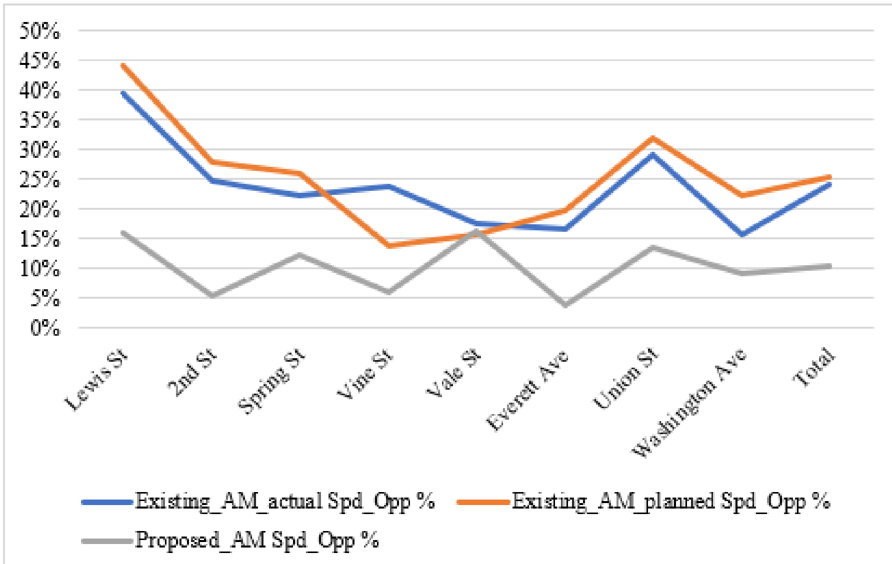
Table 10 shows the percentage of speeding opportunities in the whole network. The percentage of speeding opportunities decreased by 59% in the morning and 56% in the afternoon.

Table 9- % Speeding opportunities in network

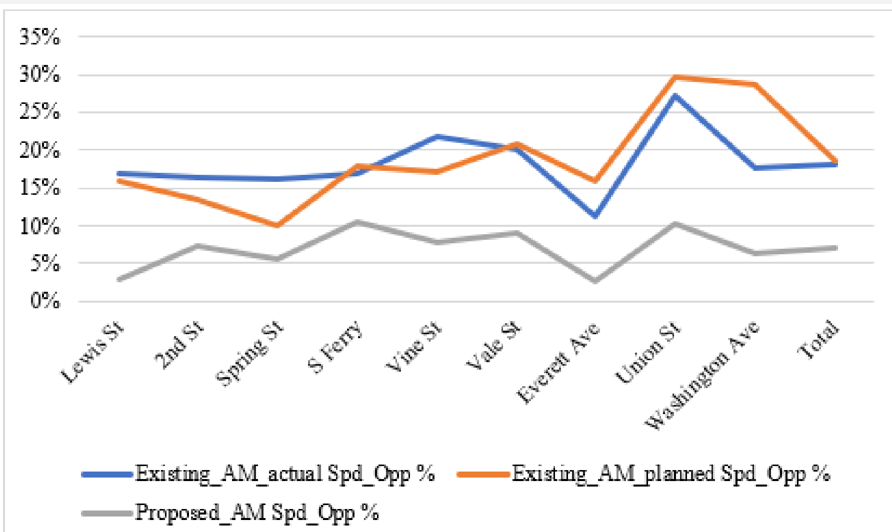
Period of day	Existing actual	Existing planned	Proposed	% Change from Planned
AM	21%	22%	9%	59%
PM	16%	16%	7%	56%

Figure 82 and Figure 83 show the percentage of speeding opportunities for all the intersections and networkwide. Results are quite similar for the existing actual and existing planned cases, while speeding opportunities are far lower for the proposed timing plan.

An interesting result is that the proportion of speeding opportunities is always higher when there is less traffic. In the AM, the eastbound direction has significantly less traffic and a greater proportion of that traffic is speeding opportunities.

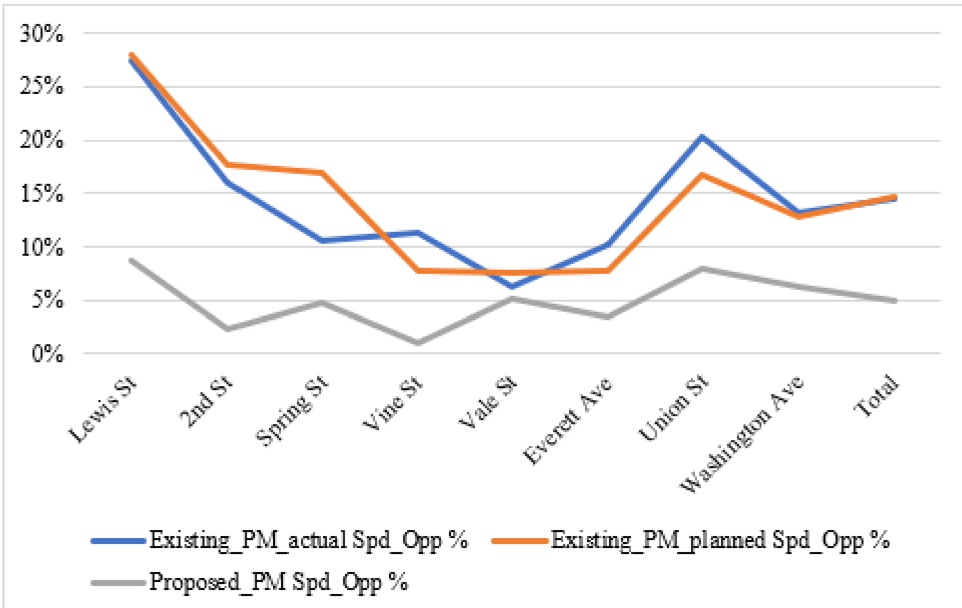


a) Eastbound direction

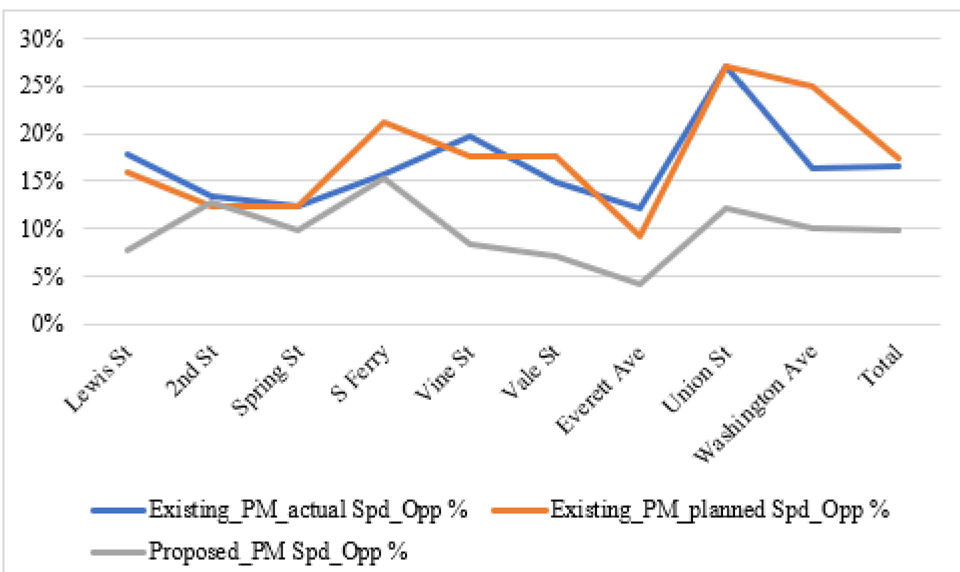


b) Westbound direction

Figure 82- Percentage of vehicles that are speeding opportunities (AM)



a) Eastbound direction



b) Westbound direction

Figure 83- Percentage of vehicles that are speeding opportunities (PM)

8.4 Results - Delay

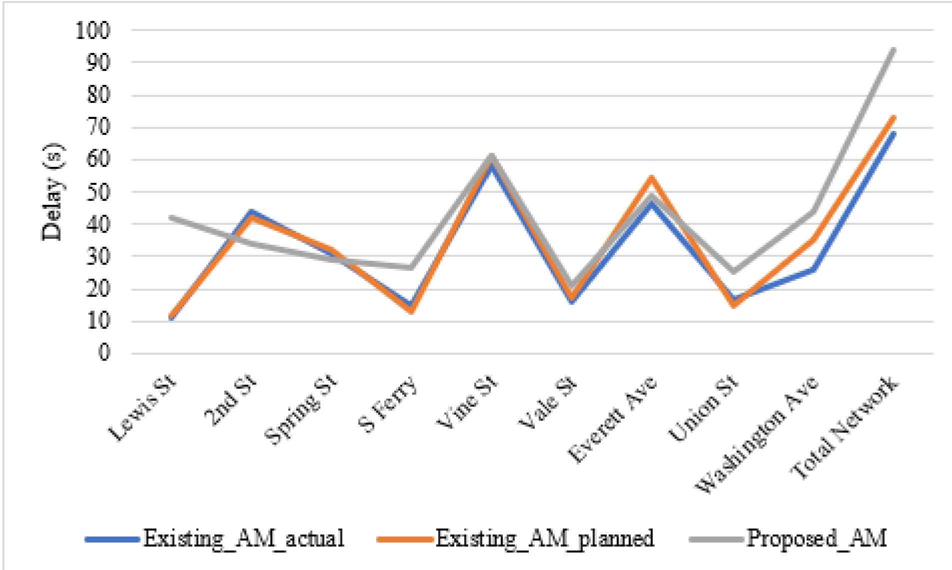
Table 10 shows how network delay (average delay per vehicle) changed by period. Values shown are per intersection, knowing that the average vehicle passes through 5.0 intersections in the AM peak and 5.3 intersections in the PM peak. One can see that, averaging over the two periods, delay per intersection increased by 2.7 s.

Table 10- Average delay per vehicle and per intersection (s) (Rt.16)

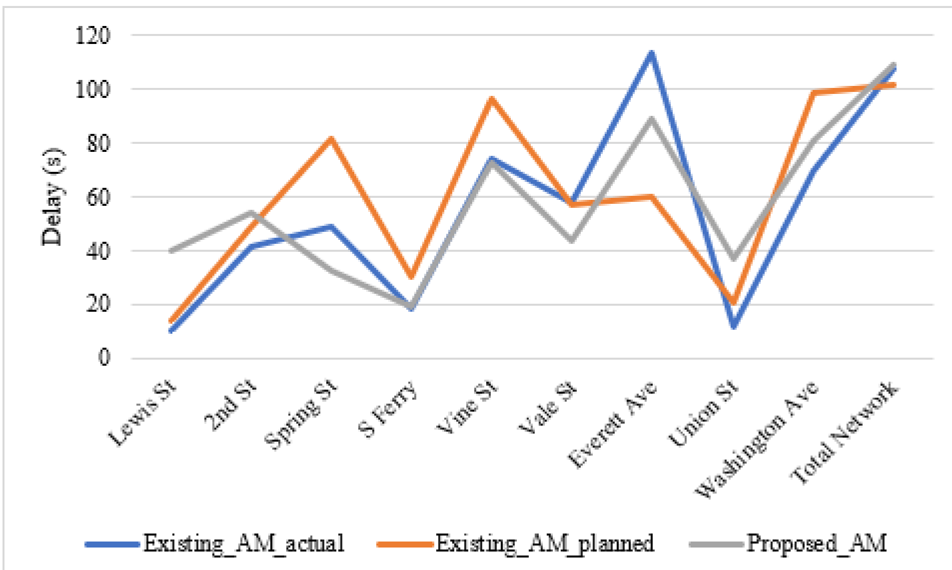
Period of day	Existing actual	Existing planned	Proposed	Change from planned
AM	13.6	14.6	18.8	4.2
PM	20.2	19.2	20.6	1.4
Average	17.1	17.1	19.8	2.7

Figure 84 shows the delay results for every intersection and networkwide. Similar to graphs for proportion of speeding opportunity, the orange and blue lines represent the delay for existing planned and existing actual and gray line represents the delay for proposed plan. Part a) of the figure shows the AM period delay and part b) shows the PM period delay.

The delay measured for all the intersections and throughout the network. The relationship between changes in delay is like speeding opportunity and while the network is more crowded the less change in delay happens. Therefore, in PM period which is more crowded delay increased by 2 seconds in the whole network.



a) AM delay



b) PM delay

Figure 84- Delay results, Rt.16

8.5 Results - Average Stops Per Vehicle

One of the concerns for applying the Safe Waves signal timing techniques to reduce the number of speeding opportunities was that the number of stops may increase, causing driver aggravation and increased fuel consumption.

Table 11 illustrates the average number of stop per vehicles in all three alternatives. In this table it can be seen that on average for both periods the stop per vehicles per intersections increased by less than 0.2 stops per vehicle per intersection which is a small change in a section that is slightly more than 1 mile.

Table 11- Average stops per vehicle per intersection (Rt.16)

Time of day	Existing actual	Existing planned	Proposed	Change form planned
AM	0.42	0.42	0.61	0.19
PM	0.67	0.54	0.74	0.20
Average	0.55	0.48	0.68	0.20

8.6 Results - Changes in Pedestrian Delay

At all locations with an arterial crossing, the pedestrian phase runs exclusively. In the proposed signal timing by using Safe Waves techniques, pedestrians are treated with different methods based on the geometry, demand and safety considerations. At Washington Ave at the eastern edge of the study segment, a multistage crossing is provided, because of the wide median and high number of lanes that a pedestrian should travel to get to another side of the Rt.16. This method changed the average pedestrian delay from 140 to 47 seconds (66% reduction). At two intersections of Second St and Vale St, because of low pedestrian demand and high turning speed due to the skew intersection geometry, an exclusive pedestrian phase was needed, which would typically require very long cycle lengths. Hence, undersized phase methods were used to achieve shorter cycle length, with pedestrian phases on demand (as they are currently). At other intersections, pedestrian phases are concurrent and on recall.

Table 12 shows the average pedestrian delay in network for both before and after changes and in AM and PM periods. It can be seen that delay fell by 93.2 s at the intersection with Washington Ave, and by 27.7 s at other intersections on average.

Figure 85 illustrates the intersection-by-intersection delay for pedestrians for both AM and PM periods. It can be seen in this figure that by shorter cycle length delay in all intersections decreased.

Table 12- Pedestrian delay (s) in network (Rt.16)

Location	Before	After	Change in delay
Washington Ave	139.9	46.7	-93.2
All other intersections	64.4	36.6	-27.7

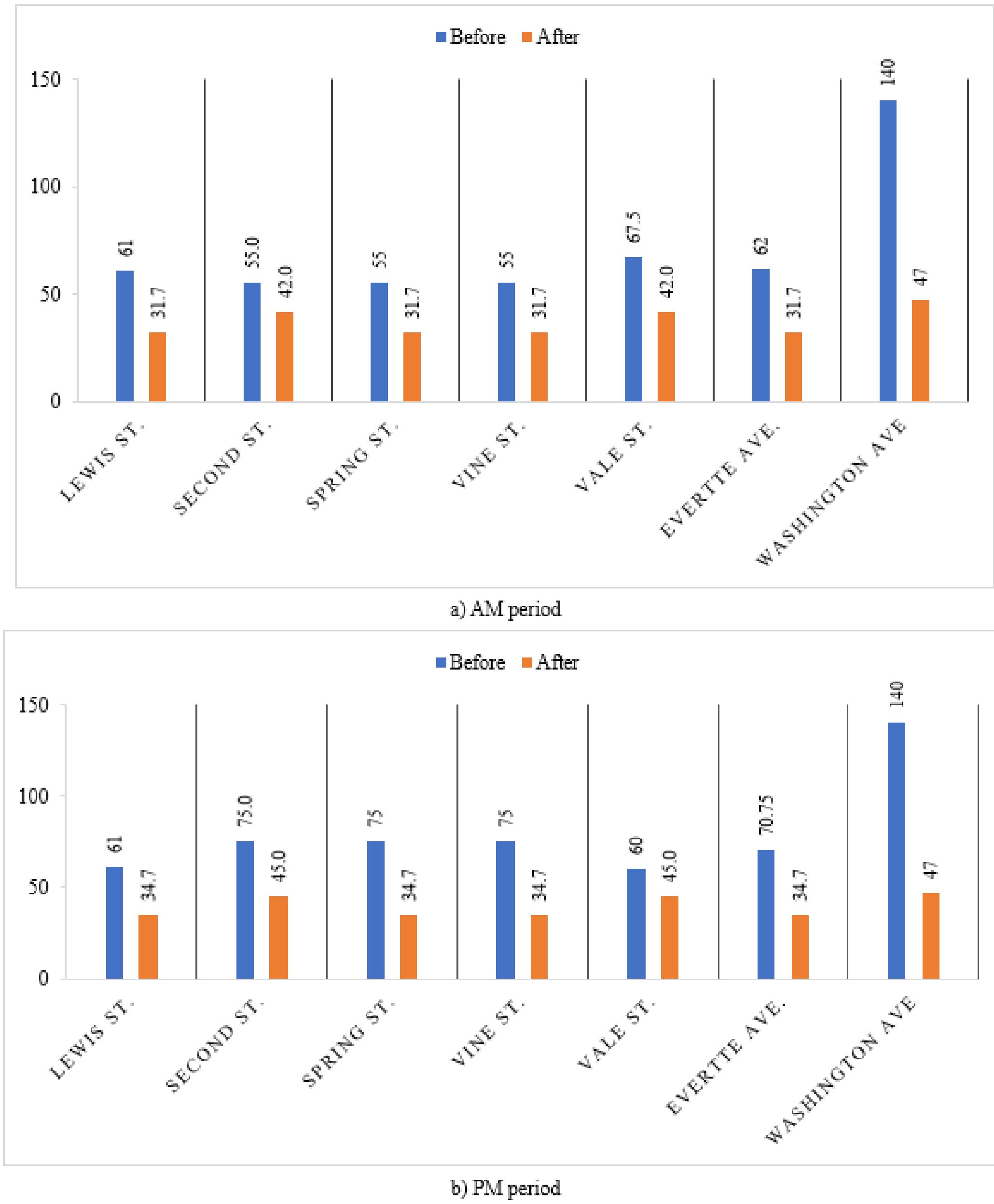


Figure 85- Pedestrian delay (s) (Rt.16)

9 Conclusions

Speed control has always been a challenge on multilane arterials because common speed control like horizontal and vertical deflection which are very effective on local streets is not applicable or suitable. The idea of using traffic signals as an effective tool to reduce the number of speeding opportunities and speeding to make it safer for all the road users especially vulnerable road users while still providing good service for vehicles in the corridors was the fundamental idea that led to conducting this research.

This study had three main tasks which are developing a comprehensive guide for engineers to use the Safe Wave techniques in traffic signal timing, conducting the Safe Waves techniques in the field to examine the impact of them on reducing the speeding opportunities and speeding, and developing the tool to facilitate the measuring of speeding opportunities in different design alternatives for engineers.

One of the tasks in this study was to test the safe waves signal timing approach on two arterials to examine the impact of it on reduction in number of speeding opportunities and speeding. Route 114 in Danvers, MA including 6 signalized intersections and Route 16 in Everett and Chelsea, MA including 9 signalized intersections were chosen to implement the tests to collect data before and after the signal changes. The field test was completed in Route 114 and the result from collected data with radar counters showed a 75% reduction in number of speeding vehicles. Also, travel time was used from INRIX data in the segment between first and last intersections, which showed the delay on average increased by only 1.8 seconds per intersection. At the same time pedestrian delay decreased by 18.5 s at the two intersections where an arterial crossing is provided.

For Route 16, which was studied using simulation, Safe Waves signal timing reduced the number of speeding opportunities by than 50% in both AM and PM periods. Vehicle delay per intersection increased a little, by 4.2 s in the AM and by 1.4 s in the PM. Pedestrian delay fell by 93 s (from 140 to 47 s) at one intersection with a two-stage crossing; at the other intersections, pedestrian delay fell by an average of 28 s.

The second objective was to develop a guideline for Safe Waves signal timing, which is provided as the second chapter of this report. Those guidelines draw from the experience of doing Safe Waves signal timing for this project's case studies as well as from earlier studies.

Current intersection analysis software is not able to directly measure the number of speeding opportunities as a performance measure to compare different alternatives. The third task in this study was to develop a tool to make it possible to measure the number of speeding opportunities through a corridor for alternative timing plans, called the Safe Waves Analysis Tool (SWAT), a web-based app. Users prepare a Microsoft Excel input file filled with signal timing, geometric, and traffic flow parameters; it produces a progression diagram as a visualization tool and the table of results including speeding opportunities, delay, and arterial travel time to facilitate comparing different alternatives after design. It uses deterministic simulation logic.

To validate the Safe Waves signal timing designs more field tests are needed in the future. Five arterials in Boston and its suburbs have been studied and the simulation models showed that using safe wave signal timing techniques help reduce the number of speeding opportunities significantly. However, the field test only implemented and examined Route 114 in Danvers, MA which showed promising results.

Another need for further study is continuing development of SWAT. Currently, SWAT receives the input file for one direction at the same time and provides the output for that direction. It would be more convenient for engineers to see both directions results at the same time to make sure the progression and number of speeding opportunities are at optimum level for both directions. Also, the conducting of more filed tests will be valuable for calibration of results provided by SWAT. At this time, SWAT is using a linear probability model to calculate the probability of being speeding opportunity which is theoretical and needs more empirical data to be calibrated to provide more accurate results.

It is mentioned in the guide for Safe Waves signal timing techniques that short cycle length is effective for reducing speeding opportunities. Undersized phases that don’t accommodate pedestrian needs while demand is low for parallel of side streets could be helpful technique to achieve shorter cycles in coordination zones. This method needs more study in order to provide a guideline for engineers to use it in their design.

10 References

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