Application of Transit AVL/APC Data for Network Wide Monitoring of the Performance of Signalized Intersections

by

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AUTHOR'S DECLARATION

I hereby declare that I am the sole author of this thesis. This is a true copy of the thesis, including any required final revisions, as accepted by my examiners.

I understand that my thesis may be made electronically available to the public.

Abstract

The quality of service in urban transportation networks is determined mainly by the performance of the intersections. In particular, signalized intersections play a significant role in regulating the traffic in urban transportation networks. As a result, it is essential for transportation authorities to have a system, which can locate poorly operating intersections in the network and rank them for potential improvements.

In practice, intersection performance is typically evaluated through the use of models such as HCS (Highway Capacity Software) or Synchro. These models estimate measures of performance (e.g. average vehicle delay, queue length, or level of service) on the basis of determinist and/or stochastic queueing theory.

Another approach is to directly estimate intersection performance on the basis of delays experienced by vehicles. One source for such data is public transit bus fleets which are equipped with automatic vehicle location (AVL) systems and automatic passenger counting (APC) systems. These systems use GPS to record where and when a bus stops and the duration of the stop.

The purpose of this research was to compare the intersection performance measures produced by Synchro and those estimated from archived AVL and APC data.

An empirical evaluation was conducted using 28 intersections in the Region of Waterloo. Average delay and queue length were estimated using Synchro and estimated from archived AVL/APC data. The results show that the estimation of mean delay from the two methods are highly correlated. The estimation of queue length show larger differences, and in general, Synchro underestimated the queue length when compared to the AVL/APC data.

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Dedication

To my father, because without whom, I would not be able to write this thesis.

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Chapter 1
Introduction

1.1 Background

The quality of service in urban transportation networks is determined mainly by the performance of the intersections. In particular, signalized intersections play a significant role in regulating the traffic in urban transportation networks. A poorly performing signalized intersection adversely affects the traffic conditions beyond its proximities, which may eventually disrupt overall traffic conditions in the network. Moreover, intersections with improved traffic conditions such as reduced delay, increased travel speed, and alleviated traffic congestion are preferred for commuters. As a result, it is essential for transportation authorities to have a system, which can locate such poorly operating intersections in the network and rank them for potential improvements ranging from adjustment of signalization parameters to modification of intersection geometry.

The performance of signalized intersections is generally determined through evaluation of well-established performance measures of effectiveness such as delay and queue length. Measures of effectiveness play an important role, as they are measures of not only the level of service that is offered to the drivers but also the fuel consumption and air pollution linked to traffic operations.

Control delay at a signalized intersection constitutes the larger part of the travel time on an arterial link. It is the component of total delay that accounts for traffic signal operation at a signalized intersection. It represents the difference between the travel time of a road segment without an intersection, and the travel time of the same road segment with a traffic signal.

There are different terms used in the literature to describe queues at signalized intersections. These terms include maximum queue length and queue extents (queue reach). Queue length is the distance from the stop line at an intersection to the rear of the queued vehicles at the end of the red interval. Whereas queue extent is the distance from the stop line at an intersection to the rear of the queued vehicles at the time when the queue dissipates. Typically, queue extent is larger than queue length.

Control delay and queue length are key measures in evaluating intersection performance. However, direct measurement of delay and queue length at every intersection in the network is impractical due to the time consuming and labor-intensive field data collection and analysis procedures.

Alternatively, delay and queue length at signalized intersections can be estimated using analytical models such as Highway Capacity Manual (Transportation Research Broard, 2000). Synchro (Trafficware, 2011) is a HCM based computer package for intersection design and performance evaluation widely used by traffic engineers and transportation planners in the industry. Synchro applies HCM analytical models (with some adjustments) to estimate the delay and queue length at signalized intersections. Intersection performance evaluation using Synchro still requires some field data such as traffic volumes for all the movements and signalization parameters. Transportation agencies often conduct periodic traffic count surveys and keep records of the signal timing schemes for the intersections in the network, which can serve as inputs for intersection performance evaluation using Synchro. However, depending on the timing of the data collection and the performance evaluation analysis, traffic count data provided by the agencies might be outdated. In addition, detailed signalization data for intersections with actuated or adaptive signal controls are seldom available. Synchro provides a feasible solution for large-scale network wide intersection performance evaluation; nonetheless, the reliability of the analysis heavily depends on the accuracy of the input data.

In recent years, availability of new data collection technologies has provided the opportunity to estimate the intersection performance measures directly from autonomously collected empirical data. Examples of such technologies are Automatic Vehicle Location (AVL) and Automatic Passenger Count (APC) data collection systems. AVL/APC data are collected autonomously and continuously by transit vehicles and have several applications in transit planning and operations. A recent study by Hellinga et al. (2011) demonstrated potential application of AVL/APC data for the estimation of the delay and queue length at signalized intersections with far-sided bus stops. The methodology applies archived AVL/APC data to evaluate the performance of signalized intersections over an extended period. Therefore, it eliminates the assumptions, approximations, and additional data collection efforts involved in the majority of analytical models such as Synchro.

Although different traffic models and technologies that estimate intersection performance measures have been developed, their definition of the measures and validity of the results varies. Some approaches are theoretical and based on simulating intersections while others are based on real life observations. The objective of this study is not to develop another method to estimate these performance measures but to understand how they are estimated by Synchro and AVL/APC data. This research demonstrates the feasibility of using AVL/APC data and Synchro for large-scale performance evaluation of intersections in urban transportation networks. The analysis methods, calibration effort and data requirements are compared and the accuracy of the estimated delay and queue length by Synchro is evaluated by comparing the results with corresponding values derived using AVL/APC data based on the methodology proposed by Hellinga et al. (2011).

1.2 Motivation

Software packages have made it easier for traffic engineers to perform complex intersection performance analysis. Synchro is a software package that is able to perform macroscopic intersection analysis and optimization. It is widely used among traffic practitioners. A great deal of research in the literature addresses the use of Synchro as the reference to evaluate the performance of other software packages. The literature does not provide sufficient research in comparing estimated intersection performance measures by Synchro with the values measured in the field. In other words, few studies are centered upon validating Synchro's estimates with real life data. The gap in the literature reflects the need for additional research effort to evaluate the accuracy and validity of Synchro intersection analysis results. Transit AVL/APC data provides another opportunity to estimate performance measures at signalized intersection which can be used as a benchmark to evaluate Synchro analysis results.

Moreover, AVL/APC data has not been typically stored or processed for consequent analysis, and its main application has been for real-time traffic monitoring and transit operations (Furth et al., 2006). Hellinga et al. (2011) proposed a methodology to estimate delays to transit vehicles at signalized intersections. The proposed methodology is new and needs to be endorsed by comparing its estimates to estimates from another approach of estimating the measures of effectiveness.

1.3 Goals and Objectives

This thesis seeks to answer the following two research questions:

1. Can the delay and queue length estimates obtained from AVL/APC data be used as a reliable estimate of intersection performance?

   Traffic engineering agencies widely accept using macroscopic models such as Synchro in evaluating intersection performance. This research attempts to examine the potential of using delay and queue length estimated from AVL/APC data in order to provide alternative approach for estimating intersection performance measures for traffic engineering purposes.

2. Can the results from Synchro be used as a reliable estimate of the delay that transit vehicles are expected to experience at signalized intersections?

   When planning new bus routes, transit agencies commonly dispatch test bus trips to estimate the delay experienced by transit vehicles at signalized intersections. Such practice is time consuming and not cost effective. This research attempts to explore the possibility of using delay and queue length estimates obtained from Synchro for various transit planning activities including:

   a. within a transit route planning tool to estimate route travel times; and
   b. to identify intersections at which to implement transit priority measures.

To answer these research questions, this thesis has the following objectives:

1. Understand how delay and queue length are estimated by Synchro and AVL/APC data.
2. Investigate the correlation between intersection performance measures estimated using AVL/APC data and Synchro.
3. Find a way in which both approaches can be used interchangeably to define the measures of effectiveness for the same intersection. In other words, investigate whether both approaches can be used interchangeably and find which approach is more suitable for estimating certain intersection performance measure i.e. delay estimation or queue estimation.

4. Discuss the similarity and discrepancies in the results and what each approach can and cannot capture.
5. Investigate the causes of the difference in intersections performance measures obtained from Synchro and from AVL/APC.
6. Provide recommendations based on the analysis results about a robust and reliable method for estimating intersection performance measures.

1.4 Thesis Outline

This thesis has five chapters. After a brief introduction in the first chapter, Chapter 2 provides a literature review on previous work on delay and queue length estimation at signalized intersections and how they are determined. The literature review on previous work on Synchro and AVL/APC systems are also presented in Chapter 2. The methodologies for estimating delay and queue length using HCM, Synchro, and AVL/APC data are present in Chapter 3. The analysis results are presented in Chapter 4 as well as discussing the possible causes of inconsistency in the analysis results performed by Synchro and AVL/APC data. Finally, Chapter 5 concludes the study and provides some recommendations.

Chapter 2

Literature Review

This chapter reviews the previous work on delay and queue length measurements and estimation at signalized intersections and how they are typically estimated. Furthermore, previous work on Synchro evaluation and its uses in estimating measures of effectiveness is reviewed. The implementation of AVL/APC data in traffic and transit operation is reviewed as well.

2.1 Intersection Performance Measures

The performance of the intersections along road segments highly affects the quality of service in urban transportation networks. The deterioration in urban mobility resulting from traffic congestion has become a major concern for transportation community and the general public. Measurements of intersection level of performance is significant for transportation planners for evaluating and improving traffic operations in arterials. Delay, queue length, volume to capacity ratio, and number of stops are the major measures of effectiveness (MOE) used for evaluating the performance of signalized intersections.

2.1.1 Delay at Signalized Intersections

Delay at signalized intersections is defined in the Highway Capacity Manual as the difference between the actual travel time along a road segment with intersection and the travel time in the absence of any intersection. The HCM uses average control delay for determining the level of service at signalized intersections (Transportation Research Broard, 2000). Delay is directly related to what motorists experience while attempting to cross an intersection, which increases its practicality as a performance indicator of signalized intersections. Three types of delay are commonly used by engineers (Sharma et al., 2007). Namely: stopped delay, which is the delay incurred while the vehicle is completely stationary; approach delay, which is the delay incurred upstream of the stop line due to deceleration and stopping; and control delay, which is the delay incurred from the start of deceleration until the vehicle reaches the free flow speed downstream of the stop line. Figure 1 illustrates these different types of delay.

Delay is probably the primary parameter used in traffic engineering industry to evaluate the performance of signalized intersections. A great deal of research has been conducted in developing methods for estimating control delay at signalized intersections. Each method used for estimating delay has its own definition for delay components and its own approach. Methods of estimating delay at signalized intersections are typically categorized into three types (Abdy, 2010): Field measurements, microscopic simulation, and analytical models for quantifying delay. Nonetheless, it is challenging to determine delay at an intersection.

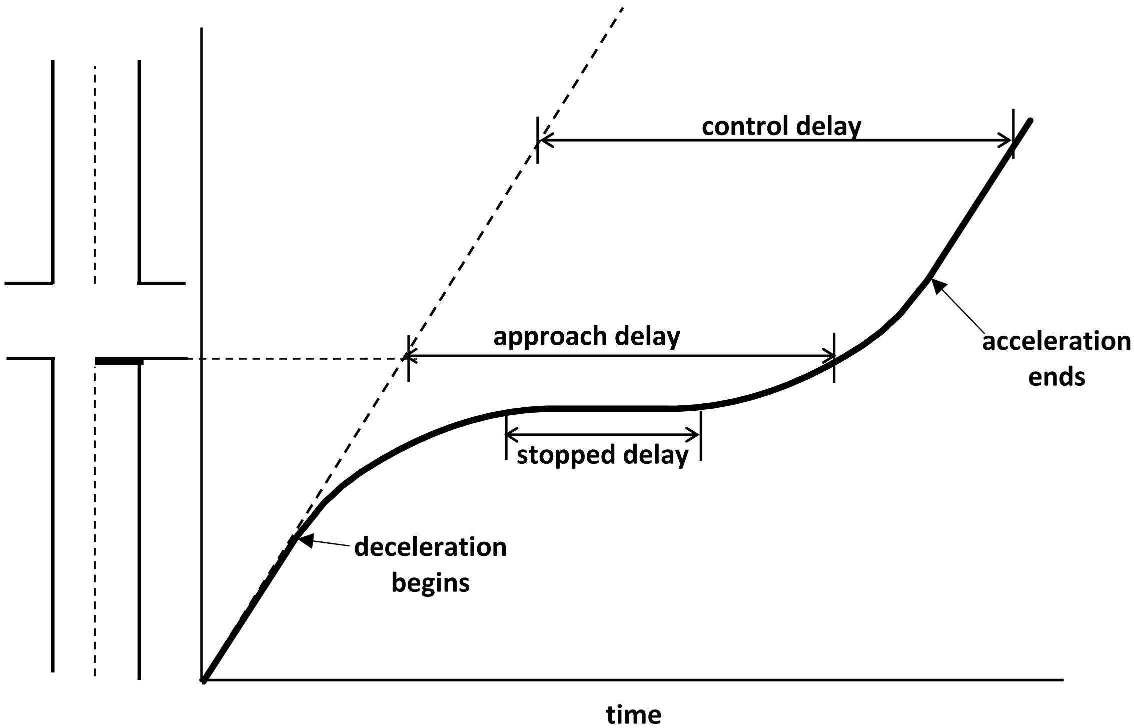


Figure 1: Graphical representation of different nuances of delay used by traffic engineers.

Field measurement of delay can be conducted traditionally in which observers are used to record delays at signalized intersections. However, it is labor intensive and time consuming. It is also difficult to control measurement errors since it depends on the skills and attentiveness of the observer. Mazloumi et al. (2010) used observers to track the trajectories of some vehicles. To overcome the difficulty of capturing all vehicles at the intersection, observers only considered vehicles with a specific color. Giving the difficulty of the traditional technique, advanced technology has been developed to collect field data. GPS equipped vehicles can capture high-resolution speed profile. Ko et al. (2008) used this technique to automate delay measurement. Deceleration, stopped, and acceleration delay can then be calculated by observers precisely.

Microscopic traffic simulation models are progressively becoming an important tool for traffic engineering analysis and management. Using microscopic traffic simulation models, trying and testing controversial and new techniques for improving intersections performance can be conducted without any interruption to traffic in a real network (Hidas, 2002). Traffic flow theory algorithms can be modelled using computer software packages such as VISSIM and Paramics which enables the analyst to simulate the movement of vehicles. Vehicle delay can be easily obtained because of the ability to trace simulated vehicle movement.

VISSIM is a microscopic model developed to analyze roadways and public transit operations. It primarily consists of a simulator that generates traffic and a signal state generator that emulates the type and parameters of controller. VISSIM is a time step and behaviour-based microscopic model that can estimate intersection performance measures.

Paramics is a microscopic stochastic simulation model that has the potential for application and modelling network situations. Every aspect of the transportation network can potentially be examined including integrated urban and freeway networks including signal control and transit operation. Intersection performance measures at network level, on a link by link basis, or at specific location can be estimated using Paramics.

Mousa (2003) developed a microscopic stochastic simulation model to emulate the traffic movement at signalized intersections. He examined the effect of cycle length, approach speed, and degree of saturation on vehicular delays. The simulation was applied to an existing signalized intersection and then the simulated delay was validated by comparing it to a set of field observed delay. It was found that the proposed simulation model produces estimates of delay that are similar to those observed in the field. Although very effective, micro simulation models generally require calibration and extensive data collection before coding a network, which limits the spatial scope that can be considered (Jones et al., 2004).

Analytical models are commonly used to estimate delay at signalized intersections. Popular capacity references such as the HCM and Canadian Capacity Guide (CCG) are solely based on such expressions (Transportation Research Broard, 2000), (Teply et al., 2008). The original work by Webster and Cobbe (1966) forms the basis of most signalized intersection delay models used in these references. The mathematical expressions assume vehicles arrive at the intersection according to the Poisson process and service times are deterministic. Such expressions are easily programmed into software packages; for instance Synchro and HCS. These software packages expedite the calculation of delays; however, they require initial data collection efforts and professional software expertise.

In most cases, it is unexpected to find perfect match between delay measured at the field and delay estimated using analytical formulas (Teply, 1989). Dion et al. (2004) state that it is similarly unexpected to find perfect match between delays estimated from different delay models. To address this problem, the authors conducted a comparative study of the delay estimated by different analytical delay models, including deterministic queuing, shock wave, steady state stochastic, and time-dependent stochastic delay models. Furthermore, the authors compared these estimates to the delays that are produced by a microscopic traffic simulation model. They observed that there is a general similarity between all the analytical delay models considered when these are applied to the analysis of under saturated signalized intersections. However, consistency of results for more complex situations should be evaluated since the study only considered simple intersections.

2.1.2 Queue Length at Signalized Intersections

Queue length is another important measure of signalized intersections' performance. Queues that overflow the available storage space for turning movements adversely affect the overall operation of the intersection. Several terms are used in the literature to describe queues such as queue length and maximum extent of queue. On one hand, the Highway Capacity Manual

(2000) defines the queue size as the number of vehicles that are queued depending on the arrival patterns of vehicles and on the number of vehicles that do not clear the intersection during a given green phase and overflowed to the next cycle. Consequently, the queue length is the distance from the stop line to tail of the queue at the end of the red interval. On the other hand, queue extent or queue reach represents the distance from the stop line to the tail of the last vehicle when the queue dissipates.

Queue length is a measure of the quality of service offered to motorists as well as the air pollution and fuel consumption. Queue extent, on the other hand, is used to determine the adequacy of the available storage. The difference between the two definitions is because vehicles continue to join the back of the queue after the end of the red phase and a shockwave is formed and moving backwards.

Queue length at signalized intersections can be obtained from field measurements, or it can be estimated using analytical procedures or micro simulation models. Field measurements are typically resource intensive; therefore, it is widely accepted in the industry to use simulation or analytical procedures to obtain queue length. Many models have been developed to estimate queue length at signalized intersections; however, their estimates may vary based on their definition of each measure (Viloria et al., 2000). Some models use the terms queue length and queue extents interchangeably without distinguishing the difference between them.

Microscopic simulation can calculate the average queue and the maximum queue at signalized intersection by tracing the vehicle path and stops. Kang (2000) compared queue extent estimates from a microscopic simulation model, INTEGRATION, to analytical models and found a consistency in the predictions for both undersaturated and oversaturated signalized intersections.

Analytical models generally utilize queuing analysis or shockwave analysis. Queuing analysis, also known as point queue or stacking queue, involves computing the queue length assuming vertical queue where vehicles are assumed to be stacking on top of each other at the intersection stop line. On the other hand, shockwave analysis takes into consideration the physical space occupied by each vehicle, i.e., vehicles are queued horizontally. Viloria et al. (2000) compared several analytical models estimates of queue length and found that the majority of them provide estimates that are more analytically defensible than those provided by the simpler theoretical models. Simple theoretical models are unjustifiably optimistic when demand approaches capacity.

2.1.2.1 Queuing Theory Analysis

Stacking queue is considered as a deterministic queuing analysis. It is relatively simple macroscopic approach in estimating queue length. "First in, first out" system is assumed in the queue in order to compute the estimates of the queue length.

Figure 2 illustrates the concept of the stacking queue in undersaturated condition. A vertical queue is assumed to form at the approach stop line, which matches the number of arriving vehicles throughout the red interval. At the beginning of the green interval, vehicles at the front of the queue start to discharge at the saturation flow rate, while at the back of the queue, new vehicles start to join the queue at arrival flow rate until the queue dissipates after the saturated green time. Saturated green time, g_s, is the time required to serve the accumulated queue while the remainder of the green time is called the unsaturated green time, g_u. The number of vehicles arriving during red interval equals the maximum queue size in vehicles. The maximum queue length can be expressed in distance units after multiplying it by the average space occupied by one vehicle.

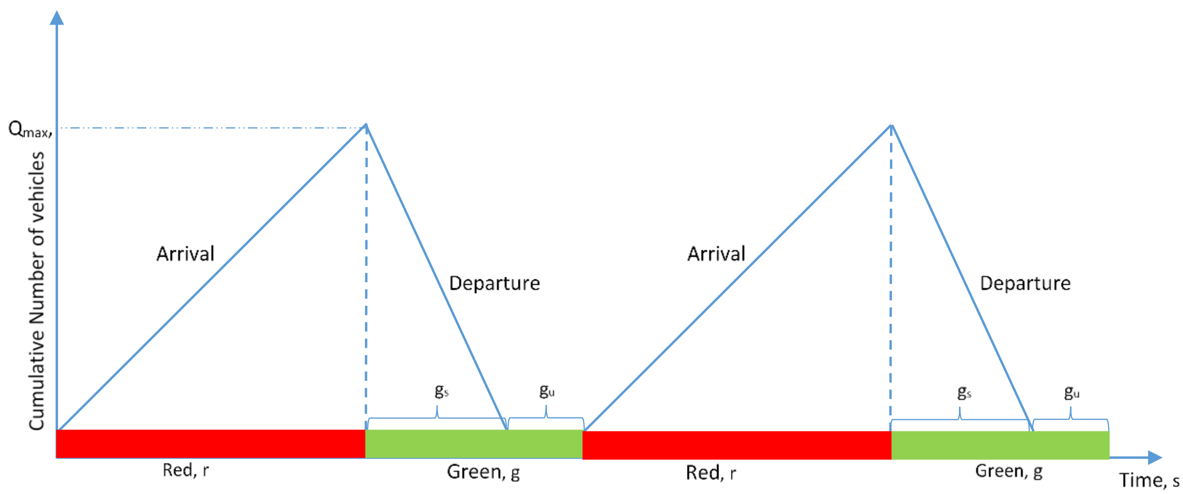


Figure 2: Vertical deterministic queuing diagram for undersaturated condition

When the demand exceeds the capacity of a lane group, a remaining queue is underserved at the end of the green interval. For demonstration, Figure 3 shows two cycles. A residual queue at the end of a cycle is caused by the difference between the arrival flow rate and the capacity of the lane group. The peaks of the triangles represent the queue length at the end of the red interval in each cycle. The points where triangles meet represents the residual queue at the end of each cycle. In oversaturated intersections, all the green time becomes a saturated green time, and it is not sufficient to serve the accumulated queue.

Furthermore, queuing theory assumes that vehicles accelerate and decelerate instantaneously. This assumption consequently results in underestimation of the delay and maximum queue as the vehicles are assumed to arrive at the tail of the queue later than they would in reality. Since queuing theory does not account for the arriving vehicles during the saturated green time, shockwave theory can be used to estimate more accurate measures of effectiveness.

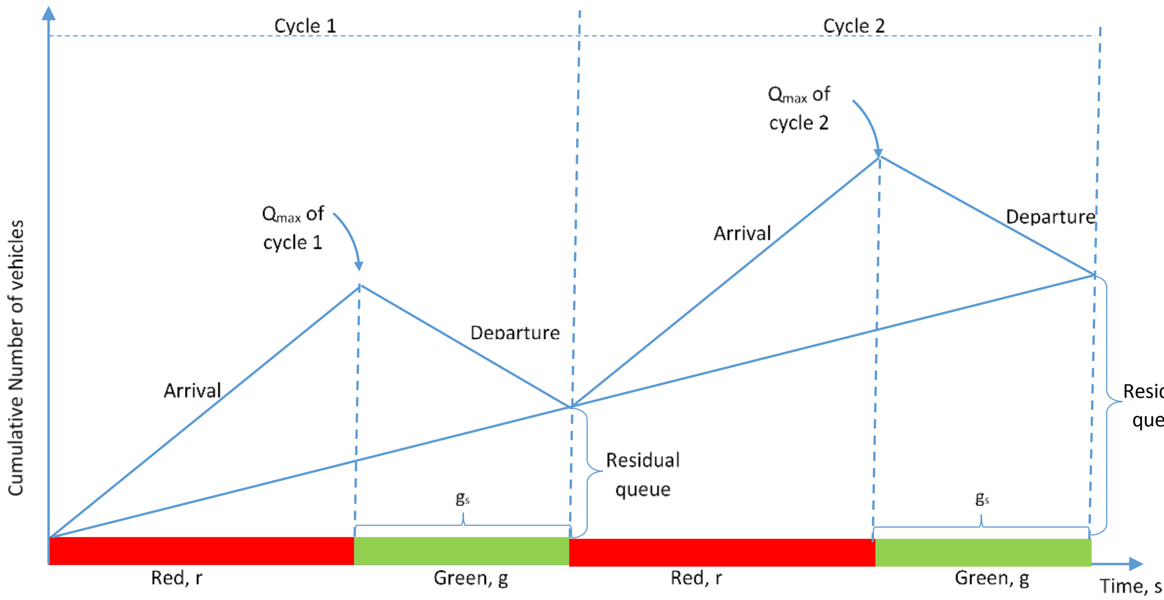


Figure 3: Vertical deterministic queuing diagram for oversaturated condition

2.1.2.2 Shockwave Theory Analysis

While queuing theory estimates the maximum queue size only, shockwave theory can provide more realistic estimates in terms of the maximum queue extent. Shockwave theory uses the relation between traffic flow and density to describe the upstream or downstream propagation of traffic status.

In reality, vehicles stop away from the stop line further than the queue size. Shockwave theory observes the maximum reach of the queue, which is more useful in terms of planning or designing traffic signals and intersection infrastructure. Figure 4 shows the creation and dissipation of shockwaves resulting from the traffic signal operation in undersaturated condition. SW_1 represents the back of the queue, which is the edge between the arriving vehicles and the stopped vehicles. SW_1 starts to form at the beginning of the red interval and moves backward. At the beginning of the green time, two shockwaves start to form. One moves downstream and it represents the first vehicle that leaves the intersection. The other one, SW_2, represents the edge between departing vehicles and stopped vehicles. It moves backward until it reaches SW_1 when the queue dissipates. SW_3 starts to form after the last stopped vehicle accelerates.

As Figure 4 shows, the maximum queue reach is more realistic than the maximum queue size since what happens in reality follows the explanation of the shockwave theory. It is more complicated to compute the maximum reach of queue in oversaturated conditions; however, it is important particularly for planning the storage lane for turning movements to prevent spillback.

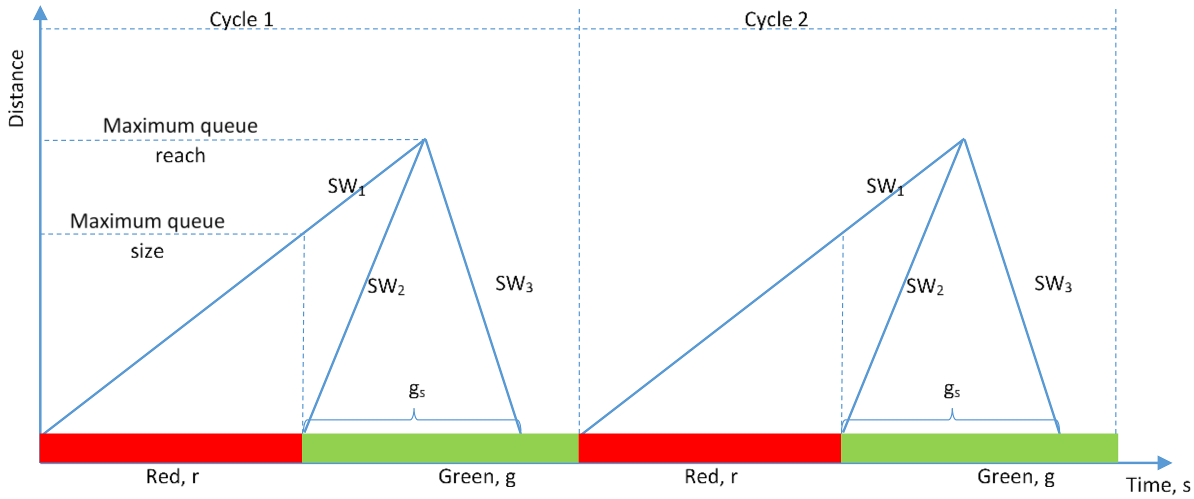


Figure 4: Horizontal shock wave diagram for undersaturated condition

2.2 Synchro

Synchro Plus suite includes:

- Synchro: a macroscopic analysis and optimization program.
- SimTraffic: a powerful, easy to use traffic simulation application.
- 3D Viewer: a three dimensional view of SimTraffic simulations, and
- SimTraffic CI: an application that interacts with controller interface (CI) device connected to a controller to simulate the operation of the controller with simulated traffic.

According to Synchro 8.0 Users Guide (Trafficware, 2011), Synchro implements the Intersection Capacity Utilization (ICU) 2003 method for determining the intersection capacity. This method is very straightforward to implement since it compares the current volume to the intersection ultimate capacity. Synchro is a macroscopic traffic software program that replicates the signalized intersection capacity analysis as specified in the Highway Capacity Manual (2000) and (2010). Macroscopic level models represent traffic in terms of aggregate measures for each movement at the intersections. Equations are used to determine measures of effectiveness such as delay and queue length. It should be noted that Synchro does not account for “bottleneck” situations where upstream capacity deficiencies reduce the amount of traffic reaching downstream intersections.

Synchro is widely used by traffic practitioners to evaluate the performance of signalized intersections (Yang, 2001). A number of studies compared Synchro with other traffic simulation models in terms of the ability to estimate the performance measures at signalized intersections. Most of these studies admits that estimated performance measures through different software might significantly vary and highlight the important factors that should be considered when comparing the results.

Benekohal et al. (2002) compared control delays computed by HCM methodology (using HCS software), Synchro and other traffic simulation software for an urban arterial with several signalized intersections. Estimated control delays from Synchro were significantly different from HCS results considering different operating conditions and signal timing schemes. Authors discussed certain precautions to be taken into account when comparing analysis results between HCS and Synchro.

Washburn and Larson (2002) compared the control delay, estimated by Synchro, TRANSYT-7F (TRC, 1999), and HCS. They concluded, even with identical input data, differences between HCS, TRANSYT-7F and Synchro are expected. The only situation in which the results were identical was a pre-timed intersection with random arrivals on all approaches, and protected movements. They claim that no single model is ideal for every situation; Synchro is the best fit for intersections that are actuated. However, at the time, public agencies did not accept its method of delay calculation for traffic impact studies (Washburn and Larson, 2002).

Mulandi et al. (2010) evaluated the performance of signal timing plans calculated by several macroscopic and microscopic traffic simulation tools including Synchro, TRANSYT-7F, CORSIM and VISSIM. They found obvious differences in the performance of the signal timing plans optimized using these simulation tools. The optimization of signal timing plans is generally achievable through minimization of the intersection performance measures such as delay, queue length and number of stops. The differences observed in optimized signal timing plans by different software indicate the significance of the approaches used by different software for estimating intersection performance measures. The authors indicate that the highest quality of signal timing was produced by VISSIM and Synchro software packages. They stated exceptionally similar performance was delivered by these programs.

2.3 Transit AVL/APC Data

Recently, AVL/APC systems have become a prominent method for data collection by transit agencies. AVL/APC systems are able to track equipped vehicles in a fleet and record a variety of information for each vehicle, which allows for numerous applications of the data.

The main applications of AVL/APC data are in real-time transit operations monitoring and control (Furth et al., 2006). Typically, AVL data has not been stored or processed for consequent analysis; however, it has a substantial potential in improving service planning, scheduling, and performance analysis practices at transit agencies. AVL /APC systems can collect large amount of data at a low cost with ability to integrate with other data sources.

AVL/APC data have been used by researchers for different applications such as real-time traveler information (Farhan et al. 2002); transit signal priority (Lin, 2002; Liu et al., 2007); transit route performance measurement, (Liao and Liu, 2010); and ridership and operational performance analysis (Golani, 2007).

Furth et al. (2006) provided an insightful description of AVL/APC systems and quantified some of the benefits of AVL and APC implementation comprehensively. They developed guidance for the effective collection and use of archived AVL/APC data in order to improve decision-making and managing transit fleets. These technologies can be used to improve route design and scheduling.

Farhan et al. (2002) used AVL/APC data in developing a travel time model able to provide real time traveler information services. Their goal was to develop and compare performance of several procedures for modelling bus arrival models. They used three techniques to develop bus travel time prediction models that were able to update the model based on new data that reflect the changing characteristics of the transit-operating environment.

Transit AVL/APC data can also be used to estimate stopped delays and queue lengths, and can be used to investigate the effect of Transit Signal Priority (TSP) for determining areas where TSP is beneficial. Liao et al. (2010) proposed a data processing framework to analyze transit performance on stop (loading and disembarking efficiency) and route (travel time monitoring) levels based on time point records.

Lin (2002) indicated that implementing TSP technologies could reduce delay at intersections, although knowledge of the existing delay of an intersection must be considered to determine the possible reduction in delay. In their analysis using AVL/APC data, Liao and Liu (2010) determined areas that would benefit from TSP, and they recognized the prospective for AVL data to be used for real-time TSP implementation. Liu et al. (2007) conducted a simulation study that demonstrated the potential of using real-time AVL data and how it can facilitate the implementation of TSP by determining the bus arrival time.

Transit AVL/APC data systems can be applied for performance analysis of ridership and transit operation. In a study by Golani (2007), AVL/APC data was proven to be of great help in improving transit service since it provides huge amount of data with the ability to automate the process of analysis. AVL/APC data was used to visualize the boarding, alighting, and delay at each bus stop.

Given the challenges involved in the field measurement of delay and queue length at signalized intersections, Hellinga et al. (2011) proposed a methodology for direct measurement of delay and queue length at signalized intersections using AVL/APC data collected by transit vehicles. The proposed methodology is suitable to be applied to most AVL/APC databases, and it is able to explain up to 96% of the variation in delay (Hellinga et al., 2011). It provides new opportunities to apply archived AVL/APC data for performance evaluation of signalized intersections, which will be discussed in the next chapter.

2.4 Conclusion

There are few studies centered upon comparing estimated intersection performance measures by Synchro with the values measured in the field (Petraglia, 1999). On the other hand, accurate measurement of delay and queue length at signalized intersections is costly and challenging, as it requires application of advanced technologies and analysis methods. As well, there is no widely accepted standard methodology for field measurement of delay and queue length at signalized intersections. As a result, field measurements of delay and queue length depend on the technology and the methodology applied. Consequently, majority of the existing research in this area revolve around comparing Synchro with other widely used computer packages and traffic simulation software.

Although AVL/APC has a great potential in signalized intersection performance analysis, the literature is focused on its applications in transit operation and control and does not provide sufficient research in implementing AVL/APC data for estimating intersection performance measures. In other words, the literature shows that capturing transit delay using AVL/APC systems are possible; however, it does not provide any conclusive studies that demonstrate the ability to estimate transit vehicle delays caused by signalized intersections from AVL/APC data (Hellinga et al., 2011).

Chapter 3
Analysis Method

This chapter presents the analysis methodologies applied in this research and the description of available data:

- The methodology for obtaining stopped delay and queue length suggested by the Highway Capacity Manual (2000).
- Synchro's methodology for obtaining stopped delay and queue length at signalized intersections.
- Demonstration of coding a signalized intersection in Synchro.
- The proposed method for determining transit vehicle stopped delays at signalized intersections from archived AVL and APC data (Hellinga et al., 2011).
- An analysis to demonstrate the relationship between stopped delay and control delay in transit vehicles.
- Description of the available data for this research.

3.1 Analysis Method Overview

In this research, the methodology proposed by Hellinga et al. (2011) is applied to estimate the delay and queue length at signalized intersections from archived AVL/APC data and compared the results with the values computed by Synchro. The analysis covers a considerable number of intersections in a relatively large urban transportation network. A flowchart of the work is presented in Figure 5.

Estimated delay and queue length from AVL/APC data realistically represent average delays and queues experienced by the road users at signalized intersections over an extended period. Therefore, it provides a robust benchmark for evaluating the performance of Synchro in computing delay and queue length at signalized intersections.

Synchro uses basic HCM models to estimate delay and queue length at signalized intersections. However, Synchro's methodology includes some additional procedures and modifications, which are addressed in this section. The proceeding sections briefly review HCM and Synchro methodologies for estimating delay and queue length at signalized intersections followed by a brief introduction of the methodology used for estimating intersection delay and queue length using AVL/APC data.

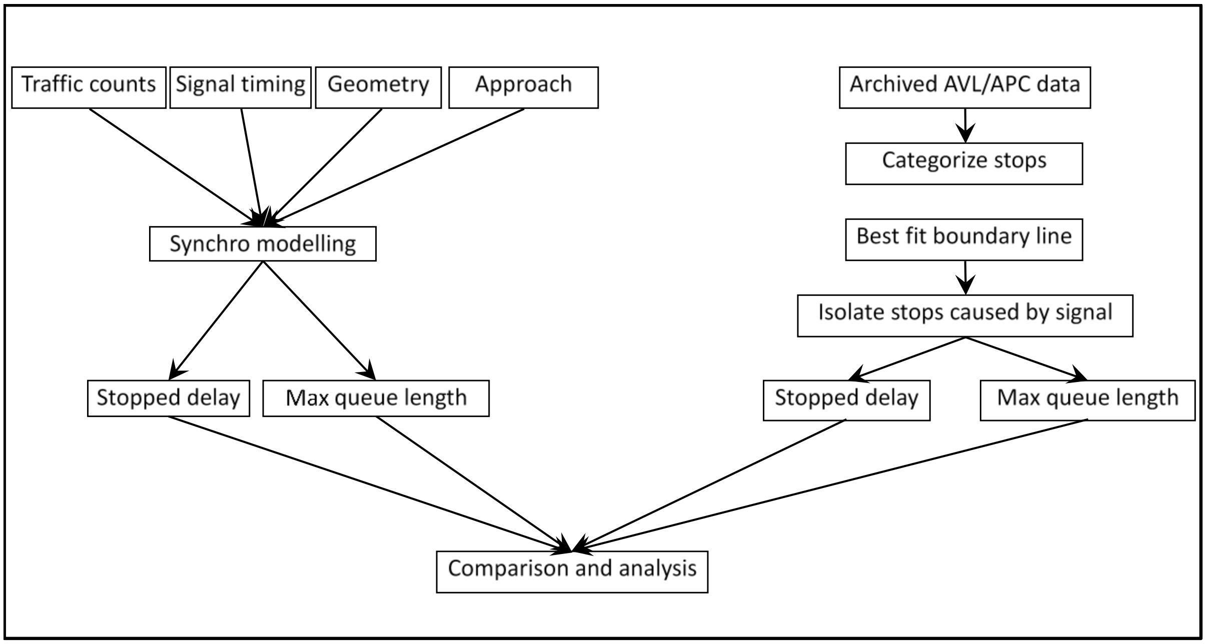


Figure 5: Workflow of the research.

3.2 HCM Methodology

HCM methodology for analyzing the capacity and level of service of signalized intersections is described in Chapter 16 of the HCM. The methodology addresses the capacity, level of service, and other performance measures for lane groups and intersection approaches and the level of service for the whole intersection. The methodology does not take into account the potential impact of downstream congestion on intersection operation. Nor does the methodology detect and adjust for the impacts of turn-pocket overflows on through traffic and intersection operation (Transportation Research Broard, 2000). This section, describes delay and queue length calculation in the HCM.

3.2.1 Delay Calculation

The Highway Capacity Manual applies Webster's delay formulation (Webster and Cobbe, 1966) for calculating average control delay for a lane group at signalized intersections. Control delay represents the additional travel time experienced by vehicles at slower speeds as well as the vehicles that stop at intersection approaches as they move up in queue position or slow down upstream of a signalized intersection. The values derived from the delay calculations represent the average control delay experienced by all vehicles that arrive in the analysis period, including delays incurred beyond the analysis period when the lane group is oversaturated. As shown by Equation (1), the control delay per vehicle (D) encompasses uniform delay (D_1), incremental delay (D_2), and the initial queue delay (D_3).

$$D = D_1 \times PF + D_2 + D_3 \tag{1}$$

Uniform delay is the delay caused assuming uniform arrivals, stable flow, and no initial queue. It is based on the first term of Webster's delay formulation and is widely accepted as an accurate depiction of delay for the idealized case of uniform arrivals. The progressions factor (*PF*) accounts for the effects of coordination on uniform delay. Good signal progression will result in a high proportion of vehicles arriving during the green interval. Poor signal progression will result in a low proportion of vehicles arriving during the green interval. Progression primarily affects uniform delay, and for this reason, the adjustment is applied only to D_1. Uniform delay component of control delay at signalized intersections is estimated using Equation (2).

The incremental delay includes the delay due to non-uniform arrivals and temporary cycle failures (random delay) as well as the delay caused by sustained periods of oversaturation. It is sensitive to the degree of saturation of the lane group (X), the duration of the analysis period (T), the capacity of the lane group (c), and the type of signal control. Incremental delay component of control delay at signalized intersections is estimated using Equation (3). The equation assumes that there is no unmet demand that causes initial queues at the start of the analysis period, which causes initial queue delay.

$$D_1 = 0.5 \times C \times \frac{\left[1 - \left(\frac{g}{C}\right)\right]^2}{1 - \left[\min(X,1) \times \left(\frac{g}{C}\right)\right]} \tag{2}$$

$$D_2 = 900 \times T \times \left[(X-1) + \sqrt{(X-1)^2 + \frac{8 \times k \times I \times X}{c \times T}}\right] \tag{3}$$

Where, C is the cycle length in seconds, T is the duration of analysis in hours, g is effective green time in seconds, X is volume to capacity ratio (v/c), c is the capacity in vehicle per hour, k is the incremental delay factor, and I is the upstream filtering factor. Chapter 16 of HCM (2000) provides additional instructions for estimating the progression factor (*PF*), incremental delay factor (k) and upstream filtering factor (I). The duration of analysis, (T) is typically 15 minutes (0.25 hr) unless when volume to capacity ratio is greater than 1.0. For oversaturated conditions, HCM (2000) recommends to prolong the analysis period to account for the period of oversaturation assuming that the average flow during analysis period is constant.

3.2.2 Queue Length Calculation

HCM (2000) defines the back of queue as the number of vehicles that are queued depending on the arrival patterns of vehicles and on the number of vehicles that do not clear the intersection during a given green interval. HCM (2000) provides procedures to calculate the average back of queue as well as 70th, 85th, 90th, and 98th percentile back of queue.

The average back of queue is the number of vehicles in the queue that is estimated using Equation (4) by combing uniform queue (Q_1) and incremental queue (Q_2).

$$Q = Q_1 + Q_2 \tag{4}$$

Uniform queue is the average back of queue, determined through Equation (5) assuming a uniform arrival pattern and then adjusting for the effects of progression for a given lane group using a progression factor (PF_2). Incremental queue is associated with randomness of flow and overflow queues that may result because of temporary failures, which can occur even when demand is below capacity. HCM (2000) provides Equation (6) for estimating incremental queue. Initial queue at the start of the analysis period is also considered in calculation of incremental queue.

$$Q_1 = PF_2 \frac{\frac{v_L C}{3600}\left(1 - \frac{g}{C}\right)}{1 - \left[\min\left(X_L, 1\right)\frac{g}{C}\right]} \tag{5}$$

$$Q_2 = 0.25 \times c_L \times T\left[\left(X_L - 1\right) + \sqrt{\left(X_L - 1\right)^2 + \frac{8 \times k_B \times X_L}{c_L \times T} + \frac{16 \times k_B \times Q_{bL}}{\left(c_L \times T\right)^2}}\right] \tag{6}$$

Where, PF_2 is the adjustment factor for effects of progression, v_L is lane group flow rate per lane (veh/h), c_L is the lane group capacity per lane (veh/h), X_L is v_L to c_L ratio, k_B is the second-term adjustment factor related to early arrivals, and Q_{bL} is the initial queue at start of analysis period (veh). Other parameters are similar to those already defined in equations (2) and (3). Appendix G in Chapter 16 of HCM (2000) provides additional equations and instructions to estimate progression factor (PF_2) and second-term adjustment factor (k_B).

The percentile back of queue ($Q_{\%}$) is computed using Equation (7) by applying the percentile back of queue factor ($f_{B\%}$) to the average back of queue estimated from Equation (4).

$$Q_{\%} = Q \times f_{B\%} \tag{7}$$

The percentile back of queue factor is calculated using Equation (8).

$$f_{B\%} = p_1 + p_2 e^{\frac{-Q}{p_3}} \tag{8}$$

Where, p_1, p_2 and p_3 are first, second, and third parameter for percentile back of queue factor, respectively. HCM (2000) provides default values for parameters of back of queue for pre-timed and actuated signals (Chapter 16, Appendix G).

3.3 Estimation of Intersection Delay and Queue Length Using Synchro

In this research, Synchro 8.0 is used to estimate delay and queue length at signalized intersections. This section provides in depth details regarding the underlying calculations found within Synchro. Software features described in this section pertain to Synchro 8.0.

3.3.1 Delay Calculation

Synchro is capable of computing average control delay for a lane group at signalized intersections. Synchro computes control delay by adding up uniform, incremental, and initial queue delays in a similar fashion as described through Equation (1).

Synchro computes uniform delay using Percentile Delay Method. The uniform delay formula in HCM (2000) assumes uniform arrivals when estimating volume to capacity ratio (X). However, it is very likely that traffic do not arrive at an intersection uniformly during the analysis period. Moreover, HCM (2000) calculates uniform delay using a single value for green times. However, green times may vary significantly in actuated traffic signals during analysis period. To account for variations in traffic flow, and variable green times in actuated signals, Synchro models traffic volume by considering five different percentiles scenarios, and computes uniform delay by taking a volume-weighted average of estimated delays for each percentile traffic flow (v_p). The five scenarios are the 10th (v_{10}), 30th (v_{30}), 50th (v_{50}), 70th (v_{70}), and 90th (v_{90}) percentile traffic flows assuming a Poisson distribution with average arrival rate of λ which is the hourly demand divided by the number of cycles in one hour. Synchro's user manual (Trafficware, 2011) provides Equation (9) for estimating the traffic flow for a given percentile.

$$v_p = (\lambda + z\rho) \times \frac{3600}{C} \tag{9}$$

Where, z is the number of standard deviations needed to reach a percentile from the mean, and ρ is the standard deviation, which is the square root of λ assuming Poisson distribution. Values of -1.28, -.052,0 , 0.52, and 1.28 are suggested in Synchro's user manual for 10th (v_{10}), 30th (v_{30}), 50th (v_{50}), 70th (v_{70}), and 90th (v_{90}) percentile traffic flows. Each traffic flow scenario represents 20% of the cycles actually occurring during the analysis period.

For actuated signals, Synchro estimates the green time for a particular traffic flow scenario by determining the probability of skipping or gapping out each phase during analysis period. Synchro skips a phase if the possibility that no vehicle arrives during the red interval is greater than 50%. On the other hand, phase gap out occurs when the possibility that no vehicle is going to be detected during the effective gap time is greater than 50%. Effective gap time is set by signal controller to define the maximum headway between successive vehicles that warrants the phase gap out (Trafficware, 2011).

The uniform delay corresponding to each percentile traffic flow is estimated by applying the first term of Webster's delay formulation as presented in Equation (2). Average uniform delay in Percentile Delay Method is estimated using Equation (10).

$$D_1 = \frac{VD_{10} + VD_{30} + VD_{50} + VD_{70} + VD_{90}}{\left(v_{10} + v_{30} + v_{50} + v_{70} + v_{90}\right) \times \frac{3600}{C}} \tag{10}$$

Where, VD_{10}, VD_{30}, VD_{50}, VD_{70}, and VD_{90} represent 10th, 30th, 50th, 70th, and 90th percentile delays, respectively. The progression factor (PF) is used in Equation (1) to account for the effects of coordination on uniform delay. Synchro calculates the progression factor explicitly by comparing uniform delays with and without coordination. Traffic flow variations can be modelled more reasonably using five traffic flow scenarios for estimating uniform delay. However, Synchro user can choose to estimate uniform delay using HCM methodology without considering traffic flow scenarios.

Synchro applies HCM (2000) methodology to compute incremental delay (D_2) using Equation (3). Incremental delay factor and upstream filtering factor are calculated using HCM (2000) instructions.

3.3.2 Queue Length Calculation

Synchro does not consider vehicles delayed by less than 6 seconds to be part of the queue because, according to Synchro's manual, these vehicles slow down but do not stop completely (Trafficware, 2011). Synchro computes 50th and 95th percentile queues. The 50th percentile queue represents the average queue length while the 95th percentile queue defines the maximum queue length. The 50th percentile queue length (Q) is estimated in feet using average arrival flow rate (v) through Equations (11) and (12) for under and over saturated conditions, respectively.

$$Q = \frac{v}{3600} \times \left(R - 6\right) \times \left[1 + \frac{1}{s/v - 1}\right] \times \frac{L}{\left(n \times f_{LU}\right)} \qquad \frac{v}{c} < 1 \tag{11}$$

$$Q = \left[\left(v \times \left(C - 6\right)\right) + \left(v - \left(s \times \frac{g}{C}\right)\right) \times \frac{C}{3600}\right] \times \frac{L}{n} \qquad \frac{v}{c} \geq 1 \tag{12}$$

Where, R is duration of red interval in seconds, s is saturation flow rate in vehicle per hour, v is arrival rate in vehicle per hour, L is length of vehicles including the space between in feet, n is number of lanes, and f_{LU} is lane utilization factor, respectively. Other variables are similar to those defined in equations (1) and (2). If volume to capacity ratio (v/c) exceeds one for extended period, the queue length is mathematically infinite. Thus, Synchro calculates the queue length as the maximum queue after two cycles for oversaturated lane groups.

To account for the impact of traffic volume fluctuations on queue length Synchro calculates 95th percentile queue length by increasing the arrival rate to 95th percentile traffic volume (v_{95}). The 95th percentile arrival rate is estimated using Equation (13).

$$v_{95} = v \times PHF_x \left[1 + 1.64 \frac{\sqrt{v_c}}{v_c} \right] \tag{13}$$

Where, v is arrival rate in vehicle per hour, PHF_x is the minimum of peak hour factor (PHF) or 0.9, and v_c is vehicle arrival per cycle ($v \times 3600 / c$). It should be noted that the arrival rate (v) is unadjusted by peak hour factor (PHF) because the 95th percentile traffic volume (v_{95}) accounts for traffic fluctuations. The 95th percentile queue is computed using 95th percentile traffic volume (v_{95}) through equations (11) and (12) considering volume to capacity ratio.

3.3.3 Queue Interactions

Queue interactions is a traffic analysis looking at how queue can reduce capacity through spillback, starvation, and storage blocking between lane groups. Queue interactions have the potential to reduce capacity and increase delay even on movements that are undersaturated. In other words, less storage space than one full cycle worth of traffic causes higher delays along with reduction in capacity. Spillback and starvation are tightly interrelated. Depending on which intersection is more capacity constrained, reduction in capacity due to starvation will eventually lead to spillback upstream (Trafficware, 2011).

Spillback is caused when a queue formed downstream intersection uses all the space on a link and prevents vehicles from entering the upstream intersection on green. Figure 6.a shows consecutive intersections with a spillback problem.

Starvation, on the other hand, occurs when a downstream signal is green, but the signal cannot service full capacity efficiency because the upstream signal is red. Figure 6.b shows consecutive intersections with a starvation problem.

Storage bay blocking occurs at an intersection where one movement is blocking other movement. Spillback and starvation concept is applied on blocked movements. Figure 6.c shows the storage bay for the right turn movement blocked by the through movement.

The queue interactions calculations generally begins by determining if the ratio of volume per cycle to distance is critical. Queue interactions cause a reduction in capacity only when the storage space is less than one cycles worth of traffic.

To determine reduction caused by queue interactions, Synchro defines *capDist* as the distance per lane used by one cycle capacity, and *volDist* as the distance per lane used by one cycle of the 90th percentile volume (*v90*). *volDist* and *capDist* are given by Equation (14) and (15) respectively.

$$capDist = L \times \left[2 + C \times \frac{c}{n \times 3600} \right] \tag{14}$$

$$volDist = L \times \left[2 + v_{90} \times \frac{c}{n \times 3600} \right] \tag{15}$$

Where L is the average vehicle length, C is the cycle length, c is the lane group capacity, and n is the number of lanes. A link is subjected to spillback and starvation whenever the minimum of *volDist* and *capDist* is larger than the downstream link distance. A storage bay is subjected to blocking whenever the minimum of *volDist* and *capDist* is larger than the storage bay length.

The time that the movement is blocked or starved is then determined. For storage blocking and spillback, the capacity during this time is zero. For starvation, the lane group capacity during this time is reduced to the upstream saturation flow rate active at the time. The reduced capacity is then used to find the incremental delay given by Equation (3).

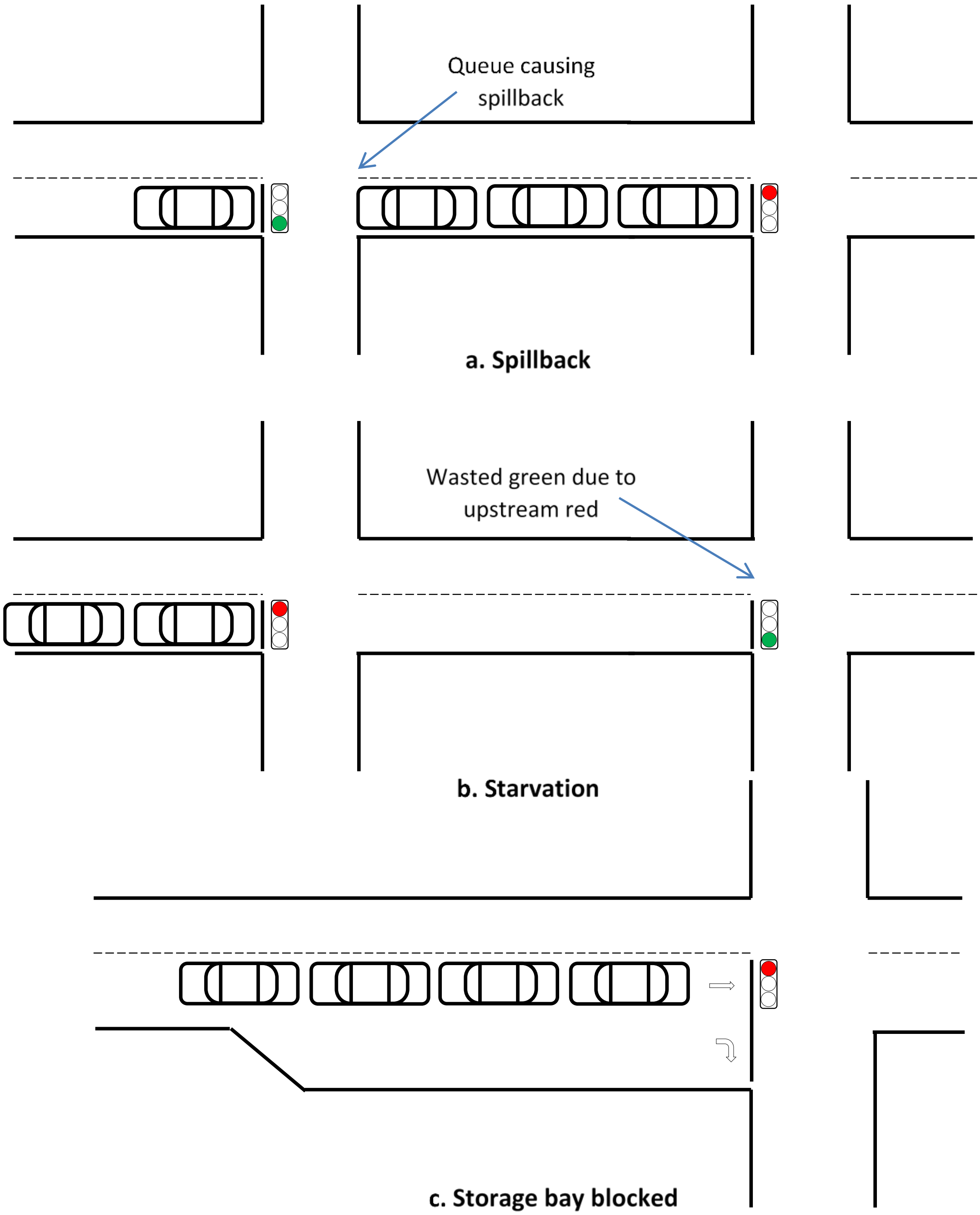


Figure 6: Spillback, starvation, and storage bay blocking.

3.3.4 Coding Intersections in Synchro

In this section, the procedure of coding an intersection in Synchro is presented. Input parameters that should be fed to the model are explained and illustrated through coding of a simple hypothetical intersection that will be used for sensitivity analysis later in the next chapter.

One of the most appealing features of Synchro is its user interface. It is very easy to use and the majority of required input is self-explanatory. The analyst creates a network by drawing links and nodes on a given pallet.

The hypothetical example is a simple 4-leg pretimed signalized intersection for which each approach consists of a single lane and only through movements are permitted. The links are first drawn using the pallet as crossing lines. Once the intersection is drawn, single lane on each approach is created by default. Figure 7 shows the intersection coded in Synchro. Changing lane characteristics takes place in the lane settings window. Most windows in the program are activated when a link is selected. It should be noted that highlighted numbers using blue color in all windows are calculated based on the input. In other words, the analyst should not override them unless he or she has good reasons.

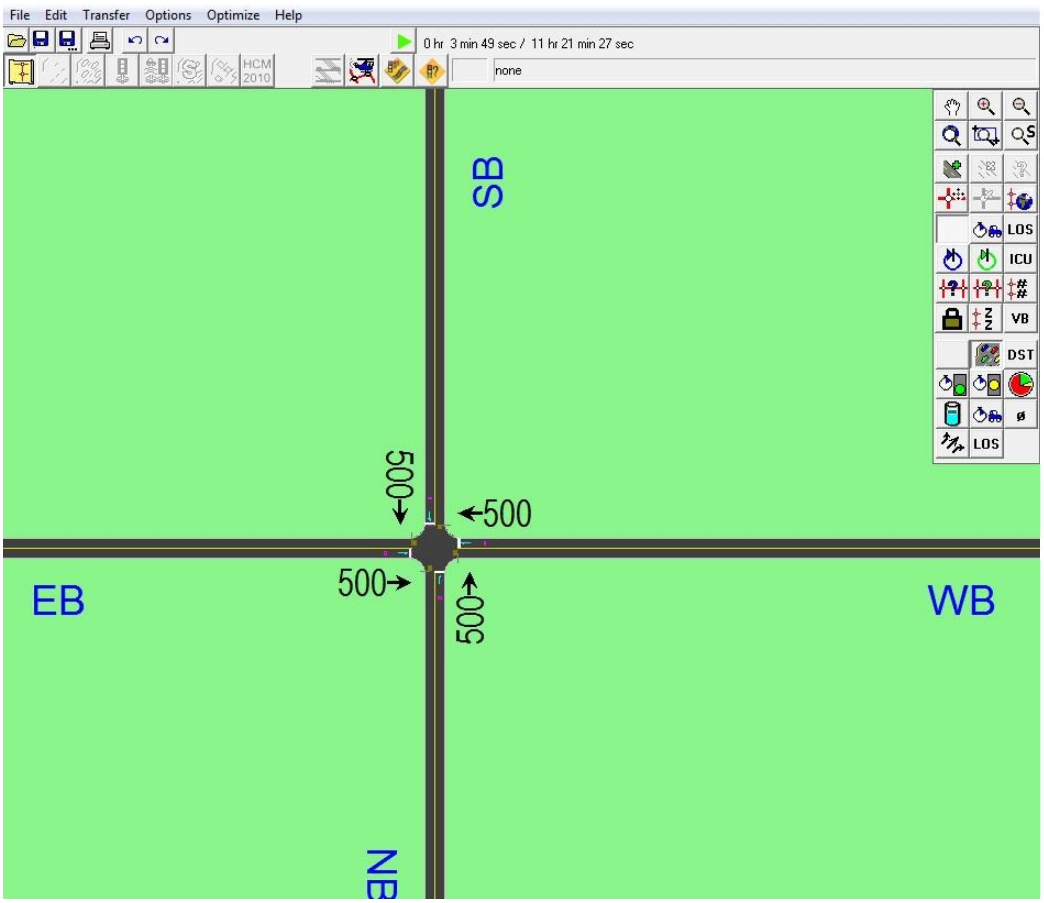


Figure 7: Hypothetical intersection coded in Synchro.

The mandatory parameters to be input in the lane settings window for each approach include lanes number and configuration, traffic volume, ideal saturation flow rate, lane width, area type, and number of storage lanes and their length. Other parameters such as link speed and right turn curb radius are optional for isolated intersections. Figure 8 shows the lane settings window in Synchro.

LANE SETTINGS	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lanes and Sharing (#RL)		↑			↑			↑			↑	
Traffic Volume (vph)	0	500	0	0	500	0	0	500	0	0	500	0
Street Name	EB			WB			NB			SB		
Link Distance (m)	—	265.5	—	—	375.0	—	—	230.3	—	—	257.2	—
Links Speed (km/h)	—	50	—	—	50	—	—	50	—	—	50	—
Set Arterial Name and Speed	—	EB	—	—	WB	—	—	NB	—	—	SB	—
Travel Time (s)	—	19.1	—	—	27.0	—	—	16.6	—	—	18.5	—
Ideal Satd. Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Lane Width (m)	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
Grade (%)	—	0	—	—	0	—	—	0	—	—	0	—
Area Type CBD	—	☐	—	—	☐	—	—	☐	—	—	☐	—
Storage Length (m)	0.0	—	0.0	0.0	—	0.0	0.0	—	0.0	0.0	—	0.0
Storage Lanes (#)	—	—	—	—	—	—	—	—	—	—	—	—
Right Turn Channelized	—	—	None	—	—	None	—	—	None	—	—	None
Curb Radius (m)	—	—	—	—	—	—	—	—	—	—	—	—
Add Lanes (#)	—	—	—	—	—	—	—	—	—	—	—	—
Lane Utilization Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Right Turn Factor	—	1.000	—	—	1.000	—	—	1.000	—	—	1.000	—
Left Turn Factor (prot)	—	1.000	—	—	1.000	—	—	1.000	—	—	1.000	—
Saturated Flow Rate (prot)	—	1900	—	—	1900	—	—	1900	—	—	1900	—
Left Turn Factor (perm)	—	1.000	—	—	1.000	—	—	1.000	—	—	1.000	—
Right Ped Bike Factor	—	1.000	—	—	1.000	—	—	1.000	—	—	1.000	—
Left Ped Factor	—	1.000	—	—	1.000	—	—	1.000	—	—	1.000	—
Saturated Flow Rate (perm)	—	1900	—	—	1900	—	—	1900	—	—	1900	—
Right Turn on Red?	—	—	☑	—	—	☑	—	—	☑	—	—	☑
Saturated Flow Rate (RTOR)	—	0	—	—	0	—	—	0	—	—	0	—
Link Is Hidden	—	☐	—	—	☐	—	—	☐	—	—	☐	—
Hide Name in Node Title	—	☐	—	—	☐	—	—	☐	—	—	☐	

Figure 8: Lane settings window in Synchro.

The volume settings window shares some parameters with the lane settings window such as number of lanes, lane configuration and traffic volume. Other mandatory parameters include conflicting pedestrians, conflicting bicycles, peak hour factor, heavy vehicles, bus blockage, adjacent parking lane, and parking maneuvers. Input parameters in lane settings window and volume settings window are used to determine the adjusted saturation flow rate. Figure 9 shows the volume settings window in Synchro.

In the node setting part of timing setting window, the only parameter to be entered is the control type. Other fields are automatically filled based on input in different setting windows. The mandatory parameters to be input in the timing settings window for each approach include turning type (protected, permitted… etc.) and phase assignment. Next, the split times are entered (Total Split = green + yellow + all red time). The minimum and maximum split times are needed only be provided for actuated signal control, which is not the case in this example. The lost time adjustment equals the yellow time reduced by the movement lost time. Lagging Phase is used to decide phase order for protected phase and protected + permitted phase. Allow Lead/ Lag Optimization is checked if the analyst would like to optimize phase order later. Figure 10 shows the timing settings window in Synchro.

VOLUME SETTINGS	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lanes and Sharing (#RL)		↑			↑			↑			↑	
Traffic Volume (vph)	0	500	0	0	500	0	0	500	0	0	500	0
Conflicting Peds. (#/hr)	0	—	0	0	—	0	0	—	0	0	—	0
Conflicting Bicycles (#/hr)	—		0	—	—	0	—	—	0	—	—	0
Peak Hour Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Growth Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Heavy Vehicles (%)	0	0	0	0	0	0	0	0	0	0	0	0
Bus Blockages (#/hr)	0	0	0	0	0	0	0	0	0	0	0	0
Adj. Parking Lane?	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
Parking Maneuvers (#/hr)	—	—	—	—	—	—	—	—	—	—	—	—
Traffic from mid-block (%)	—	0	—	—	0	—	—	0	—	—	0	—
Link OD Volumes	—	—	—	—	—	—	—	—	—	—	—	—
Adjusted Flow (vph)	0	500	0	0	500	0	0	500	0	0	500	0
Traffic in shared lane (%)	—	—	—	—	—	—	—	—	—	—	—	—
Lane Group Flow (vph)	0	500	0	0	500	0	0	500	0	0	500	0

Figure 9: Volume settings window in Synchro.

Timing settings window and phasing settings window share several fields and these do not need to be filled again. Other parameters to be entered include walk time, flash do not walk time, and pedestrian calls, which is the summation of the pedestrian volume for the associated phase. Pedestrian calls is important for actuated signal control, which is not the case for this example. Figure 11 shows phasing settings window in Synchro.

Simulation settings are mostly used for three-dimensional animation, which is a nice feature of Synchro. Typically, these settings do not affect the estimation of measures of effectiveness. Detector settings are beyond the scope of this example; however, they are important for actuated signals. Most of the fields to be filled related to the detector settings are self-explanatory.

TIMING SETTINGS	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR	PED	HOLD
Lanes and Sharing (#RL)		↑			↑			↑			↑		—	—
Traffic Volume (vph)	0	500	0	0	500	0	0	500	0	0	500	0	—	—
Turn Type	—	—	—	—	—	—	—	—	—	—	—	—	—	—
Protected Phases	—	4	—	—	4	—	—	6	—	—	6	—		
Permitted Phases	—		—	—		—	—		—	—		—	—	—
Detector Phases	—	4	—	—	4	—	—	6	—	—	6	—	—	—
Switch Phase	—	0	—	—	0	—	—	0	—	—	0	—	—	—
Leading Detector (m)	—	10.0	—	—	10.0	—	—	10.0	—	—	10.0	—	—	—
Trailing Detector (m)	—	0.0	—	—	0.0	—	—	0.0	—	—	0.0	—	—	—
Minimum Initial (s)	—	1.0	—	—	1.0	—	—	1.0	—	—	1.0	—	—	—
Minimum Split (s)	—	30.0	—	—	30.0	—	—	30.0	—	—	30.0	—	—	—
Total Split (s)	—	30.0	—	—	30.0	—	—	30.0	—	—	30.0	—	—	—
Yellow Time (s)	—	3.0	—	—	3.0	—	—	3.0	—	—	3.0	—	—	—
All-Red Time (s)	—	1.0	—	—	1.0	—	—	1.0	—	—	1.0	—	—	—
Lost Time Adjust (s)	—	0.0	—	—	0.0	—	—	0.0	—	—	0.0	—	—	—
Lagging Phase?	—	—	—	—	—	—	—	—	—	—	—	—	—	—
Allow Lead/Lag Optimize?	—	—	—	—	—	—	—	—	—	—	—	—	—	—
Recall Mode	—	Max	—	—	Max	—	—	Max	—	—	Max	—	—	—
Actuated Effct. Green (s)	—	26.0	—	—	26.0	—	—	26.0	—	—	26.0	—	—	—
Actuated g/C Ratio	—	0.43	—	—	0.43	—	—	0.43	—	—	0.43	—	—	—
Volume to Capacity Ratio	—	0.61	—	—	0.61	—	—	0.61	—	—	0.61	—	—	—
Control Delay (s)	—	17.0	—	—	17.0	—	—	17.0	—	—	17.0	—	—	—
Queue Delay (s)	—	0.0	—	—	0.0	—	—	0.0	—	—	0.0	—	—	—
Total Delay (s)	—	17.0	—	—	17.0	—	—	17.0	—	—	17.0	—	—	—
Level of Service	—	B	—	—	B	—	—	B	—	—	B	—	—	—
Approach Delay (s)	—	17.0	—	—	17.0	—	—	17.0	—	—	17.0	—	—	—
Approach LOS	—	B	—	—	B	—	—	B	—	—	B	—	—	—
Queue Length 50th (m)	—	42.3	—	—	42.3	—	—	42.3	—	—	42.3	—	—	—
Queue Length 95th (m)	—	70.2	—	—	70.2	—	—	70.2	—	—	70.2	—	—	—
Stops (vph)	—	369	—	—	369	—	—	369	—	—	369	—	—	—
Fuel Used (l/hr)	—	27	—	—	33	—	—	26	—	—	27	—	—	—
Dilemma Vehicles (#/hr)	—	0	—	—	0	—	—	0	—	—	0	—	—	—

Figure 10: Timing settings window in Synchro.

PHASING SETTINGS	4-EBWB	6-NBSB
Minimum Initial (s)	1.0	1.0
Minimum Split (s)	30.0	30.0
Maximum Split (s)	30.0	30.0
Yellow Time (s)	3.0	3.0
All-Red Time (s)	1.0	1.0
Lagging Phase?	—	—
Allow Lead/Lag Optimize?	—	—
Vehicle Extension (s)	3.0	3.0
Minimum Gap (s)	3.0	3.0
Time Before Reduce (s)	0.0	0.0
Time To Reduce (s)	0.0	0.0
Recall Mode	Max	Max
Pedestrian Phase	☑	☑
Walk Time (s)	5.0	5.0
Flash Dont Walk (s)	11.0	11.0
Pedestrian Calls (#/hr)	0	0
Dual Entry?	☑	☑
Fixed Force Off?	☑	☑
90th %ile Green Time (s)	26 mr	26 cd
70th %ile Green Time (s)	26 mr	26 cd
50th %ile Green Time (s)	26 mr	26 cd
30th %ile Green Time (s)	26 mr	26 cd
10th %ile Green Time (s)	26 mr	26 cd

Figure 11: Phasing settings window in Synchro.

3.4 Estimation of Intersection Delay and Queue Length Using AVL APC Data

Hellinga et al. (2011) proposed a methodology to estimate delay and queue length experienced by transit vehicle at signalized intersections using AVL/APC data collected by transit vehicles. It is assumed that the delay experienced by transit vehicles is representative of the delay experienced by all vehicles because the location and the duration of time that vehicles stop in the queues caused by traffic signals are independent of vehicle types (except when a transit vehicle makes an stop only to serve the passengers). The methodology is applicable only to intersections which do not have a near side transit stop. The methodology is refined and explained with more details in Yang's thesis (2012).

AVL/APC data contain the time and location of stop event data. Stop events are categorized into scheduled and unscheduled stops. Scheduled stops occur at the bus stop or its close proximity for serving passengers. Unscheduled stops may occur due to several reasons such as traffic signals, congestion, on-street parking maneuvers by other vehicles, and road geometry. However, AVL/APC data do not provide any information regarding the type and cause of the unscheduled stop event.

Considering an urban transportation network with AVL/APC data available for each bus route, AVA/APC data points were allocated to certain segments along corridors, and the distance from the downstream stop line was associated to each stop point. A boundary line is fitted for each segment to differentiate unscheduled stops caused by downstream traffic signal control from other unscheduled stops affected by other causes. The delay measurements are then estimated for the stops caused by downstream traffic signal control. The following steps were proposed by Hellinga et al. (2011) to estimate the delay and queue length at each signalized intersection using AVL/APC data:

Step 1: Define route segments. Each route segment is bounded by a signalized intersection at upstream and downstream ends.

Step 2: For each stop event within the defined segment, obtain the stopped delay and its associated distance from the downstream intersection.

Step 3: Plot stopped delay versus distance for each route segment as in the schematic diagram in Figure 12. Then remove the scheduled stops.

Step 4: Define "mileposts" and fit boundary line candidates connecting the mileposts on the diagram obtained in Step 3 as in Figure 13.

Step 5: Choose the optimum boundary line from the candidates obtained in Step 4 to divide stop events into unscheduled stops due to downstream traffic signal, and other types of unscheduled stops as in Figure 14.

Step 6: The delay envelop boundary identifies unscheduled stop events that occurred due to the traffic signal downstream of a particular route segment. Once relevant unscheduled stop events are identified, various performance measures such as average delay, 95th percentile delay, and maximum queue length can be estimated for a particular approach of an intersection.

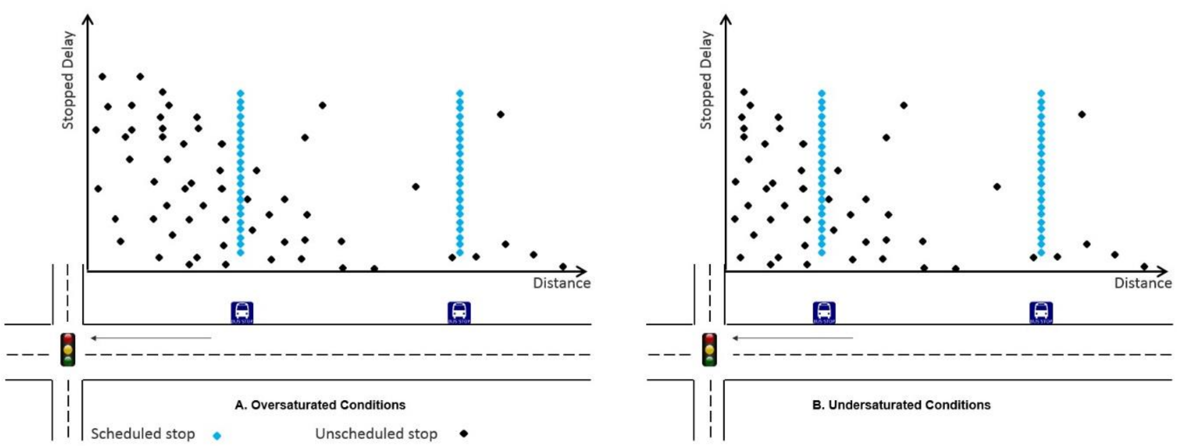


Figure 12: Recorded stops along defined road segment.

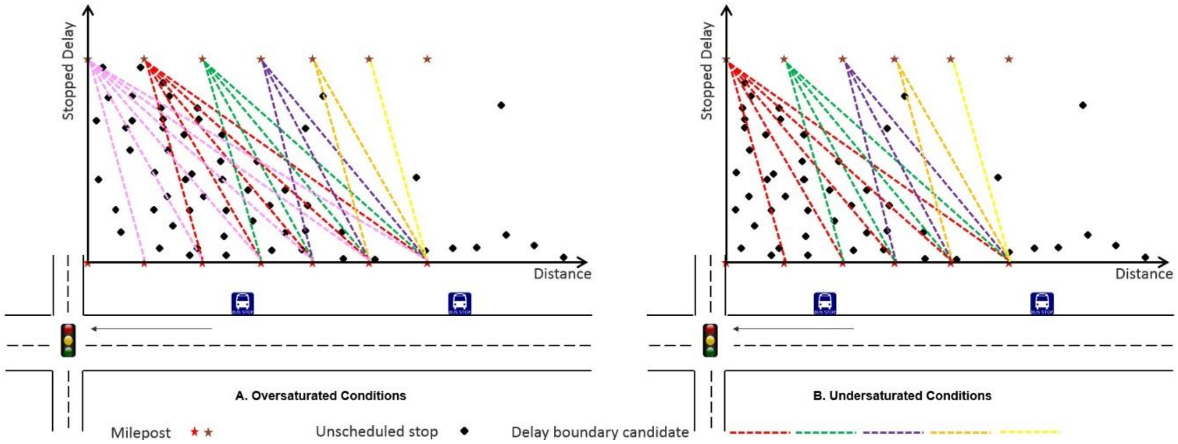


Figure 13: Boundary line candidates connecting mileposts.

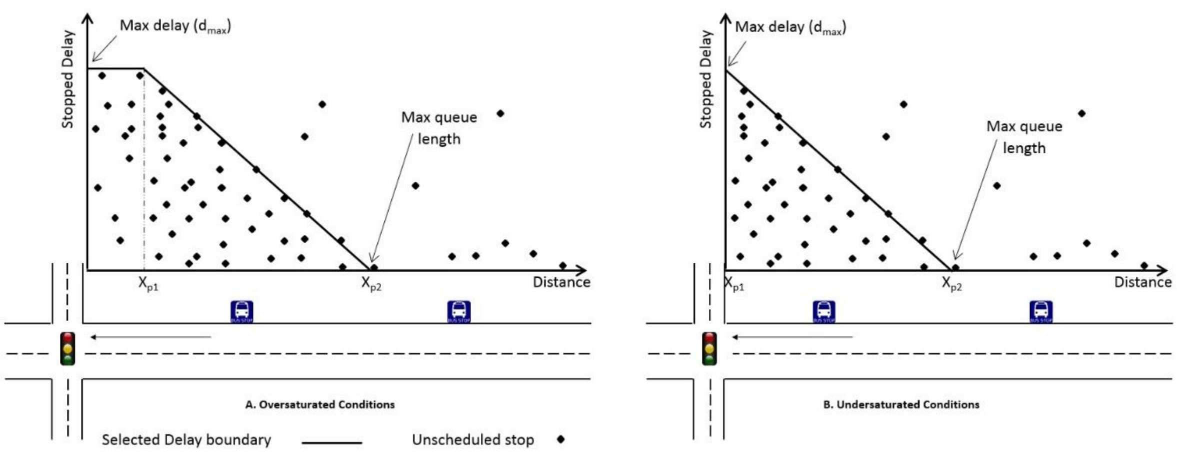


Figure 14: Selected boundary line dividing stop events into stops due to traffic signal and due to other causes.

As shown in Figure 14, the delay envelope boundary in Step 5 is represented by a piece-wise linear function with intercept d_{max} which is the maximum stopped delay that occurs when a vehicle stops at the stop line at the end of the green interval. Equation (16) defines the locus of the delay envelop boundary.

$$d = \begin{cases} d_{\max} & 0 \le x \le X_{p1} \\ a + bx & x > X_{p1} \end{cases} \tag{16}$$

Where, d and x are delay and the distance to the stop line, respectively. If X_{p1} equals zero, the intersection is under-saturated and the delay envelope boundary is linear. However, if X_{p1} is greater than zero, the intersection is oversaturated and the delay boundary envelope is piece-wise linear. A series of candidate boundary lines connecting X_{p1} and X_{p2} within the feasible region are evaluated. Candidate boundary lines are created by connecting feasible "milepost" as demonstrated in Figure 13. For each candidate boundary line, the density of stopped delay event (*DS*) is defined as the cumulative number of stopped delay observations (N_s) divided by the area defined by the delay envelope boundary line (*A*) i.e. $DS = N_s / A$.

The optimum delay envelop boundary line is the one which minimizes the change in the density of stopped delay observations contained within the boundary line. X_{p2} represents the maximum queue length, and average delay is calculated as the summation of stopped delay observations contained within the optimum boundary line divided by the total number of service trips passing the intersection approach during the analysis period.

The GPS unit on-board the transit vehicle is designed essentially for use in transit scheduling and route planning. For these applications, the GPS estimates are likely accurate enough. However, here it is used to estimate delays and queues in real-time, and depending on the nature of the on-board GPS unit, error in location can occur. This is especially important where enroute land use and traffic conditions affect the number and position of satellites in view (e.g. tall buildings near the intersection, vehicle occlusion, etc.). Erroneous GPS data will appear as outliers in AVL data. The proposed methodology accounts for outliers by fitting the envelope boundary line to observations for each intersection, which filters out unscheduled stops and outliers that are not legible or not acceptable.

3.5 The Relationship between Stopped Delay and Control Delay

Synchro calculates control delay (*D*) at signalized intersections. However, the delay estimated using AVL/APC data represents stopped delay (D_s). To compare analysis results, it is necessary to apply a factor (r_d) to convert stopped delay to control delay or vice versa as presented in Equation (17).

$$D = r_d \times D_s \tag{17}$$

Figure 15 demonstrates the difference between control delay and stopped delay. Stopped delay refers to additional travel time experienced by a vehicle when stopping in the queue upstream of a signalized intersection (e.g. P_2 to P_3). Control delay consists of the stopped delay and the additional travel times due to deceleration and acceleration before and after stopping in the queue (e.g. P_1 to P_4).

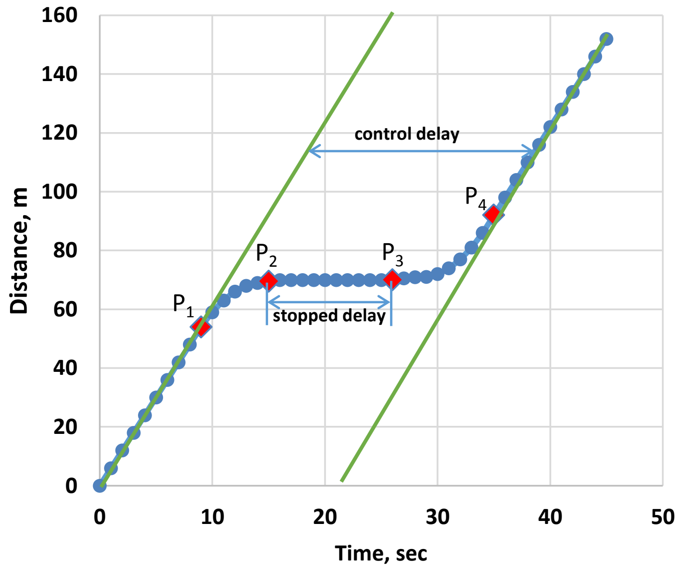


Figure 15: Control delay and stopped delay defined on a vehicle trajectory.

In Figure 15, points before P_1 represents the trajectory of the vehicle when it is at free flow speed and therefore the trajectory has a constant slope. Points between P_1 and P_2 represent the trajectory of the vehicle when it is decelerating and therefore the trajectory has a decreasing slope. Points between P_2 and P_3 represent the trajectory of the vehicle when it is not moving and therefore the trajectory has a zero slope. Points between P_3 and P_4 represent the trajectory of the vehicle when it is accelerating and therefore the trajectory has an increasing slope. Points after P_4 represents the trajectory of the vehicle when it is at free flow speed again and therefore the trajectory has a constant slope.

Synchro user manual (Trafficware, 2011) suggests that control delay is generally 1.3 times the stopped delay. However, as denoted by other researchers (Mousa, 2002), control delay might be up to 2 times the stopped delay. Other studies found that this factor should be variable rather than just a constant value (Olszewski, 1993), (Quiroga and Bullock, 1999).

The stopped delay estimated from AVL/APC data is based on the movements of transit vehicles in the network. Transit vehicles have different acceleration and deceleration capabilities compared to other vehicles. Limitations in the performance and maneuverability of transit vehicles influence the relationship between the control delay and stopped delay estimated from AVL/APC.

The stopped delay obtained from AVL/APC data is assumed to be representative of the whole traffic including heavy vehicles and passenger cars. However, control delay differs based on the physical characteristics of the vehicles, which is adjusted by applying the conversion factor (r_d) in equation (1). Figure 16 illustrate the trajectories of a bus and a passenger car. Although control delay is clearly different for each vehicle, the stopped delay is the same.

A study was conducted to collect field data to reconstruct vehicle trajectories at signalized intersections to investigate the relationship between stopped delay and control delay estimated using different vehicle types. GPS loggers were used to collect the time and location data of different vehicle types while traveling through signalized intersections. GPS loggers record the time stamp and location coordinates at every second.

Using collected GPS data, trajectories of 30 buses and 30 passenger cars were reconstructed. The time-space diagram in Figure 17 shows sample bus and passenger car trajectories. Control delay and stopped delay were estimated for each vehicle trajectory considering the definitions provided using the schematic vehicle trajectory in Figure 15. Consequently, the ratio of control delay to stopped delay (D/D_s) was estimated for each vehicle trajectory. Table 1 shows the total delay, the stopped delay, and the ratio of control delay to stopped delay (D/D_s) for each trajectory.

Table 2 shows the average and variance of D/D_s ratios estimated for buses and passenger cars. The average D/D_s ratio for passenger cars is 1.3 while for buses the average ratio is 1.4. As shown in Table 2, statistical tests were applied to demonstrate whether the difference between average D/D_s ratio for passenger cars and buses was statistically significant.

The F-test results show that two samples have equal variance at 95% confidence level. However, T-test results demonstrate that two samples have different means at 95% confidence level. Consequently, D/D_s ratio of 1.4 was used in this study.

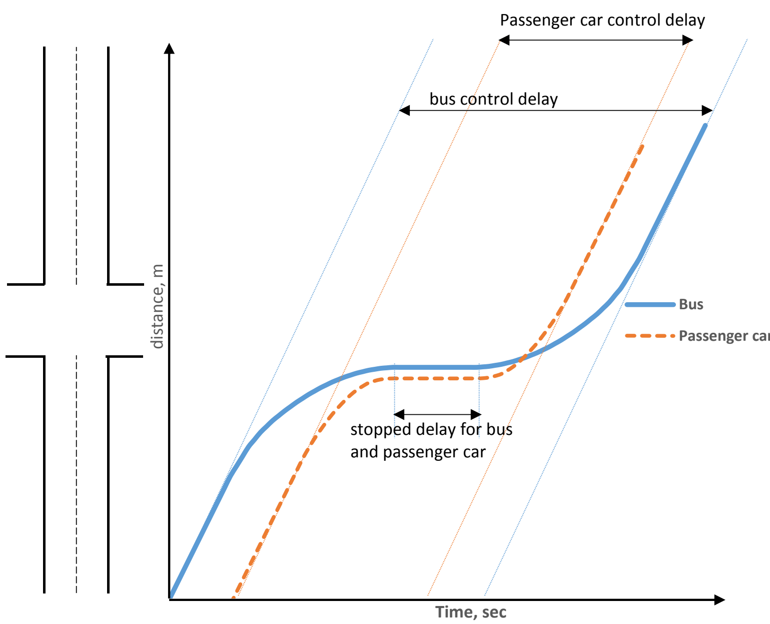


Figure 16: Control delay and stopped delay for bus and passenger car

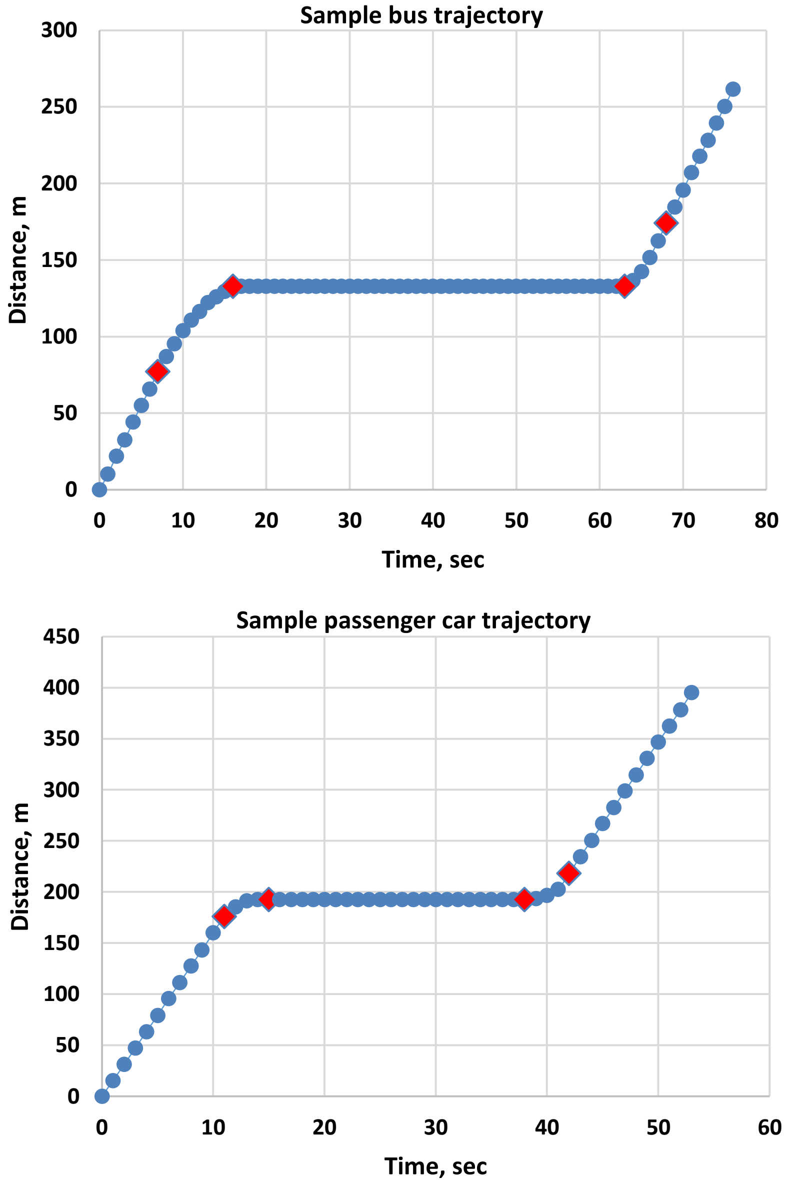


Figure 17: Sample bus and passenger car trajectories.

Table 1: Observed total delay and stopped delay for trips using passenger cars and buses.

#	Passenger Cars			Bus		
	D Sec.	D_s Sec.	$r_d=D/D_s$	*D* Sec.	D_s Sec.	$r_d=D/D_s$
1	29	21	1.3810	38	27	1.4074
2	31	23	1.3478	46	31	1.4839
3	27	23	1.1739	87	66	1.3182
4	112	96	1.1667	29	19	1.5263
5	9	6	1.5000	48	34	1.4118
6	57	40	1.4250	57	40	1.4250
7	25	18	1.3889	48	36	1.3333
8	99	84	1.1786	59	45	1.3111
9	31	25	1.2400	34	24	1.4167
10	39	30	1.3000	49	36	1.3611
11	19	16	1.1875	73	56	1.3036
12	9	7	1.2857	39	28	1.3929
13	39	37	1.0541	40	28	1.4286
14	98	68	1.4412	61	45	1.3556
15	27	21	1.2857	44	35	1.2571
16	27	19	1.4211	59	45	1.3111
17	25	14	1.7857	26	17	1.5294
18	44	41	1.0732	46	34	1.3529
19	48	45	1.0667	47	32	1.4688
20	23	21	1.0952	60	44	1.3636
21	58	42	1.3810	19	10	1.9000
22	25	18	1.3889	37	28	1.3214
23	99	84	1.1786	43	32	1.3438
24	27	21	1.2857	61	47	1.2979
25	39	30	1.3000	43	33	1.3030
26	15	11	1.3636	37	28	1.3214
27	16	10	1.6000	42	31	1.3548
28	39	37	1.0541	35	24	1.4583
29	98	90	1.0889	46	33	1.3939
30	30	21	1.4286	29	19	1.5263
Average			**1.2956**			**1.3993**
STD			**0.7228**			**0.1199**

Table 2: Evaluation of the relationship between control delay and stopped delay considering different vehicle types.

	Passenger Car Trips	Bus Trips
Mean of D/D_s	1.296	1.399
Variance	0.030	0.014
Number of samples	30	30

Two-tailed F-test	Two-tailed T-test (equal variance))
Confidence level= 95% (α=0.05)	Confidence level: 95% (α=0.05)
Degree of freedom of the first sample (df_1)= 29	Degree of freedom of the first sample (df)= 58
Degree of freedom of the second sample (df_2)= 29	$T(1- \alpha/2, df)$= -2.002
$F(1- \alpha/2, df_1, df_2)$= 0.476	$T(\alpha/2, df)$= 2.002
$F(\alpha/2, df_1, df_2)$= 2.101	Rejection region: F> 2.002 or F< -2.002
Rejection region: F> 2.101 or F<0.475	Test Statistics = 2.707**
Test Statistics = 2.065*	
* *The F test statistics is not in rejection areas. Therefore, no evidence to conclude that variances are different.*	
** *The T test statistics is in rejection areas indicating the two samples have different means at 95% confidence level.*	

3.6 Available Data

The study was conducted on 28 major signalized intersections in the Region of Waterloo in Southern Ontario. Grand River Transit (GRT), who is the provider of transit services in the Region of Waterloo, provided the AVL/APC data for this research. These data included AVL/APC records collected in September, October, November, and December 2011 and 2013. The delay and queue length were estimated for 29 approaches at these selected signalized intersections using AVL/APC data by applying the methodology proposed by Hellinga et al. (2011). The analysis period was the PM peak period (4:30 PM to 6:00 PM) on non-holiday weekdays. Table 3 shows AVL/APC data records and estimated mean delay and maximum queue length. Figure 18 shows a map of the Region of Waterloo with the considered intersections highlighted. Each intersection is marked with its unique identification number from Table 3.

As shown in Figure 19 for two sample intersections, a series of candidate boundary lines were considered to separate unscheduled stop events caused by traffic signals from other types of stop events. The optimum boundary line was selected considering the change in the density of stop events contained within the boundary line.

Table 3: AVL/APC data records

Intersection Id	Route	Upstream intersection	Down Stream Intersection	Movement *	Data Obtained in	Mean Delay, Sec	STD, Sec	Proportion of Trip with delay	Total number of trips	90 Percentile Delay, Sec	Max Queue Length, m	Max Delay, Sec	Portion of stops under boundary line
1	52	DUNDAS AT Easton	HESPELER And WATER AT Coronation And Dundas	T	2011	50.2	38	85.2%	102	127.0	127	325	98.5%
2	52	KING AT River	FAIRWAY AT King	L	2013	39.4	35	69.9%	249	91.0	106	142	97.2%
3	11	OTTAWA AT Alpine	OTTAWA AT Homer Watson	T	2011	39.1	24	84.5%	64	189.0	189	268	81.0%
4	51	PINEBUSH AT Walmart & Home Depot	HESPELER AT Eagle And Pinebush	L	2011	39.0	44	57.6%	101	102.0	102	280	92.9%
5	5	ERB AT Beechwood And Gateview	FISCHER HALLMAN AT Thorndale	R	2011	37.8	33	75.6%	87	76.0	76	258	65.1%
6	16	HOMER WATSON AT Doon South Rd	HOMER WATSON AT Conestoga College	L	2011	36.6	41	57.2%	98	121.0	121	134	94.8%
7	10	DOON VILLAGE AT Pioneer	HOMER WATSON AT Manitou And Doon Village	T	2011	34.4	25	80.6%	69	91.0	91	201	88.6%
8	10	WILSON AT Kingsway	FAIRWAY AT Wilson	T	2011	32.1	28	71.0%	65	120.0	120	258	60.5%
9	8	Terminal	FAIRWAY AT Fairview Park Mall	L	2011	32.1	28	34.0%	258	162.0	162	194	59.3%
10	15	LACKNER AT Keewatin	VICTORIA AT Natchez	L	2011	31.7	28	76.6%	73	76.0	76	234	70.9%
7	10	MANITOU AT Wabanaki	HOMER WATSON AT Manitou And Doon Village	T	2013	28.2	28	56.1%	157	66.0	106	73	51.2%
11	200	SHELDON AT Conestoga	PINEBUSH AT Conestoga	T	2011	26.0	31	55.0%	77	91.0	91	463	72.4%
12	7	Terminal	KING AT Conestoga Mall	L	2013	18.5	21	37.3%	427	83.4	107	102	96.0%
13	51	HOLIDAY INN AT Groh	QUEEN AT Goebel	L	2011	23.1	22	66.1%	53	164.0	164	389	77.1%
14	23	Terminal	CHARLES AT Ontario	T	2011	21.1	21	51.7%	145	54.2	76	65	60.3%
15	35	BRIDGEPORT AT Ellis	WEBER AT Bridgeport	T	2011	19.8	19	58.9%	45	74.0	74	160	90.8%
16	5	ERB AT Roslin	WESTMOUNT AT Erb	T	2011	19.3	20	73.0%	125	56.8	118	75	48.8%
17	9	WEBER AT Albert	WEBER AT Parkside	L	2011	18.1	20	51.9%	46	76.0	76	201	60.2%
18	53	MAIN AT Elgin	DUNDAS AT Main	T	2011	17.6	21	54.7%	52	90.0	90	131	94.7%
19	21	KING AT Home Depot	KING AT Northfield	T	2011	25.6	21	77.4%	54	119.0	119	147	93.9%
20	8	HIGHLAND AT Belmon	VICTORIA AT Belmont	T	2011	9.1	11	26.8%	220	30.0	76	70	63.6%
21	5	ERB AT Amos	ERB AT Fischer Hallman	R	2013	9.0	1	2.1%	146	45.5	30	50	70.0%
22	200	WEBER AT Parkside	NORTHFIELD AT Parkside	R	2013	4.0	9	21.4%	393	17.0	76	46	73.0%
23	200	HESPELER AT CanAmera and YMCA	HESPELER AT Dunbar	R	2013	3.9	9	19.6%	393	17.6	103	45	82.8%
24	35	BRIDGE AT Lexington	NORTHFIELD AT Bridge	L	2011	10.9	13	62.0%	32	91.0	91	328	43.3%
25	29	FISCHER HALLMAN AT Keatsway	UNIVERSITY AT Keatsway	L	2011	27.5	26	71.1%	67	106.0	106	207	90.3%
26	13	WESTMOUNT AT Columbia	FISCHER HALLMAN AT Columbia	T	2011	26.8	19	75.7%	51	273.0	91	250	69.6%
27	13	FISCHER HALLMAN AT Columbia	WESTMOUNT AT Columbia	T	2011	25.7	21	74.0%	57	91.0	273	185	94.6%
28	201	FISCHER HALLMAN AT Thorndale	FISCHER HALLMAN AT University	T	2011	23.6	23	68.7%	63	89.0	89	226	97.8%

* T: Through movement; L: Left turn; R: Right turn.

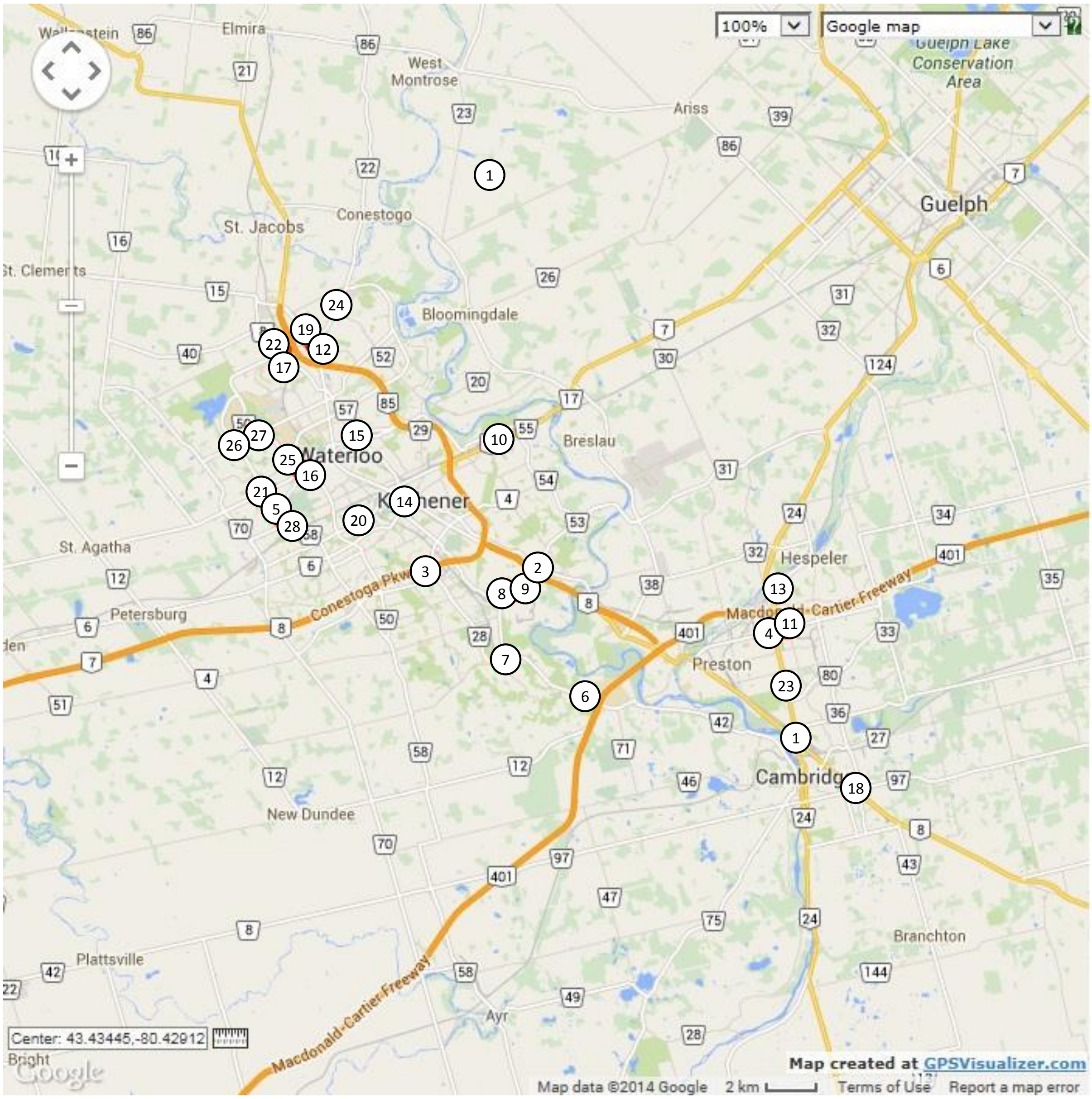


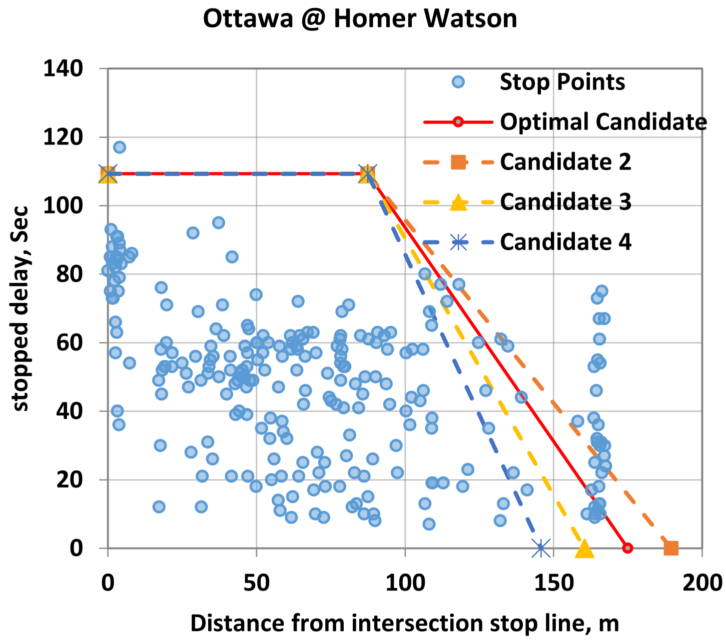
Figure 18: Intersections considered in the study. (Source: GPSVisualizer.com)

The Transportation and Environmental Services at the Regional Municipality of Waterloo provided the necessary data for modeling the selected intersections using Synchro for the same analysis periods in 2011 and 2013. The provided data included the turning movement counts, traffic signal timing parameters, ratio of heavy vehicles, conflicting pedestrian counts, ideal saturation flow rate, and peak hour factors. Intersection geometries were assessed using Google Maps and through site visits. The delay and queue length at selected approaches of the signalized intersections were computed using Synchro and the results were compared with corresponding values estimated from AVL/APC data.

Appendix A provides intersection information such as name, defined segment, and the bus route. It also includes lane configuration, turning movement counts, year in which counts collected, pedestrian counts, peak hour factors, ratio of heavy vehicles, and ideal saturation flow rate. Considered movement (lane group) is highlighted for each intersection. Appendix B includes signal-timing settings for each intersection.

Similar to other jurisdictions, the region does not collect turning movement counts data for an intersection each year. Consequently, the year of collection of the turning movement counts data varies across the intersections. In order to provide an unbiased comparison, the delay and queue length estimates were selected from either the 2011 or 2013 AVL/APC data which ever most closely corresponded with the year at which the turning movement counts data were collected.

Appendix C and D provides the associated AVL/APC data. Appendix C includes the recorded unscheduled stop events for each defined segment plotted against the distance from the downstream intersection. The selected boundary line is also fitted for each defined segment. Appendix D includes visual representation of the recorded stop events on the map of each defined segment.



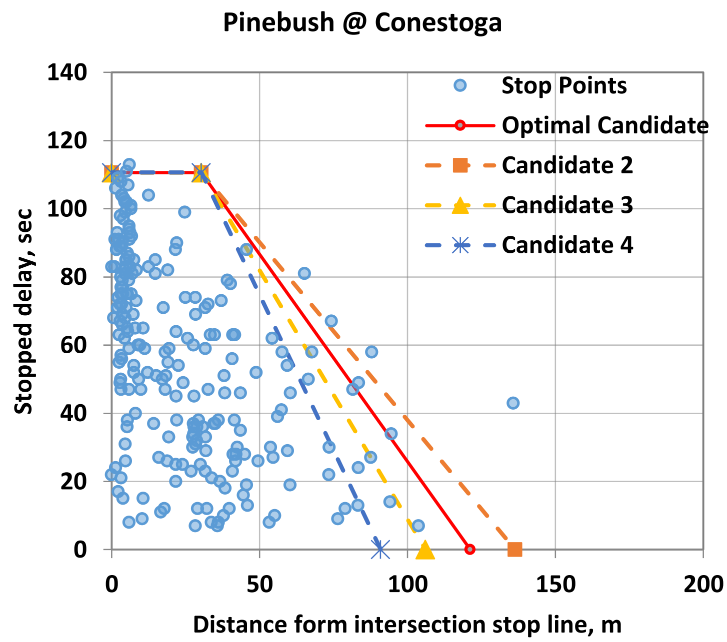


Figure 19: Recorded stop events and different candidate boundary lines for sample intersections

Chapter 4
Results and Discussion

In this chapter, the results from this study are presented and discussed. The mean stopped delay and the maximum queue length obtained from AVL/APC data and from Synchro for the studied intersections are compared and analyzed. Discussion of the causes of differences in results are also presented in this chapter.

4.1 Analysis Results

Table 4 provides the delay and queue length estimates obtained from the AVL/APC data and the Synchro analysis. The table is divided into four groups of columns. The first group of columns is the ID, name, and turning movement of the subject lane group of the intersection approach.

The second group of columns is associated with estimates of mean stopped delay (MSD). The mean stopped delay is obtained directly from the AVL/APC data using the method discussed in Section 3.4.

The mean stopped delay from Synchro is obtained from Equation 17 as:

$$D_s = \frac{D}{r_d}$$

Where D is the control delay provided by Synchro and r_d = 1.4 as calibrated from field data (Section 3.5).

Percentage of change is computed as:

$$\%change = \frac{MSD_{Synchro} - MSD_{AVL/APC}}{MSD_{Synchro}} \times 100\% \tag{18}$$

Difference is computed as:

$$Difference = MSD_{Synchro} - MSD_{AVL/APC} \tag{19}$$

The third group of columns is associated with the maximum extent of the queue. The estimates from the AVL/APC data represents X_p as defined in Section 3.4. The estimates from Synchro are the 95th percentile queue length.

The fourth group pf columns is associated with level of service (LOS). The HCM defines LOS in terms of control delay (Table 5). Control delay was estimated from the AVL/APC data using Equation 17 as:

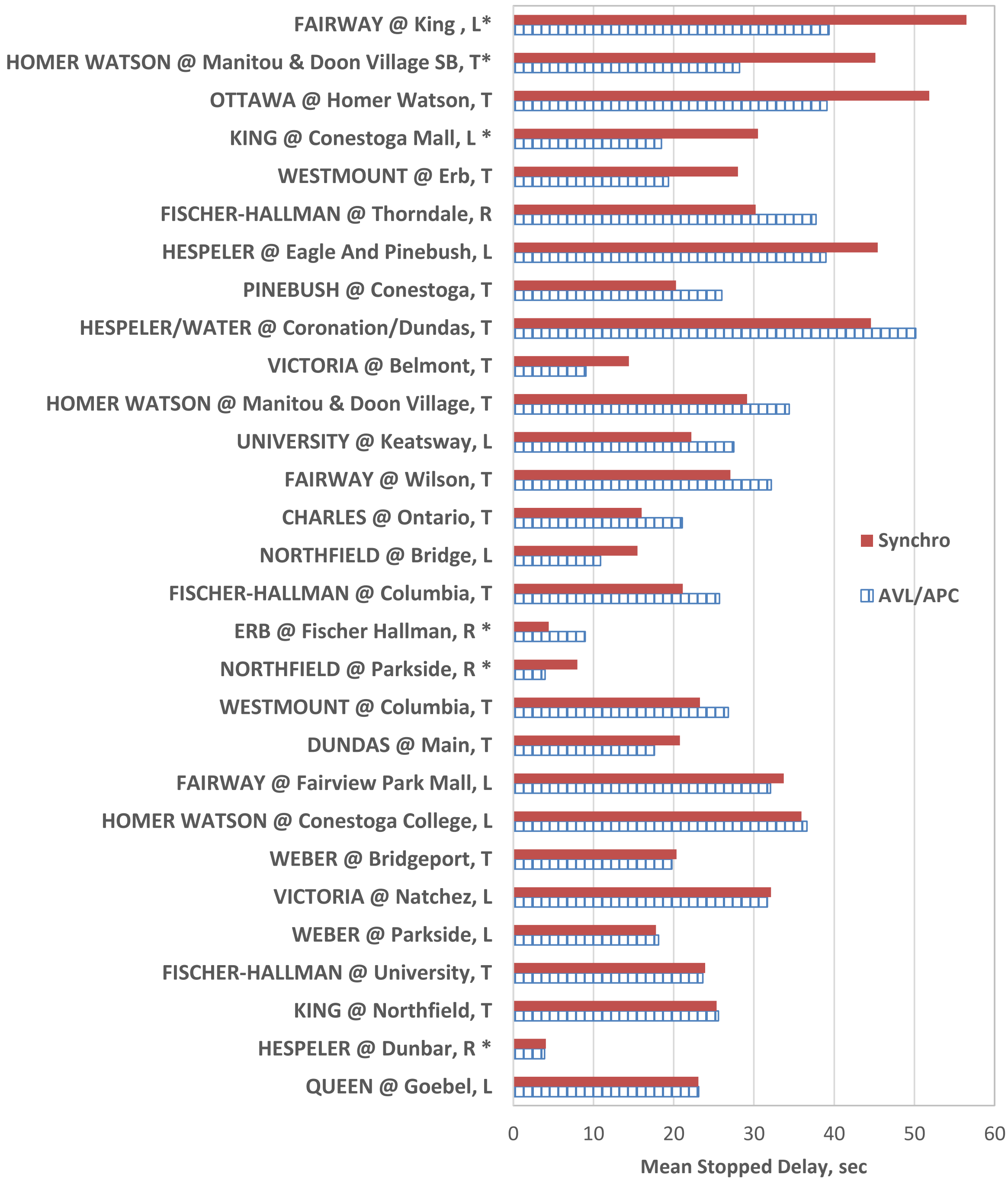
$$D = MSD_{AVL/APC} \times r_d$$

Where $r_d = 1.4$.

Figure 20 and Figure 21 compare the mean stopped delay (MSD) and maximum queue length estimated using AVL/APC data and Synchro (i.e. the data from Table 4). In Figure 20, the approaches are sorted in ascending order of the difference between the mean stopped delay estimated from the AVL/APC data and Synchro.

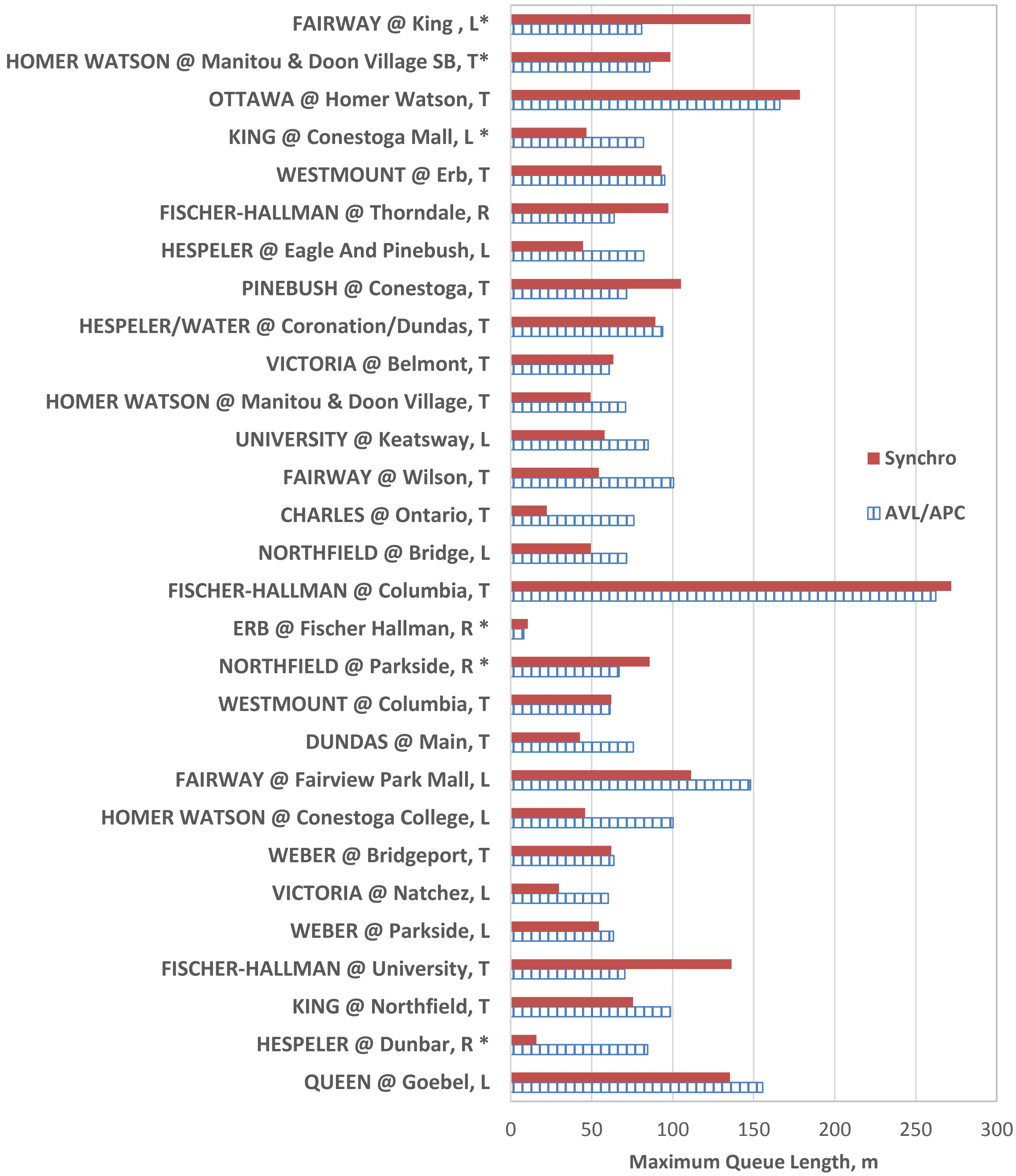
The absolute differences range from almost zero to a maximum of 17 seconds. The approaches with largest differences are Fairway Rd. @ King St., Ottawa St. @ Homer Watson Blvd., and Homer Watson Blvd. @ Manitou Dr. and Doon Village Rd. southbound.

The average estimated stopped delay for all intersections were 25.2 and 26.6 seconds for AVL/APC data and Synchro, respectively. However, estimated maximum queue length from AVL/APC data and Synchro varies drastically at different intersections. The difference in estimated maximum queue length ranges from 0.73 meter to 68.7 meter with largest differences observed at Hespeler Rd. @ Dunbar Rd., Fairway Rd. @ Wilson Dr., and King St. @ Conestoga Mall.



T: Through movement; L: Left turn; R: Right turn.
* 2013 data; approaches without an asterisk based on 2011 data.

Figure 20: Comparison of stopped delay estimated using AVL/APC data and Synchro.



T: Through movement; L: Left turn; R: Right turn.
* 2013 data; approaches without an asterisk based on 2011 data.

Figure 21: Comparison of maximum queue length estimated using AVL/APC data and Synchro.

Table 4: Summary of intersection performance measures estimated from AVL/APC data and Synchro

Intersection ID	Intersection Name (GIS Lay)	movement	Mean Stopped Delay				Maximum Queue Length				Level of Service	
			AVL/APC, Sec.	Synchro, Sec.	% Change	Difference, Sec.	AVL/APC, m.	Synchro, m.	% Change	Difference, m.	AVL/APC	Synchro
1	HESPELER/WATER @ Coronation/Dundas	Through	50.2	44.6	-12.58%	-5.6	93.8	89.3	-5.02%	-4.5	E	E
2	FAIRWAY @ King *	Left	39.4	56.5	30.34%	17.1	80.8	148.0	45.40%	67.2	E	E
3	OTTAWA @ Homer Watson	Through	39.1	51.9	24.56%	12.7	166.1	178.5	6.96%	12.4	D	E
4	HESPELER @ Eagle And Pinebush	Left	39.0	45.4	14.22%	6.5	82.0	44.7	-83.49%	-37.3	D	E
5	FISCHER-HALLMAN @ Thorndale	Right	37.8	30.2	-24.96%	-7.5	63.9	97.3	34.32%	33.4	D	D
6	HOMER WATSON @ Conestoga College	Left	36.6	35.9	-1.84%	-0.7	100.1	46.0	-117.6%	-54.1	D	D
7	HOMER WATSON@Manitou& Doon Village Nb	Through	34.4	29.1	-18.09%	-5.3	70.9	49.3	-43.87%	-21.6	D	D
8	FAIRWAY @ Wilson	Through	32.1	27.1	-18.76%	-5.1	100.4	54.4	-84.58%	-46.0	D	D
9	FAIRWAY @ Fairview Park Mall	Left	32.1	33.7	4.93%	1.7	148.0	111.4	-32.84%	-36.6	D	D
10	VICTORIA @ Natchez	Left	31.7	32.1	1.46%	0.5	60.1	29.7	-102.4%	-30.4	D	D
7	HOMER WATSON@Manitou& Doon Village Sb*	Through	28.2	45.1	37.51%	16.9	85.7	98.6	13.06%	12.9	D	E
11	PINEBUSH @ Conestoga	Through	26.0	20.3	-28.13%	-5.7	71.6	105.2	31.94%	33.6	D	C
12	KING @ Conestoga Mall*	Left	18.5	30.5	39.44%	12.0	82.0	46.8	-75.21%	-35.2	C	D
13	QUEEN @ Goebel	Left	23.1	23.1	-0.18%	0.0	155.6	135.4	-14.90%	-20.2	C	C
14	CHARLES @ Ontario	Through	21.1	16.0	-31.71%	-5.1	76.0	22.3	-240.8%	-53.7	C	C
15	WEBER @ Bridgeport	Through	19.8	20.4	2.85%	0.6	63.7	62.1	-2.58%	-1.6	C	C
16	WESTMOUNT @ Erb	Through	19.3	28.0	30.96%	8.7	95.0	93.1	-2.04%	-1.9	C	D
17	WEBER @ Parkside	Left	18.1	17.8	-1.86%	-0.3	63.3	54.4	-16.35%	-8.9	C	C
18	DUNDAS @ Main	Through	17.6	20.8	15.33%	3.2	75.6	42.8	-76.68%	-32.8	C	C
19	KING @ Northfield	Through	25.6	25.4	-0.82%	-0.2	98.6	75.5	-30.60%	-23.1	D	D
20	VICTORIA @ Belmont	Through	9.1	14.4	37.14%	5.4	60.9	63.3	3.82%	2.4	B	C
21	ERB @ Fischer Hallman*	Right	9.0	4.4	-102.7%	-4.6	8.0	10.5	23.81%	2.5	B	A
22	NORTHFIELD @ Parkside*	Right	4.0	8.0	50.25%	4.0	66.8	85.8	22.09%	19.0	A	B
23	HESPELER @ Dunbar*	Right	3.9	4.1	3.96%	0.2	84.5	15.8	-435.1%	-68.7	A	A
24	NORTHFIELD @ Bridge	Left	10.9	15.5	29.70%	4.6	71.5	49.4	-44.82%	-22.1	B	C
25	UNIVERSITY @ Keatsway	Left	27.5	22.2	-23.68%	-5.3	84.8	58.0	-46.17%	-26.8	D	C
26	WESTMOUNT @ Columbia	Through	26.8	23.3	-15.09%	-3.5	61.4	62.1	1.17%	0.7	D	C
27	FISCHER-HALLMAN @ Columbia	Through	25.7	21.1	-21.60%	-4.6	262.4	272.0	3.52%	9.6	D	C
28	FISCHER-HALLMAN @ University	Through	23.6	23.9	1.19%	0.3	70.4	136.4	48.40%	66.0	C	C
		Average	**25.2**	**26.6**	**21.6%**[+]	**5.1**[++]	**89.8**	**80.6**	**58.3%**[+]	**27.1**[++]		
		Maximum	**50.2**	**56.5**	**103%**[+]	**17.1**[++]	**262.4**	**272.0**	**435.1%**[+]	**68.7**[++]		
		Minimum	**3.9**	**4.1**	**0.2%**[+]	**0.04**[++]	**8.0**	**10.5**	**1.2%**[+]	**0.73**[++]		

* 2013 data; approaches without an asterisk based on 2011 data.

[+] The average, maximum and minimum are based on the absolute relative error.

[++] The average, maximum and minimum are based on the absolute error.

As shown in Table 6, estimated LOS for 16 out of 29 approaches (55%) were identical regardless of the methodology used for computing the control delay. For 8 of the 29 approaches (27%) the LOS estimated by Synchro was worse than the LOS estimated from the AVL/APC data. For 5 of the 29 approaches (17%) the LOS estimated by Synchro was better than the LOS estimated from the AVL/APC data. As evidence by Table 6, at most the LOS estimated from the two methods differed by only a single level.

Table 5: LOS criteria for signalized intersections. Source: HCM (2000)

LOS	Control Delay, sec
A	≤ 10
B	> 10 – 20
C	> 20 – 35
D	> 35 – 55
E	> 55 – 80
F	> 80

Table 6: Comparison of the level of service estimated from AVL/APC data and Synchro.

AVL/APC LOS	Synchro LOS						Total
	A	B	C	D	E	F	
A	1	1					2
B	1		2				3
C			6	2			8
D			4	7	3		14
E					2		2
F							0
Total	2	1	12	9	5	0	29

Figure 22 illustrates the correlation between estimated average stopped delays from Synchro and AVL/APC data. Figure 23 illustrates the correlation between estimated and maximum queue lengths from Synchro and AVL/APC data. In Figure 22, the dashed line is the line of perfect correlation. The Pearson correlation coefficient, R, is equal to 0.85 (Table 7) suggesting a relatively strong positive correlation between the stopped delay estimated from Synchro and stopped delay estimated from the AVL/APC data.

A least squares linear regression was also fit to the data in which the independent variable is the stopped delay from Synchro and the dependent variable is the delay form the AVL/APC data.

$$D_{AVL/APC} = aD_{Synchro} + b \tag{20}$$

Table 7 shows that both *a* and *b* are statistically significant at the 95% level. The fact that *a* is not equal to zero and *b* is not equal to 1.0 suggests some bias in the estimates such that an average AVL/APC delays are larger than Synchro delays when delays are small and less than Synchro delays when delays are large.

Table 7: Results of stopped delay linear regression

Coefficients	Value	Standard Error	P-value
b	5.461 sec.	2.591	0.044
a	0.742	0.088	4.67×10^{-9}
Pearson correlation coefficient, R = 0.851			
Coefficient of determination, R^2 = 0.725			

A similar analysis is done for queue length estimates (Figure 23). The correlation (R = 0.797) is not as strong as for stopped delay. A least squares linear regression was fit to the data in which the independent variable is the queue length from Synchro and the dependent variable is the queue length form the AVL/APC data.

$$Q_{AVL/APC} = aQ_{Synchro} + b \qquad (21)$$

Table 8 shows that the regression results indicate both *a* and *b* are statistically significant. Similar to results for stopped delay, these results indicate a bias. When queue length is short, the AVL/APC estimates are larger than the Synchro estimates. When queue length is larger than approximately 150 m (as indicated by Synchro), the AVL/APC data estimates are smaller than the Synchro estimates.

Table 8: Results of queue length linear regression

Coefficients	Value	Standard Error	P-value
b	36.71 m	9.3112	5.157×10^{-4}
a	0.658	0.096	2.278×10^{-7}
Pearson correlation coefficient, R = 0.797			
Coefficient of determination, R^2 = 0.635			

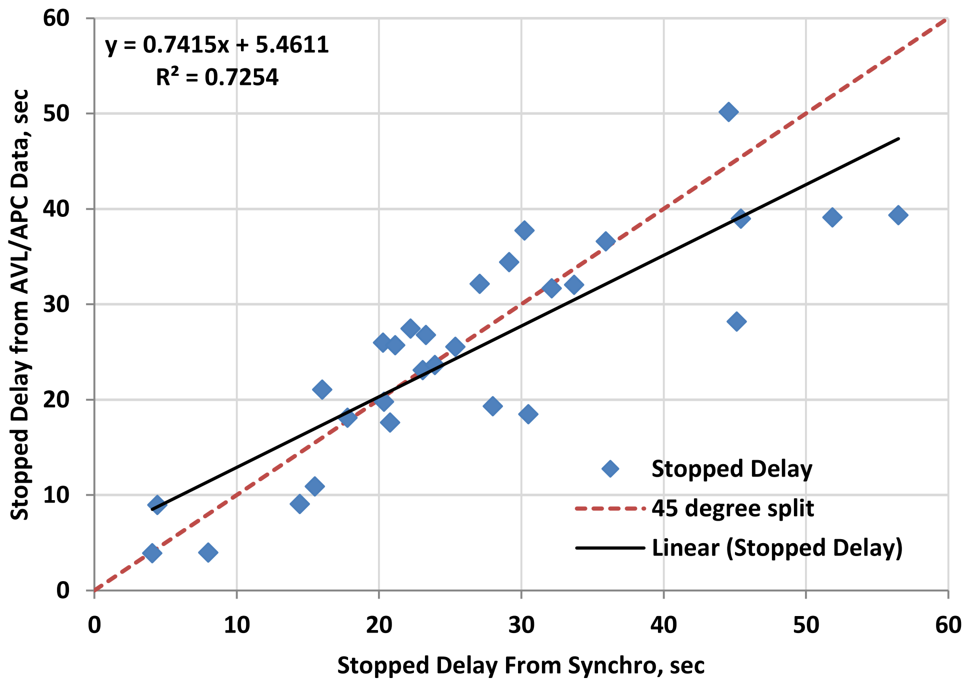


Figure 22: Correlation between stopped delays estimated from AVL/APC data and Synchro.

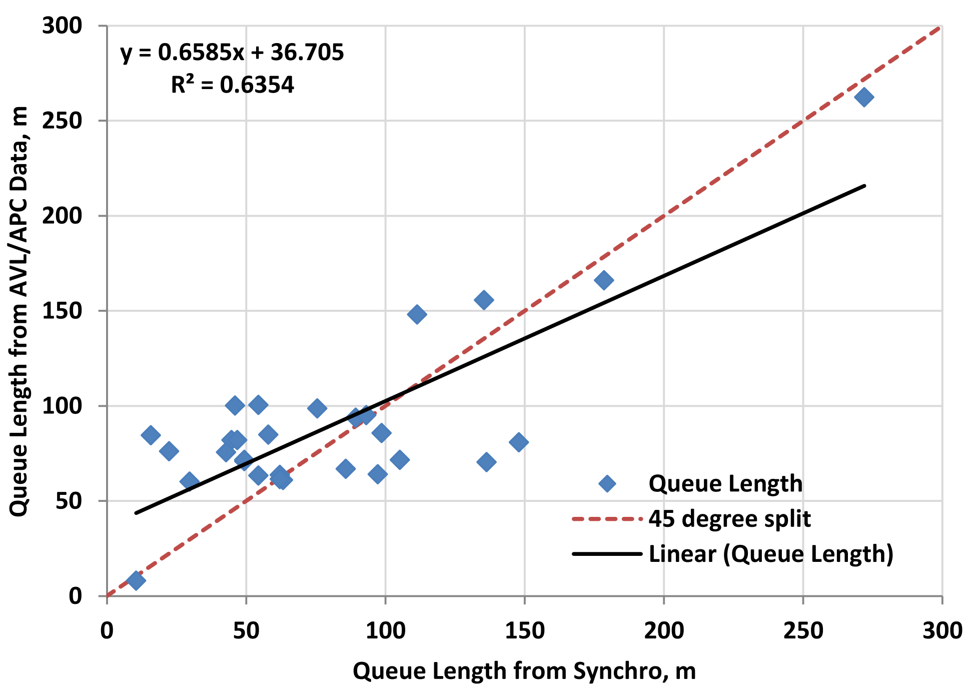


Figure 23: Correlation between queue lengths estimated from AVL/APC data and Synchro

Figure 24 and Figure 25 illustrate the relationship between stopped delay and maximum queue length for the AVL/APC data and Synchro respectively. A least squares linear regression was fit to the AVL/APC data in which the independent variable is the queue length and the dependent variable is the stopped delay.

$$D_{AVL/APC} = aQ_{AVL/APC} + b \tag{22}$$

The coefficient *b* was found to be insignificant; as a result, it was set to zero. The coefficient *a* is statistically significant as shown in Table 9. The Pearson correlation coefficient, R, is equal to 0.87 suggesting a relatively strong positive correlation between the stopped delay and queue length estimated from the AVL/APC data.

Table 9: Results of linear regression for AVL/APC data

Coefficients	Value	Standard Error	P-value
b	0	N/A	N/A
a	0.238	0.0256	5.379×10^{-10}
Pearson correlation coefficient, R = 0.868			
Coefficient of determination, R^2 = 0.753			

Similarly, a least squares linear regression was fit to the Synchro analysis results in which the independent variable is the queue length and the dependent variable is the stopped delay.

$$D_{Synchro} = aQ_{Synchro} + b \tag{23}$$

Similar to AVL/APC data, the coefficient *b* was also found to be insignificant; as a result, it was set to zero. The coefficient *a* is statistically significant as shown in Table 10. The Pearson correlation coefficient, R, is equal to 0.83 suggesting a relatively strong positive correlation between the stopped delay and queue length estimated from the AVL/APC data.

Table 10: Results of linear regression for Synchro analysis results

Coefficients	Value	Standard Error	P-value
b	0	N/A	N/A
a	0.254	0.032	1.053×10^{-10}
Pearson correlation coefficient, R = 0.834			
Coefficient of determination, R^2 = 0.695			

It should be noted that the expected relationship between delay and queue length is not linear. The sensitivity analysis in Section 4.2.1.1 demonstrates the relationship between these measures. Figure 29 illustrates the relationship mean delay and maximum queue length as the degree of saturation (v/c) increases.

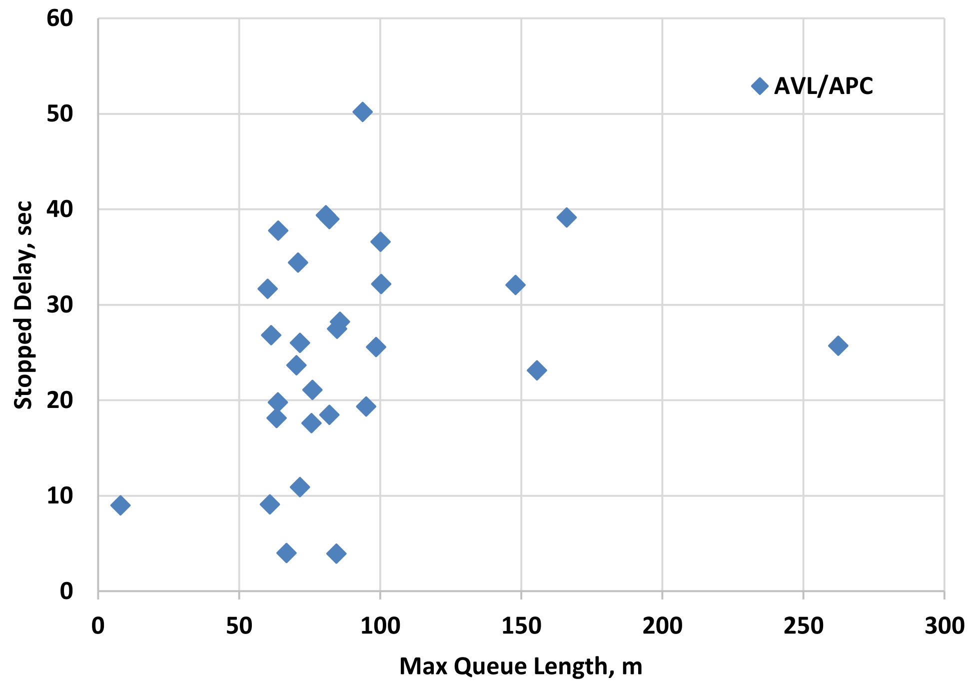


Figure 24: Relationship between stopped delay and queue length for AVL/APC data.

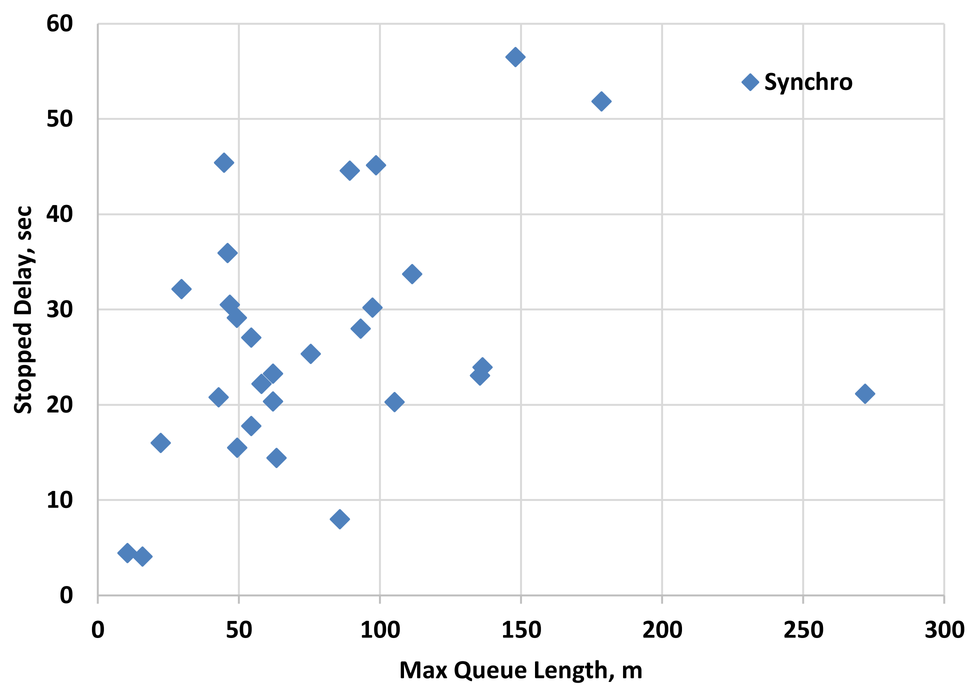


Figure 25: Relationship between stopped delay and queue length for Synchro analysis.

4.2 Discussion

Comparison of estimated average stopped delay and maximum queue length from AVL/APC data and Synchro shows marginal to significant differences. This section discusses some of possible reasons for the differences in the analysis results. For better understanding of the impact of different traffic parameters on average stopped delay and queue length, this section investigates the sensitivity of Synchro estimates of mean stopped delay and maximum queue length to degree of saturation, cycle length, and g/C (green interval duration to cycle length) ratio and validated the results using HCM (2000) methodology.

4.2.1 Sensitivity Analysis

A hypothetical isolated intersection with four single-lane approaches as shown in Figure 26.a was used to investigate the sensitivity of Synchro estimates. The intersection serves only through movement with equal demand on each approach. The saturation flow rate was assumed 1900 vph, and the average vehicle occupancy is eight meters. The duration for analysis period is assumed to be 15 minutes; however, when lane group is oversaturated, Synchro considers the maximum queue length after two cycles. For comparison reason, the analysis period considered in HCM queue length calculation is adjusted to be the equivalent of two cycles only when the lane group becomes oversaturated. The intersection is controlled by pretimed signal with two phases only as shown in Figure 26.b. Section 3.3.4 of this thesis describes the modelling of this intersection.

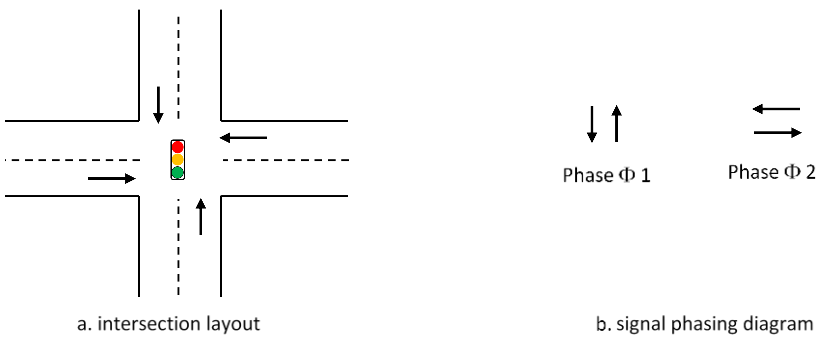


Figure 26: Intersection layout and phasing diagram for sensitivity analysis.

4.2.1.1 Sensitivity to Degree of Saturation

Sensitivity of mean delay to degree of saturation is illustrated in Figure 27. In this analysis, the mean delay was calculated for different values of degree of saturation. However, all other parameters are kept unchanged.

The results are shown in Figure 27 for three different cycle lengths with g/C ratio of 0.46. Although delay is sensitive to cycle length as discussed in Section 4.2.1.3, the results for the specified cycle lengths in Figure 27 are very close. That is because the cycle lengths used are in the linear part as in Figure 32.

Delay is almost insensitive to demand until the demand approaches 80% of the capacity. At *v/c* of 0.8, delay slightly shows an increase as the demand increases and rapidly increases once the demand reaches the capacity. The observed trend is consistent with the sensitivity analysis shown in the HCM (2000). Synchro and HCM results are relatively identical in terms of delay sensitivity to volume to capacity ratio.

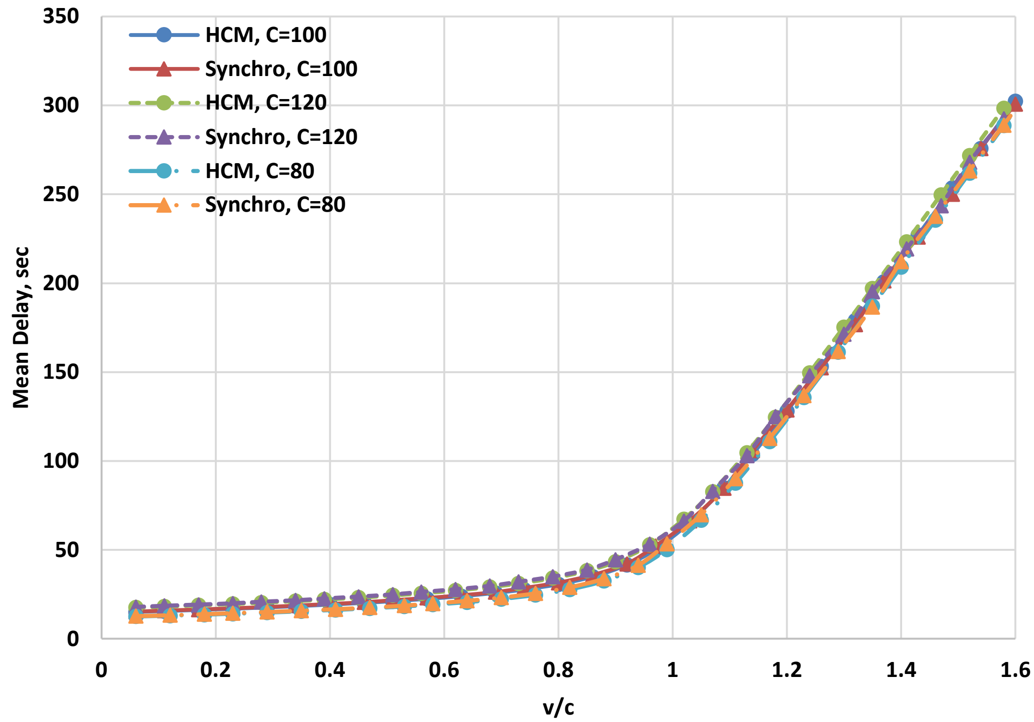


Figure 27: Sensitivity of mean delay to degree of saturation.

Sensitivity of queue length to degree of saturation is illustrated in Figure 28. Queue length increases as the degree of saturation increases and the rate of increase escalates as *v/c* passes 0.8. However, queue length rate of increase after v/c of 1.0 is not as rapid as delay increase.

The relationship between mean delay and maximum queue length for increasing degree of sat is illustrated in Figure 29. The mean delay increases slightly as maximum queue length increases until the degree of saturation reaches 0.2. When degree of saturation surpasses o.2, the rate of mean delay increase escalates higher the rate of maximum queue length increase.

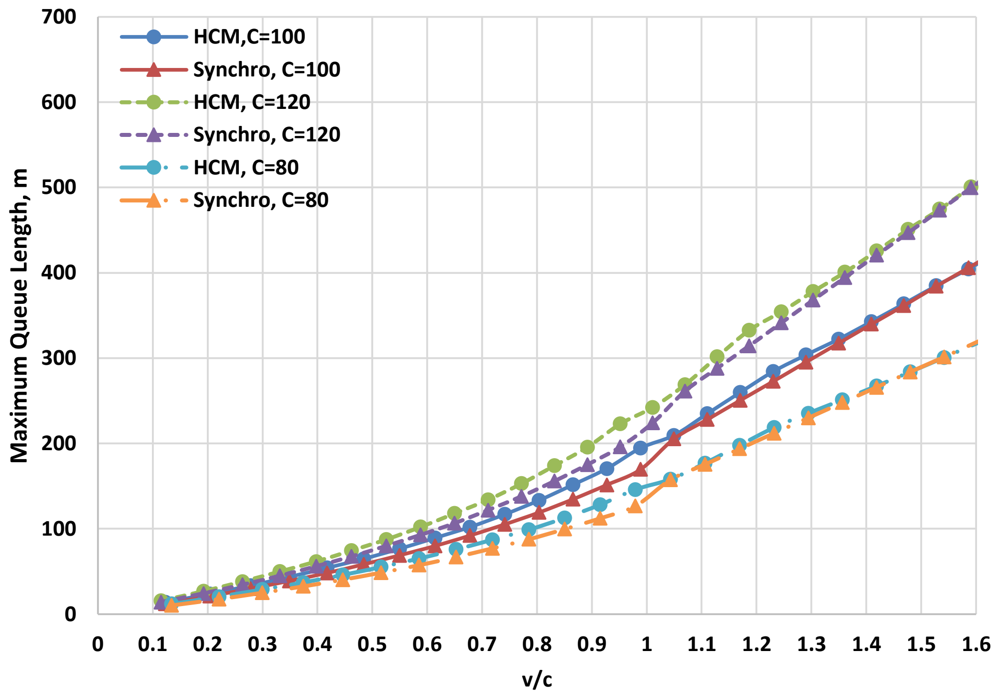


Figure 28: Sensitivity of queue length to degree of saturation.

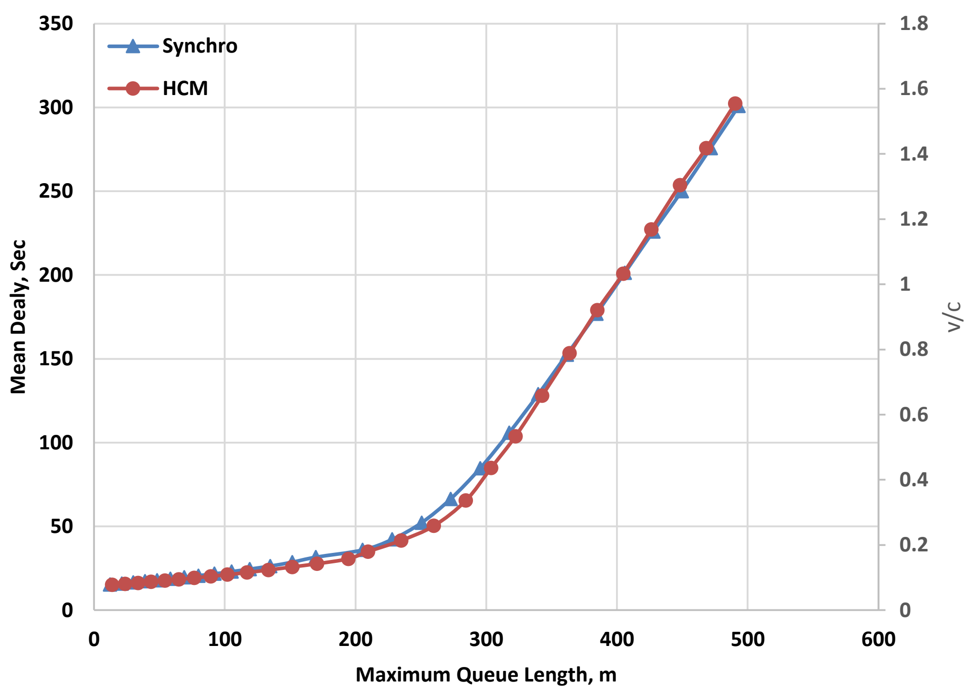


Figure 29: Relationship between mean delay and maximum queue length for increasing v/c

4.2.1.2 Sensitivity to g/C Ratio

Figure 30 illustrates the sensitivity of mean delay to green interval duration to cycle length (g/C) ratio. When demand exceeds 80% of capacity, delay becomes sensitive to signal control parameters. Small values of g/C can cause excessive delay because smaller g/C value do not provide sufficient capacity to serve the demand. However, if there is sufficient g/C to serve the demand, delay becomes less sensitive to g/C.

Figure 31 illustrates the sensitivity of maximum queue length to g/C ratio. Queue length is expected to follow the same trend as in delay sensitivity if the analysis period remains unchanged. However, Synchro considers the maximum queue length after two cycles for over saturated lane groups. Therefore, HCM calculations were adjusted to follow the same procedure. The irregularity in the queue length trend occurs when the lane group becomes oversaturated and therefore the analysis period is reduced.

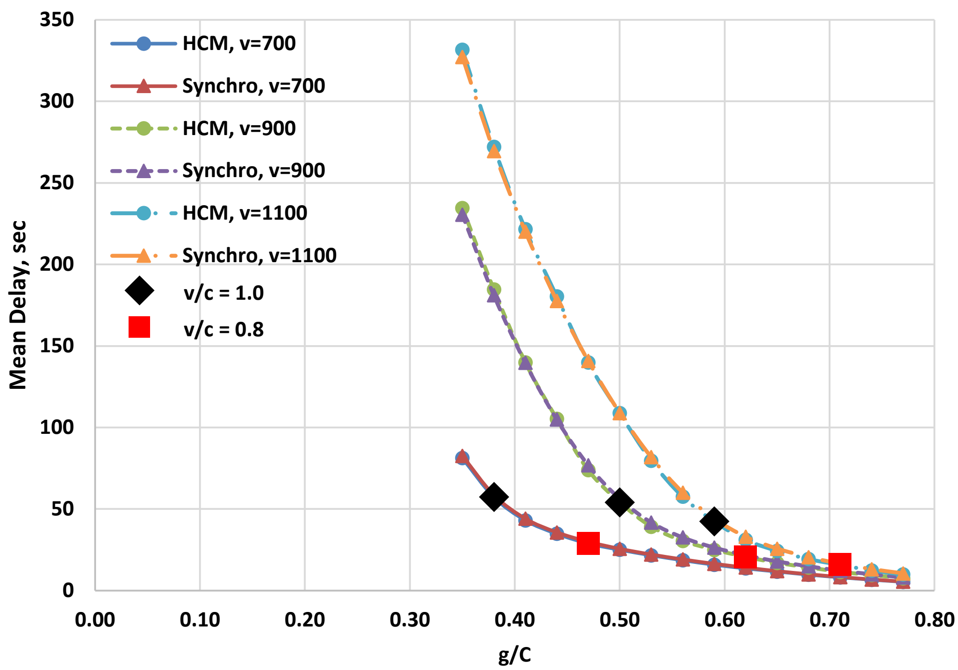


Figure 30: Sensitivity of delay to g/C ratio.

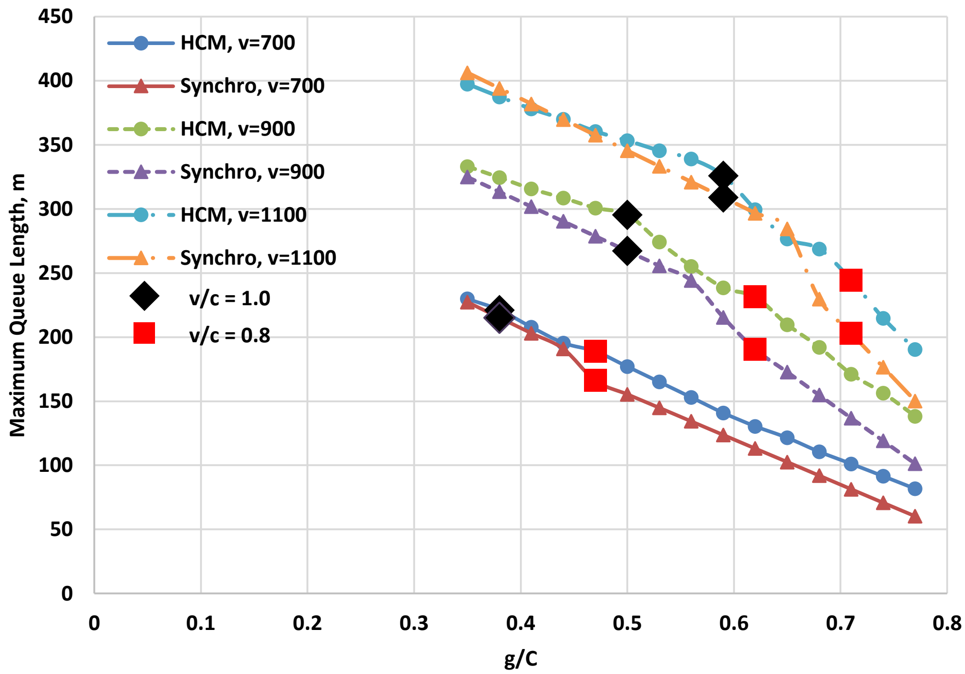


Figure 31: Sensitivity of queue length to g/C ratio.

4.2.1.3 Sensitivity to Cycle Length

Figure 32 shows the sensitivity of mean delay to cycle length. The results shown are for three different demand levels with g/C ratio of 0.46. Short cycle lengths do not provide sufficient capacity result in much larger delays. Very long cycle lengths also increase delay; however, the increase is relatively small smaller as compared to short cycle lengths.

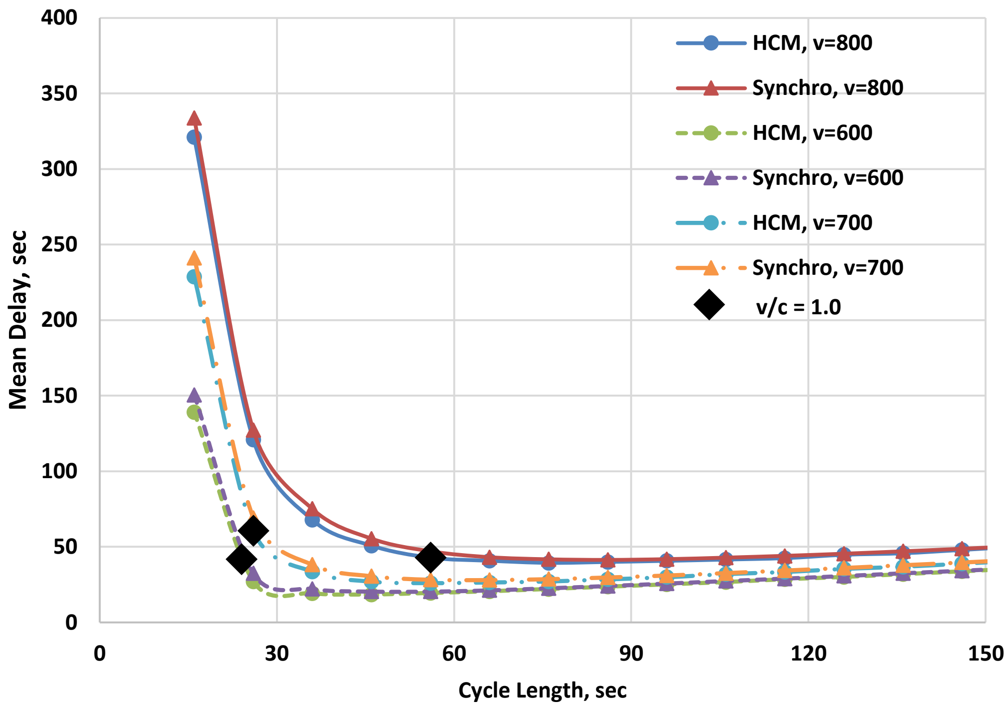


Figure 32: Sensitivity of mean delay to cycle length.

Figure 33 shows the sensitivity of maximum queue length to cycle length. Queue length results from Synchro and HCM are relatively identical when demand exceeds capacity; however, a slight difference can be observed when the lane group is undersaturated. A distinct break point can be observed in the trend when demand reaches capacity. This is due to the change applied to the duration of analysis period. It should be noted that the analysis period is adjusted to be equivalent to two cycles only when v/c exceeds 1.0. Lanes with small g/C values or short cycle lengths are most likely to be oversaturated. As a result, the queues accumulated in short duration of two cycles are by far less than the queues accumulated in a 15 minutes interval. This explains the short queues even for oversaturated lanes.

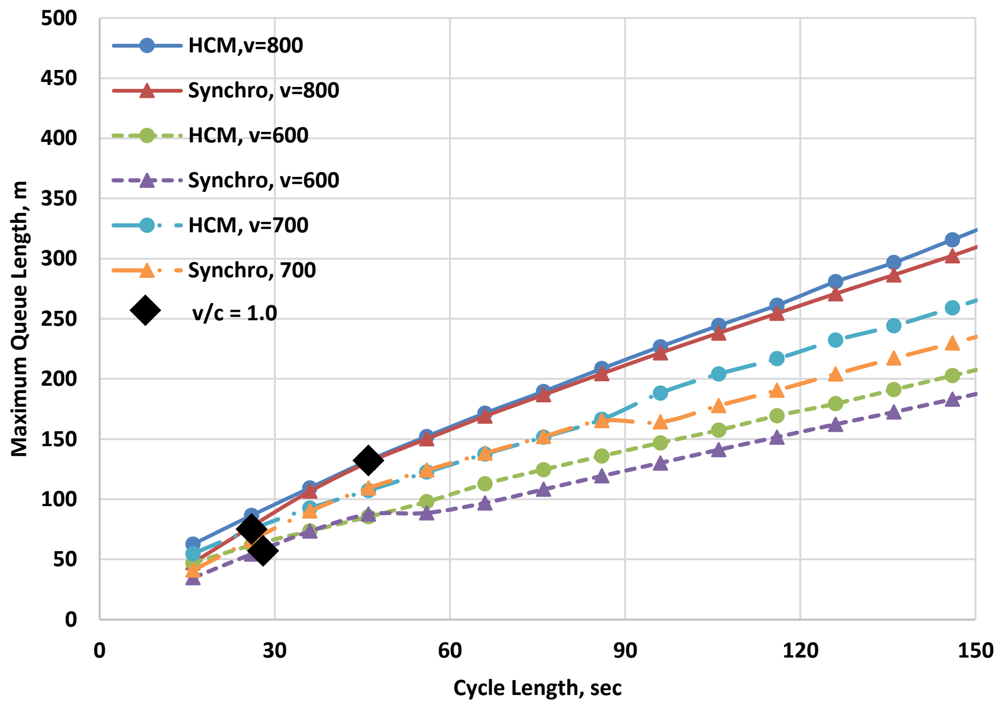


Figure 33: Sensitivity of maximum queue length to cycle length.

4.2.2 Impact of Demand Variation

The delay and queue length estimated using AVL/APC data are representative of what has been experienced by road users over time. However, Synchro calculations are based on a single peak period demand. The peak period demand is measured through periodic traffic count surveys. It is well documented in the literature that the peak period demand may vary significantly on daily basis. However, traffic count surveys do not consider day-to-day variation of the peak hour demand.

Hellinga and Abdy (2008) showed that the impact of day-to-day variations in peak hour demand is significant and should not be ignored. They used Coefficient of Variation (COV) to analyze day-to-day variation in peak hour traffic in the city of Waterloo. Their observations showed that peak hour traffic COV varies within the range of 5.4% to 13.1% with the mean of 8.7%.

A sensitivity analysis was conducted to demonstrate the impact of peak hour traffic variations on intersection performance. The signalized intersection at Homer Watson Blvd. @ Manitou Dr. & Doon Village Rd. was considered for the sensitivity analysis. The movement of interest is the southbound through movement. The average peak hour volume for through traffic on the considered approach at this intersection was 480 vph. Other parameters such as signal timings were kept unchanged.

Considering the mean COV of 8.7% suggested by Hellinga and Abdy (2008), Monte Carlo (MC) simulation was used to generate a range of feasible traffic volumes for the peak period traffic. HCM (2000) methodology was applied to estimate the average stopped delay and maximum queue length for each simulation trial. For oversaturated periods, the queue length was estimated only for two cycles.

Based on 100 simulation runs, analysis results for average delay and maximum queue length are demonstrated through Figure 34 and Figure 35, respectively. Synchro and MC simulation produced relatively similar stopped delay because they use the same computational process, and they are based on a fixed peak demand. On the other hand, AVL/APC data capture the real life experienced delay over long period and averaged based on the number of trips.

By considering 8.7% variation in peak hour traffic, the average delay and maximum queue length varied from less than 40 seconds to 70 seconds and from 75 meters to 120 meters, respectively. The analysis results imply that even slight fluctuations in peak hour traffic can result in significant variations in average delay and maximum queue length.

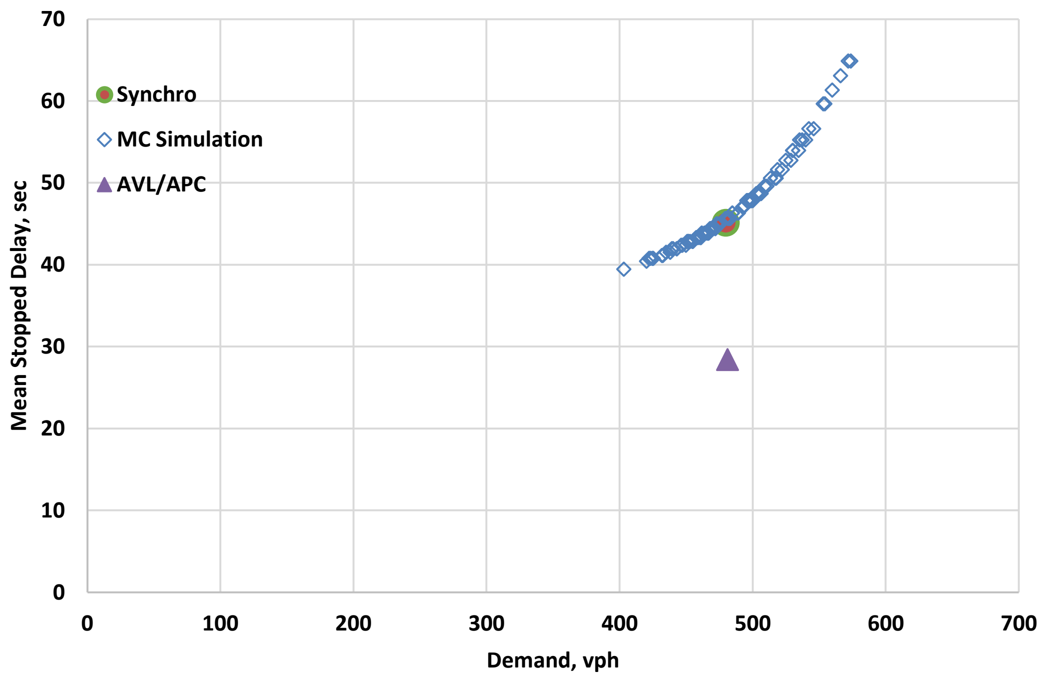


Figure 34: Impact of day-to-day demand variation on stopped delay.

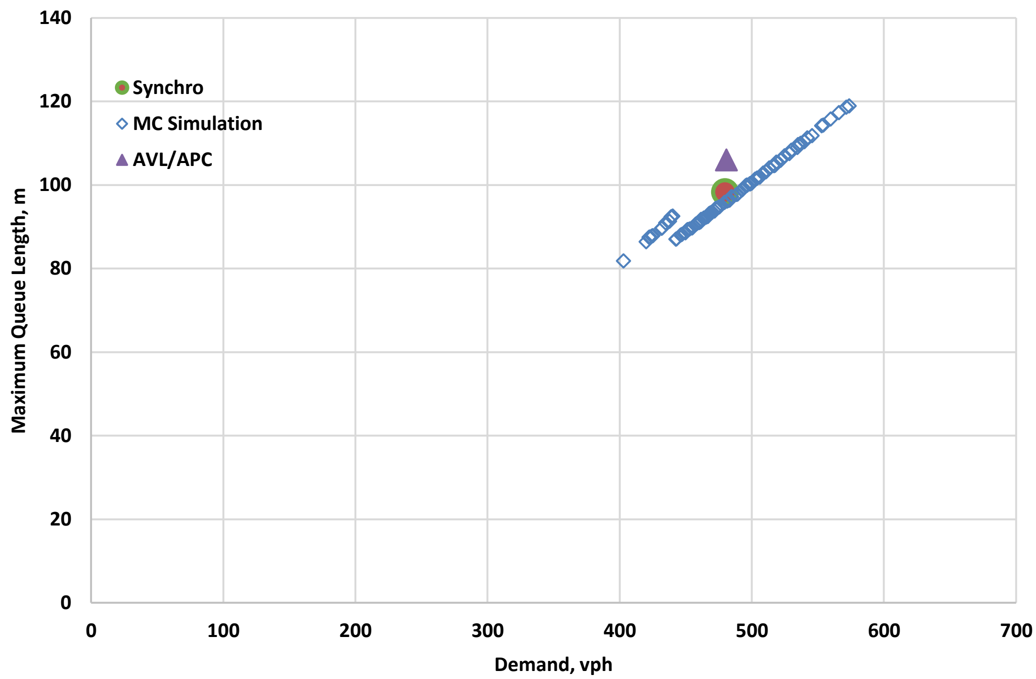


Figure 35: Impact of day-to-day demand variation on queue length.

4.2.3 Limitation of Synchro

Synchro has limited capabilities in modelling some complex intersection geometries. For instance, there are limited modeling options in Synchro for intersections with midblock left turn lanes immediately downstream or upstream of the intersection. The limitation of Synchro in modeling such intersections creates the possibility that Synchro might underestimate the intersection performance.

Figure 36 shows the intersection at Hespeler Rd. @ Dunbar Rd. The movement of interest at this intersection is the northbound right turning movement. the bus route providing the AVL/APC data at this intersection performs a left turn just 50 meters after turning right at the intersection. Queues may form frequently and reach the intersection as buses are waiting for a gap to turn left downstream of intersection. This scenario cannot be captured by Synchro; however, AVL/APC data record stop events at that location as if it is caused by the signal. For this particular movement, Synchro computed the maximum queue length of 15 meters, while the maximum queue length estimated from AVL/APC data was 85 meters.

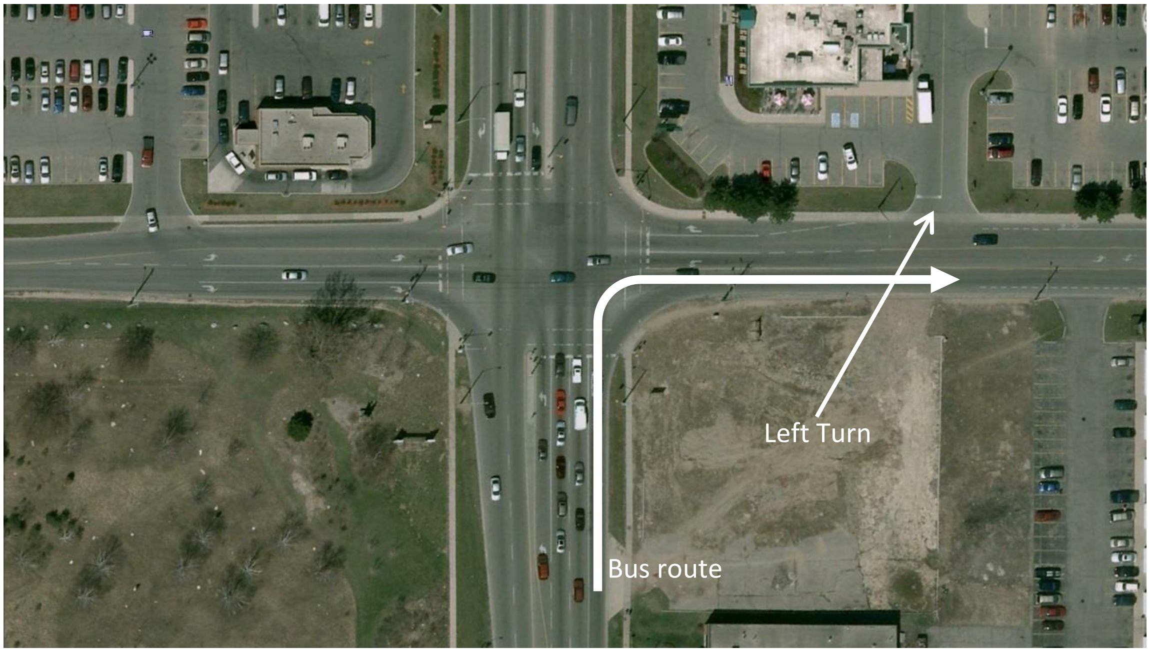


Figure 36: Impact of the left turn downstream of intersection.

The lane configuration of midblock turning movements upstream of the intersection cannot be modelled in Synchro, which might lead to intersection performance underestimation. Figure 37 shows the intersection at Fairway Rd. @ King St., where the movement of interest is the northbound left turning movement. There are two midblock left turns immediately upstream of the intersection. The vehicles waiting for a gap to turn left may cause queues and additional delays for through traffic, which cannot be modelled realistically in Synchro. Fortunately, these complex situations can be addressed if AVL/APC data are used for estimating intersection performance measures.

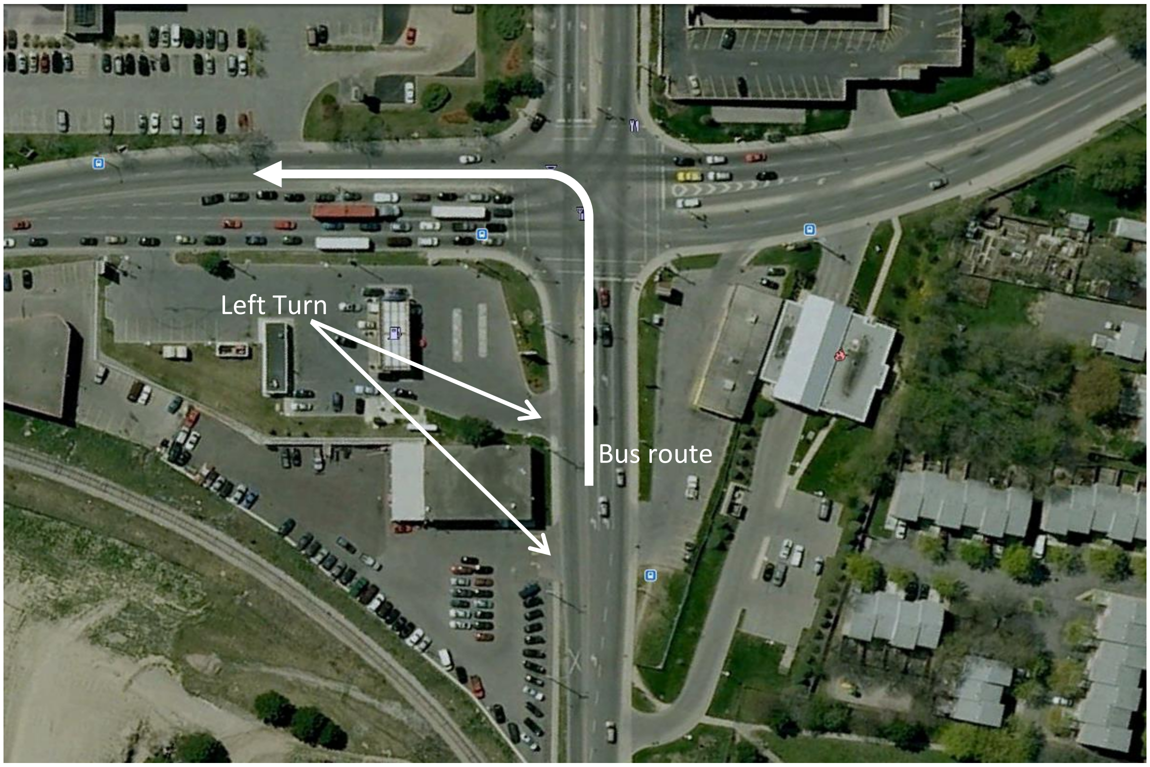


Figure 37: Impact of the left turn upstream of intersection.

4.2.4 Delay and Queue Length Estimation Methodology in Synchro

Based on the definitions of maximum queue length and maximum queue extent given in section 2.1.2, queue length is essentially less than queue extent. Vehicles arriving at the end of red interval and before the queue completely dissipates will accumulate at the rear of the queue. While the queue size is decreasing, the rear of the queue moves backward and the queue extent is increasing. Shockwave theory is more realistic in term of queue length since it captures not the maximum queue size but the maximum queue reach.

The methodology implemented in HCM (2000) and Synchro for estimating delay and queue length at signalized intersections is based on queuing theory, which considers the cumulative arrival and departure curves to estimate delay and queue length. Queuing theory does not consider the physical queue and assumes that queuing vehicles stack on top of each other at the head of the queue, which is also known as point queue. It is well understood that point queue methodology generally underestimates the queue length. In other words, Synchro estimates the queue length not the queue extent.

On the other hand, AVL/APC data reflect real life traffic conditions including the shockwaves and physical queues experienced by road users. Buses joining the queue after the traffic signal has turned to green will stop even though the queue size is decreasing from downstream. Simultaneously, the queue reach is moving further away from the stop line. AVL/APC data take into account the queue extent while Synchro does not, and therefore Synchro underestimates the queue reach. This suggests that estimation of delay and queue length at signalized intersections using AVL/APC data is more realistic and accurate since it captures the queue extent not the queue length.

The difference between queue length and queue extent should be considered when comparing intersections performance measures estimated using AVL/APC data and Synchro.

4.2.5 Delay and Queue Length Estimation Methodology Using AVL/APC Data

Considering the large volume of AVL/APC data, a GPS software is used to find the stop events that correspond to the bus routes going through each signalized intersection. Some intersections were used by more than one bus route.

The defined road segments on bus routes were modeled using intersection midpoints as reference. As a result, the location of stop events were always estimated in reference to intersection center points. In other words, the maximum queue length was estimated as the distance from the queue tail to the middle of the intersection. However, Synchro computes the queue length based on the distance between the back of queue and the stop line. The difference between estimated queue using AVL/APC data and Synchro is at least equal to the distance between the stop line and the midpoints of the intersection.

The boundary line fitted for each road segment defines the average stopped delay and the maximum queue length. Imprecision of choosing the best-fit boundary line might result in inaccurate estimates. In other words, estimated delay and queue length from AVL/APC data is sensitive to the shape of the boundary line fitted to the stop event data to identify the delay envelop.

The boundary line fitting process is based on the minimization of the change in the density of the points contained within the candidate boundary line. Although the procedure yields the optimum solution in most cases, occasionally it may find a local optimum solution rather than the global optimum solution.

Figure 38 provides an example using the stop event data collected at King Street @ Conestoga Mall. The selected optimum boundary line (boundary candidate 1) yields the maximum queue length of 107 meters. The distance from the stop line to the middle of the intersection is 25 meters. As a result, the maximum queue length estimated using AVL/APC data is 82 meters. The maximum queue length computed by Synchro is 47 meters, which is much smaller than the queue length estimated using AVL/APC data. If boundary candidate 2 is considered as the optimum solution, the maximum queue length will be 76 meters. The adjusted queue length considering the distance from the stop line to the middle of the intersection will be 51 meters, which is very similar to what is computed by Synchro.

This suggests that it may be possible to improve the accuracy of the AVL/APC estimation method by changing the boundary fitting method. Though the development of an improved method is outside the scope of this thesis, it is hypothesized that the sensitivity of the method results from discretization of distance in order to define the set of finite candidate boundary lines. A method that avoids this discretization may provide more robust performance.

It should be noted that the queue lengths estimates from the AVL/APC data and presented in Section 4.1 are adjusted by subtracting the distance from the middle of the intersection to the approach's stop line from the queue length.

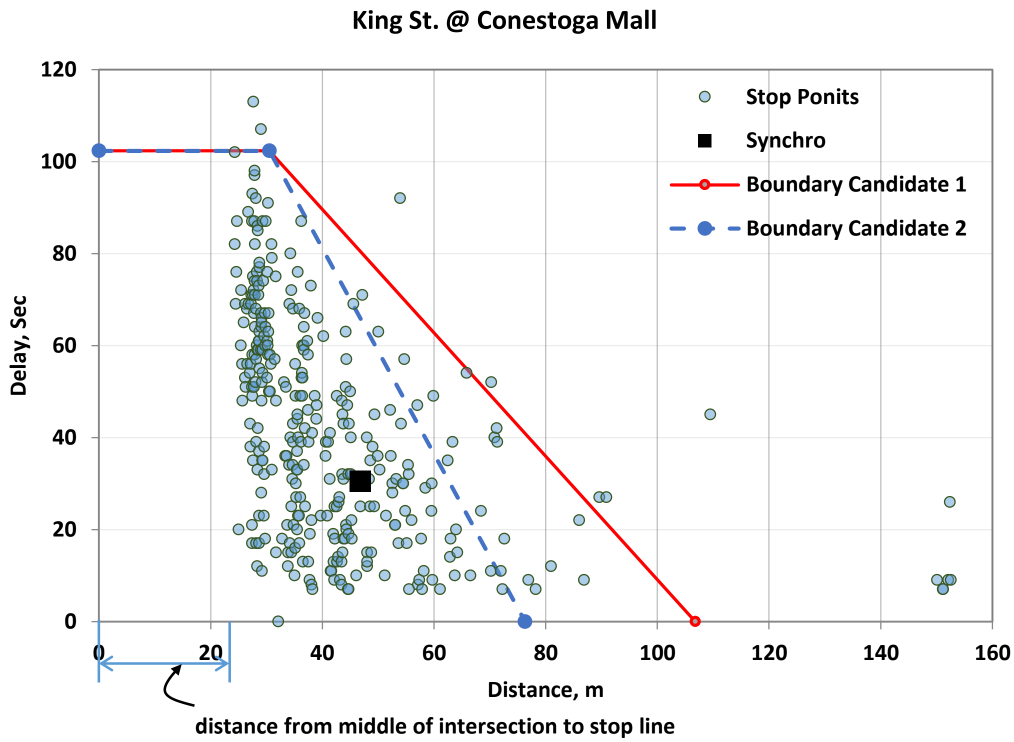


Figure 38: Impact of alternative candidate boundary lines on delay and queue length

Chapter 5

Conclusions and Recommendations

The feasibility of using AVL/APC data and Synchro for large-scale performance evaluation of intersections in urban transportation networks was evaluated. The evaluation of intersection performance using estimates from AVL/APC data and from Synchro was compared and analyzed to find the correlation between the two methods.

Intersection performance evaluation using Synchro heavily relies on empirical input data, and therefore estimated delays and queue lengths are sensitive to variations or errors in these input data such as turning movement counts and signal timing parameters. In particular, if input data, which are typically observed on a single day, are not representative of conditions experienced over the larger period of interest (i.e. several moths), then the Synchro estimates may not be accurate. On the other hand, AVL/APC data reflect actual traffic conditions experienced by road users over an extended period. Therefore, the delay and queue length estimated using AVL/APC data are more robust, reliable and less sensitive to occasional variations in traffic conditions.

Overall, a strong correlation was observed between estimates from AVL/APC data and Synchro. Over the 29 intersection approaches examined, the mean absolute relative error (MARE) was 21.6% and 58.3% for stopped delay and maximum queue length respectively. The absolute relative error (ARE) ranged from a minimum of 0.2% to a maximum of 103% for stopped delay and from a minimum of 1.2% to a maximum of 435% for maximum queue length.

Possible reasons for differences in delay and queue length estimations from Synchro and AVL/APC data were discussed including Synchro's limitations in modelling complex intersection geometries and limitations of the queuing theory implemented in Synchro for estimating delay and queue length at signalized intersections.

Based on the findings of this study, using AVL/APC data for estimating performance measures for signalized intersections is recommended where applicable. The most significant advantages of this approach are:

1. The estimates are based on actual observation of delay and queues.
2. The estimates automatically reflects variations (day to day as well as time of day) in traffic demands.
3. There is no need to collect additional data (i.e. turning movement counts).

The most significant limitations are:

1. Can only be used to estimate conditions for approaches and lane groups on which bus routes currently exists.
2. The methodology is not applicable for approaches with nearside transit stops.

On the basis of the work carried out in this thesis, the following recommendations are made:

1. Agencies with access to archived transit AVL/APC data can use these data to estimate intersection measures of performance.
2. These estimates are consistent with estimates form standard signalized intersection analysis methods such as Synchro. However, these estimates tend to be more robust as they reflect actual observations.
3. Estimates of delay and queue length can be obtained from methods such as Synchro when AVL/APC data are not available (e.g. transit route planning).
4. The methodology is sensitive to the shape of the boundary line fitted to stop event data. Additional research and analysis is necessary for extending the AVL/APC methodology for intersections with nearside bus stops and refining the existing boundary line fitting methodology.

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Appendix A

Intersections Information

Appendix A. encloses intersections information such as intersection name, defined segment, and the bus route. It also encloses lanes configuration, turning movements' volume, pedestrian counts, peak hour factors, ratio of heavy vehicles, and ideal saturation flow rate. The movement of interest is highlighted with yellow color for each intersection.

Intersection Information			
Intersection:	HESPELER Rd. /WATER St. @ Coronation Blvd. /Dundas St.	**Bus Route:**	52
Segment:	DUNDAS @ Easton to HESPELER/WATER @ Coronation/Dundas		

Dates of used Data			
AVL/APC data:	2011	**Turning Movement Counts:**	2011

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	50.2	102	127
2013	37	95.2	135

Synchro Input Data												
Street	**Water St.**			**Hespeler Rd.**			**Coronation Blvd.**			**Dundas St.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖↖	↑↑	Shared	↖↖	↑↑	↗	↖	↑↑	↗	↖	↑↑	↗
Traffic Volume	331	694	7	381	1071	107	153	604	461	73	406	230
Pedestrians	4	-	17	17	-	4	5	-	14	14	-	5
PHF	0.95	0.93	0.58	0.91	0.91	0.76	0.98	0.88	0.84	0.91	0.88	0.86
Heavey Veh. %	3	6	29	2	2	2	1	2	1	1	1	3
Ideal Sat. Flow	1775	1775	1775	1775	1775	1775	1775	1900	1750	1775	1900	1750

Intersection Information			
Intersection:	FAIRWAY Rd. @ King St.	**Bus Route:**	52
Segment:	KING St. @ River St. to FAIRWAY Rd. @ King St.		

Dates of used Data			
AVL/APC data:	2013	**Turning Movement Counts:**	2013

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	41	83.9	106
2013	39.4	91	106

Synchro Input Data												
Street	**King St.**			**King St.**			**Fairway Rd.**			**Fairway Rd.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖↖	↑	Shared	↖	↑	↗	↖↖	↑↑	Shared	↖	↑↑	Shared
Traffic Volume	287	413	27	43	299	591	552	934	368	21	669	63
Pedestrians	12	-	63	63	-	12	37	-	14	14	-	37
PHF	0.77	0.77	0.6	0.68	0.96	0.87	0.87	0.73	0.86	0.42	0.89	0.61
Heavey Veh. %	1	1	4	2	0	3	1	2	2	0	3	18
Ideal Sat. Flow	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900

Intersection Information												
Intersection:	Ottawa St. @ Homer Watson Blvd.									**Bus Route:**		11
Segment:	OTTAWA St. @ Alpine St. to OTTAWA St. @ Homer Watson Blvd.											
Dates of used Data												
AVL/APC data:			2011			**Turning Movement Counts:**				2011		
AVL/APC Data												
	Mean Stopped Delay				**90th Percentile Delay**				**Maximum Queue Length**			
2011	39.1				64				189			
2013	39.3				74.6				175			
Synchro Input Data												
Street	**Homer Watson Blvd.**			**Homer Watson Blvd.**			**Ottawa St.**			**Ottawa St.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	↑↑	↗	↖	↑↑	↗	↖	↑↑	↗	↖	↑↑	Shared
Traffic Volume	319	584	693	719	774	33	291	909	284	210	655	29
Pedestrians	3	-	10	10	-	3	1	-	0	0	-	1
PHF	0.85	0.9	0.88	0.89	0.88	0.83	0.76	0.97	0.76	0.89	0.88	0.73
Heavey Veh. %	1	1	3	1	2	0	0	2	2	0	2	10
Ideal Sat. Flow	1775	1900	1750	1775	1900	1750	1775	1900	1750	1775	1775	1775

Intersection Information			
Intersection:	Hespeler Rd @ Pinebush Rd & Eagle St.	**Bus Route:**	51
Segment:	PINEBUSH Rd. @ Walmart & Home Depot to HESPELER Rd. @ Eagle St. & Pinebush Rd.		

Dates of used Data			
AVL/APC data:	2011	**Turning Movement Counts:**	2011

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	39	101	102
2013	64.3	107	102

Synchro Input Data												
Street	**Hespeler Rd.**			**Hespeler Rd.**			**Eagle St.**			**Pinebush Rd.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	↑↑↑	↗	↖↖	↑↑↑	Shared	↖↖	↑↑	Shared	↖↖	↑↑	Shared
Traffic Volume	128	928	305	408	1317	343	406	363	174	197	339	223
Pedestrians	3	-	2	2	-	3	6	-	5	5	-	6
PHF	0.73	0.83	0.9	0.89	0.95	0.96	0.96	0.87	0.89	0.84	0.91	0.79
Heavey Veh. %	1	3	1	3	2	8	8	2	1	2	2	5
Ideal Sat. Flow	1775	1900	1750	1775	1775	1775	1775	1775	1775	1775	1900	1750

Intersection Information			
Intersection:	Fischer-Hallman Rd. @ Thorndale Dr.	**Bus Route:**	5
Segment:	ERB St. @ Beechwood & Gateview to FISCHER-HALLMAN Rd. @ Thorndale Dr.		

Dates of used Data			
AVL/APC data:	2011	**Turning Movement Counts:**	2011

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	37.8	87	76
2013	Not Available	Not Available	Not Available

Synchro Input Data												
Street	**Fischer Halman Rd.**			**Fischer Halman Rd.**			**Thorndale Dr.**			**Thorndale Dr.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	↑↑	Shared	↖	↑↑	Shared	Shared	↑	↗	Shared	↑	Shared
Traffic Volume	163	579	19	5	653	142	52	4	88	7	2	7
Pedestrians	6	-	3	3	-	6	9	-	2	2	-	9
PHF	0.85	0.86	0.95	0.63	0.93	0.85	0.87	0.5	0.73	0.58	0.5	0.88
Heavey Veh. %	1	2	5	0	3	1	4	0	5	0	0	14
Ideal Sat. Flow	1775	1775	1775	1775	1775	1775	1775	1650	1650	1550	1550	1550

Intersection Information												
Intersection:	Conestoga College Blvd. @ Homer Watson Blvd.									**Bus Route:**		16
Segment:	HW Blvd. @Doon South Rd & Monarch Tr to HW Blvd. @ Conestoga College Blvd.											

Dates of used Data			
AVL/APC data:	2011	**Turning Movement Counts:**	2011

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	36.6	98	121
2013	21.7	72.5	98

Synchro Input Data												
Street	**Homer Watson Blvd.**			**Homer Watson Blvd.**			**Conestoga College Blvd.**			**Conestoga College Blvd.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	↑↑	↗	↖↖	↑↑	↗	↖	↑↑	Shared	↖	↑	↗
Traffic Volume	220	1724	125	245	1315	20	119	69	105	477	132	281
Pedestrians	0	-	1	1	-	0	3	-	65	65	-	3
PHF	0.96	0.97	0.82	0.89	0.97	0.71	0.93	0.78	0.85	0.89	0.85	0.85
Heavey Veh. %	17	4	6	3	3	0	0	0	8	1	2	2
Ideal Sat. Flow	1775	1900	1750	1775	1900	1750	1775	1775	1775	1775	1900	1750

Intersection Information			
Intersection:	Homer Watson Blvd. @ Manitou Dr. Doon Village Rd.	**Bus Route:**	10
Segment:	DOON VILLAGE Rd. @ Pioneer to HOMER WATSON Blvd. @ Manitou Dr. & Doon Village Rd.		

Dates of used Data			
AVL/APC data:	2011	**Turning Movement Counts:**	2011

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	34.4	69.4	91
2013	35.4	83.3	90

Synchro Input Data												
Street	**Doon Village Rd.**			**Manitou Dr.**			**Homer Watson Blvd.**			**Homer Watson Blvd.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	↑↑	Shared	↖↖	↑↑	↗	↖	↑↑	↗	↖	↑↑	↗
Traffic Volume	240	240	50	561	479	181	112	810	439	47	820	340
Pedestrians	1	-	0	0	-	1	0	-	1	1	-	0
PHF	0.88	0.83	0.85	0.87	0.94	0.75	0.84	0.95	0.87	0.93	0.96	0.66
Heavey Veh. %	1	2	4	6	1	2	5	5	1	1	6	6
Ideal Sat. Flow	1775	1900	1750	1750	1900	1750	1750	1900	1750	1775	1900	1750

Intersection Information			
Intersection:	Homer Watson Blvd. @ Manitou Dr. Doon Village Rd.	**Bus Route:**	10
Segment:	MANITOU Dr. @ Wabanaki to HOMER WATSON Blvd. @ Manitou Rd. & Doon Village Rd.		

Dates of used Data			
AVL/APC data:	2013	**Turning Movement Counts:**	2013

AVL/APC Data	**Mean Stopped Delay**	**90th Percentile Delay**	**Maximum Queue Length**
2011	28.6	68	197
2013	28.2	66	106

Synchro Input Data												
Street	**Homer Watson Blvd.**			**Homer Watson Blvd.**			**Conestoga College Blvd.**			**Conestoga College Blvd.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	↑↑	Shared	↖↖	↑↑	↗	↖	↑↑	↗	↖	↑↑	↗
Traffic Volume	246	238	51	557	481	182	111	804	441	45	821	341
Pedestrians	1	-	0	0	-	1	0	-	1	1	-	0
PHF	0.88	0.83	0.85	0.87	0.94	0.75	0.84	0.95	0.87	0.93	0.96	0.66
Heavey Veh. %	0	2	4	6	1	2	5	5	1	0	6	6
Ideal Sat. Flow	1775	1900	1750	1750	1900	1750	1750	1900	1750	1775	1900	1750

Intersection Information			
Intersection:	Fairway Rd. @ Wilson Ave.	**Bus Route:**	10
Segment:	WILSON Ave. @ Kingsway Dr. to FAIRWAY Rd. @ Wilson Ave.		

Dates of used Data			
AVL/APC data:	2011	**Turning Movement Counts:**	2011

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	32.1	64.7	120
2013	23	61	105

Synchro Input Data												
Street	**Wilson Ave.**			**Wilson Ave.**			**Fairway Rd.**			**Fairway Rd.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	↑	↗	↖	↑↑	Shared	↖	↑↑	Shared	↖	↑↑	↗
Traffic Volume	121	260	193	111	204	245	310	616	78	307	711	140
Pedestrians	35	-	31	31	-	35	31	-	28	28	-	31
PHF	0.8	0.84	0.86	0.87	0.81	0.82	0.96	0.96	0.81	0.94	0.96	0.9
Heavey Veh. %	2	1	5	4	2	2	1	3	6	6	1	5
Ideal Sat. Flow	1775	1900	1750	1775	1775	1775	1775	1775	1775	1775	1900	1750

Intersection Information												
Intersection:	Fairway Rd. @ Fairview Mall										**Bus Route:**	8
Segment:	Terminal to FAIRWAY Rd. @ Fairview Park Mall											

Dates of used Data			
AVL/APC data:	2011	**Turning Movement Counts:**	2011

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	32.1	74.1	162
2013	17.2	124	16

Synchro Input Data												
Street	**Fairview Mall**			**Fairview Mall**			**Fairway Rd.**			**Fairway Rd.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	↑	Shared	↖	↑	Shared	↖	↑↑	Shared	↖	↑↑	↗
Traffic Volume	26	20	31	282	13	105	116	872	15	147	1130	139
Pedestrians	6	-	5	5	-	6	6	-	0	0	-	6
PHF	0.65	0.56	0.78	0.97	0.65	0.88	0.88	0.87	0.54	0.9	0.98	0.83
Heavey Veh. %	0	5	0	9	15	0	3	4	0	1	3	12
Ideal Sat. Flow	1775	1775	1775	1775	1775	1775	1775	1775	1775	1775	1900	1750

Intersection Information			
Intersection:	Natchez Rd. & Driveway @ Victoria St.	**Bus Route:**	15
Segment:	LACKNER @ Keewatin to VICTORIA St. @ Natchez Rd.		

Dates of used Data			
AVL/APC data:	2011	**Turning Movement Counts:**	2011

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	31.7	72.1	76
2013	33.5	71	76

Synchro Input Data												
Street	**Nattchez Rd.**			**Dirveway**			**Victoria St.**			**Victoria St.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	N/A	↗	↖	N/A	↗	↖	↑↑	↗	↖	↑↑	Shared
Traffic Volume	91	0	41	3	0	4	5	1584	151	145	2007	5
Pedestrians	0	-	0	0	-	0	0	-	4	4	-	0
PHF	0.63	0	0.79	0.25	0.25	0.5	0.42	0.98	0.82	0.88	0.92	0.42
Heavey Veh. %	1	0	0	0	0	0	0	2	0	1	1	0
Ideal Sat. Flow	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900

Intersection Information			
Intersection:	Pinebush Rd. @ Conestoga Blvd.	**Bus Route:**	200
Segment:	SHELDON @ Conestoga Blvd. to PINEBUSH Rd. @ Conestoga Blvd.		

Dates of used Data			
AVL/APC data:	2011	**Turning Movement Counts:**	2011

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	26	77	91
2013	28.2	83	121

Synchro Input Data												
Street	**Conestoga Blvd.**			**Conestoga Blvd.**			**Pinebush Rd.**			**Pinebush Rd.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	↑	Shared	↖	↑	Shared	↖	↑↑	↗	↖	↑↑	↗
Traffic Volume	159	251	119	85	124	72	69	458	125	97	435	97
Pedestrians	5	-	0	0	-	5	0	-	2	2	-	0
PHF	0.9	0.87	0.88	0.69	0.79	0.78	0.66	0.91	0.87	0.73	0.91	0.64
Heavey Veh. %	2	2	6	0	4	8	9	3	6	4	2	3
Ideal Sat. Flow	1775	1775	1775	1775	1775	1775	1775	1900	1750	1775	1900	1750

Intersection Information			
Intersection:	King St. @ Conestogo Rd.	**Bus Route:**	7
Segment:	Terminal to KING St. @ Conestogo Rd.		

Dates of used Data			
AVL/APC data:	2013	**Turning Movement Counts:**	2013

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	24.7	56	143
2013	18.5	83.4	107

Synchro Input Data												
Street	**King St.**			**King St.**			**Conestogo Rd.**			**Conestogo Rd.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	↑↑↑	↗	↖	↑↑	Shared	↖	↑	Shared	↖↖	↑	Shared
Traffic Volume	57	701	262	102	947	30	35	20	125	305	13	81
Pedestrians	3	-	30	30	-	3	2	-	3	3	-	2
PHF	0.75	0.86	0.84	0.85	0.85	0.83	0.67	0.63	0.89	0.87	0.65	0.75
Heavey Veh. %	9	4	1	6	2	20	3	5	5	4	8	2
Ideal Sat. Flow	1775	1900	1750	1775	1775	1775	1775	1775	1775	1775	1775	1775

Intersection Information												
Intersection:	Queen St. @ Goebel Ave.									**Bus Route:**		51
Segment:	HOLIDAY INN @ Groh to QUEEN St. @ Goebel Ave.											
Dates of used Data												
AVL/APC data:			2011			**Turning Movement Counts:**				2011		
AVL/APC Data												
	Mean Stopped Delay				**90th Percentile Delay**				**Maximum Queue Length**			
2011	23.1				53				164			
2013	9				28.4				92			
Synchro Input Data												
Street	**Goebel Ave.**			**N/A**			**Queen St.**			**Queen St.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	N/A	↗	N/A	N/A	N/A	N/A	↑	↗	↖	↑	N/A
Traffic Volume	546	0	252	0	0	0	0	405	181	103	292	0
Pedestrians	0	-	0	0	-	0	0	-	1	1	-	0
PHF	0.98	0	0.88	0	0	0	0	0.93	0.84	0.66	0.94	0
Heavey Veh. %	5	0	2	0	0	0	0	2	2	7	0	0
Ideal Sat. Flow	1775	N/A	1750	N/A	N/A	N/A	N/A	1900	1750	2500	1900	N/A

Intersection Information			
Intersection:	Charles St. @ Ontario St.	**Bus Route:**	23
Segment:	Terminal to CHARLES St. @ Ontario St.		

Dates of used Data			
AVL/APC data:	2011	**Turning Movement Counts:**	2011

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	21.1	54.2	76
2013	16.8	46.6	62

Synchro Input Data												
Street	**Charles St.**			**Charles St.**			**Terminal**			**Otario St.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	↑	Shared	↖	↑	Shared	Shared	↑	Shared	Shared	↑	Shared
Traffic Volume	69	365	35	21	421	58	11	45	54	20	23	14
Pedestrians	37	-	28	28	-	37	122	-	100	100	-	122
PHF	0.78	0.93	0.67	0.66	0.94	0.85	0.69	0.75	0.79	0.63	0.72	0.44
Heavey Veh. %	14	6	0	0	6	31	0	18	35	0	0	0
Ideal Sat. Flow	1775	1775	1775	1775	1775	1775	1550	1550	1550	1550	1550	1550

Intersection Information												
Intersection:	Bridgeport Rd. @ Weber St.									**Bus Route:**		35
Segment:	BRIDGEPORT Rd. @ Ellis to WEBER St. @ Bridgeport Rd.											

Dates of used Data			
AVL/APC data:	2011	**Turning Movement Counts:**	2011

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	19.8	45	74
2013	Not Available	Not Available	Not Available

Synchro Input Data												
Street	**Weber St.**			**Weber St.**			**Bridgeport Rd.**			**Bridgeport Rd.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	↑↑	N/A	N/A	↑↑	Shared	N/A	N/A	N/A	Shared	↑↑↑↑	Shared
Traffic Volume	208	818	0	0	864	208	0	0	0	93	857	169
Pedestrians	15	-	0	0	-	15	0	-	0	9	-	14
PHF	0.88	0.81	0	0	0.93	0.95	0	0	0	0.8	0.87	0.92
Heavey Veh. %	0	1	0	0	1	0	0	0	0	1	1	3
Ideal Sat. Flow	1775	1900	N/A	N/A	1775	1775	N/A	N/A	N/A	1735	1735	1735

Intersection Information

Intersection:	Erb Street @ Westmount Road	**Bus Route:**	5
Segment:	:ERB St. @ Roslin to WESTMOUNT Rd. @ Erb St.		

Dates of used Data

AVL/APC data:	2011	**Turning Movement Counts:**	2011

AVL/APC Data

	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	19.3	56.8	118
2013	Not Available	Not Available	Not Available

Synchro Input Data

Street	**Westmount Rd.**			**Westmount Rd.**			**Erb St.**			**Erb St.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	↑↑	Shared	↖	↑↑	Shared	↖	↑↑	Shared	↖	↑↑	↗
Traffic Volume	240	608	46	104	771	75	78	462	84	89	684	73
Pedestrians	15	-	18	18	-	15	18	-	31	31	-	18
PHF	0.87	0.93	0.68	0.79	0.96	0.69	0.65	0.82	0.78	0.79	0.96	0.76
Heavey Veh. %	1	2	4	3	3	3	1	2	5	1	1	0
Ideal Sat. Flow	1775	1775	1775	1775	1775	1775	1775	1775	1775	1775	1900	1775

Intersection Information			
Intersection:	Weber St. @ Parkside Dr.	**Bus Route:**	9
Segment:	WEBER St. @ Albert St. to WEBER St. @ Parkside Dr.		

Dates of used Data			
AVL/APC data:	2011	**Turning Movement Counts:**	2011

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	18.1	46	76
2013	14	35	76

Synchro Input Data												
Street	**Parkside Dr.**			**Parkside Dr.**			**Weber St.**			**Weber St.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	↑	Share d	↖	↑	Share d	Share d	↑↑	Share d	Share d	↑↑	Share d
Traffic Volume	58	552	271	22	424	75	77	346	46	149	272	53
Pedestrians	8	-	5	5	-	8	3	-	0	0	-	3
PHF	0.76	0.93	0.74	0.61	0.88	0.75	0.69	0.95	0.77	0.75	0.85	0.66
Heavey Veh. %	7	1	1	14	1	3	3	3	11	1	2	2
Ideal Sat. Flow	177 5	165 0	1650	177 5	165 0	1650	1650	165 0	1650	1650	165 0	1650

Intersection Information			
Intersection:	DUNDAS St. @ Main St.	**Bus Route:**	53
Segment:	MAIN St. @ Elgin St. to DUNDAS St. @ Main St.		

Dates of used Data			
AVL/APC data:	2011	**Turning Movement Counts:**	2011

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	17.6	51.6	90
2013	19.5	48	78

Synchro Input Data												
Street	**Dundas St.**			**Dundas St.**			**Main St.**			**Main St.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	↑↑	Shared	↖	↑↑	↗	↖	↑↑	↗	↖	↑↑	Shared
Traffic Volume	188	321	25	118	453	39	71	307	193	73	434	132
Pedestrians	7	-	4	4	-	7	5	-	5	5	-	5
PHF	0.92	0.8	0.69	0.92	0.83	0.7	0.74	0.89	0.8	0.83	0.92	0.92
Heavey Veh. %	4	2	4	0	1	0	0	3	3	1	1	2
Ideal Sat. Flow	1775	1775	1775	1775	1900	1750	1775	1900	1750	1775	1775	1775

Intersection Information			
Intersection:	King St. @ Northfield Dr.	**Bus Route:**	21
Segment:	KING St. @ Home Depot to KING St. @ Northfield Dr.		

Dates of used Data			
AVL/APC data:	2011	**Turning Movement Counts:**	2011

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	25.6	54	119
2013	23.4	46	103

Synchro Input Data												
Street	**Knig St.**			**Knig St.**			**Northfield Dr.**			**Northfield Dr.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	↑↑	↗	↖	↑↑	↗	↖	↑↑	Shared	↖	↑↑	Shared
Traffic Volume	208	373	254	139	542	206	95	690	78	283	771	92
Pedestrians	31	-	10	10	-	31	2	-	13	13	-	2
PHF	0.87	0.91	0.87	0.25	0.95	0.9	0.82	0.94	0.72	0.86	0.88	0.77
Heavey Veh. %	5	3	2	1	2	0	4	2	9	2	3	2
Ideal Sat. Flow	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900

Intersection Information			
Intersection:	VICTORIA St. @ Belmont Ave.	**Bus Route:**	8
Segment:	HIGHLAND Rd. @ Belmon Ave. to VICTORIA St. @ Belmont Ave.		

Dates of used Data			
AVL/APC data:	2011	**Turning Movement Counts:**	2011

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	9.1	3	76
2013	10.5	55.6	106

Synchro Input Data												
Street	**Belmont Ave.**			**Belmont Ave.**			**Victoria St.**			**Victoria St.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	↑↑	↗	Shared	↑↑	Shared	↖	↑	↗	↖	↑	Shared
Traffic Volume	56	452	75	144	760	122	92	735	37	124	1020	108
Pedestrians	9	-	16	16	-	9	3	-	5	5	-	3
PHF	0.88	0.9	0.69	0.82	0.88	0.85	0.85	0.94	0.84	0.82	0.94	0.89
Heavey Veh. %	0	1	3	1	1	2	1	3	0	2	2	3
Ideal Sat. Flow	1775	1900	1750	1775	1650	1000	1000	1775	1750	1000	1650	1000

Intersection Information			
Intersection:	ERB St. @ Fischer Hallman Rd.	**Bus Route:**	5
Segment:	ERB St. @ Amos Ave to ERB St. @ Fischer Hallman Rd.		

Dates of used Data			
AVL/APC data:	2013	**Turning Movement Counts:**	2013

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	3.9	17	197
2013	9	45.5	30

Synchro Input Data												
Street	**Fischer Hallman Rd.**			**Fischer Hallman Rd.**			**Erb St.**			**Erb St.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	↑↑	Shared	↖	↑↑	Shared	↖	↑↑	Shared	↖	↑↑	↗
Traffic Volume	148	417	90	123	615	159	115	316	73	118	918	128
Pedestrians	21	-	9	9	-	21	54	-	16	16	-	54
PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.84	0.93	0.84	0.96	0.97	0.79
Heavey Veh. %	1	2	2	1	2	1	2	2	1	2	1	3
Ideal Sat. Flow	1775	1775	1775	1775	1775	1775	1775	1775	1775	1775	1775	1775

Intersection Information			
Intersection:	Hespeler Rd. @ Dunbar Rd.	**Bus Route:**	200
Segment:	HESPELER Rd. @ Can Amera Pkwy. &Y MCA Driveway to HESPELER Rd. @ Dunbar Rd.		

Dates of used Data			
AVL/APC data:	2013	**Turning Movement Counts:**	2013

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	3.3	14	210
2013	3.9	17.6	103

Synchro Input Data

Street	**Hespeler Rd.**			**Hespeler Rd.**			**Dunbar Rd.**			**Dunbar Rd.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	↑↑	↗	↖	↑↑	↗	↖	↑	↗	↖	↑	↗
Traffic Volume	147	1021	198	139	1424	29	226	150	33	247	138	75
Pedestrians	2	-	5	5	-	2	1	-	1	1	-	1
PHF	0.95	0.88	0.89	0.7	0.97	0.89	0.77	0.79	0.74	0.81	0.84	0.73
Heavey Veh. %	3	3	7	6	3	0	0	5	0	5	4	10
Ideal Sat. Flow	1775	1820	1000	1775	1900	1750	1775	1900	1775	1775	1900	1775

Intersection Information												
Intersection:	Northfield Dr. @ Bridge St.										**Bus Route:**	35
Segment:	BRIDGE St. @ Lexington Rd. to NORTHFIELD Dr. @ Bridge St.											

Dates of used Data			
AVL/APC data:	2011	**Turning Movement Counts:**	2011

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	10.9	32	91
2013	N/A	N/A	N/A

Synchro Input Data												
Street	**Northfield Dr.**			**Northfield Dr.**			**Bridge St.**			**Bridge St.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	↑	↗	↖	↑	Shared	↖	↑	Shared	↖	↑	↗
Traffic Volume	28	474	327	59	393	20	82	404	47	191	120	43
Pedestrians	0	-	0	0	-	0	0	-	0	0	-	0
PHF	0.78	0.9	0.95	0.74	0.91	0.63	0.76	0.77	0.56	0.94	0.75	0.6
Heavey Veh. %	0	3	1	0	3	10	2	1	0	3	5	2
Ideal Sat. Flow	1775	1900	1750	1775	1650	1775	1900	1750	1000	1775	1650	1000

Intersection Information			
Intersection:	University Ave. @ Keatway	**Bus Route:**	29
Segment:	FISCHER HALLMAN Rd. @ Keatsway to UNIVERSITY Ave. @ Keatsway		

Dates of used Data			
AVL/APC data:	2011	**Turning Movement Counts:**	2011

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	27.5	66.8	106
2013	34.7	72.4	91

Synchro Input Data												
Street	**Keatsway**			**Keatsway**			**University Ave.**			**University Ave.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	N/A	N/A	N/A	↖	N/A	↗	↖	↑↑	N/A	N/A	↑↑	↗
Traffic Volume	N/A	N/A	N/A	193	N/A	8	14	342	N/A	N/A	824	432
Pedestrians	N/A	N/A	N/A	0	N/A	0	0	-	N/A	N/A	-	0
PHF	N/A	N/A	N/A	0.85	N/A	0.66	0.7	0.94	N/A	N/A	0.94	0.91
Heavey Veh. %	N/A	N/A	N/A	4	N/A	25	0	1	N/A	N/A	0	2
Ideal Sat. Flow	N/A	N/A	N/A	1775	N/A	1750	1775	1900	N/A	N/A	1900	1775

Intersection Information			
Intersection:	Westmount Rd. @ Columbia St.	**Bus Route:**	13
Segment:	FISCHER-HALLMAN Rd. @ Columbia St. to WESTMOUNT Rd. @ Columbia St.		

Dates of used Data			
AVL/APC data:	2011	**Turning Movement Counts:**	2011

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	26.8	56.6	91
2013	23.8	58	121

Synchro Input Data												
Street	**Westmount Rd.**			**Westmount Rd.**			**Columbia Rd.**			**Columbia Rd.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	↑↑	Shared	↖	↑↑	↗	↖	↑↑	↗	↖	↑↑	Shared
Traffic Volume	339	253	87	22	336	110	117	419	48	306	1109	16
Pedestrians	5	-	9	9	-	5	9	-	16	16	-	9
PHF	0.86	0.9	0.91	0.69	0.9	0.56	0.55	0.91	0.86	0.88	0.93	0.57
Heavey Veh. %	0	2	1	14	1	1	4	1	0	0	1	0
Ideal Sat. Flow	1775	1775	1000	1775	1900	1750	1775	1900	1750	1775	1775	1000

Intersection Information			
Intersection:	FISCHER-HALLMAN Rd. @ Columbia St.	**Bus Route:**	13
Segment:	WESTMOUNT Rd. @ Columbia St. to FISCHER-HALLMAN Rd. @ Columbia St.		

Dates of used Data			
AVL/APC data:	2011	**Turning Movement Counts:**	2011

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	25.7	51.1	273
2013	0.3	0	90

Synchro Input Data												
Street	**Fischer Hallman Rd.**			**Fischer Hallman Rd.**			**Columbia St.**			**Columbia St.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	↑↑	↗	↖	↑↑	↗	↖	↑↑	↗	↖	↑	↗
Traffic Volume	77	241	101	191	525	157	110	235	5	418	75	443
Pedestrians	10	-	4	4	-	10	37	-	2	2	-	37
PHF	0.66	0.85	0.68	0.75	0.92	0.68	0.67	0.65	0.42	0.74	0.88	0.67
Heavey Veh. %	3	1	3	3	2	0	13	3	0	1	2	2
Ideal Sat. Flow	1775	1900	1750	1775	1900	1750	1775	1900	1750	1775	1900	1750

Intersection Information												
Intersection:	FISCHER-HALLMAN RD. @ University Ave.									**Bus Route:**		201
Segment:	FISCHER-HALLMAN Rd. @ Thorndale Dr. to FISCHER-HALLMAN Rd. @ University Ave.											
Dates of used Data												
AVL/APC data:			2011			**Turning Movement Counts:**				2011		
AVL/APC Data												
	Mean Stopped Delay				**90th Percentile Delay**				**Maximum Queue Length**			
2011	36				63				89			
2013	15.7				61				91			
Synchro Input Data												
Street	**Fischer Hallman Rd.**			**Fischer Hallman Rd.**			**University Ave.**			**University Ave.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	↖	↑↑	↗	↖	↑↑	↗	↖	↑↑	↗	↖	↑↑	Shared
Traffic Volume	300	888	145	29	957	129	133	174	147	535	548	34
Pedestrians	15	-	6	6	-	15	5	-	7	7	-	5
PHF	0.83	0.95	0.84	0.56	0.94	0.85	0.9	0.87	0.92	0.98	0.96	0.61
Heavey Veh. %	1	2	1	0	2	1	0	0	1	0	0	0
Ideal Sat. Flow	1775	1775	1000	1775	1775	1000	1775	1900	1750	1775	1775	1000

Intersection Information			
Intersection:	Northfield Dr. @ Parkside Dr.	**Bus Route:**	200
Segment:	WEBER St. @ Parkside Dr. to NORTHFIELD Dr. @ Parkside Dr.		

Dates of used Data			
AVL/APC data:	2013	**Turning Movement Counts:**	2013

AVL/APC Data			
	Mean Stopped Delay	**90th Percentile Delay**	**Maximum Queue Length**
2011	25.7	51.1	273
2013	0.3	0	90

Synchro Input Data												
Street	**Parkside Dr.**			**Parkside Dr.**			**Northfield Dr.**			**Northfield Dr.**		
	NBL	**NBT**	**NBR**	**SBL**	**SBT**	**SBR**	**EBL**	**EBT**	**EBR**	**WBL**	**WBT**	**WBR**
Lanes	Shared	↑	↗	↖	↑	Shared	↖	↑↑	Shared	↖	↑↑	Shared
Traffic Volume	26	0	375	16	1	1	0	1154	29	314	1516	18
Pedestrians	1	-	3	3	-	1	4	-	3	3	-	4
PHF	0.85	0.25	0.92	0.8	0.65	0.69	0.5	0.9	0.58	0.97	0.94	0.72
Heavey Veh. %	6	50	1	0	8	0	100	3	2	3	2	12
Ideal Sat. Flow	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900

Appendix B

Signal Timing for Intersections

Appendix B. includes signal timing for the intersections used in this study. It also includes the phasing and splitting settings for different approaches.

Note: Time of the day that the signal timing is in effect is represented in 24 hour format.

Intersection: **WEBER STREET at Bridgeport Road**

Fixed time operation with actuated northbound protected left turn phase.

Time in effect	14:00-19:00 Monday-Friday			
Weber Street				
WBL Green Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	11.0		
WBL Amber Arrow		3.0		
All Red		1.0		
Green		30.0	Walk	20.0
Amber		4.0	FDW	10.0
All Red		2.0		
Bridgeport Road				
Green		23.0	Walk	12.0
Amber		4.0	FDW	12.0
All Red		2.0		
Total Cycle		80.0		

Intersection: **CHARLES STREET & ONTARIO STREET**

Fixed time operation

Time in effect 14:00 - 19:00, Monday to Friday

CHARLES STREET

Green	42.0	Walk	36.0
Amber	4.0	FDW	6.0
All Red	2.0		

ONTARIO STREET

Green	26.0	Walk	19.0
Amber	4.0	FDW	7.0
All Red	2.0		
Total Cycle	80.0		

Intersection: **FISCHER-HALLMAN ROAD & COLUMBIA STREET**

Fixed time operation.

Actuated protected/permissive left-turn phases in all directions.

Time in effect	14:00-19:00 Monday-Friday			
FISCHER-HALLMAN ROAD				
N/S Green Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	14.0		
N/S Amber				
Arrow		3.0		
All Red		1.0		
Green		34.0	Walk	13.0
Amber		4.0	FDW	21.0
All Red		2.0		
COLUMBIA STREET				
E/W Green				
Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	14.0		
E/W Amber				
Arrow		3.0		
All Red		1.0		
Green		38.0	Walk	17.0
Amber		4.0	FDW	21.0
All Red		2.0		
Total Cycle		120.0		

Intersection: **COLUMBIA ST. & WESTMOUNT RD.**

Semi-Actuated Operation.

Actuated protected/permissive left-turn phases in all directions.

Time in effect 15:00 - 19:00 Monday to Friday

COLUMBIA STREET

EB Green Arrow	Min.	5.0	WB Green Arrow		Min.	5.0
	Ext.	3.0			Ext.	3.0
	Max.	8.0			Max.	14.0
E/W Amber Arrow		3.0				
All Red		1.0				
Green		48.0	Walk	24.0		
Amber		4.0	FDW	24.0		
All Red		2.0				

WESTMOUNT ROAD

SB Green Arrow	Min.	5.0	NB Green Arrow		Min.	5.0
	Ext.	3.0			Ext.	3.0
	Max.	7.0			Max.	11.0
E/W Amber Arrow		3.0				
All Red		1.0				
Green		37.0	Walk	7.0		
Amber		4.0	FDW	20.0		
All Red		2.0	SDW	10.0		
Total Cycle		130.0				

Intersection: **HOMER WATSON BLVD. at Conestoga College Blvd.**

Semi-actuated operation with actuated northbound, southbound, eastbound and westbound protected left turn phases

Time in effect 14:00-19:30 Monday- Friday

Homer Watson Boulevard

SBL Green Arrow	Min.	5.0	NBL Green Arrow		Min.	5.0
	Ext.	3.0			Ext.	3.0
	Max.	15.6			Max.	9.2
SBL Amber Arrow		4.0	NBL Amber Arrow			3.0
All Red		2.0	All Red			1.0
Green		37.6				
Amber		4.2	Walk	18.6		
All Red		1.6	FDW	19.0		

Conestoga College Boulevard

EBL Green Arrow	Min.	5.0	WBL Green Arrow		Min.	5.0
	Ext.	3.0			Ext.	3.0
	Max.	11.0			Max.	13.6
EBL Amber Arrow		3.0	WBL Amber Arrow			3.0
All Red		1.0	All Red			1.0
Green		31.0				
Amber		3.7	Walk	7.0		
All Red		2.7	FDW	24.0		
Total Cycle		120.0				

Intersection: **HESPELER RD/WATER ST at Coronation Blvd/Dundas St**
Fully-Actuated operation with protected right turns for eastbound and westbound vehicles overlapping the northbound and southbound left turn phases respectively

Time in effect 15:00-19:00 Monday to Friday

Hespeler Road/Water Street

SBL Green Arrow	Min.	7.0	NBL Green Arrow		Min.	7.0
	Ext.	3.0			Ext.	3.0
	Max.	22.9			Max.	22.9
SBL Amber Arrow		3.3	NBL Amber Arrow			3.3
All Red		5.3	All Red			5.3
Green		51.9	corssing Coronation		corssing Dundas	
Amber		3.3	Walk	18.6	Walk	18.6
All Red		4.8	FDW	19.0	FDW	19.0

Dundas Street/Coronation Blvd.

EBL Green Arrow	Min.	7.0	WBL Green Arrow		Min.	7.0
	Ext.	3.0			Ext.	3.0
	Max.	14.2			Max.	8.2
EBL Amber Arrow		3.3	WBL Amber Arrow			3.3
All Red		3.5	All Red			3.5
Green		28.4	Crossing Water			
Amber		3.3	Walk	7.0		
All Red		5.8	FDW	20.0		
Total Cycle		150.0				

Intersection: **DUNDAS STREET & MAIN STREET**

Fixed time operation
Northbound/southbound protected/permissive left-turn phases on Dundas St
Eastbound/westbound protected/permissive left-turn phases on Main St

Time in effect	15:00 - 19:00 Monday to Friday			
DUNDAS STREET				
N/S Green Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	9.2		
N/S Amber Arrow		3.0		
All Red		1.0		
Green		35.0	Walk	13.0
Amber		3.3	FDW	22.0
All Red		3.5		
MAIN STREET				
E/W Green Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	9.2		
E/W Amber Arrow		3.0		
All Red		1.0		
Green		35.2	Walk	11.2
Amber		3.3	FDW	24.0
All Red		3.3		
Total Cycle		110.0		

Intersection: **ERB STREET & FISCHER-HALLMAN ROAD**

Fixed time operation

Actuated left turn arrows for Erb Street and Fischer-Hallman Road

Time in effect	15:00 - 19:00 Monday to Friday			
ERB STREET				
N/S Green Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	10.0		
N/S Amber Arrow		3.0		
All Red		1.0		
Green		39.0	Walk	18.0
Amber		4.0	FDW	21.0
All Red		2.0		
FISCHER-HALLMAN ROAD				
E/W Green Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	12.0		
E/W Amber Arrow		3.0		
All Red		1.0		
Green		39.0	Walk	24.0
Amber		4.0	FDW	15.0
All Red		2.0		
Total Cycle		120.0		

Intersection: **WESTMOUNT ROAD & ERB STREET**

Fixed time operation

Actuated NBL and SBL on Westmount Rd. Actuated EBL and WBL on Erb. St.

Time in effect 14:30 - 19:00 Monday to Friday

WESTMOUNT ROAD

NBL Green Arrow	Min.	5.0	SBL Green Arrow		Min.	5.0
	Ext.	3.0			Ext.	3.0
	Max.	17.0			Max.	12.0
NBL Amber Arrow		3.0	SBL Amber Arrow			3.0
All Red		1.0	All Red			1.0
Green		36.0	Walk	26.0		
Amber		4.0	FDW	10.0		
All Red		2.0				

ERB STREET

E/W Green Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	7.0		
E/W Amber Arrow		3.0		
All Red		1.0		
Green		30.0	Walk	17.0
Amber		4.0	FDW	13.0
All Red		2.0		
Total Cycle		110.0		

Intersection: **Fairway Road at King Street**

Fixed time operation with actuated westbound, eastbound and northbound left-turn phasing. Note: Westbound left-turn phase is protected/permissive. Eastbound and northbound left-turn phases are fully protected. Southbound right-turn arrow overlaps with the eastbound left turn phase.

Time in effect 15:00-22:00 Monday to Friday

Fairway Road						
EBL Green Arrow	Min.	5.0	WBL Green Arrow		Min.	5.0
	Ext.	3.0			Ext.	3.0
	Max.	24.0			Max.	5.6
EBL Amber Arrow		4.0	WBL Amber Arrow			3.0
All Red		2.0	All Red			1.0
Green		28.8	Walk	8.0		
Amber		4.0	FDW	20.0		
All Red		2.0				
King Street						
NBL Green Arrow	Min.	5.0				
	Ext.	3.0				
	Max.	17.6				
NBL Amber Arrow		3.0				
All Red		1.0				
Green		27.6	Walk	6.6		
Amber		4.0	FDW	21.0		
All Red		2.0				
Total Cycle		120.0				

Intersection: **Fairway Road @ Fairview Park Mall/ Best-Buy Plaza**

Semi-actuated operation. Pushbuttons used to cross Fairway Road

Actuated Protected/Permissive eastbound/westbound left-turn phases on Fairway Road

Time in effect	16:00 - 18:00 Monday to Friday			
Fairway Road				
E/W Green Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	19.0		
E/W Amber Arrow		3.0		
All Red		1.0		
Green		52.0	Walk	28.0
Amber		4.0	FDW	24.0
All Red		2.0		
Fairview Park Mall				
Green	Min.	7.0	Walk	12.0
	Ext.	3.0	FDW	23.0
	Max.	43.0	SDW	8
Amber		4.0		
All Red		2.0		
Total Cycle		130.0		

Intersection: **Fairway Road @ Wilson Avenue**

SEMI-ACTUATED- Pushbuttons in place to cross Fairway Road
Actuated left turn phases on Fairway Rd. are fully protected
Actuated left turn phases on Wilson Ave are protected/permissive

Time in effect 16:00 - 18:00 Monday to Friday

Fairway Road				
E/W Green Arrow	Min.	7.0		
	Ext.	5.0		
	Max.	29.0		
EBL Amber Arrow		3.0		
All Red		1.0		
Green		36.0	Walk	19.0
Amber		4.0	FDW	17.0
All Red		2.0		
Wilson Avenue				
E/W Green Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	9.0		
EBL Amber Arrow		3.0		
All Red		1.0		
Green	Min.	8.0	Walk	7.0
	Ext.	3.0	FDW	25.0
	Max.	36.0	SDW	4
Amber		4.0		
All Red		2.0		
Total Cycle		130.0		

Intersection: **FISCHER-HALLMAN ROAD & UNIVERSITY AVENUE**

Fixed time operation

Actuated protected/permissive left-turn phases in all directions.

Time in effect 15:00 - 19:00 Monday to Friday

FISCHER-HALLMAN ROAD				
S/N Green Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	7.0		
S/N Amber Arrow		3.0		
All Red		1.0		
Green		50.0	Walk	25.0
Amber		4.0	FDW	25.0
All Red		2.0		
UNIVERSITY AVENUE				
E/W Green Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	7.0		
E/W Amber Arrow		3.0		
All Red		1.0		
Green		36.0	Walk	14.0
Amber		4.0	FDW	22.0
All Red		2.0		
Total Cycle		120.0		

Intersection: **FISCHER-HALLMAN ROAD at Thorndale Drive**

Semi-actuated operation with actuated northbound and southbound protected left turn phases.

Time in effect 15:00-19:00 Monday- Friday

Fischer-Hallman Road

NBL Green Arrow	Min.	5.0	SBL Green Arrow		Min.	5.0
	Ext.	3.0			Ext.	3.0
	Max.	10.0			Max.	7.0
NBL Amber Arrow		3.0	SBL Amber Arrow			3.0
All Red		1.0	All Red			1.0
Green		62.0	Walk	49.0		
Amber		4.0	FDW	13.0		
All Red		2.0				

Thorndale Drive

Green	Min.	8.0	Walk	15.0
	Ext.	3.0	FDW	17.0
	Max.	32.0		
Amber		4.0		
All Red		2.0		
Total Cycle		120.0		

Intersection: **HESPELER ROAD @ DUNBAR ROAD**

Semi-Actuated Operation.
Actuated protected/permissive NB/SB/WB left-turn phases.
Intersection equipped with Transit Signal Priority.

Time in effect 14:00 - 19:00 Monday to Friday

Hespeler Road

NBL Green Arrow	Min.	7.0	SBL Green Arrow		Min.	7.0
	Ext.	3.0			Ext.	3.0
	Max.	11.6			Max.	8.0
NBL Amber Arrow		3.0				
All Red		1.0				
Green		56.4	Walk	39.4		
Amber		4.0	FDW	17.0		
All Red		2.0				

Dunbar Road

WBL Green Arrow	Min.	7.0		
	Ext.	3.0		
	Max.	14.0		
WBL Amber Arrow		3.0		
All Red		1.0		
Green	Min.	8.0	Walk	10.0
	Ext.	3.0	FDW	23.0
	Max.	18.0	SDW	3
Amber		4.0		
All Red		2.0		
Total Cycle		120.0		

Intersection: **HESPELER ROAD at Eagle Street/ Pinebush Road**

Fully-Actuated- operates Semi-Actuated during peak periods and is part of a coordinated signal network along Hespeler Road, Eagle Street and Pinebush Road

Left Info	Actuated	SB	LT	-	Hespeler Road		Leading Fully Protected LT Phase
	Actuated	NB	LT	-	Hespeler Road		Lagging Fully Protected LT Phase
	Actuated	EB	LT	-	Eagle Street		Leading Fully Protected LT Phase
	Actuated	WB	LT	-	Pinebush Road		Lagging Fully Protected LT Phase
Right Info	Actuated	NB	RT	-	Hespeler Road	via	Overlap A (Phase 3 WBLT)

Time in effect	14:00 - 19:00 Monday to Friday					
Hespeler Road						
SBL Green Arrow	Min.	7.0	NBL Green Arrow		Min.	7.0
	Ext.	5.0			Ext.	3.0
	Max.	20.1			Max.	15.9
SBL Amber Arrow		3.7	NBL Amber Arrow			3.7
All Red		2.8	All Red			2.8
Green		25.2	Walk	10.0		
Amber		3.7	FDW	19.0		
All Red		2.8	SDW	5.8		
EAGLE STREET						
EBLT	Min.	7.0	Walk	8.0		
	Ext.	5.0	FDW	26.0		
	Max.	28.0				
Amber		3.7				
All Red		2.9				
PINEBUSH ROAD						
WBLT	Min.	7.0	Walk	8.0		
	Ext.	5.0	FDW	21.0		
	Max.	18.0				
Amber		3.7				
All Red		3.0				
Total Cycle		140.0				

Intersection: **HOMER WATSON BOULEVARD @ Doon Villiage/Manitou Drive**
Fully-actuated, semi-actuated during PM peak (co-ordinated along Homer-Watson)

Left Info	Actuated	WB LT	-	Homer Watson Blvd
	Actuated	EB LT	-	Homer Watson Blvd
	Actuated	NB LT	-	Doon Village Drive
	Actuated	SB LT	-	Manitou Drive DUAL LT (protected only)

Time in effect 14:00 - 19:00 Monday to Friday

HOMER WATSON BOULEVARD

EBL Green Arrow	Min.	5.0	WBL Green Arrow		Min.	5.0
	Ext.	3.0			Ext.	5.0
	Max.	10.0			Max.	10.0
E/W Amber Arrow		3.0				
All Red		1.0				
Green		36.9	Walk	10.0		
Amber		4.2	FDW	17.0		
All Red		2.7				

MANITOU/DOON VILLIAGE

NBL Green Arrow	Min.	5.0	SBL Green Arrow		Min.	5.0
	Ext.	3.0	protected		Ext.	5.0
	Max.	12.0			Max.	30.0
NBL Amber Arrow		3.0	SBL Amber Arrow			4.0
All Red		1.0	All Red			2.0
Green	Min.	15.0	Walk	10.0		
	Ext.	5.0	FDW	19.0		
	Max.	19.0				
Amber		3.7				
All Red		3.5				
Total Cycle		120.0				

Intersection: **KING STREET & NORTHFIELD DRIVE**

Fixed time operation
Actuated left turn arrows on King St. & Northfield Dr.

Time in effect	14:30 - 19:00 Monday to Friday			
KING STREET				
N/S Green Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	9.0		
N/S Amber Arrow		3.0		
All Red		1.0		
Green		32.0	Walk	13.0
Amber		4.0	FDW	19.0
All Red		2.0		
NORTHFIELD DRIVE				
E/W Green Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	10.0		
E/W Amber Arrow		3.0		
All Red		1.0		
Green		39.0	Walk	17.0
Amber		4.0	FDW	22.0
All Red		2.0		
Total Cycle		110.0		

Intersection: **King Street @ Conestoga Mall/Conestogo Road**

Semi-Actuated - Eastbound leg moves separately from the westbound leg due to existing dual left turn lanes at Conestoga Mall.

Any unused time from Phase 4 (Conestoga Rd.) is allocated to Conestoga Mall phase 8.

Time in effect 14:30 – 19:00 Monday to Friday

KING STREET				
N/S Green Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	10.0		
N/S Amber Arrow		3.0		
All Red		1.0		
Green		38.0	Walk	25.0
Amber		4.0	FDW	13.0
All Red		2.0		
CONESTOGO ROAD				
Green	Min.	7.0	Walk	7.0
	Ext.	3.0	FDW	17.0
	Max.	11.0		
Amber		4.0		
All Red		2.0		
CONESTOGA MALL				
Green	Min.	7.0	Walk	7.0
	Ext.	3.0	FDW	20.0
	Max.	29.0		
Amber		4.0		
All Red		2.0		
Total Cycle		110.0		

Intersection: **VICTORIA STREET at Natchez Road**

Semi-actuated operation with actuated westbound protected left turn phase

Time in effect	14:00 – 19:00 Monday to Friday			
Victoria Street				
N/S Green Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	13.0		
N/S Amber Arrow		3.0		
All Red		1.0		
Green		46.0	Walk	39.0
Amber		4.0	FDW	7.0
All Red		2.0		
Natchez Road				
Green	Min.	8.0	Walk	11.0
	Ext.	3.0	FDW	14.0
	Max.	25.0		
Amber		4.0		
All Red		2.0		
Total Cycle		100.0		

Intersection: **NORTHFIELD DRIVE & BRIDGE STREET**

Semi-Actuated Operation.

Actuated protected/permissive SB/WB left-turn phases.

Time in effect 15:00 - 19:00 Monday to Friday

NORTHFIELD DRIVE

SBL Green Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	7.0		
SBL Amber Arrow		3.0		
All Red		1.0		
Green		31.0	Walk	13.0
Amber		4.0	FDW	18.0
All Red		2.0		

BRIDGE STREET

WBL Green Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	9.0		
WBL Amber Arrow		3.0		
All Red		1.0		
Green	Min.	10.0	Walk	11.0
	Ext.	3.0	FDW	14.0
	Max.	33.0		
Amber		4.0		
All Red		2.0		
Total Cycle		100.0		

Intersection: **Northfield Drive @ Parkside Drive/NCR Driveway**

Semi-Actuated Operation.

Actuated protected/permissive westbound left-turn phase on Northfield Drive. Northbound right-turn arrow on Parkside Dr. overlaps with westbound left-turn phase on Northfield Dr.

Time in effect 16:30 - 18:00 Monday to Friday

Northfield Drive				
N/S Green Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	19.0		
N/S Amber Arrow		3.0		
All Red		1.0		
Green		52.0	Walk	41.0
Amber		4.0	FDW	11.0
All Red		2.0		
Parkside/NCR Driveway				
Green	Min.	8.0	Walk	9.0
	Ext.	3.0	FDW	15.0
	Max.	33.0	SDW	9.0
Amber		4.0		
All Red		2.0		
Total Cycle		120.0		

Intersection: **OTTAWA STREET at Homer-Watson Boulevard**

Intersection operates fixed timing with actuated eastbound, westbound, and northbound protected left turn phases. This intersection also operates with actuated northbound, southbound, and eastbound protected right turn phases overlapping with the left turn phases. Intersection runs free so no offsets are required.

Time in effect 15:00-20:00 Monday- Friday

Ottawa Street				
E/W Green Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	15.5		
E/W Amber Arrow		3.0		
All Red		1.0		
Green		35.0	Walk	14.0
Amber		3.7	FDW	21.0
All Red		3.2		
Homer-Watson Boulevard				
NBL Green Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	16.8		
NBL Amber Arrow		3.0		
All Red		1.0		
Green		41.2	Walk	21.2
Amber		4.2	FDW	20.0
All Red		2.4		
Total Cycle		130.0		

Intersection: **PINEBUSH RD @ CONESTOGA BLVD**

Semi-actuated operation. This signal operates in a co-ordinated traffic signal network along Pinebush Road. East/West protected/permissive left turn phasing on Pinebush North/South protected/permissive left turn phasing on Conestoga. Intersection equipped with Transit Signal Priority – NB/SB Conestoga Blvd. Solid Don't Walk (SDW) only when pedestrian phase extends to the maximum

Time in effect 12:00-20:00 Monday-Friday

Pinebush Road				
SBL Green Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	11.4		
SBL Amber Arrow		3.0		
All Red		1.0		
Green		58.4	Walk	41.4
Amber		4.0	FDW	17.0
All Red		2.0		
Conestoga Boulevard				
WBL Green Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	10.0		
WBL Amber Arrow		3.0		
All Red		1.0		
Green	Min.	8.0	Walk	10.0
	Ext.	3.0	FDW	18.0
	Max.	40.2	SDW	12.2
Amber		4.0		
All Red		2.0		
Total Cycle		140.0		

Intersection: **QUEEN STREET & Goebel Avenue**

Semi-actuated operation with actuated westbound protected left turn phase

Time in effect	15:30-18:00 Mon-Fri			
Queen Street				
WBL Green Arrow		5.0		
WBL Amber Arrow		3.0		
All Red		1.0		
Green		26.0	Walk	7.0
Amber		4.0	FDW	19.0
All Red		2.0		
Goebel Avenue				
Green	Min.	10.0	Walk	10.0
	Ext.	5.0	FDW	18.0
	Max.	35.0	SDW	12.2
Amber		4.0		
All Red		2.0		
Total Cycle		82.0		

Intersection: **University Avenue at Keats Way**

Semi-Actuated Operation.

Time in effect 14:30 - 19:00 Monday to Friday

University Avenue				
Green		61.0	Walk	53.0
Amber		4.0	FDW	8.0
All Red		2.0		

Keats Way				
Green	Min.	10.0	Walk	12.0
	Ext.	3.0	FDW	16.0
	Max.	37.0	SDW	9.0
Amber		4.0		
All Red		2.0		
Total Cycle		110.0		

Intersection: **VICTORIA ST & BELMONT AVENUE**

Fixed time operation.

Actuated left turn arrows on Victoria Street and southbound on Belmont Avenue.

Time in effect	15:00 - 20:00 Monday to Friday			
VICTORIA STREET				
E/W Green Arrow	Min.	5.0		
	Ext.	3.0		
	Max.	6.0		
E/W Amber Arrow		3.0		
All Red		1.0		
Green		42.0	Walk	29.0
Amber		4.0	FDW	13.0
All Red		2.0		
BELMONT AVENUE				
SBL Green Arrow		5.0		
SBL Amber Arrow		3.0		
All Red		1.0		
Green		27.0	Walk	17.0
Amber		4.0	FDW	10.0
All Red		2.0		
Total Cycle		100.0		

Intersection: **WEBER STREET @ Parkside Drive**

FIXED TIME operation

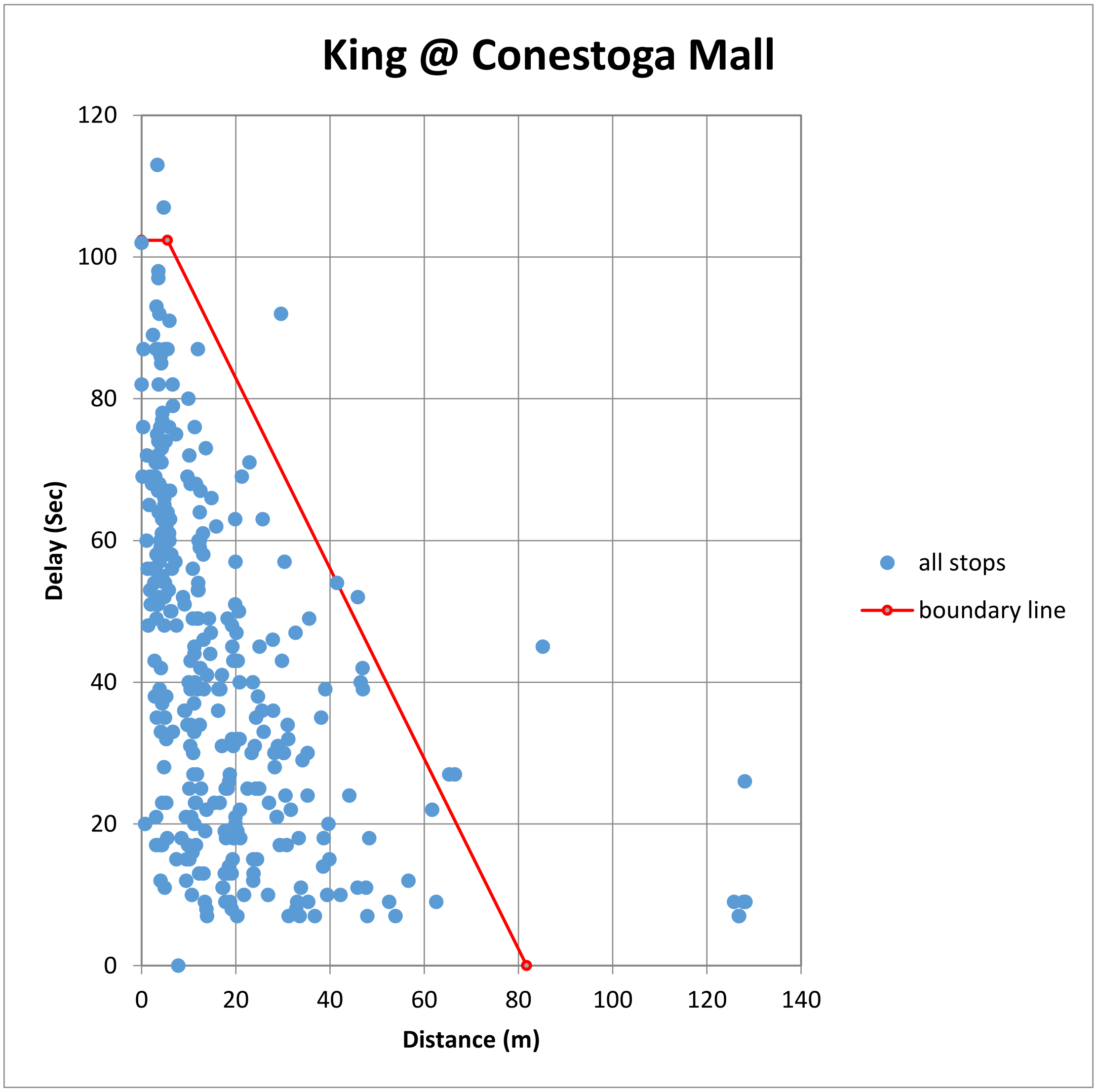
Time in effect 16:00-18:00 Monday to Friday

WEBER STREET			
Green	35.6	Walk	24.6
Amber	4.0	FDW	11.0
All Red	2.0		
Parkside Drive			
Green	32.4	Walk	22.4
Amber	4.0	FDW	10.0
All Red	2.0		
Total Cycle	80.0		

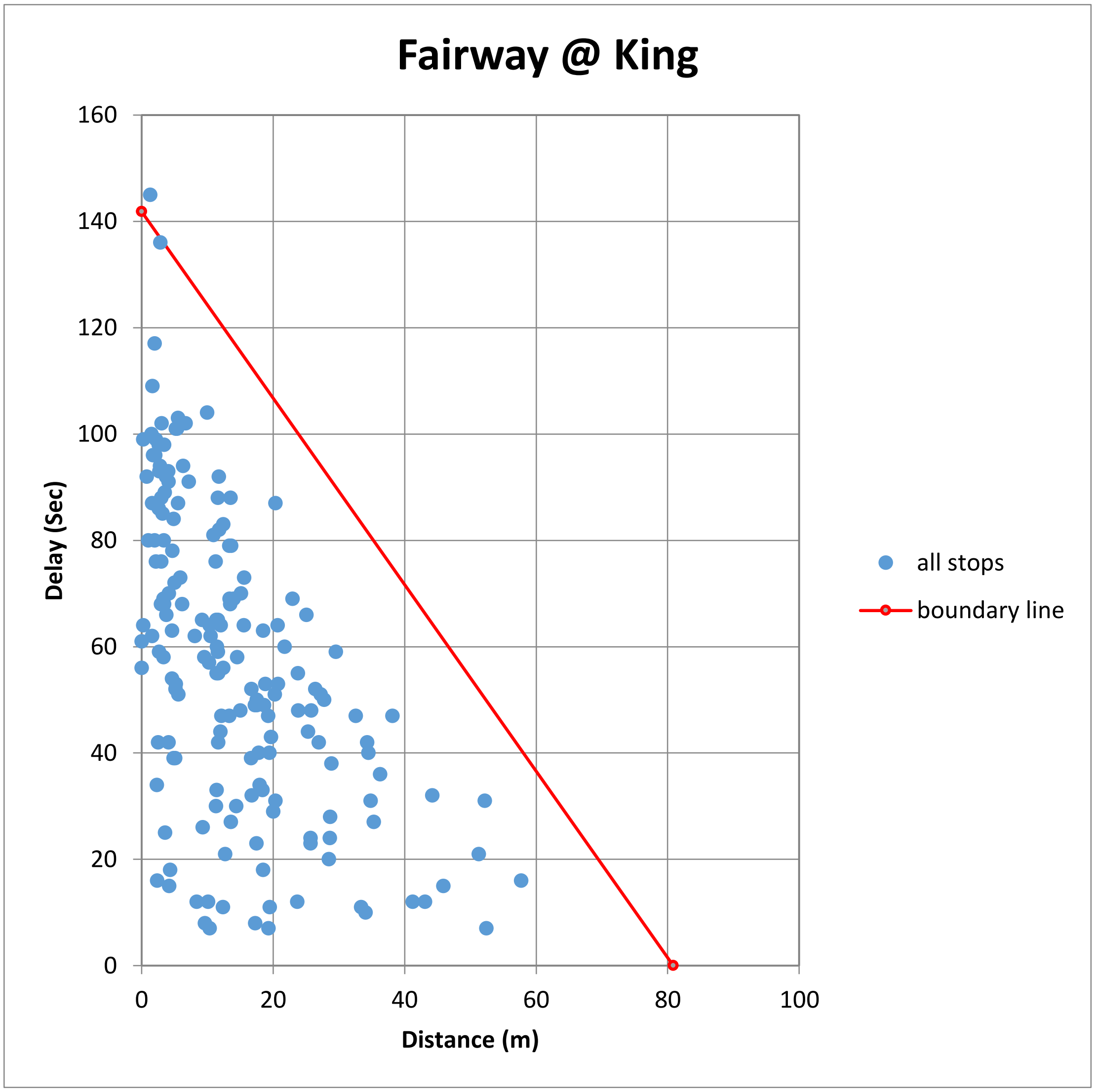
Appendix C

Fitted Delay Envelop to AVL/APC Stop Events

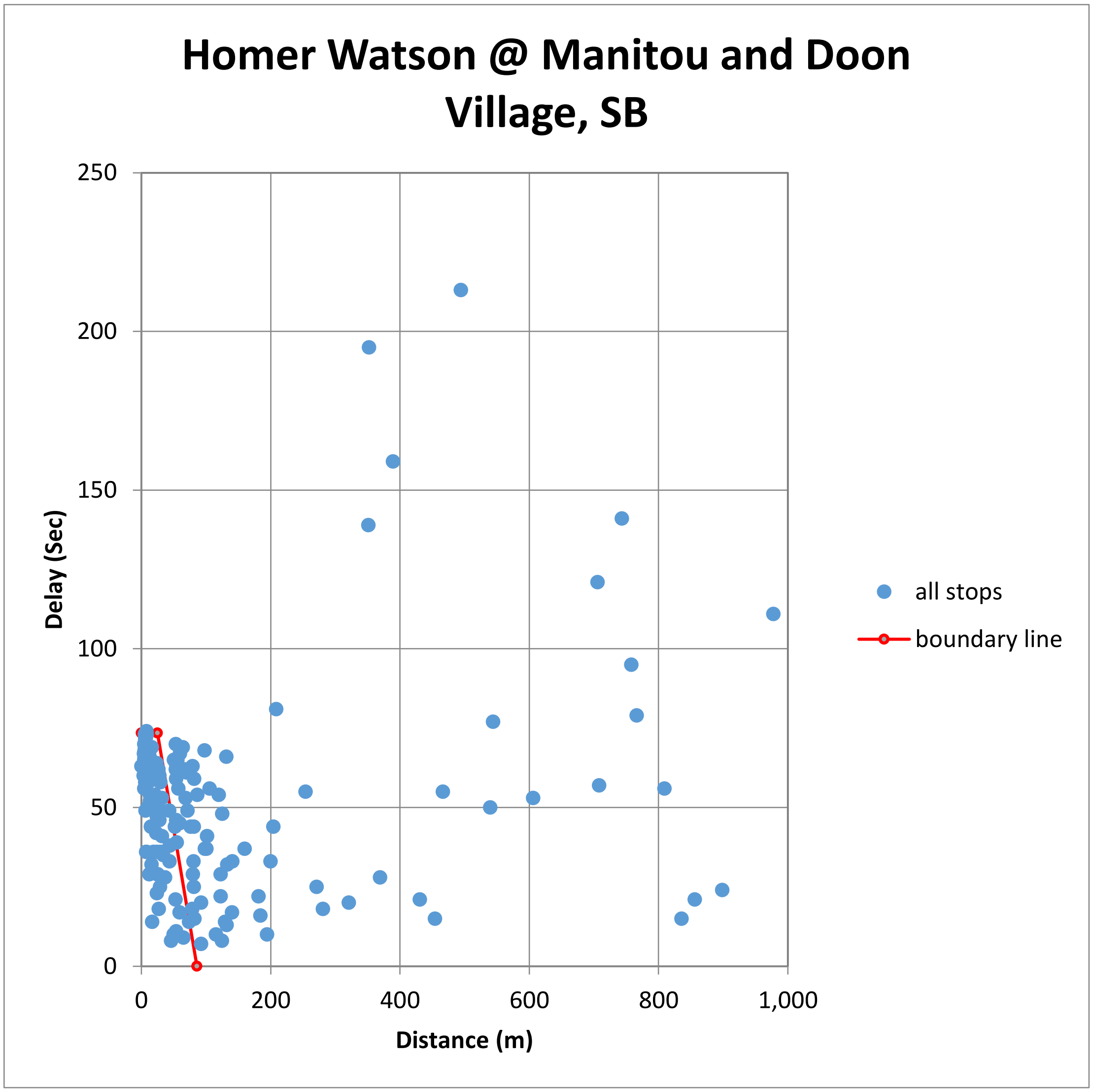
Appendix C includes the recorded unscheduled stop events for each defined segment plotted against the distance from the downstream intersection stopline. The selected boundary line is also fitted for each defined segment.


King @ Conestoga Mall
120
100
80
60
40
20
0
Delay (Sec)
0
20
40
60
80
100
120
140
Distance (m)
all stops
boundary line

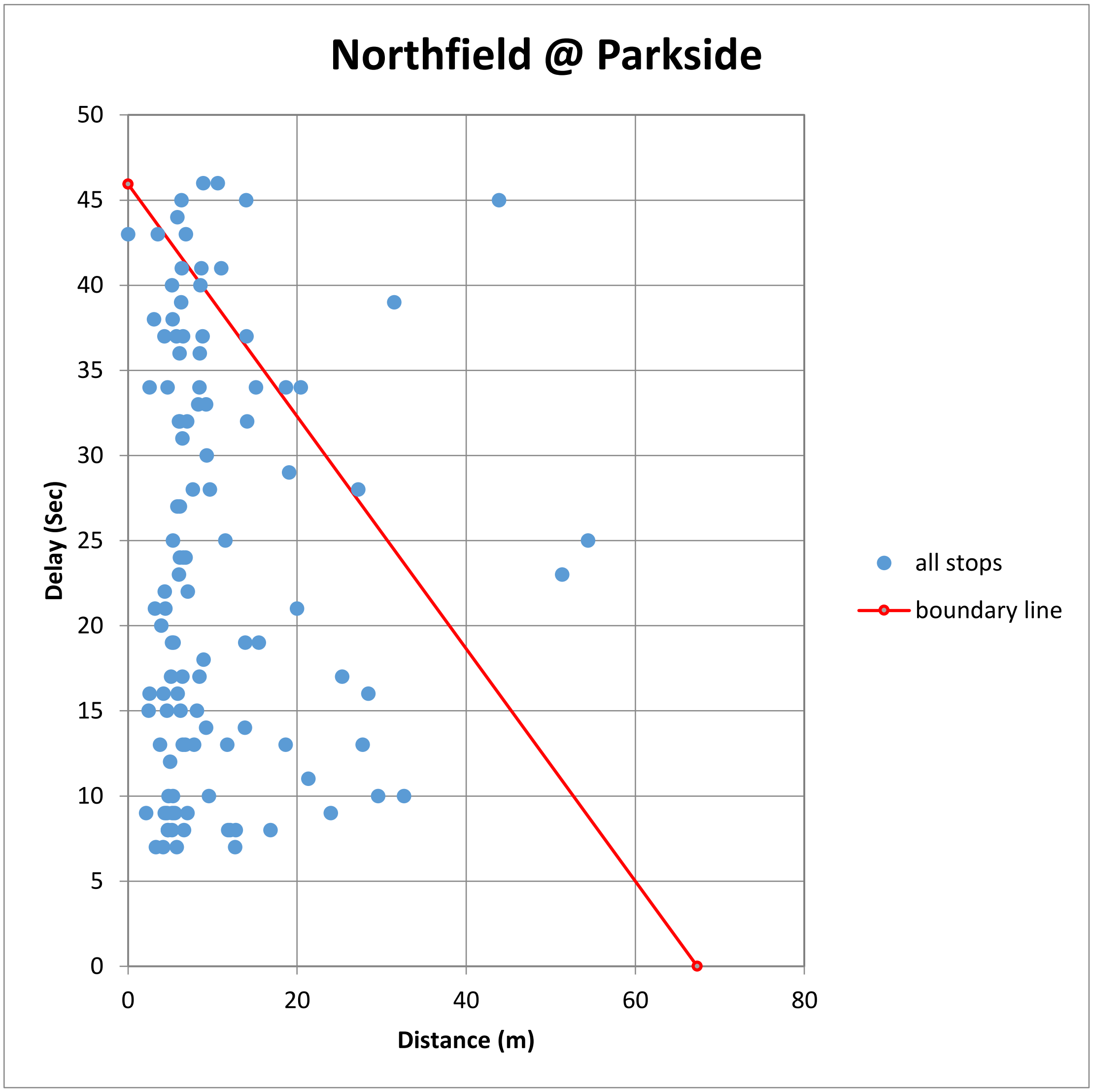
(Left Turn)


Fairway @ King
160
140
120
100
80
60
40
20
0
Delay (Sec)
0
20
40
60
80
100
Distance (m)
all stops
boundary line

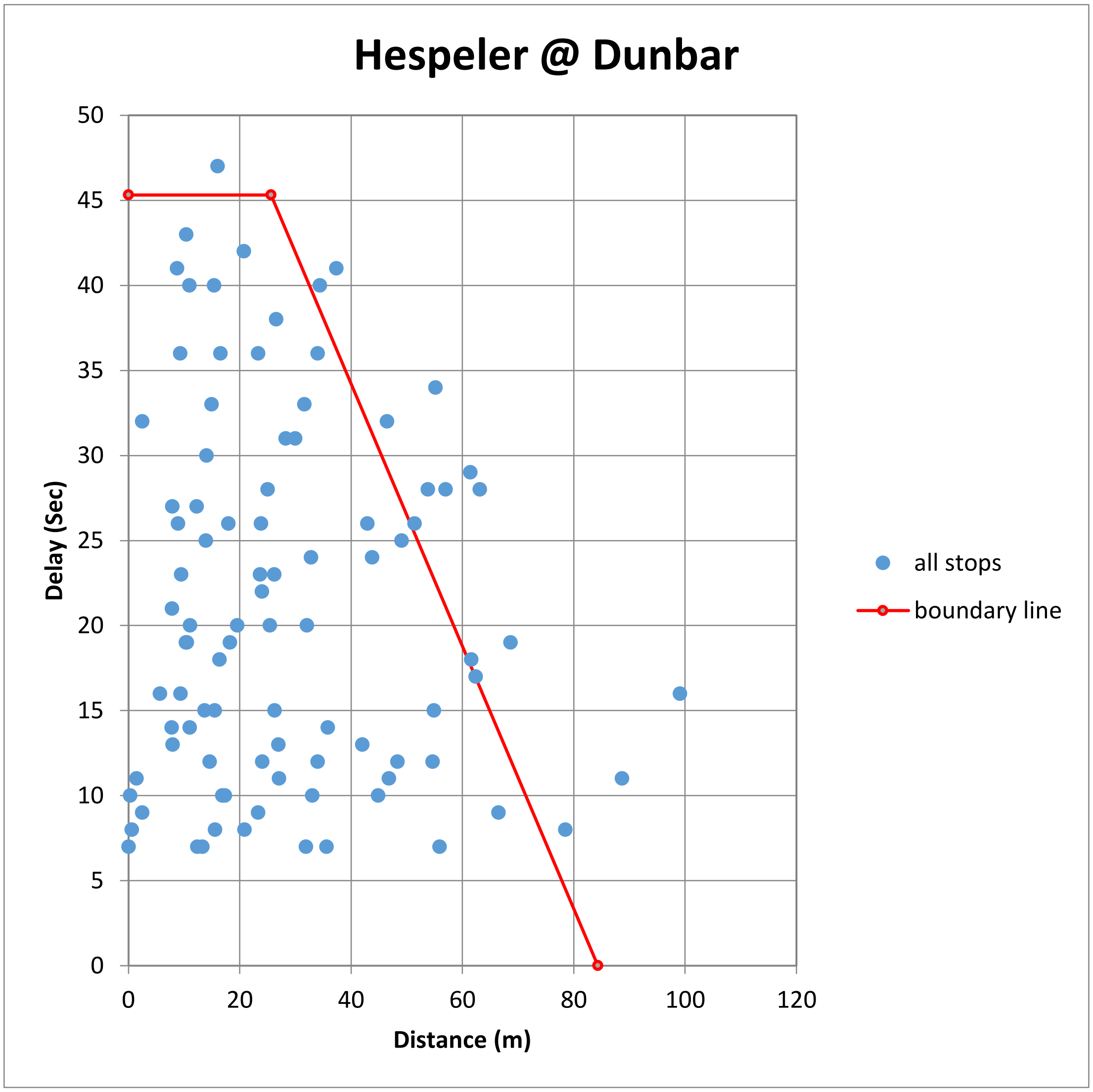
Left Turn



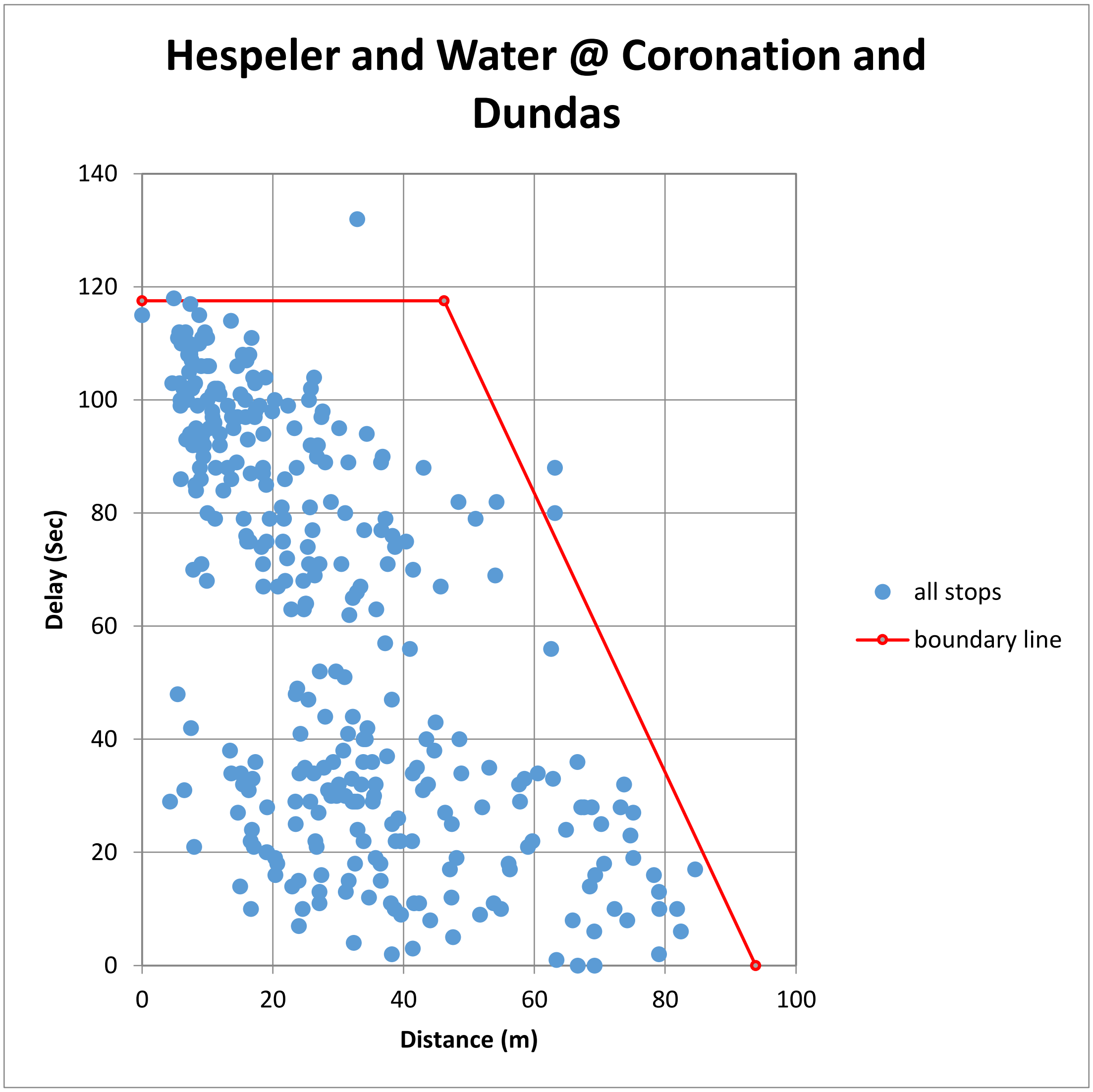
Through Movement


Northfield @ Parkside
Delay (Sec)
50
45
40
35
30
25
20
15
10
5
0
0
20
40
60
80
Distance (m)
all stops
boundary line

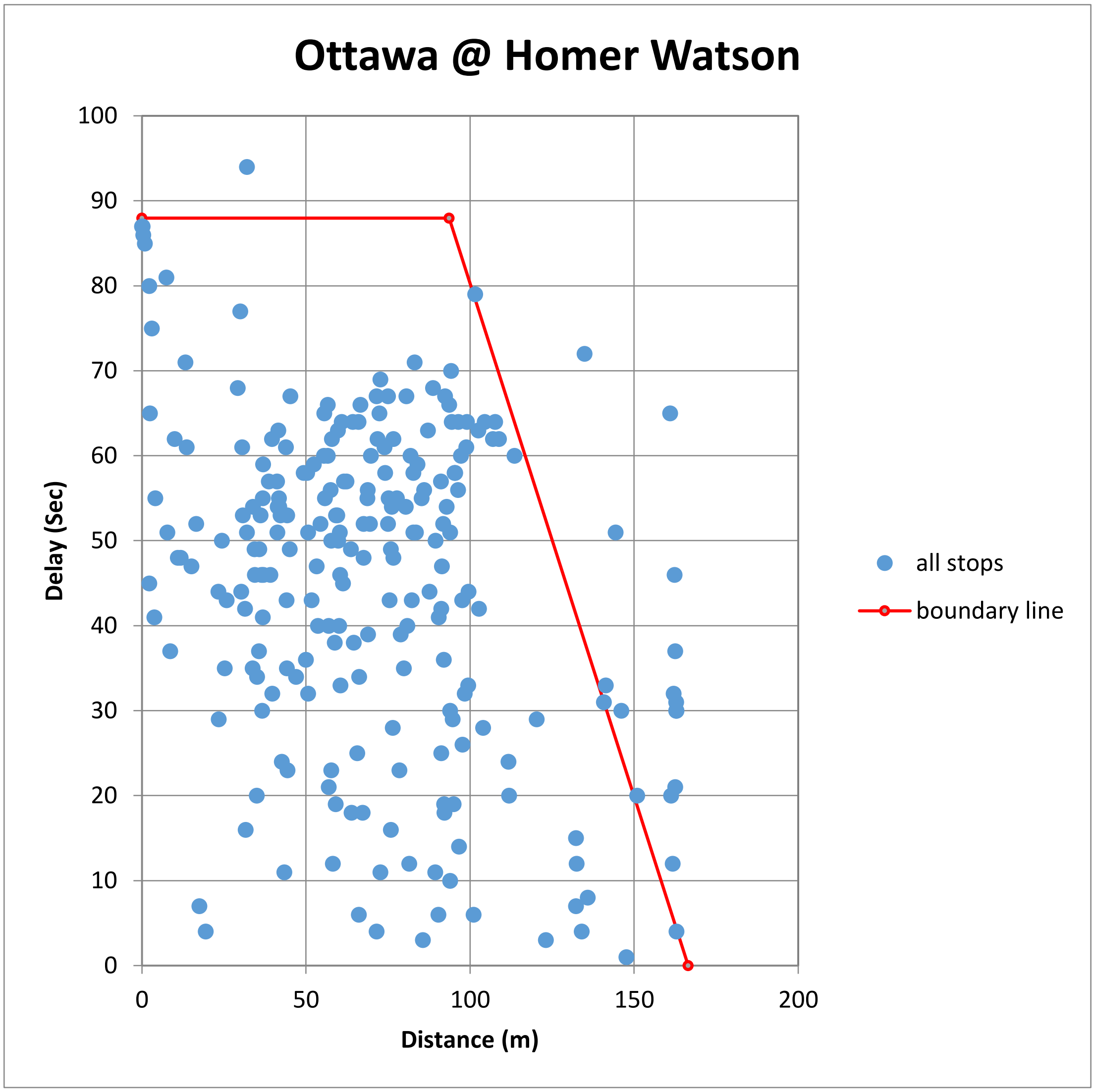
Right Turn


Hespeler @ Dunbar
Delay (Sec)
Distance (m)
0
5
10
15
20
25
30
35
40
45
50
0
20
40
60
80
100
120
all stops
boundary line

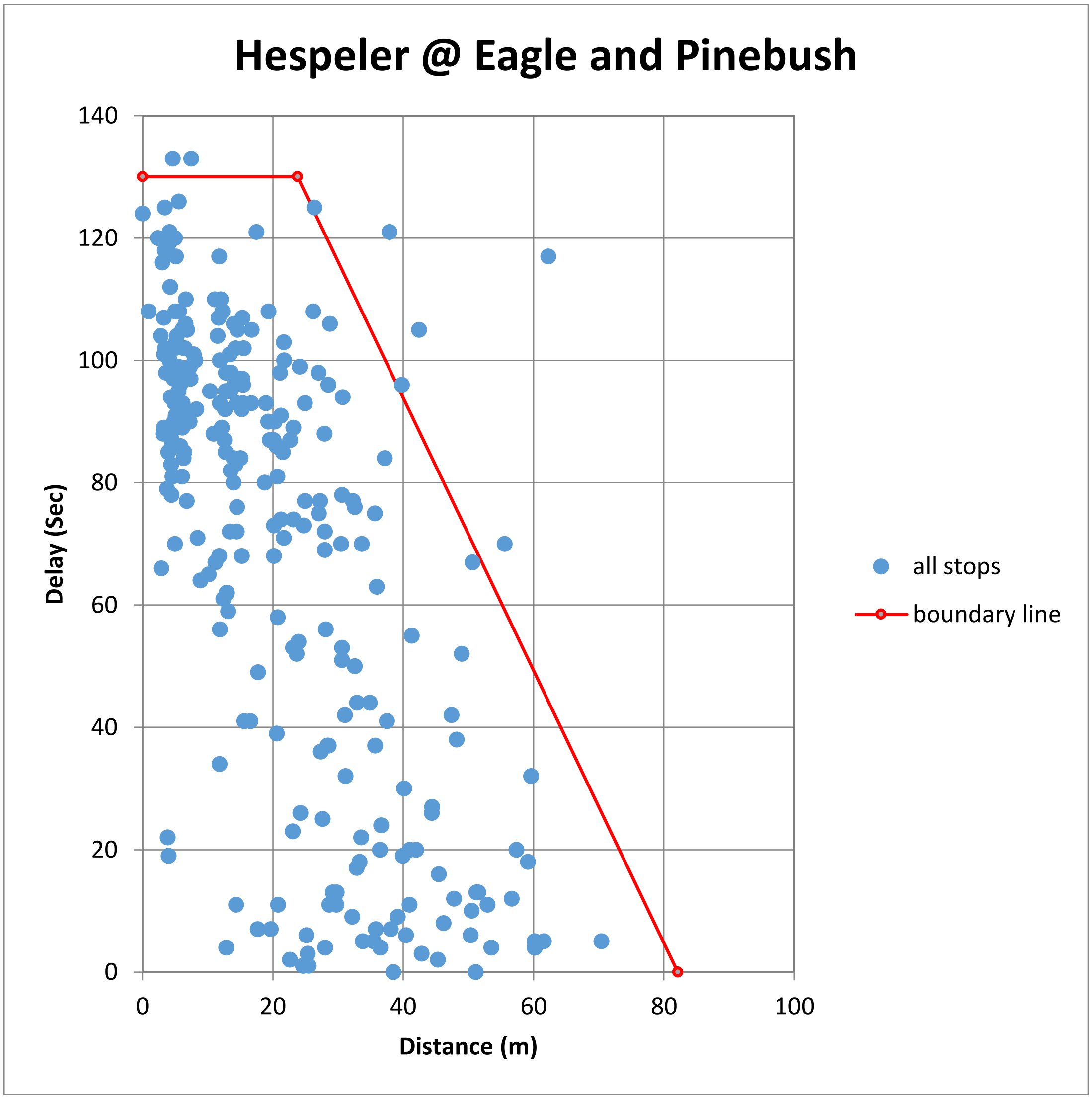
Right Turn



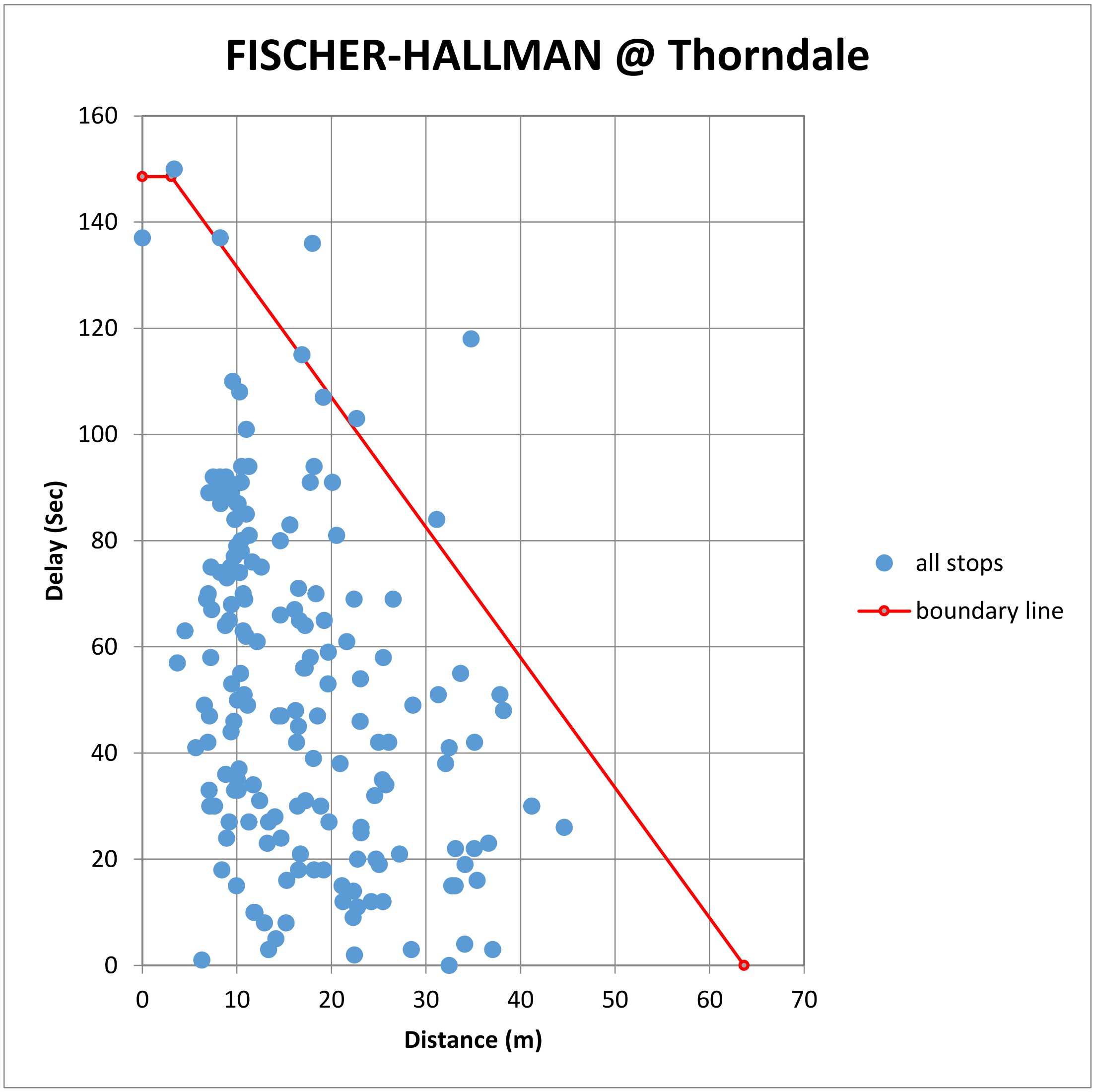
Through Movement



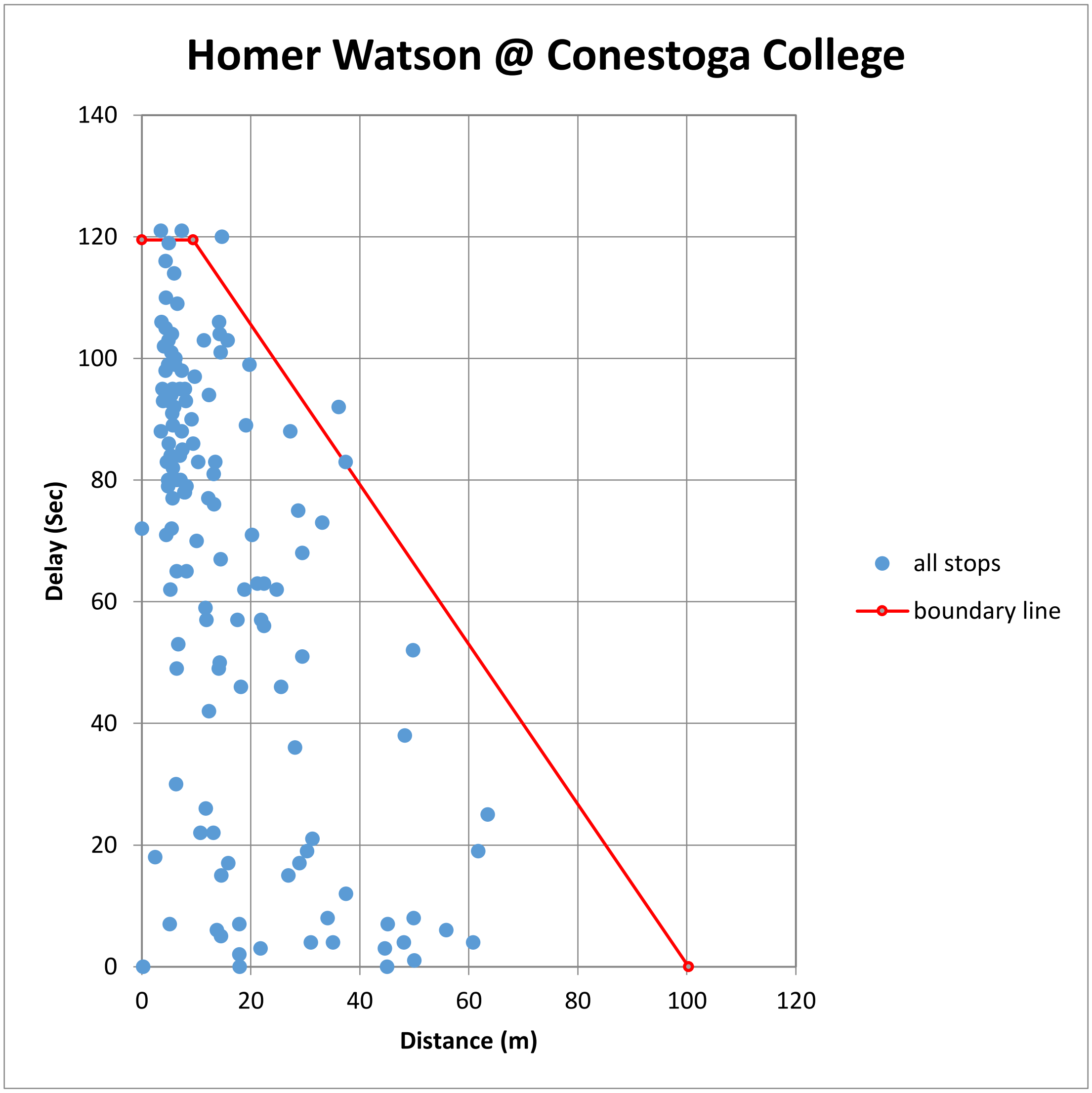
Through Movement


Hespeler @ Eagle and Pinebush
140
120
100
80
60
40
20
0
Delay (Sec)
0
20
40
60
80
100
Distance (m)
all stops
boundary line

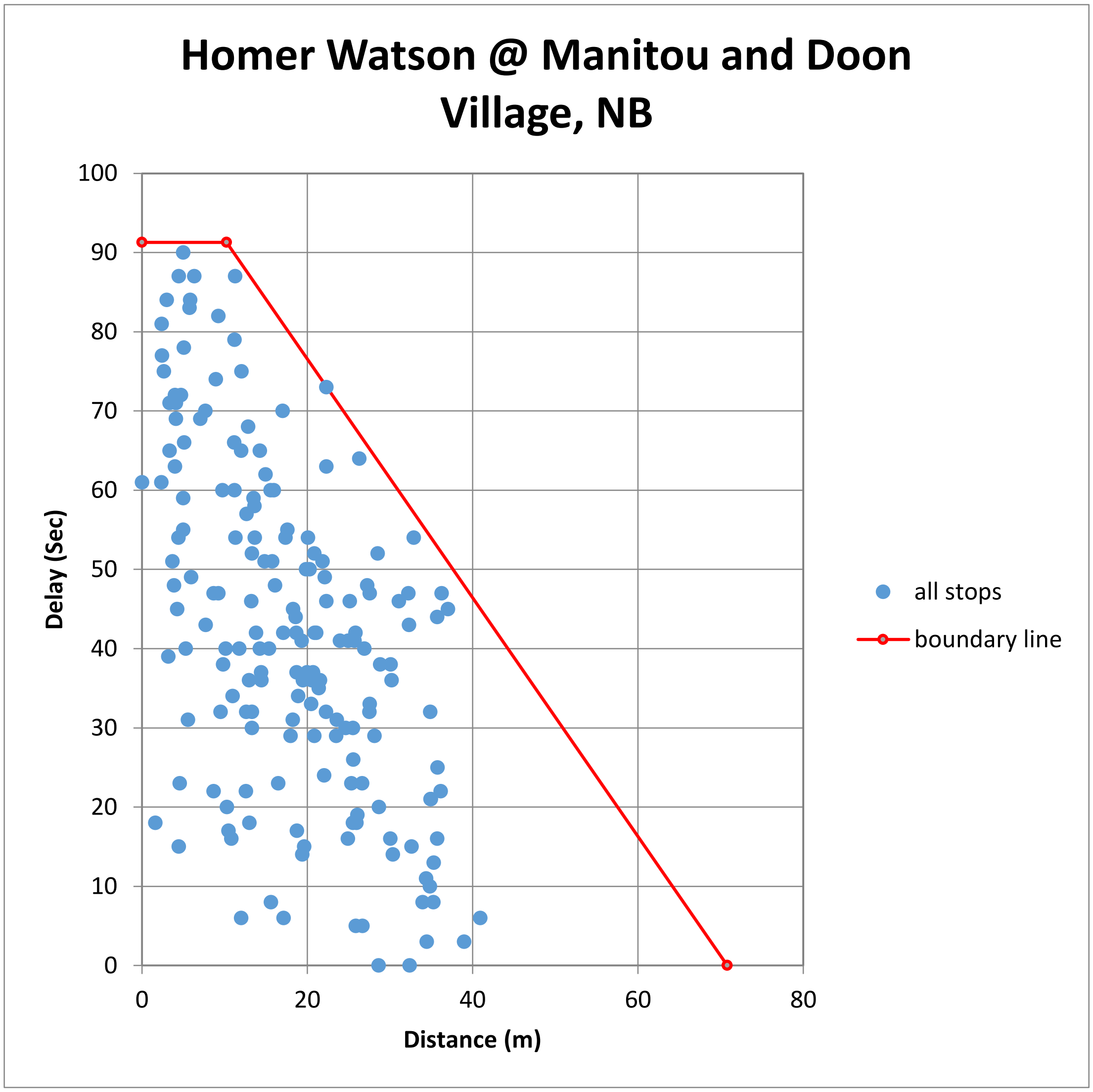
Left Turn


FISCHER-HALLMAN @ Thorndale
Delay (Sec)
Distance (m)
all stops
boundary line
160
140
120
100
80
60
40
20
0
0
10
20
30
40
50
60
70

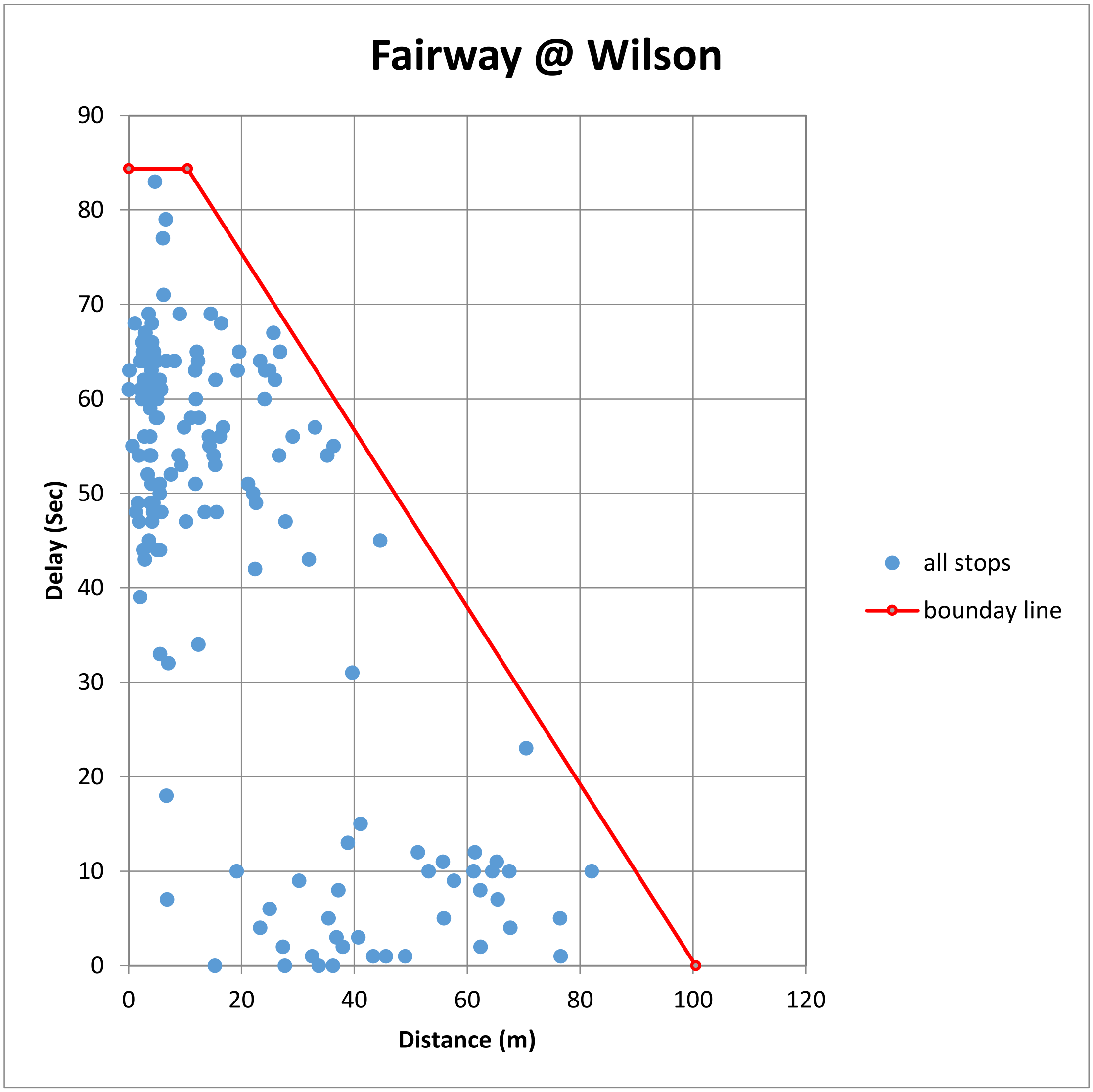
Right Turn


Homer Watson @ Conestoga College
140
120
100
80
60
40
20
0
Delay (Sec)
0
20
40
60
80
100
120
Distance (m)
all stops
boundary line

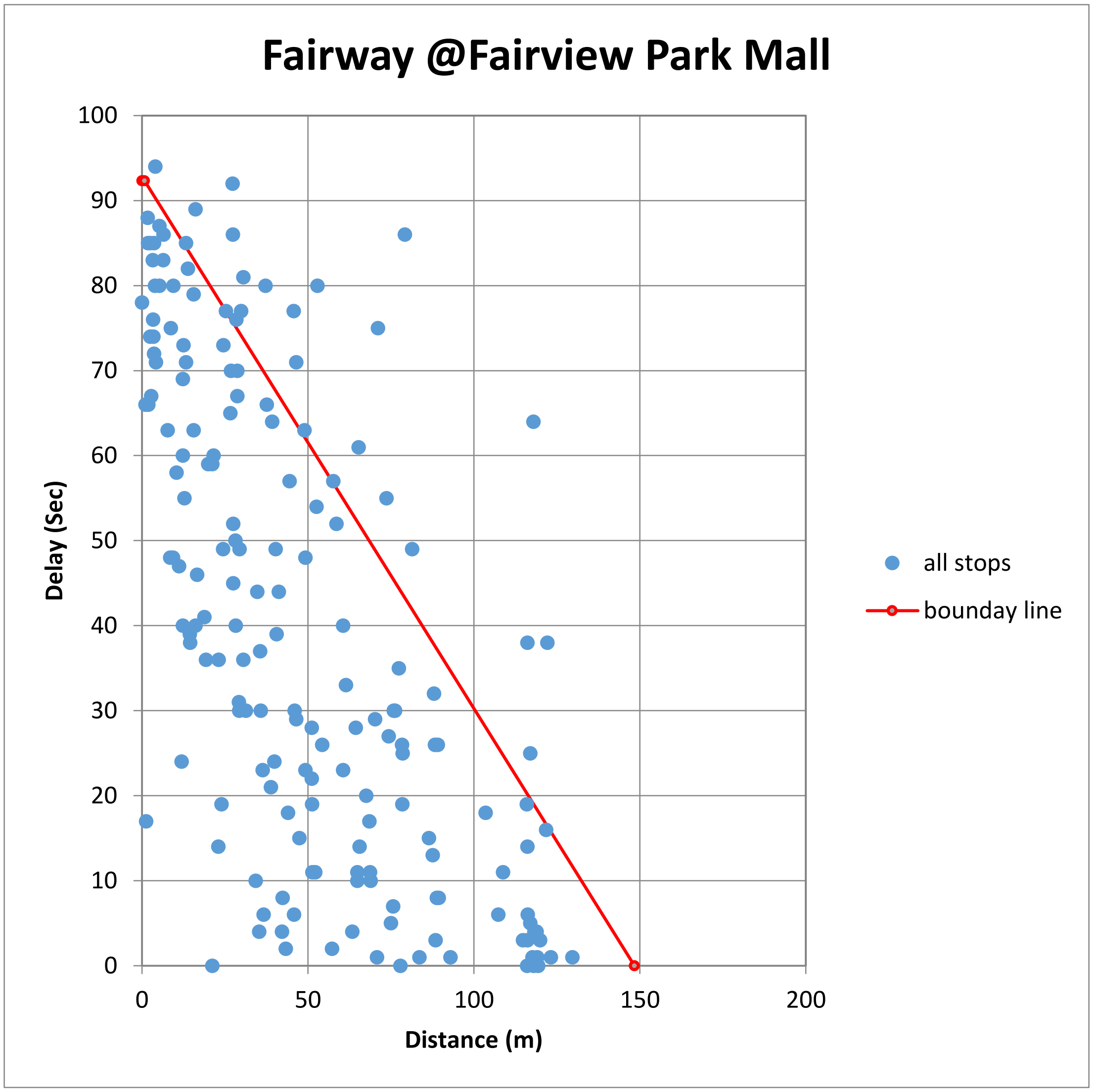
Left Turn



Through Movement

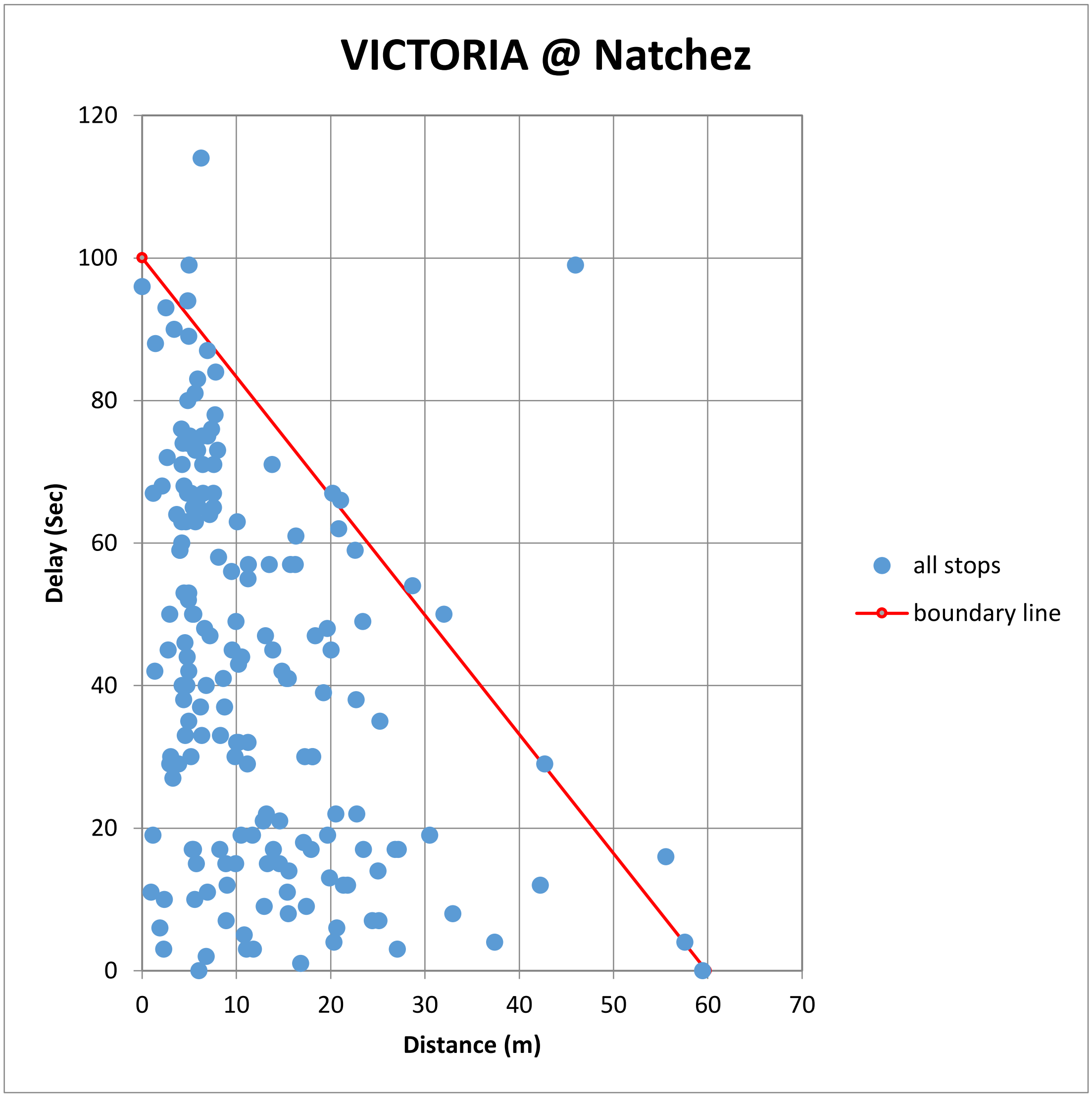


Through Movement

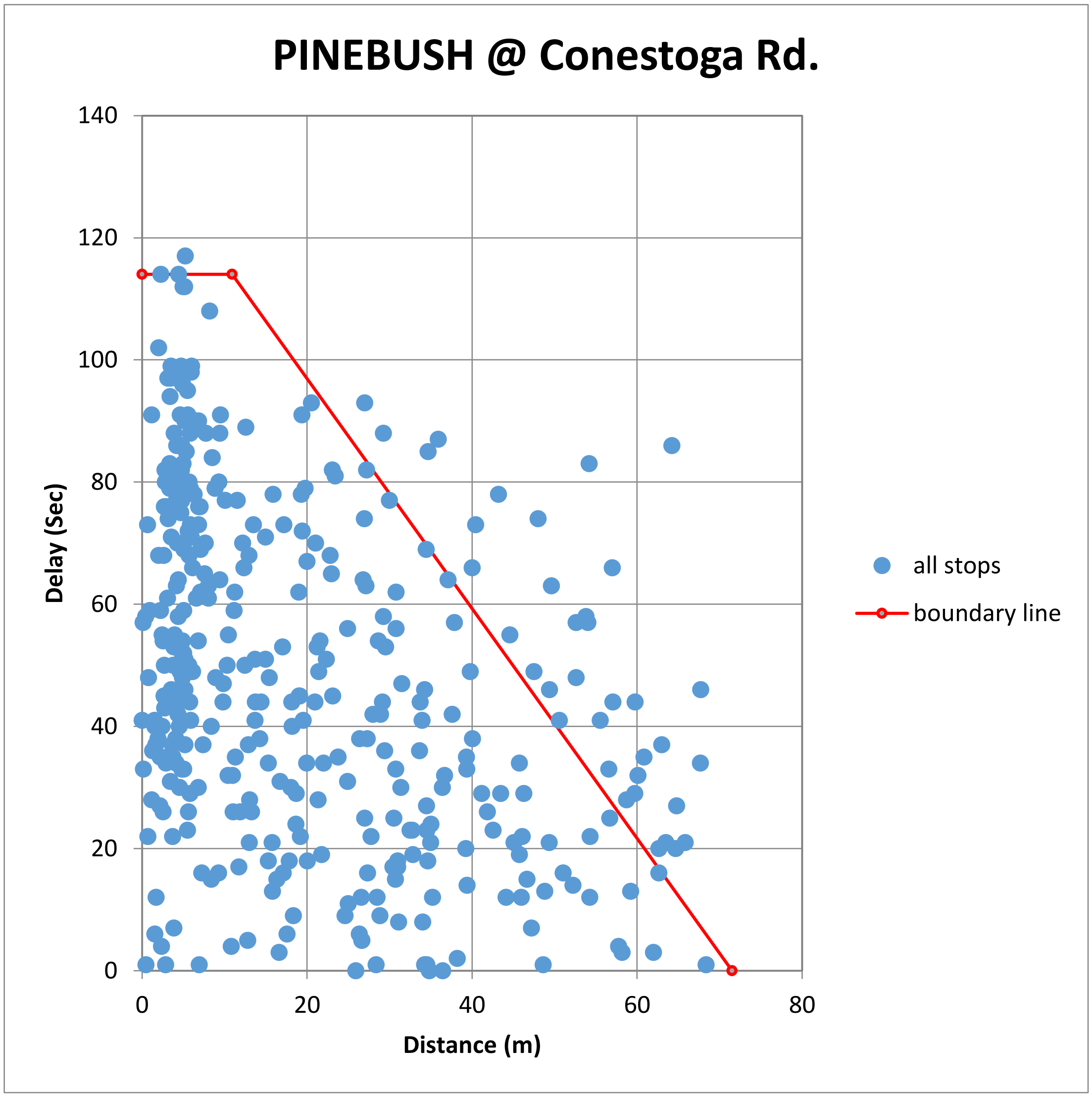


Fairway @Fairview Park Mall
Delay (Sec)
Distance (m)
0
10
20
30
40
50
60
70
80
90
100
0
50
100
150
200
all stops
bounday line

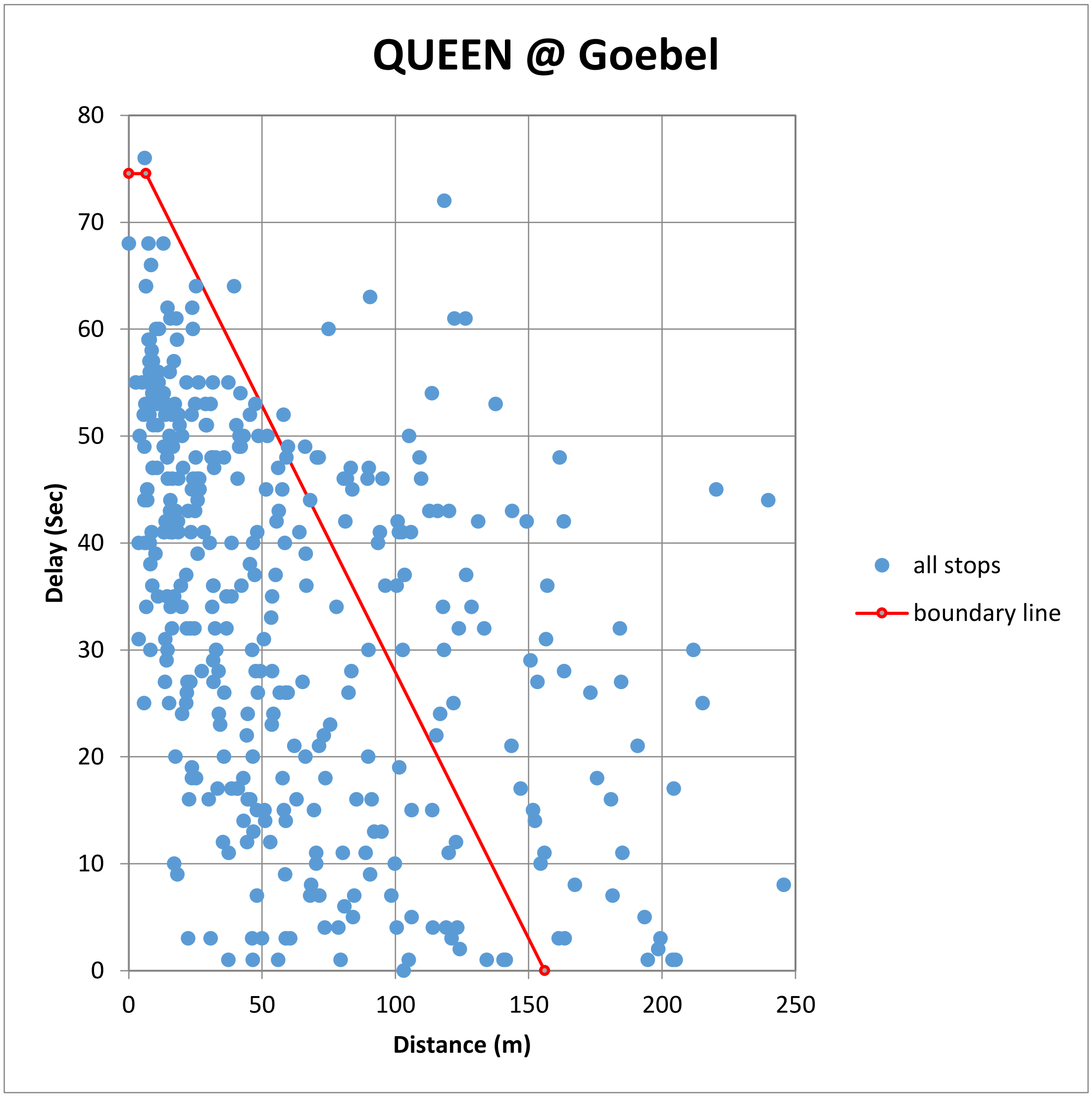
Left Turn



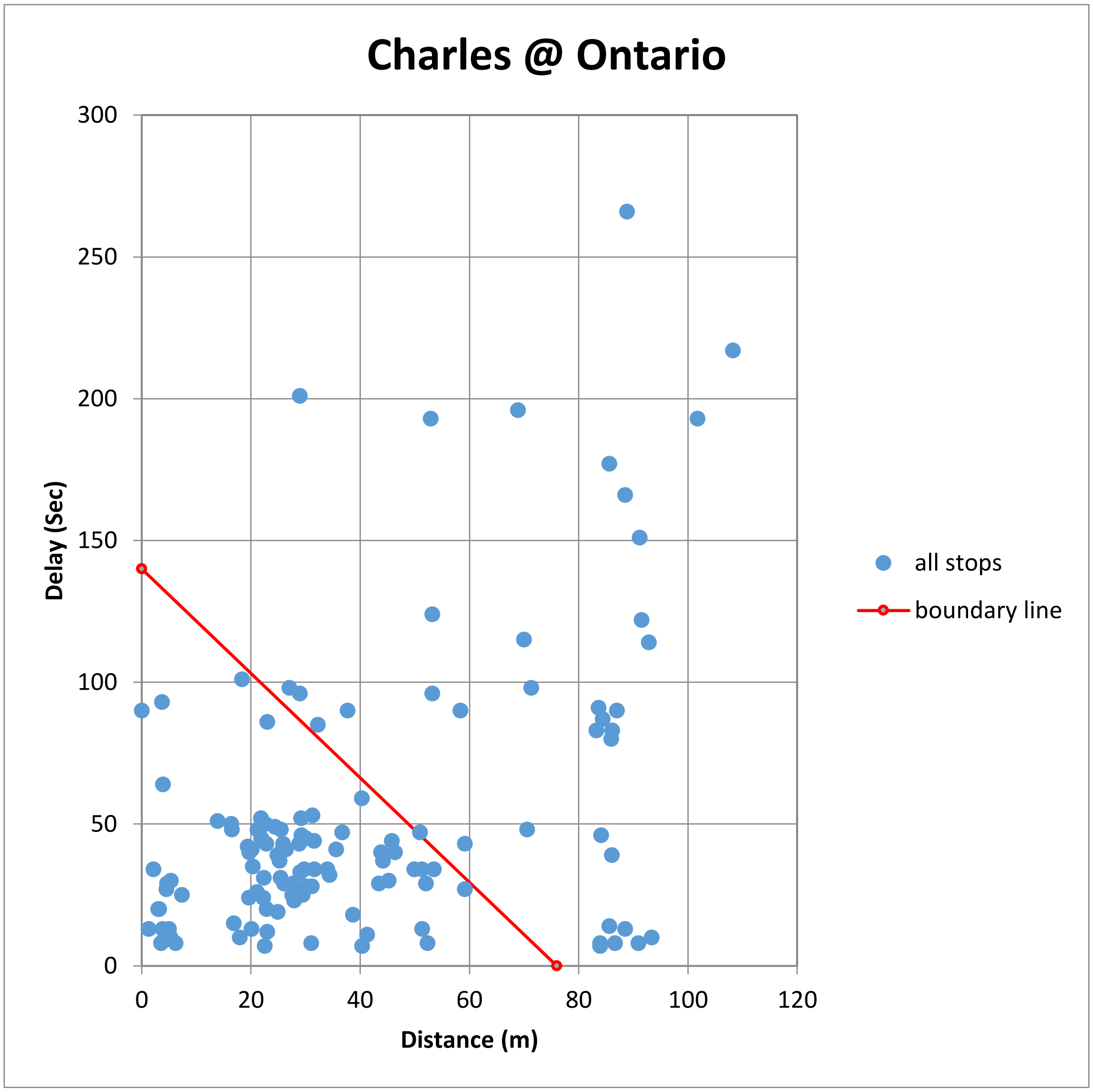
Through Movement



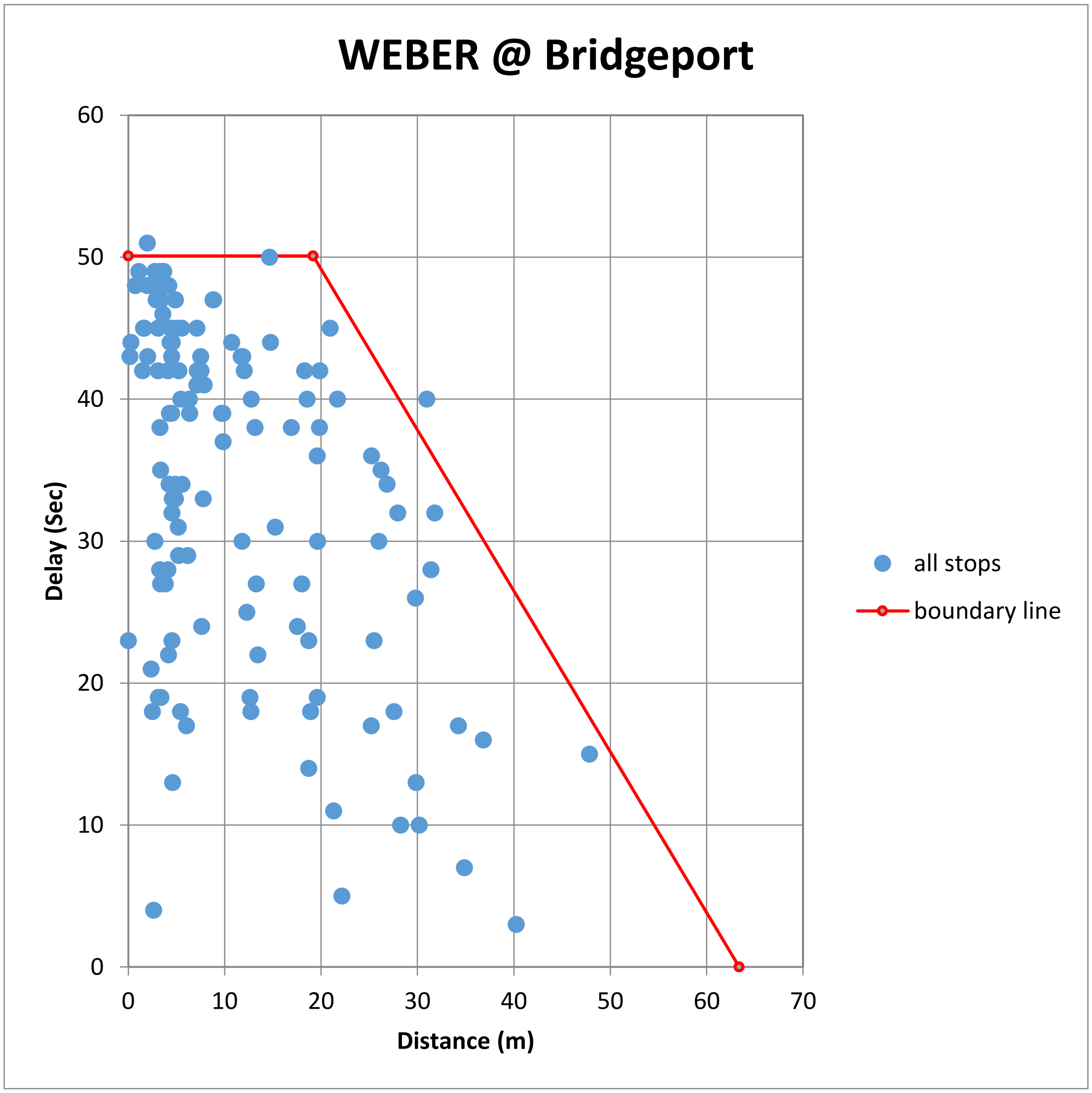
Through Movement


QUEEN @ Goebel
Delay (Sec)
80
70
60
50
40
30
20
10
0
0
50
100
150
200
250
Distance (m)
all stops
boundary line

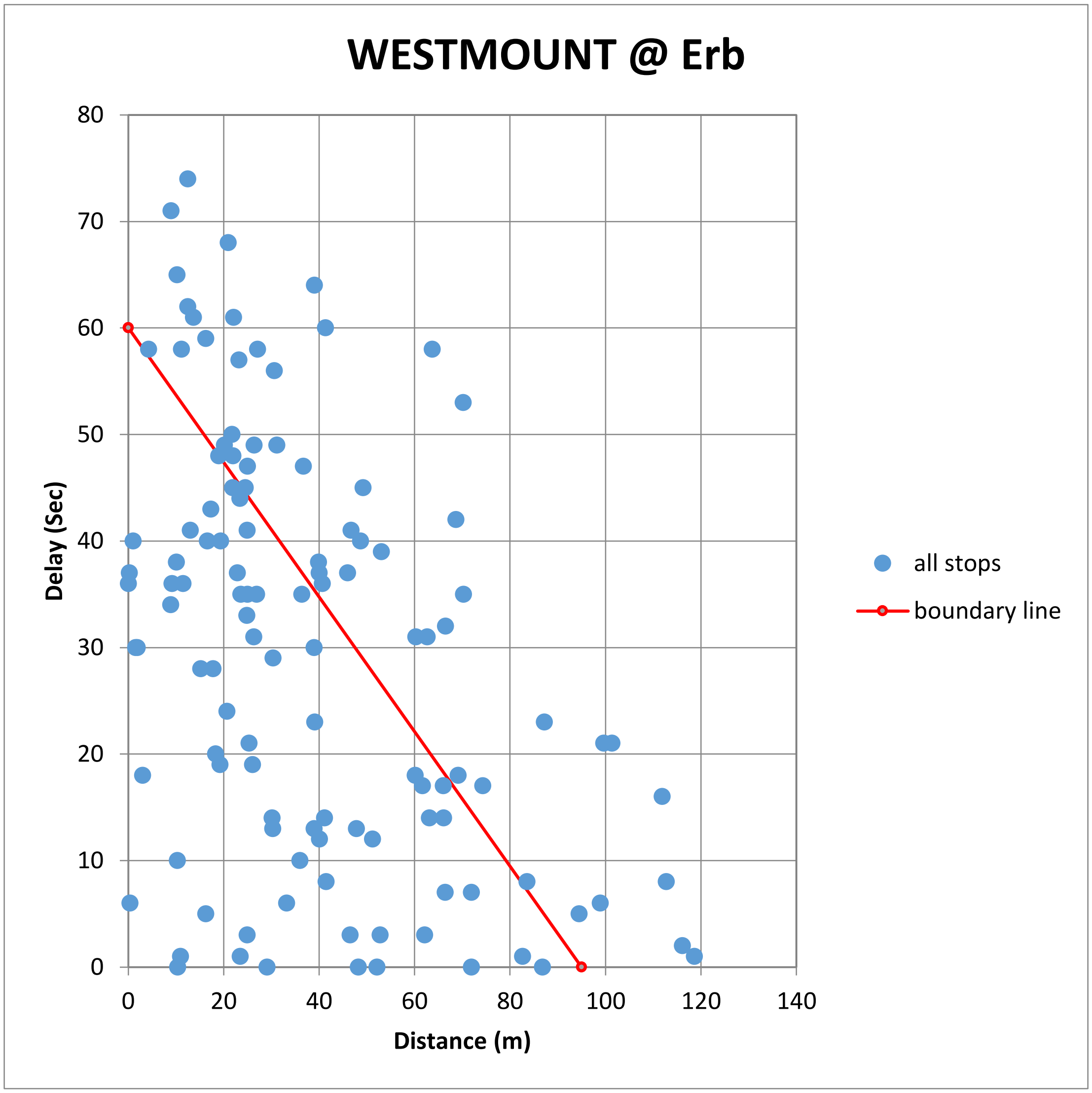
Left Turn



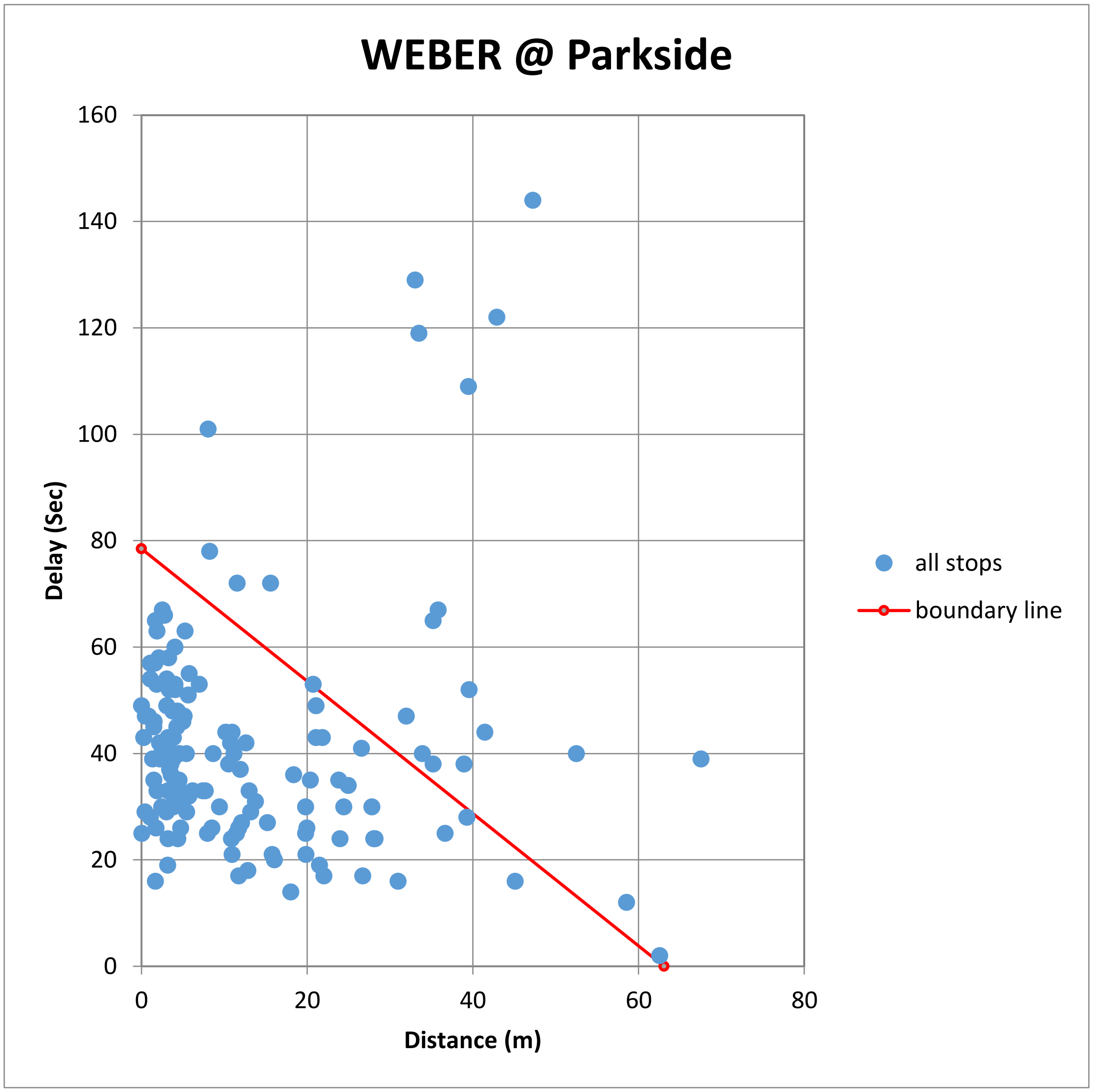
Through Movement



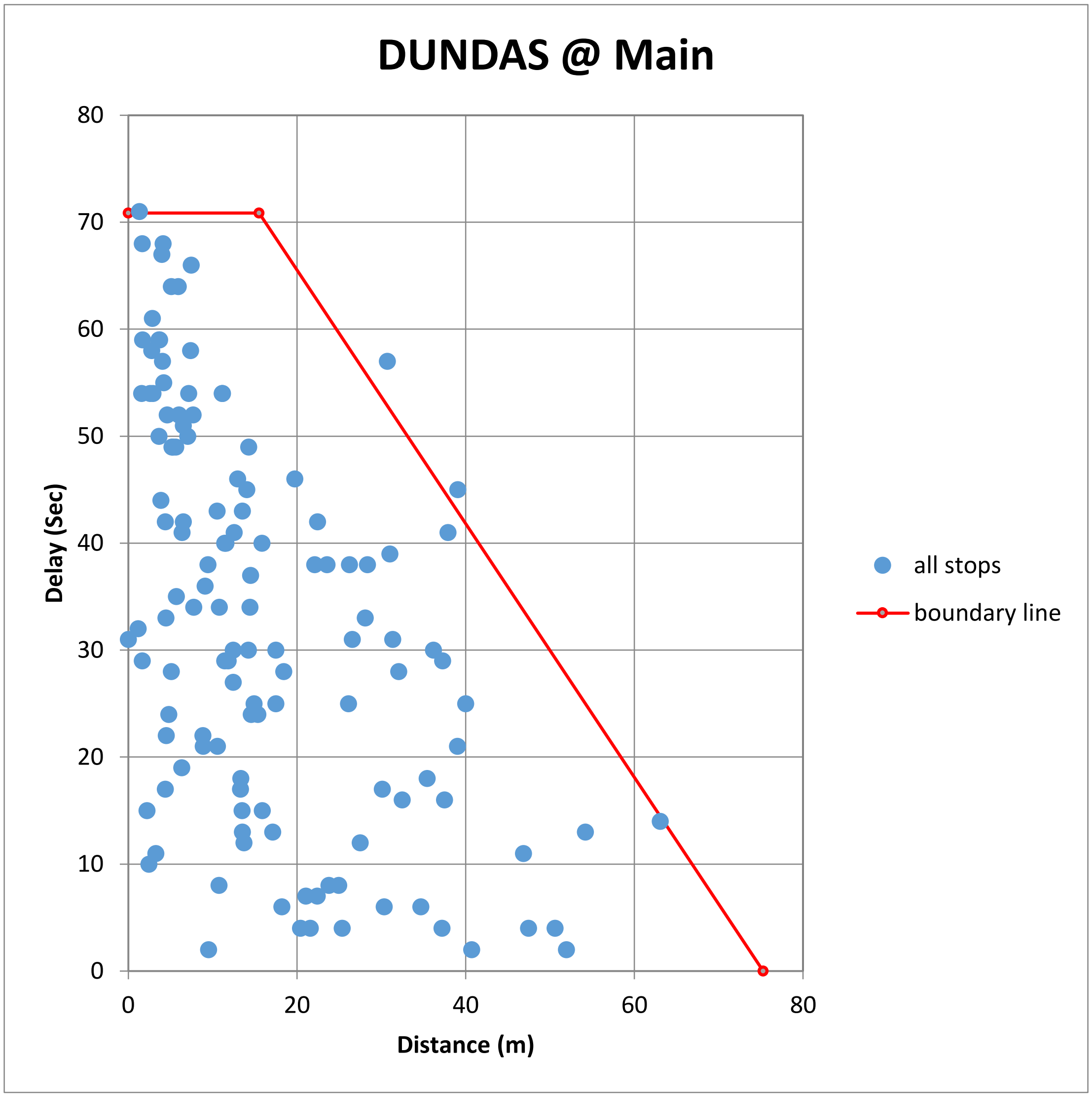
Through Movement



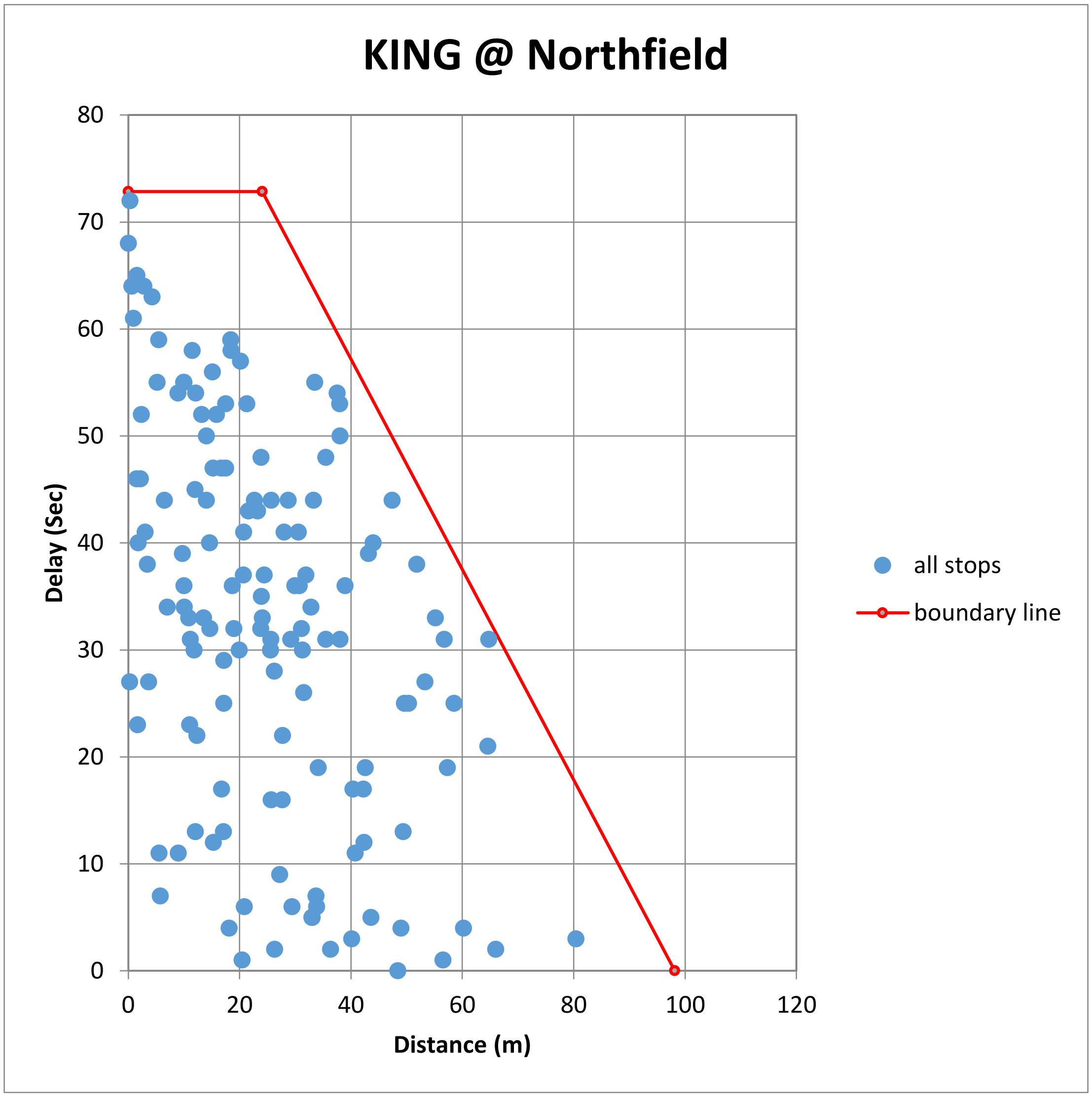
Through Movement


WEBER @ Parkside
160
140
120
100
80
60
40
20
0
Delay (Sec)
0
20
40
60
80
Distance (m)
all stops
boundary line

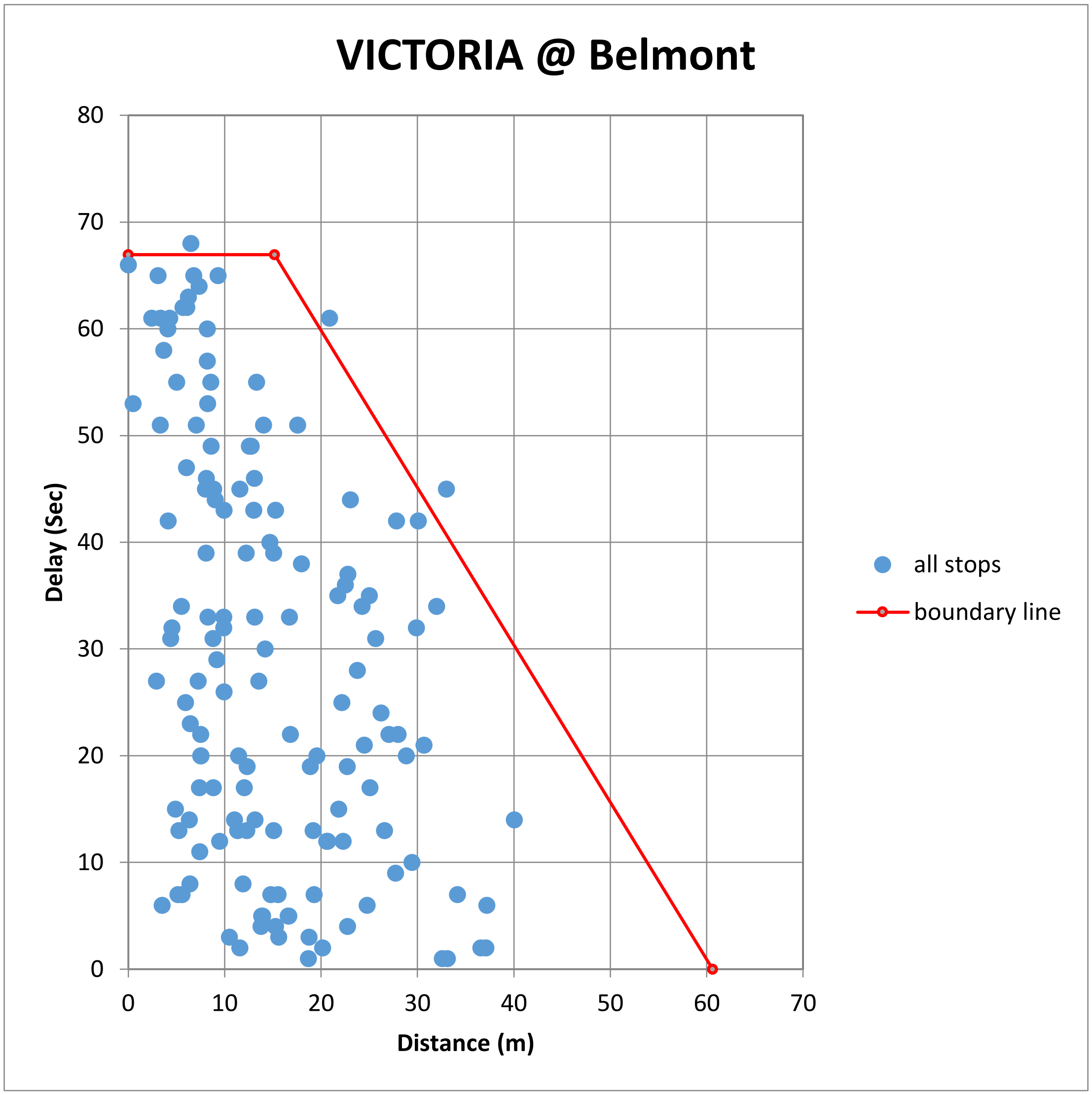
Left Turn



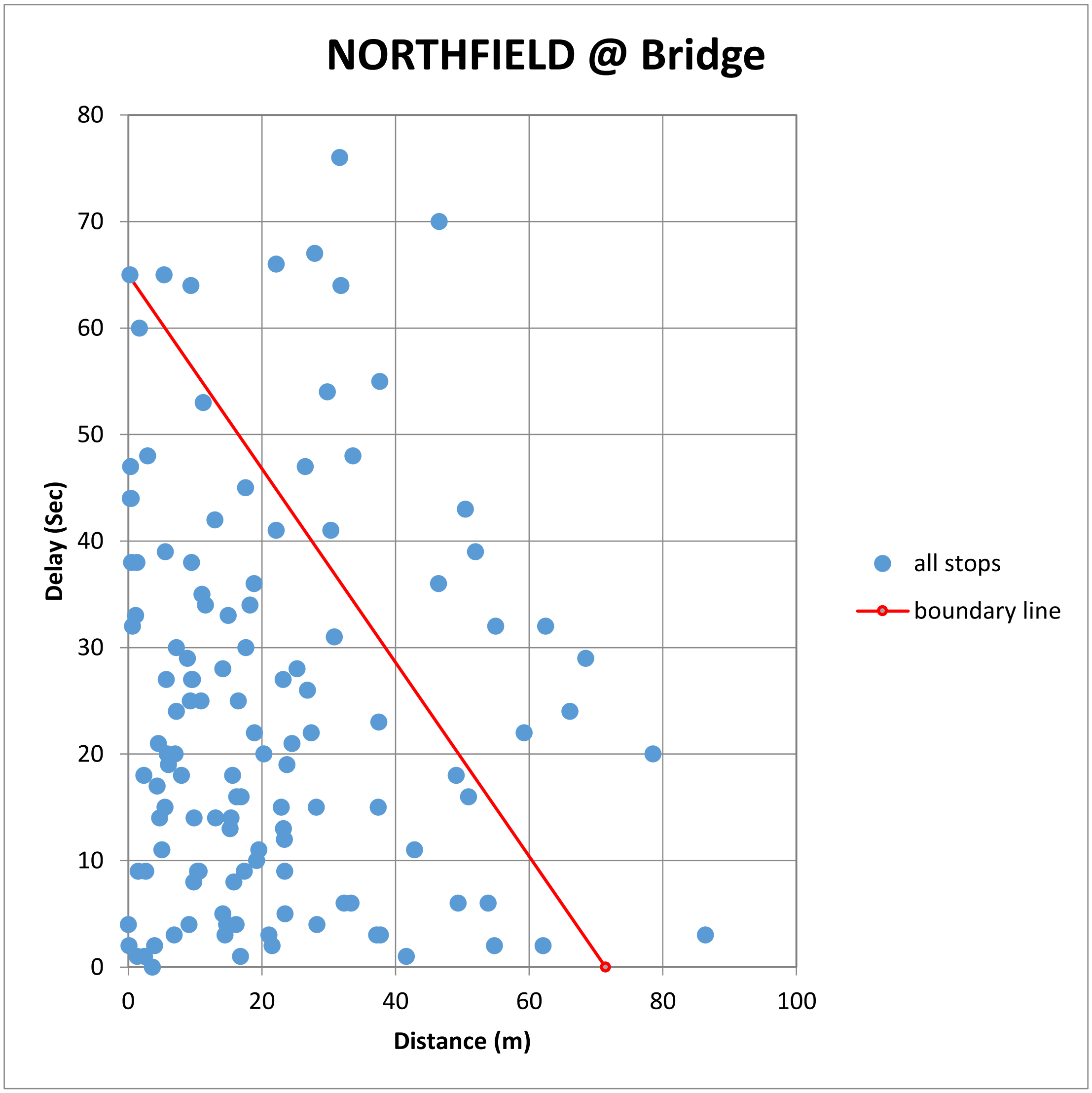
Through Movement



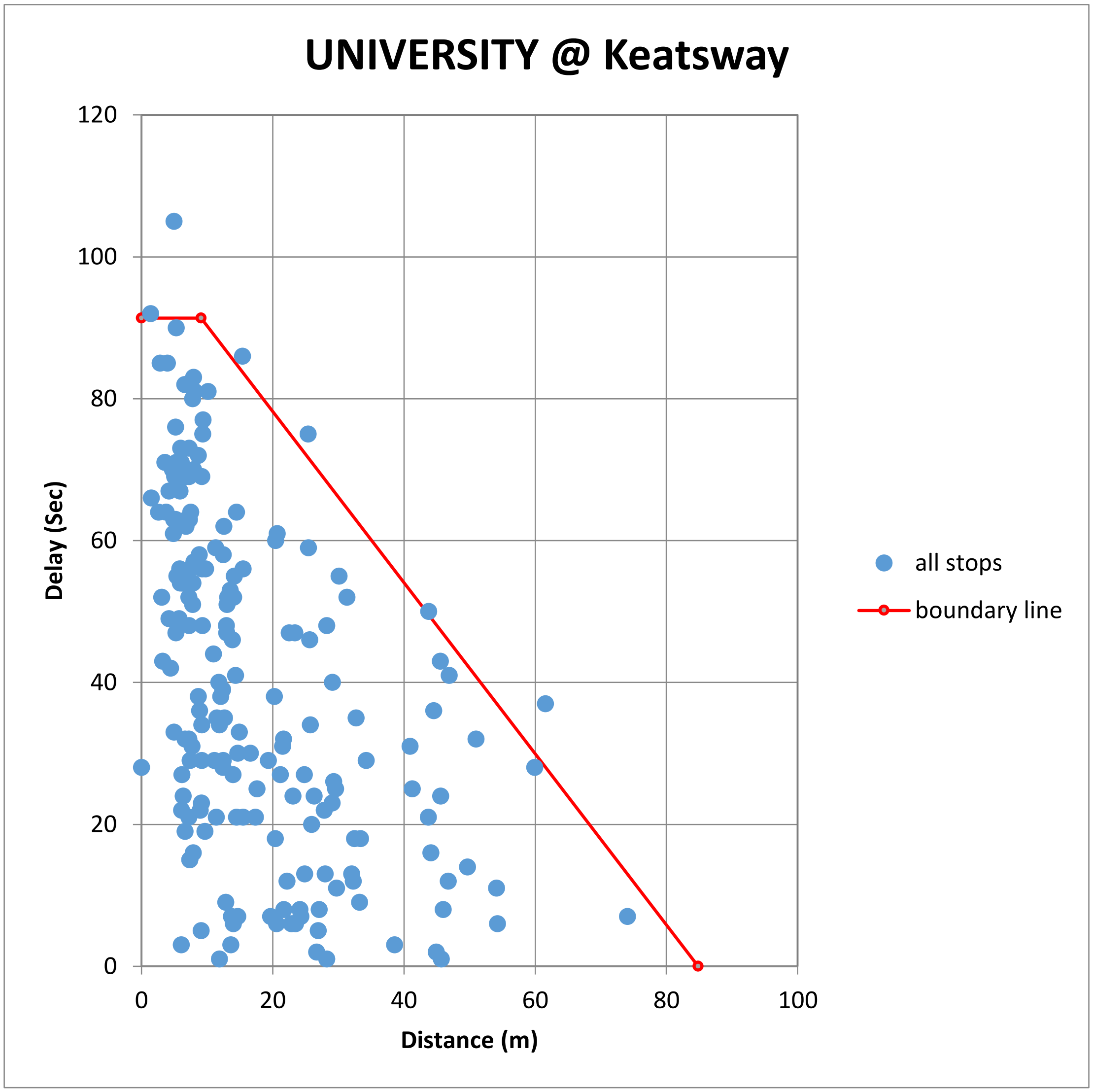
Through Movement



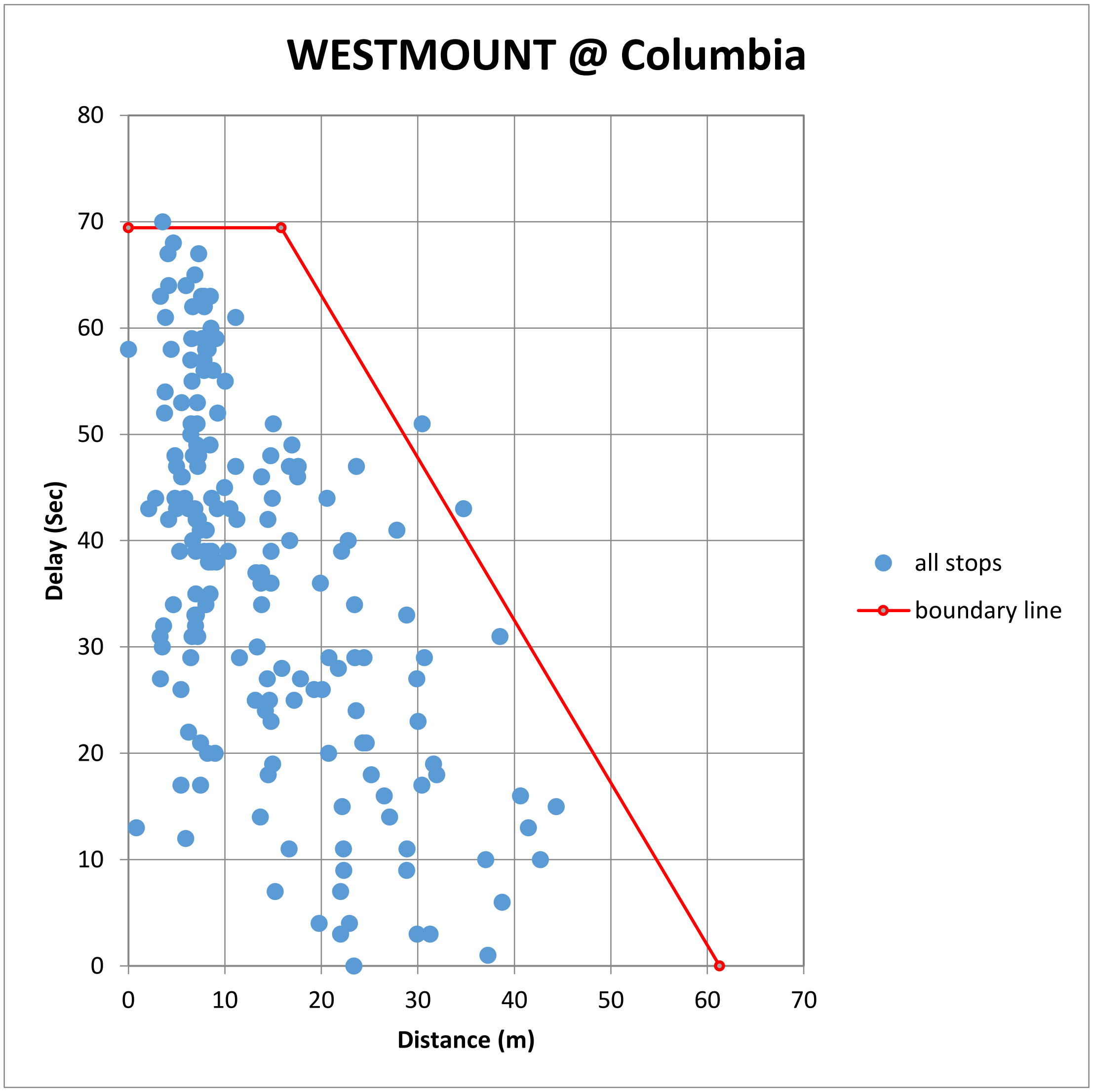
Through Movement


NORTHFIELD @ Bridge
80
70
60
50
40
30
20
10
0
Delay (Sec)
0
20
40
60
80
100
Distance (m)
all stops
boundary line

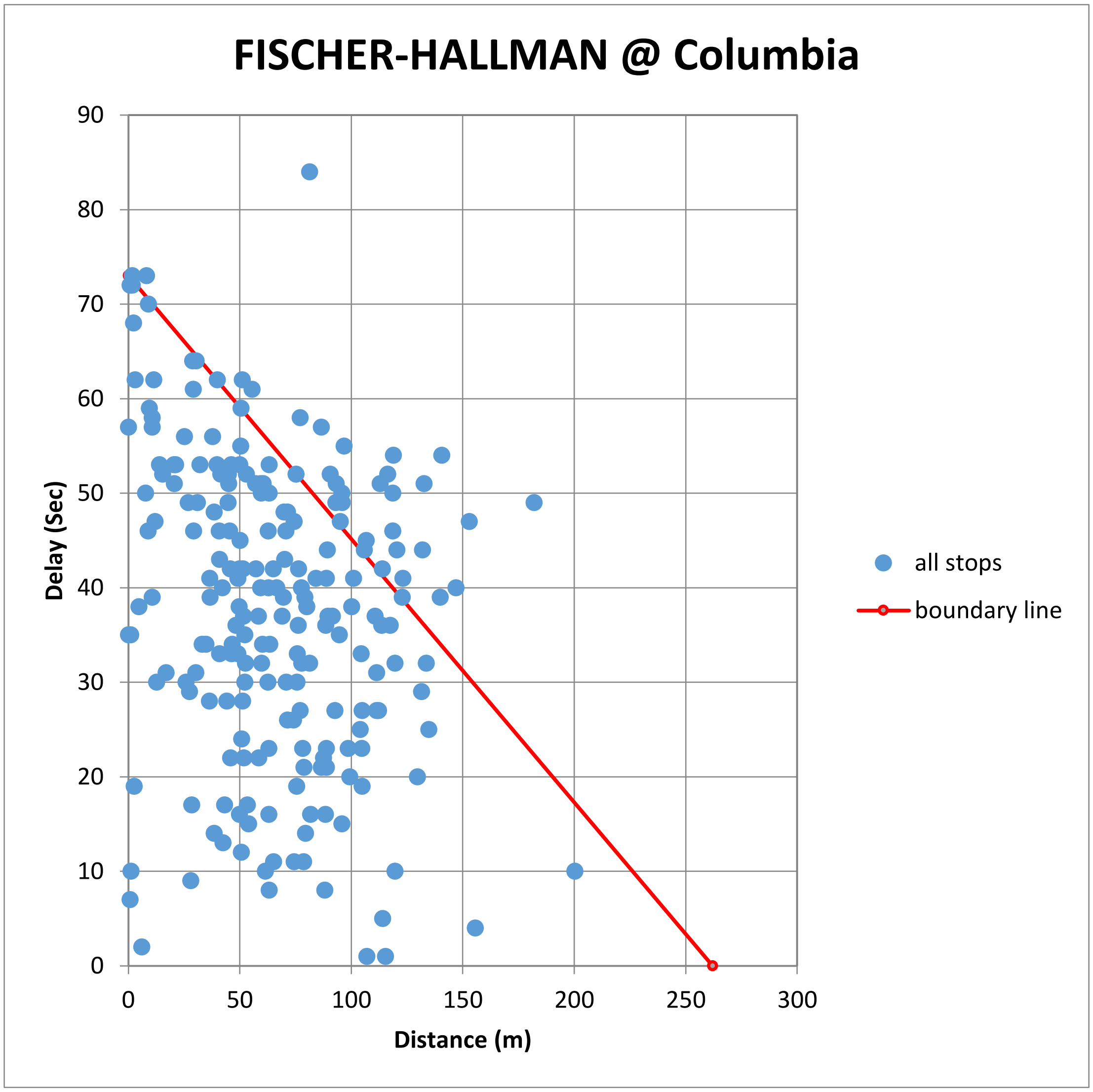
Left Turn


UNIVERSITY @ Keatsway
120
100
80
60
40
20
0
Delay (Sec)
0
20
40
60
80
100
Distance (m)
all stops
boundary line

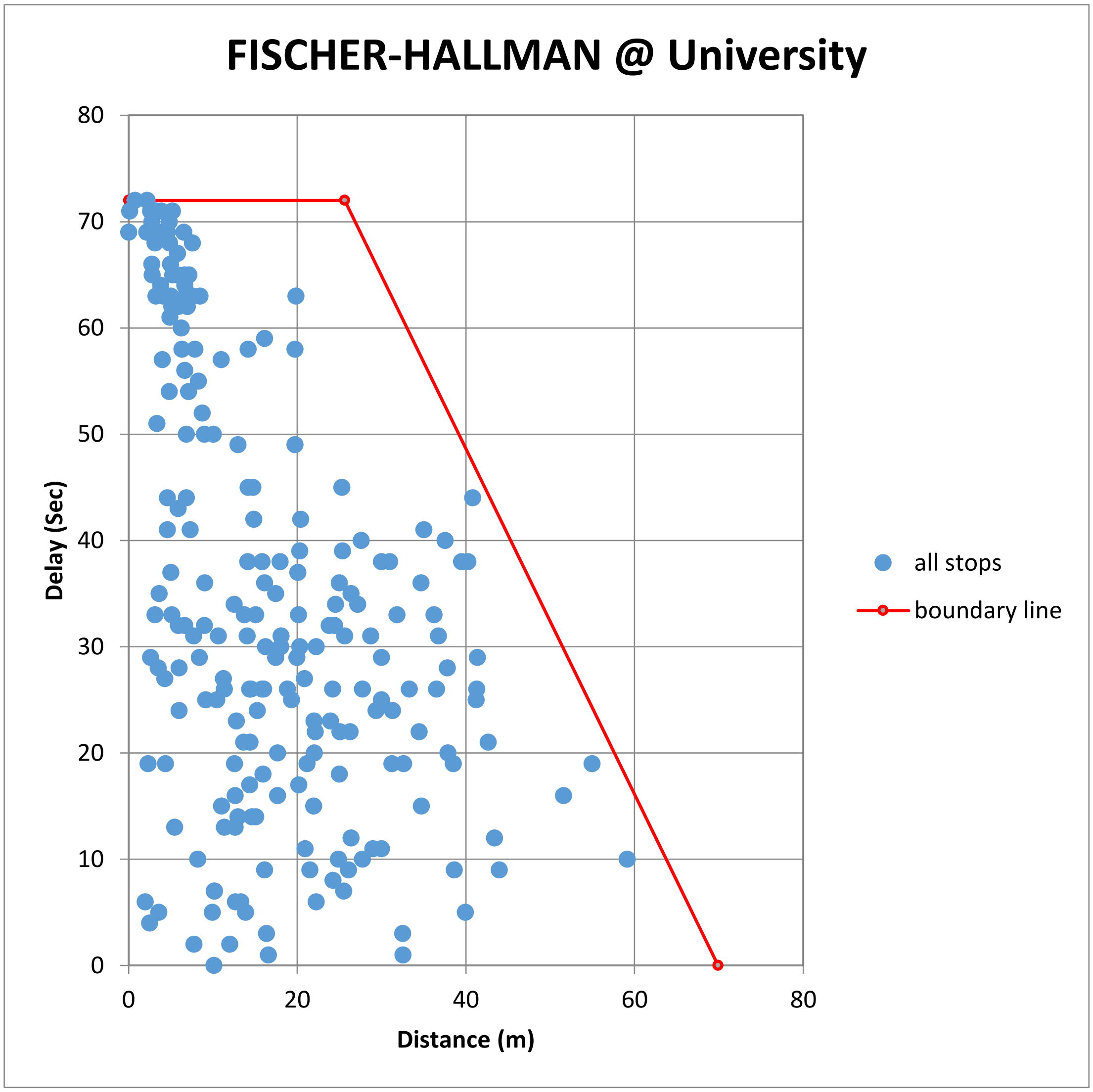
Left Turn



Through Movement



Through Movement

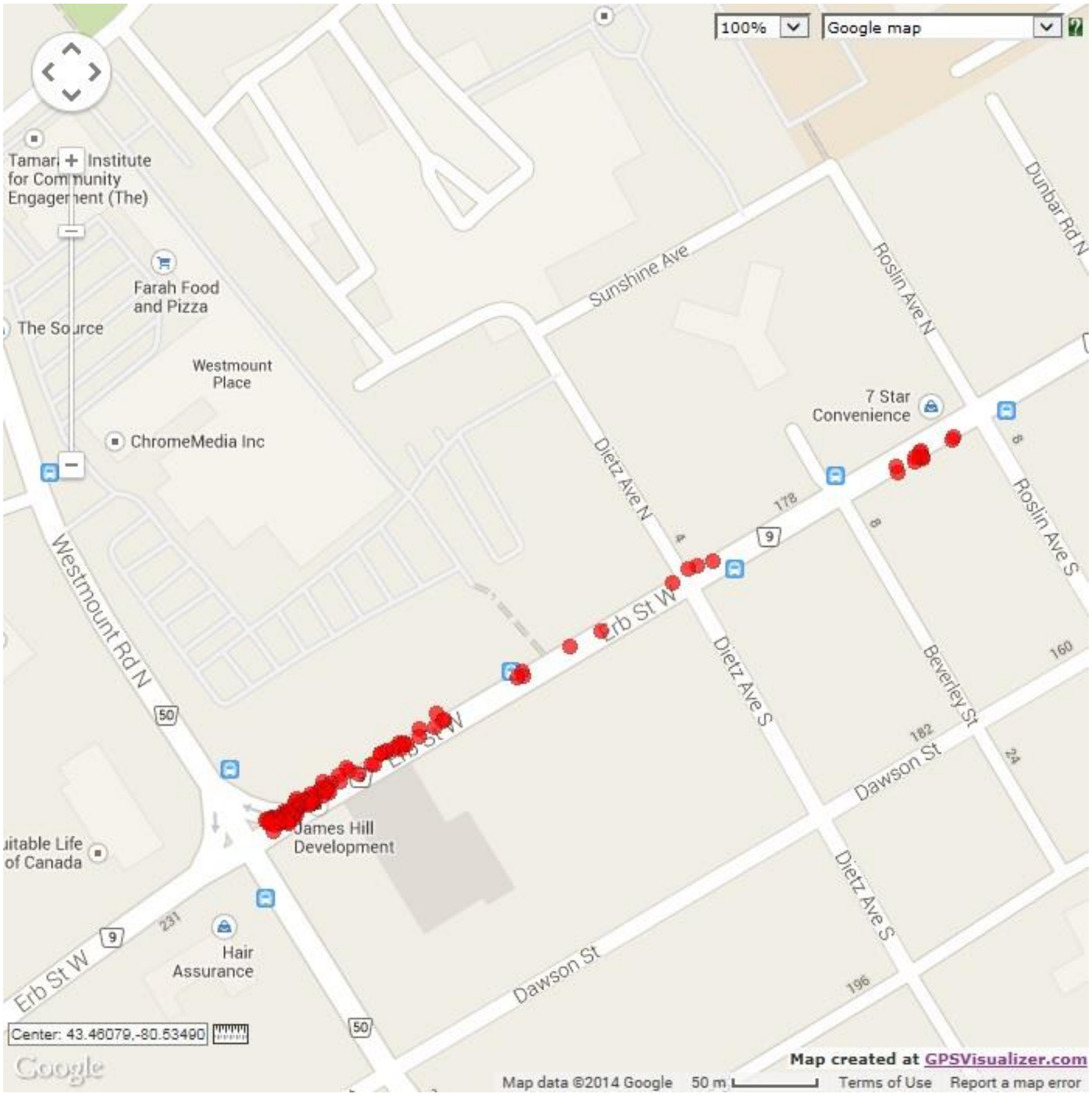


Through Movement

Appendix D

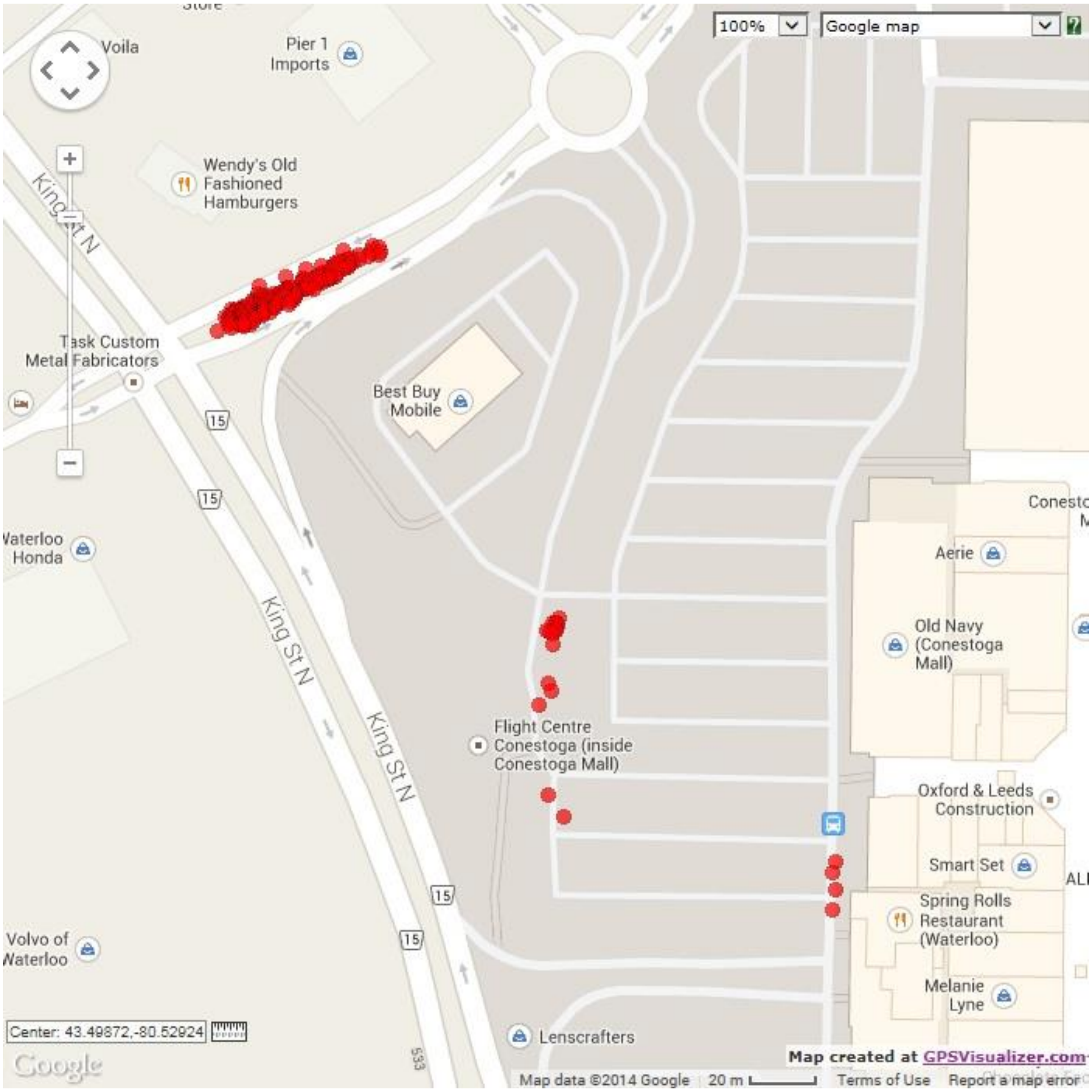
Visual Representation of the Recorded Stop Events

Appendix D includes visual representation of the recorded stop events on the map of each defined segment. Google Maps are used to visualize the intersections. The figures presented in this appendix are produced using the GPS Visualizer website. www.gpsvisualizer.com

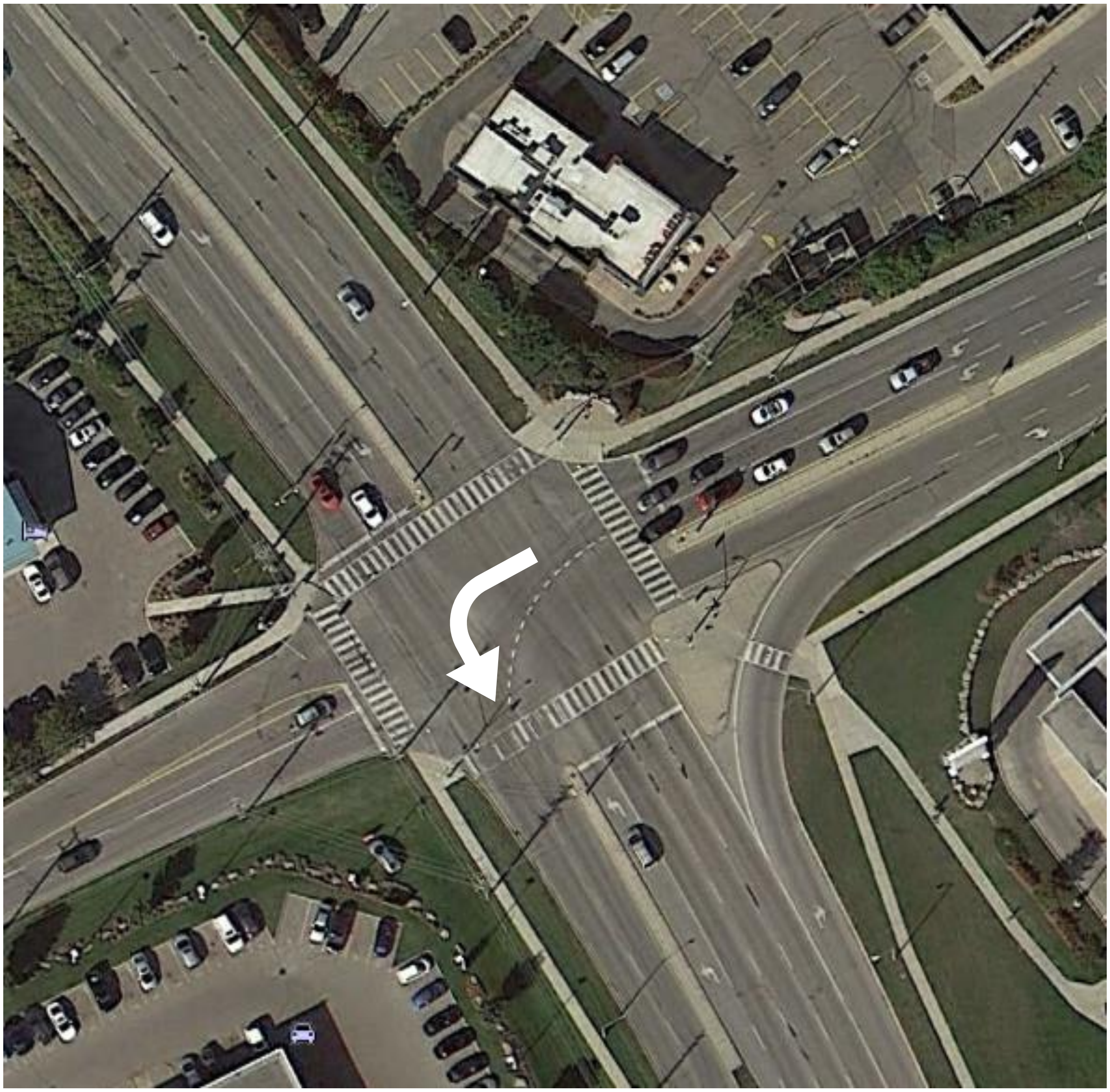


ERB @ Roslin to WESTMOUNT @ Erb

WESTMOUNT @ Erb



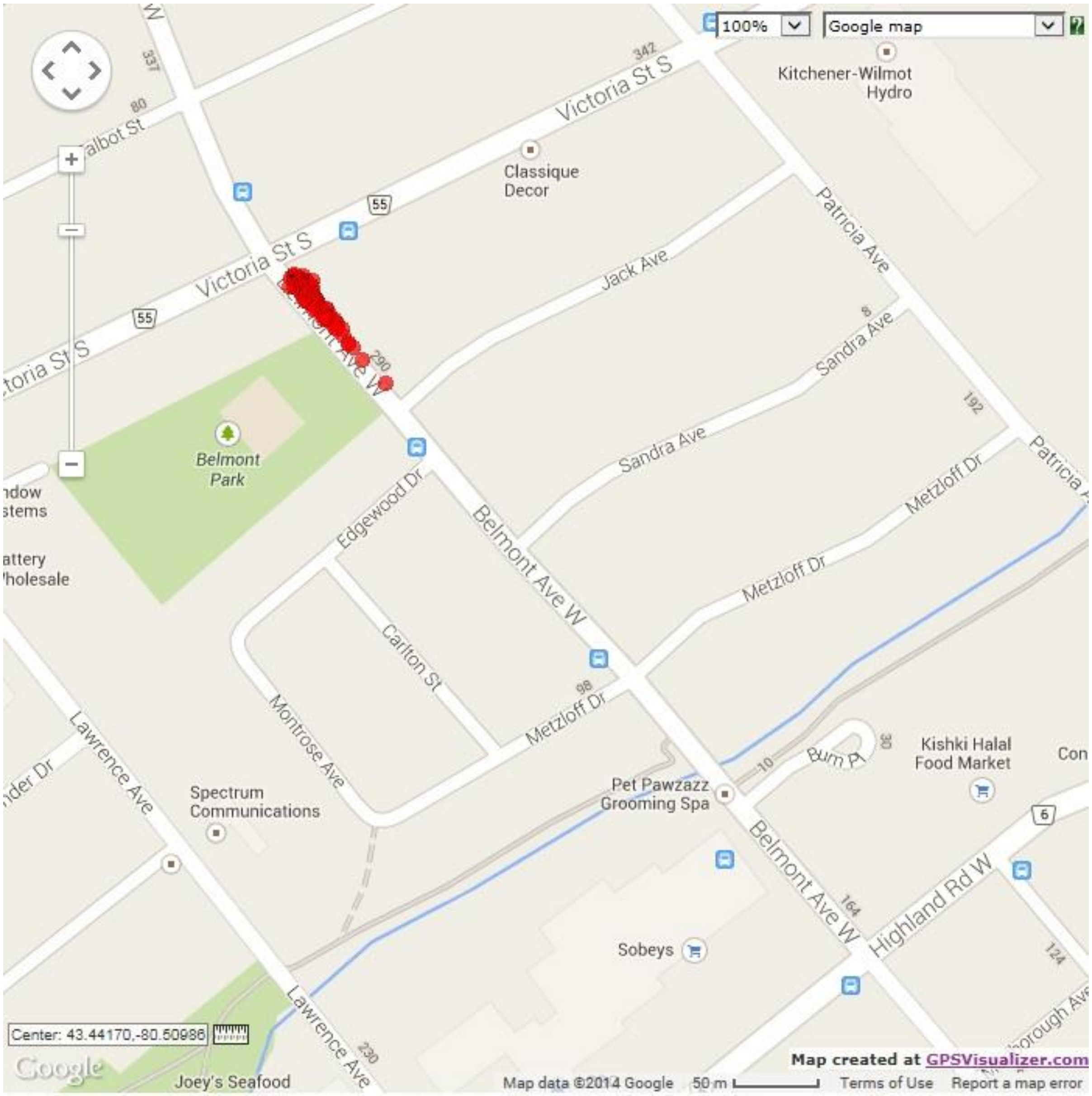
Terminal to KING @ Conestoga Mall



KING @ Conestoga Mall

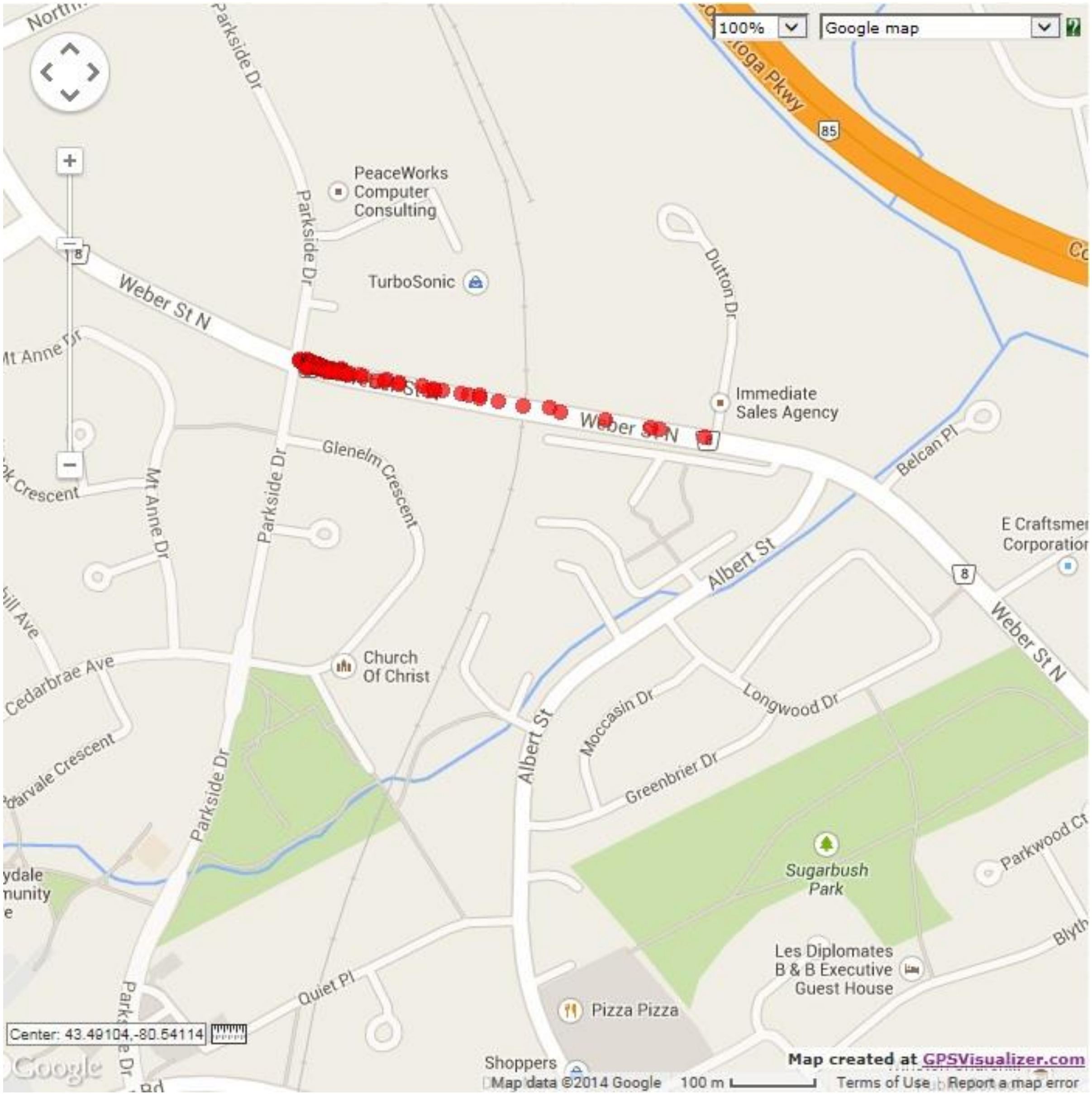
Terminal to FAIRWAY @ Fairview Park Mall

FAIRWAY @ Fairview Park Mall



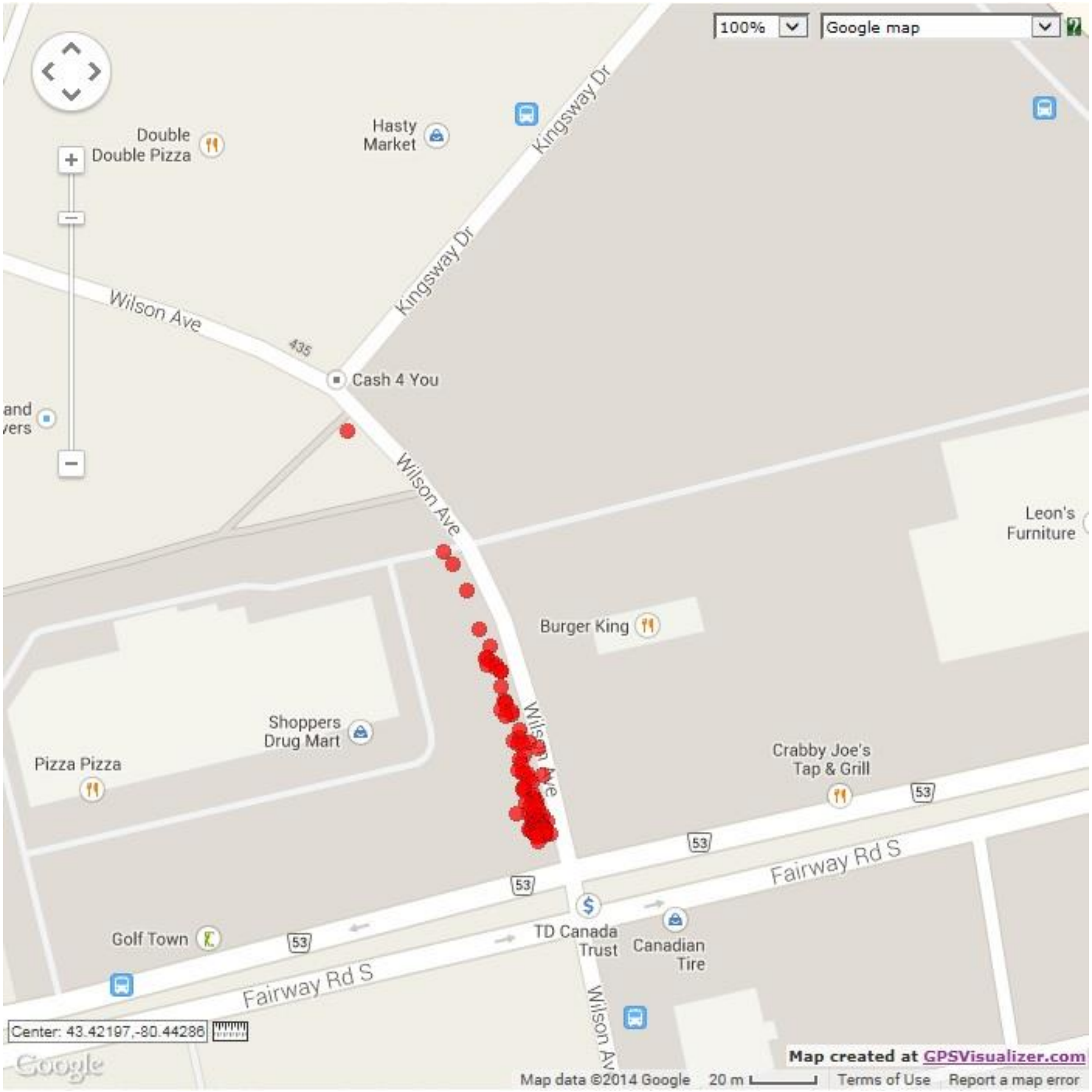
HIGHLAND @ Belmon to VICTORIA @ Belmont

VICTORIA @ Belmont



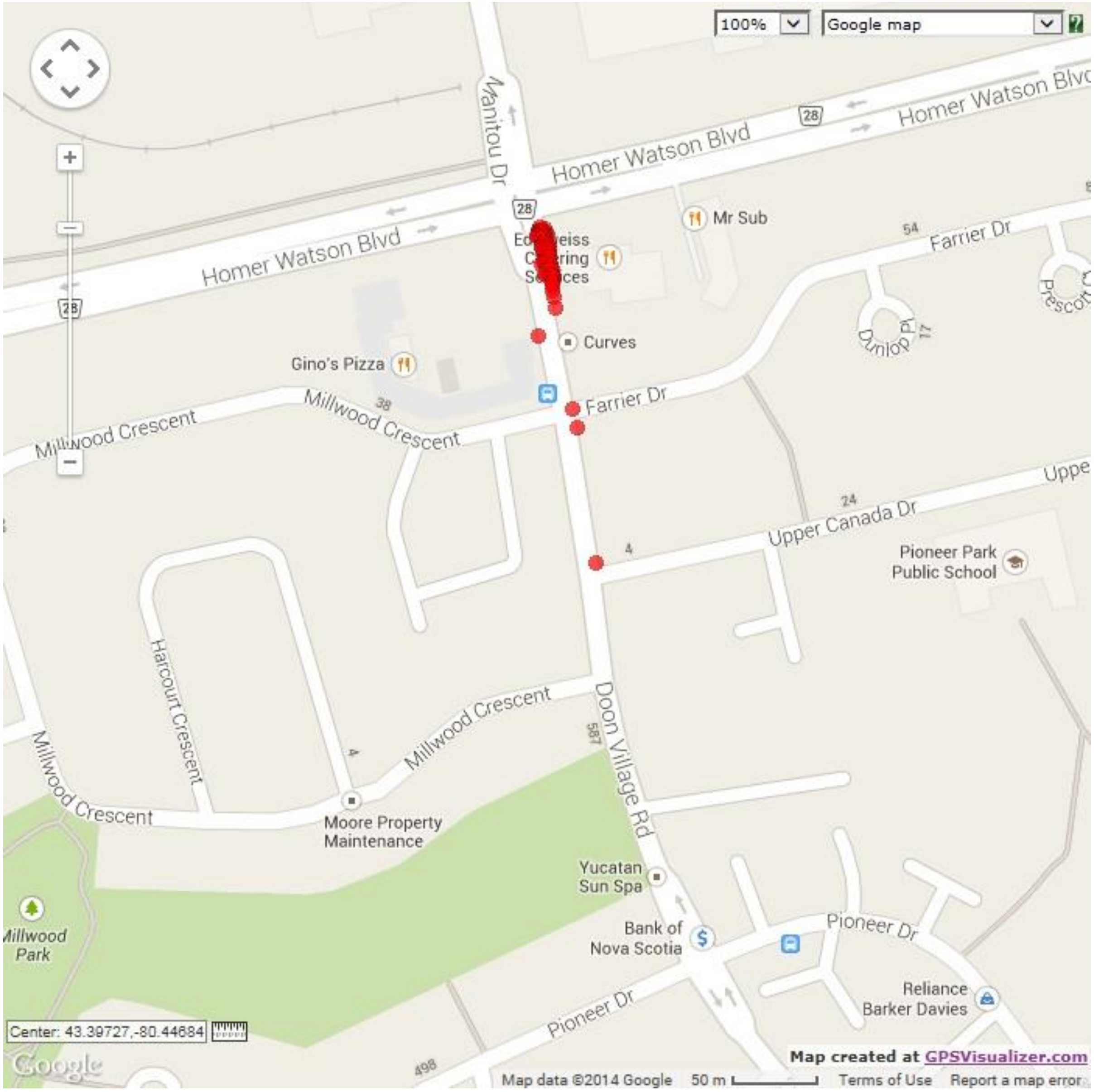
WEBER @ Albert to WEBER @ Parkside

WEBER @ Parkside



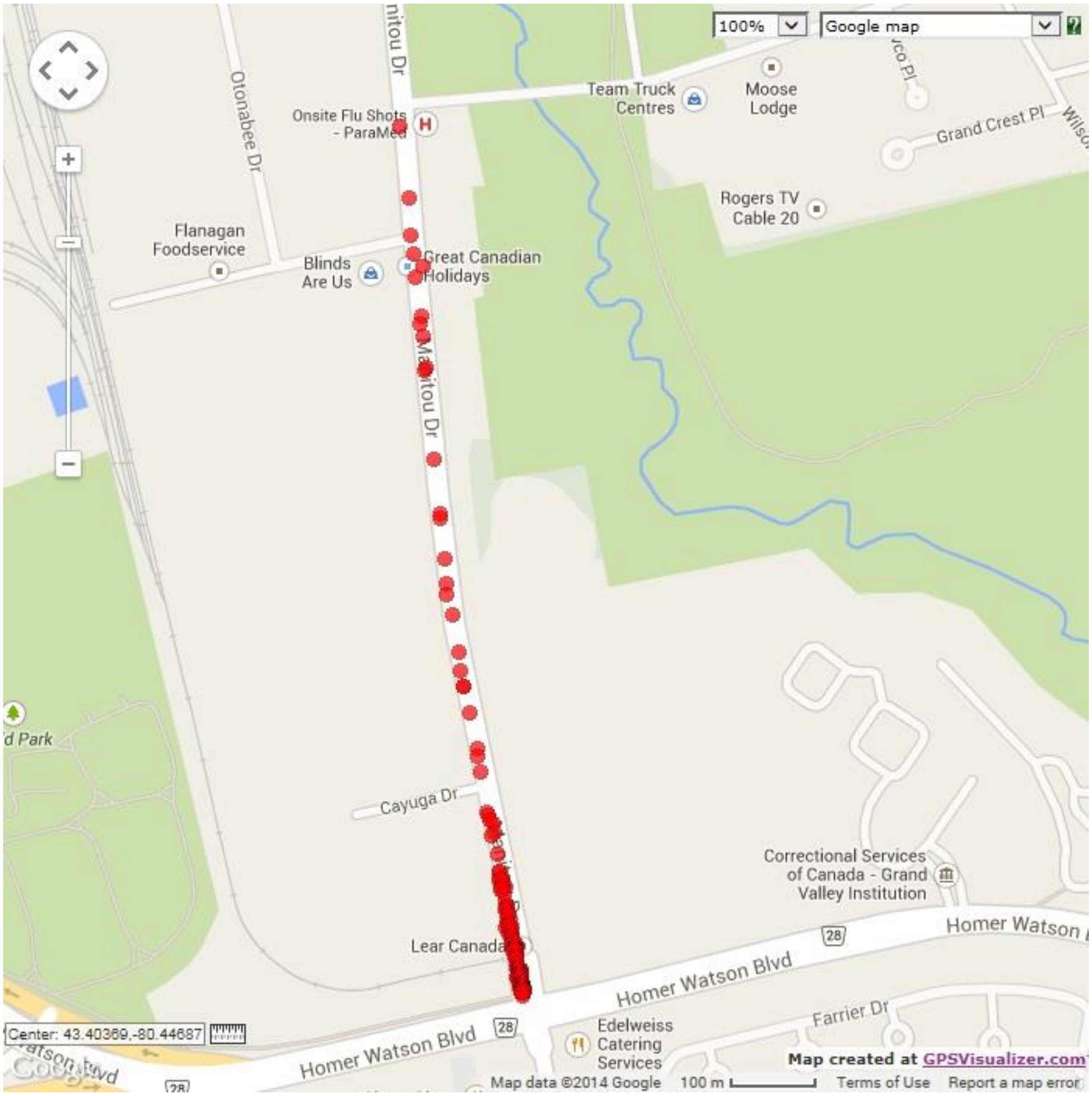
WILSON @ Kingsway to FAIRWAY @ Wilson

FAIRWAY @ Wilson



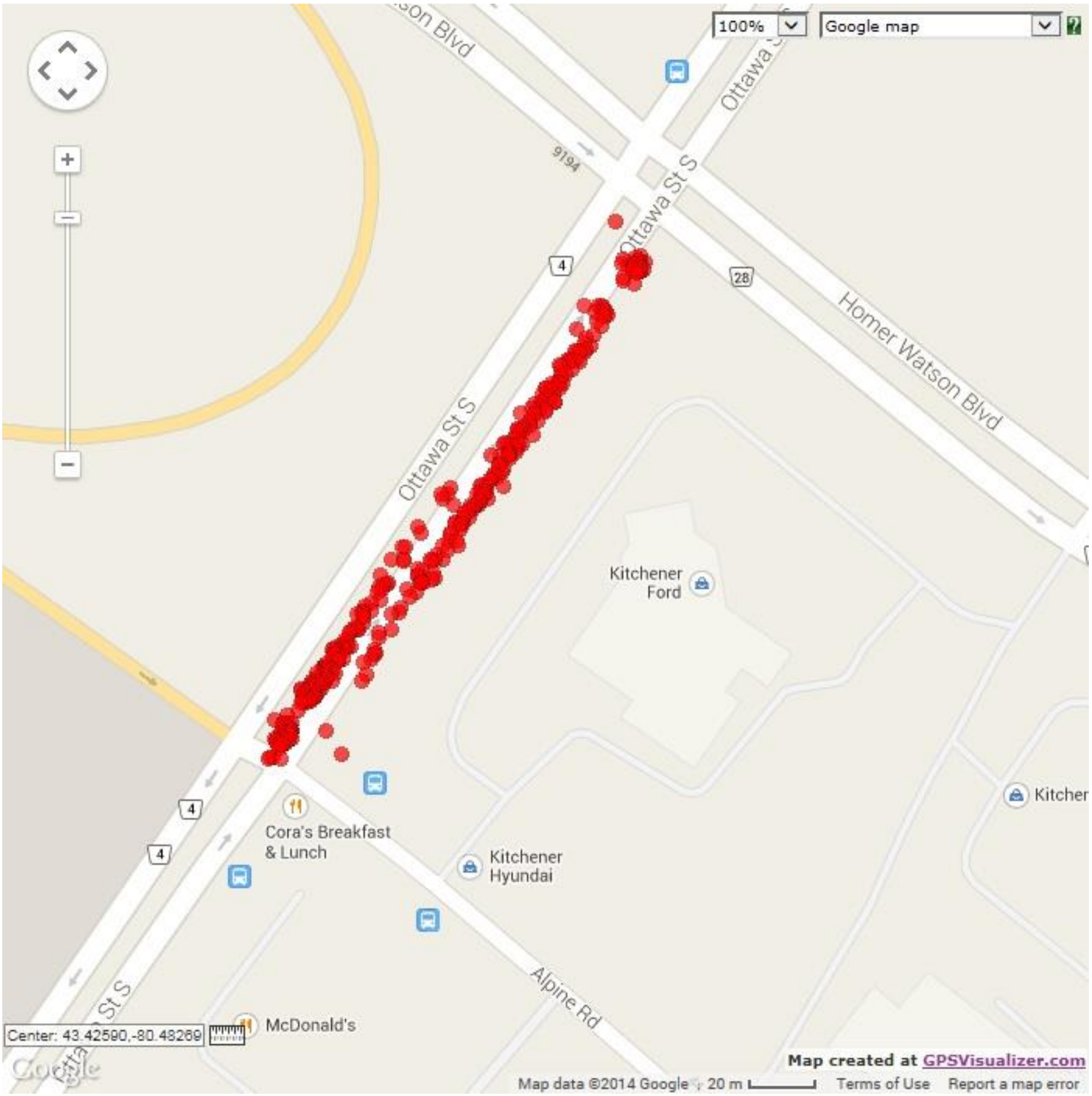
DOON VILLAGE @ Pioneer to HOMER WATSON @ Manitou and Doon Village

HOMER WATSON @ Manitou and Doon Village (Nb)



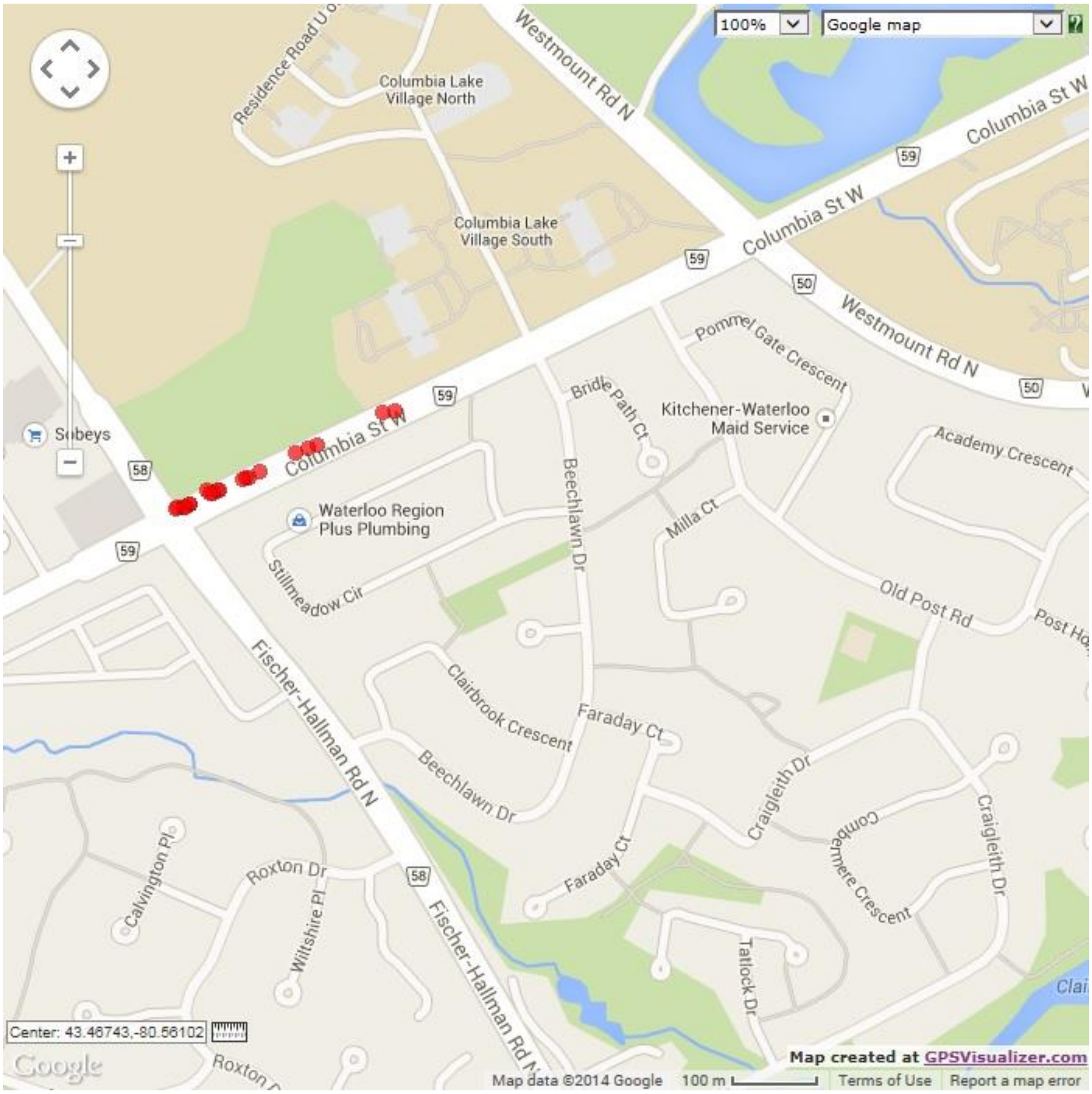
MANITOU @ Wabanaki to HOMER WATSON @ Manitou and Doon Village

HOMER WATSON @ Manitou and Doon Village (Sb)



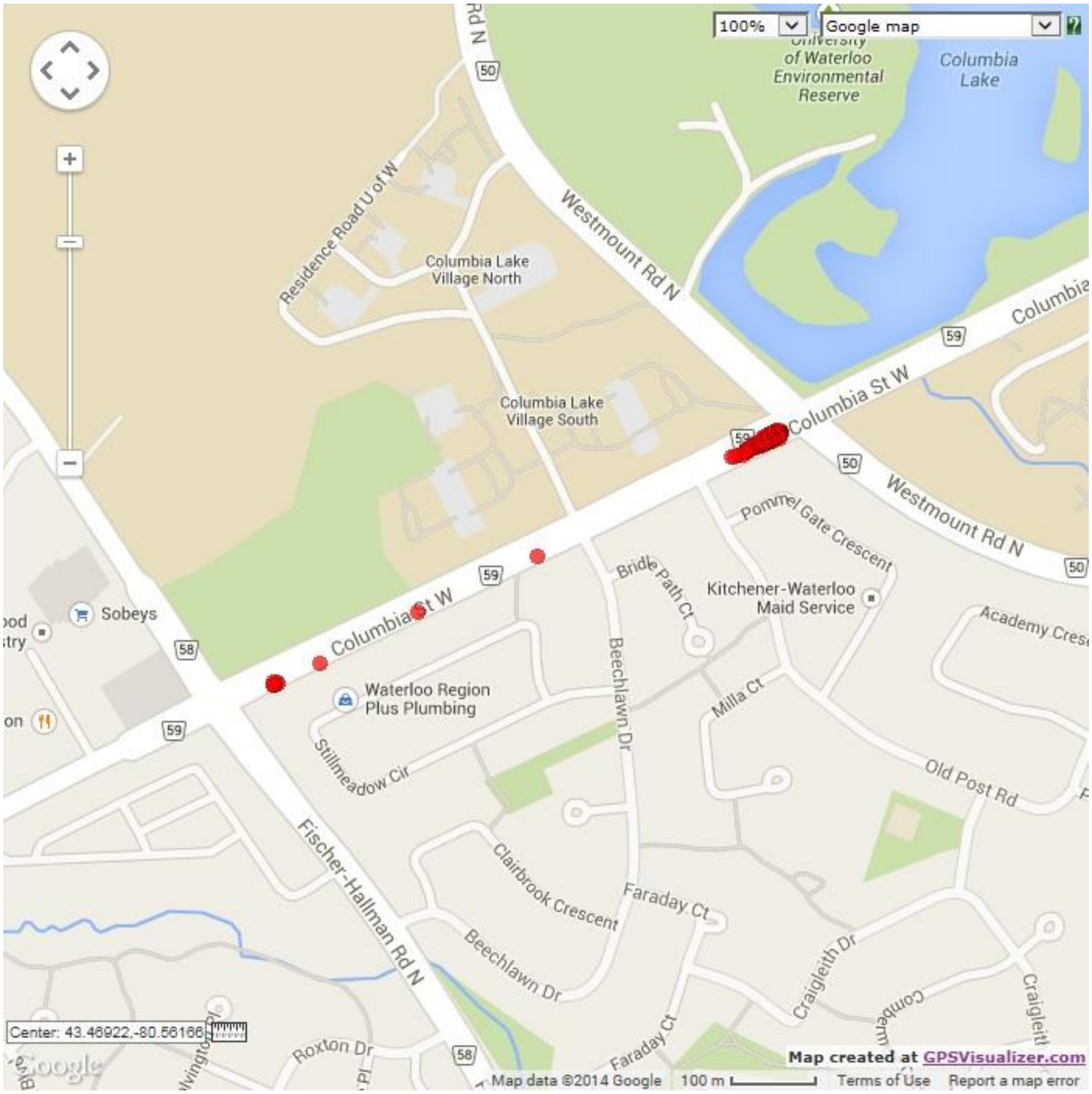
OTTAWA @ Alpine to OTTAWA @ Homer Watson

OTTAWA @ Homer Watson



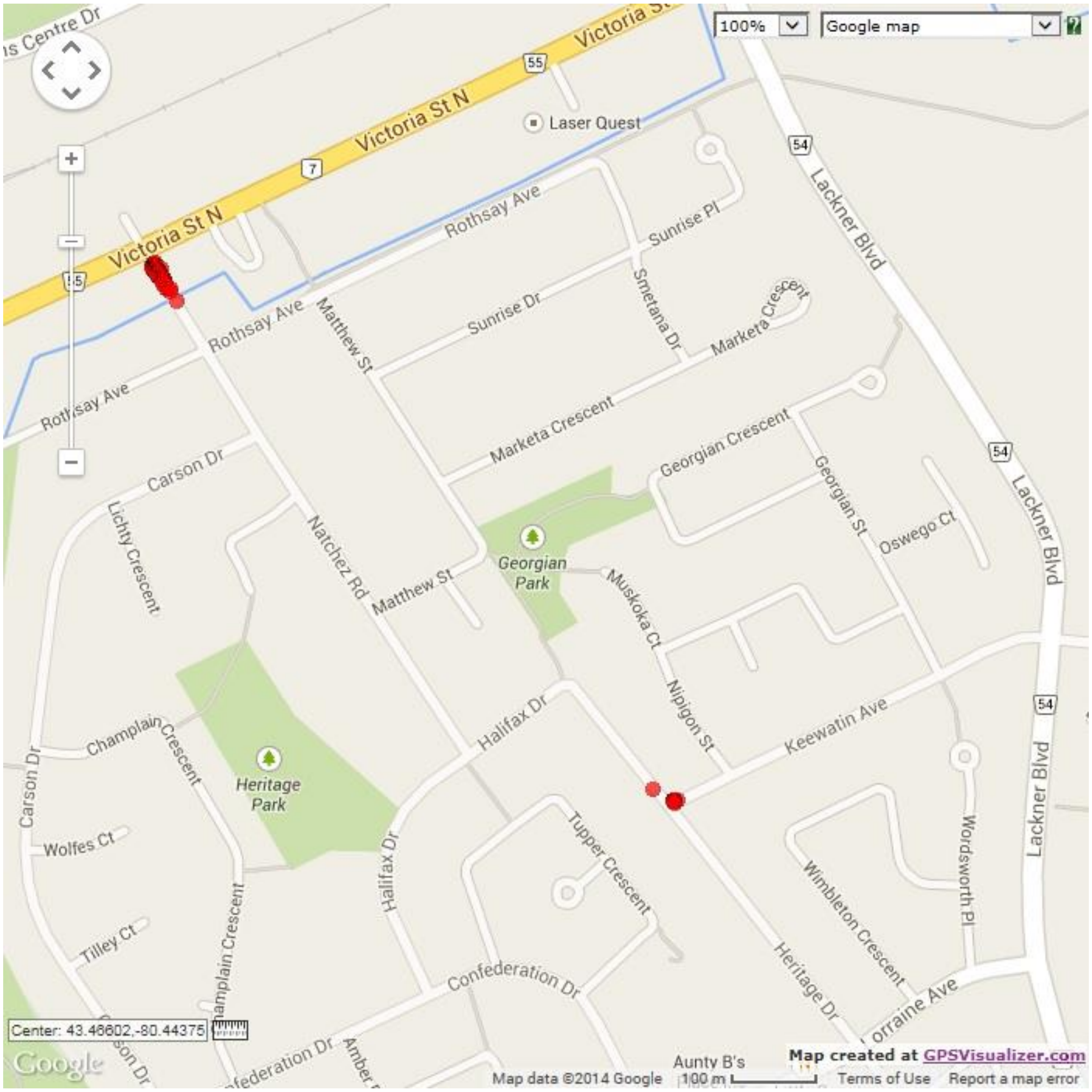
WESTMOUNT @ Columbia to FISCHER-HALLMAN @ Columbia

FISCHER-HALLMAN @ Columbia



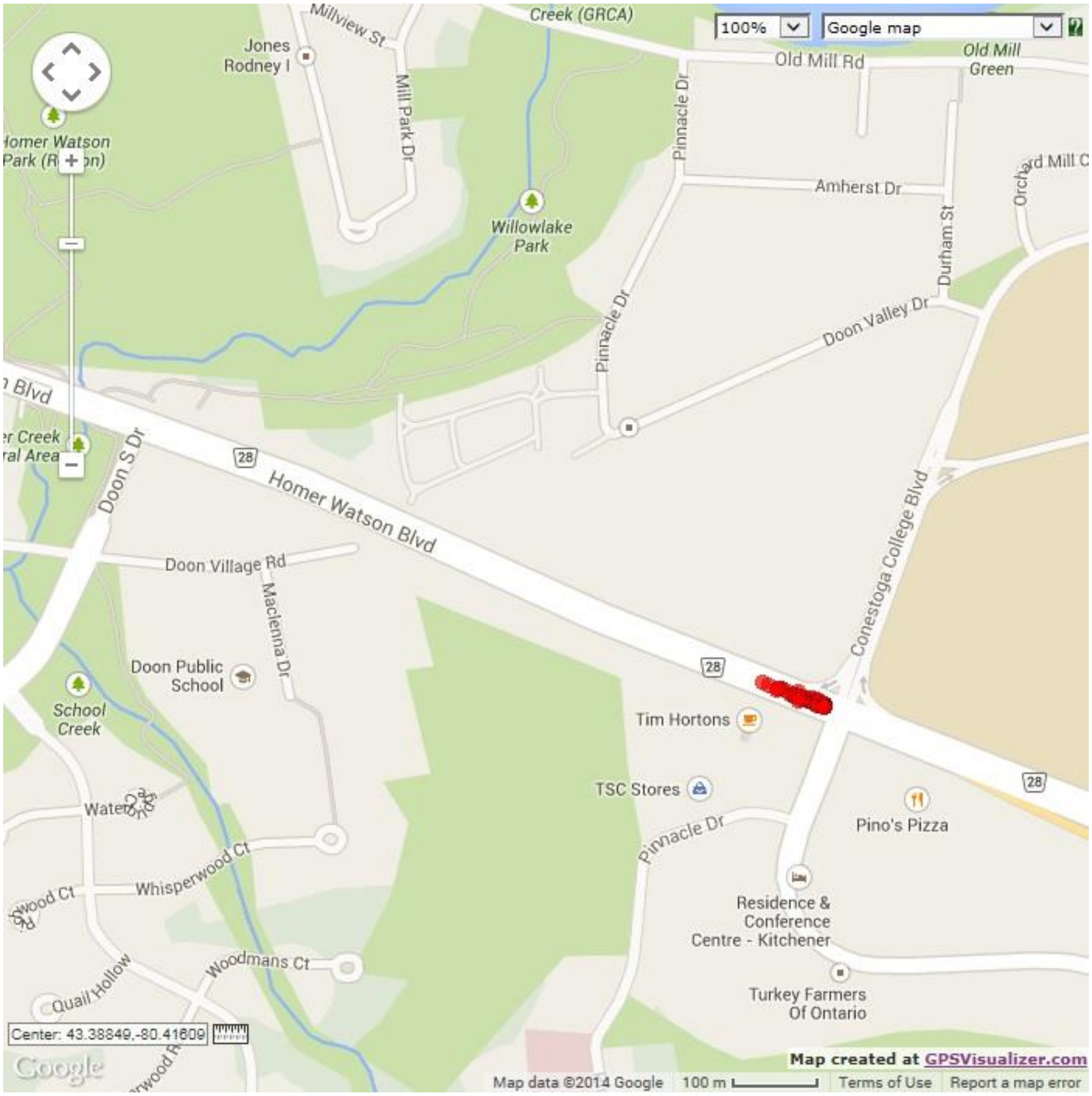
FISCHER-HALLMAN @ Columbia to WESTMOUNT @ Columbia

WESTMOUNT @ Columbia



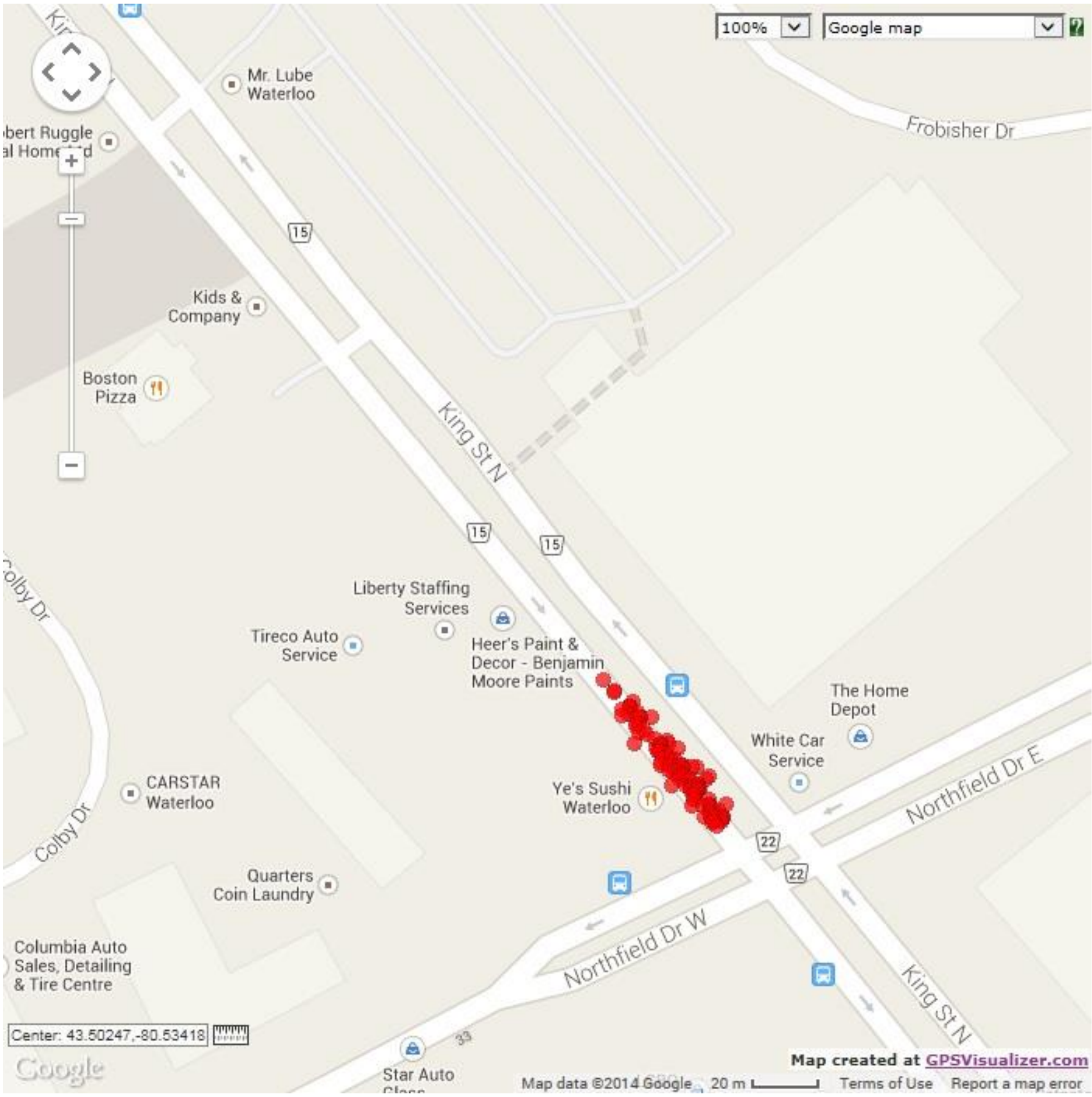
LACKNER @ Keewatin to VICTORIA @ Natchez

VICTORIA @ Natchez



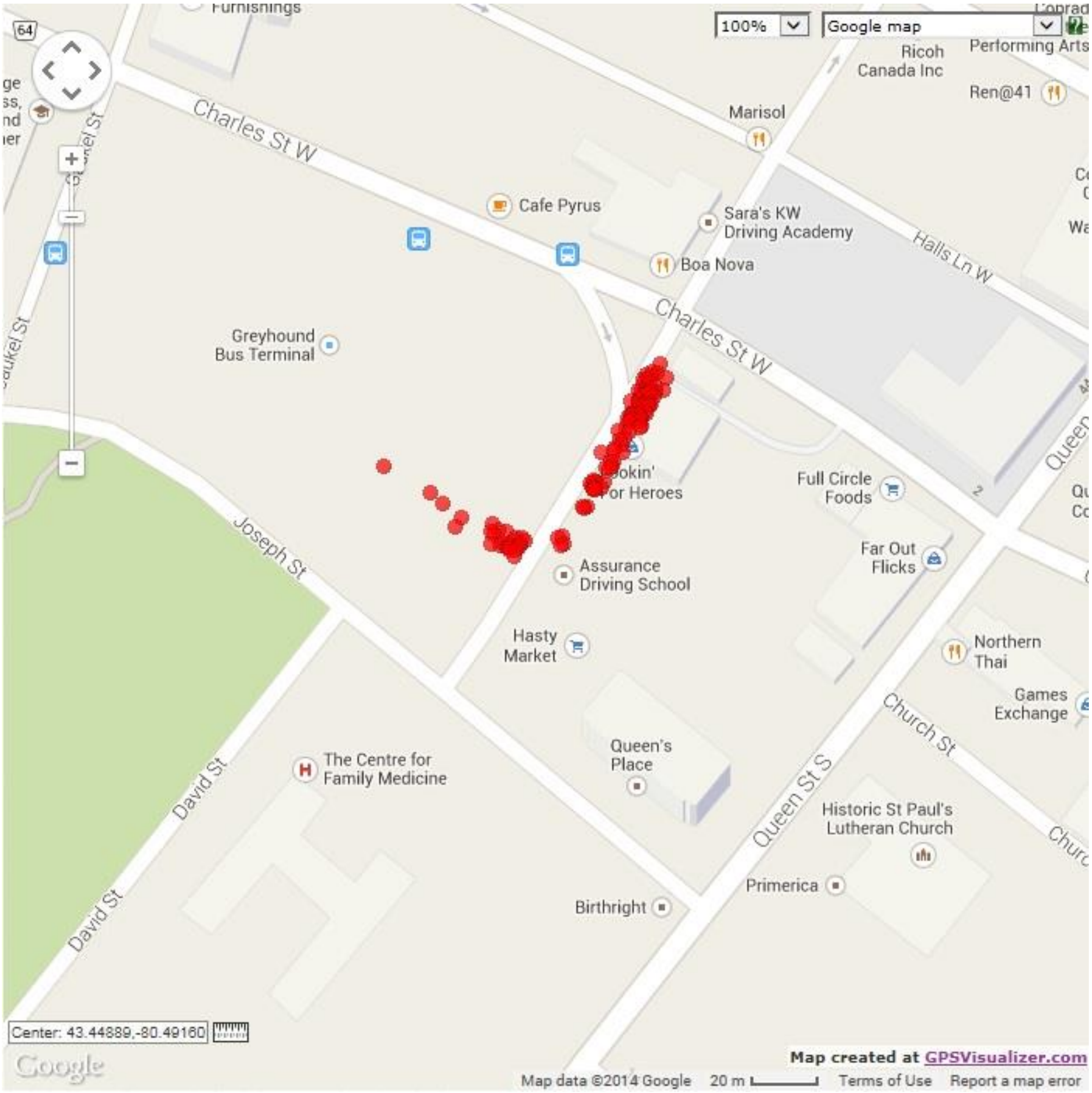
HOMER WATSON @ Doon South and Monarch to HOMER WATSON @ Conestoga College

HOMER WATSON @ Conestoga College



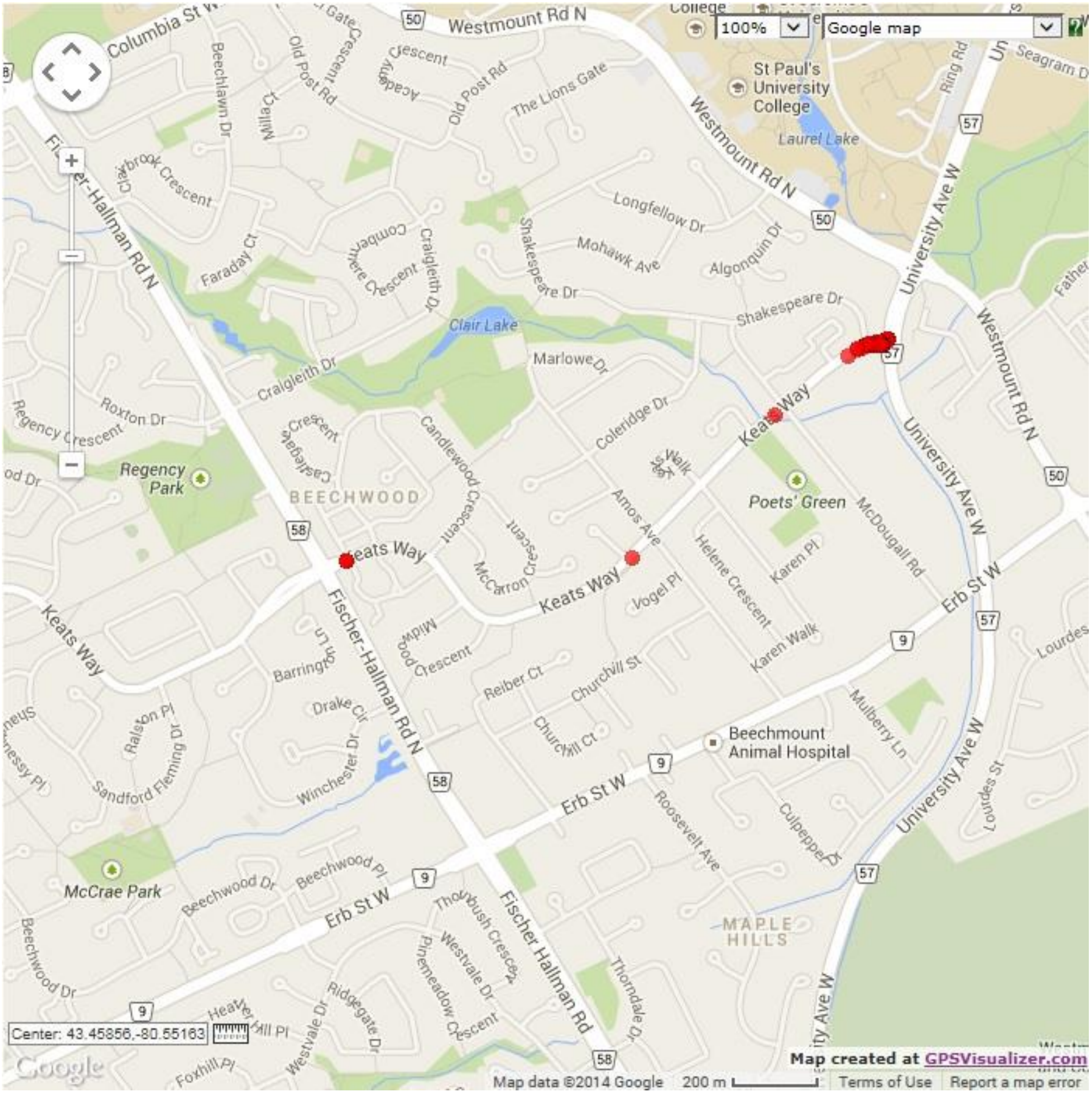
KING @ Home Depot to KING @ Northfield

KING @ Northfield



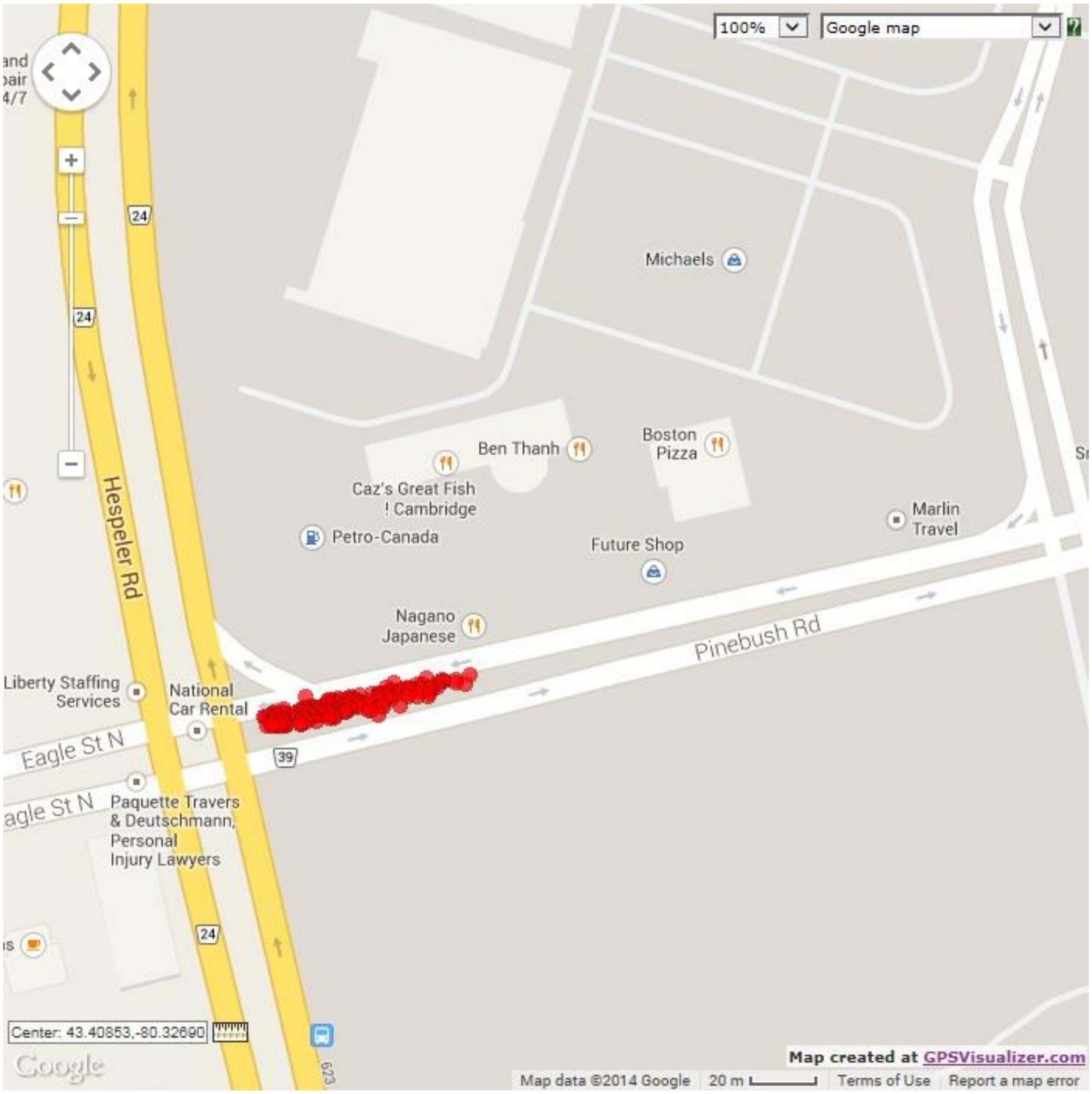
TERMINAL to CHARLES @ Ontario

CHARLES @ Ontario



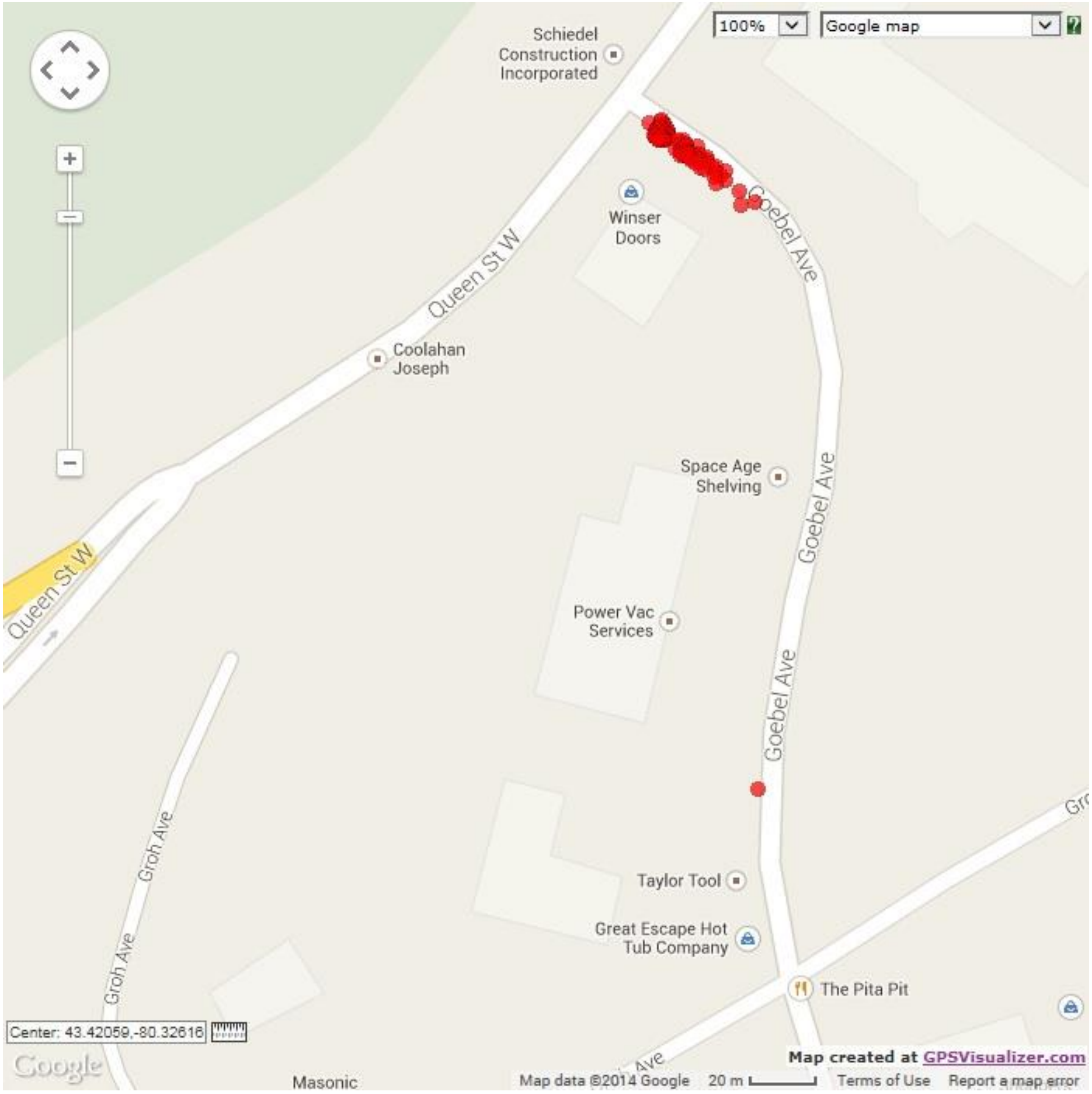
FISCHER HALLMAN @ Keatsway to UNIVERSITY @ Keatsway

UNIVERSITY @ Keatsway



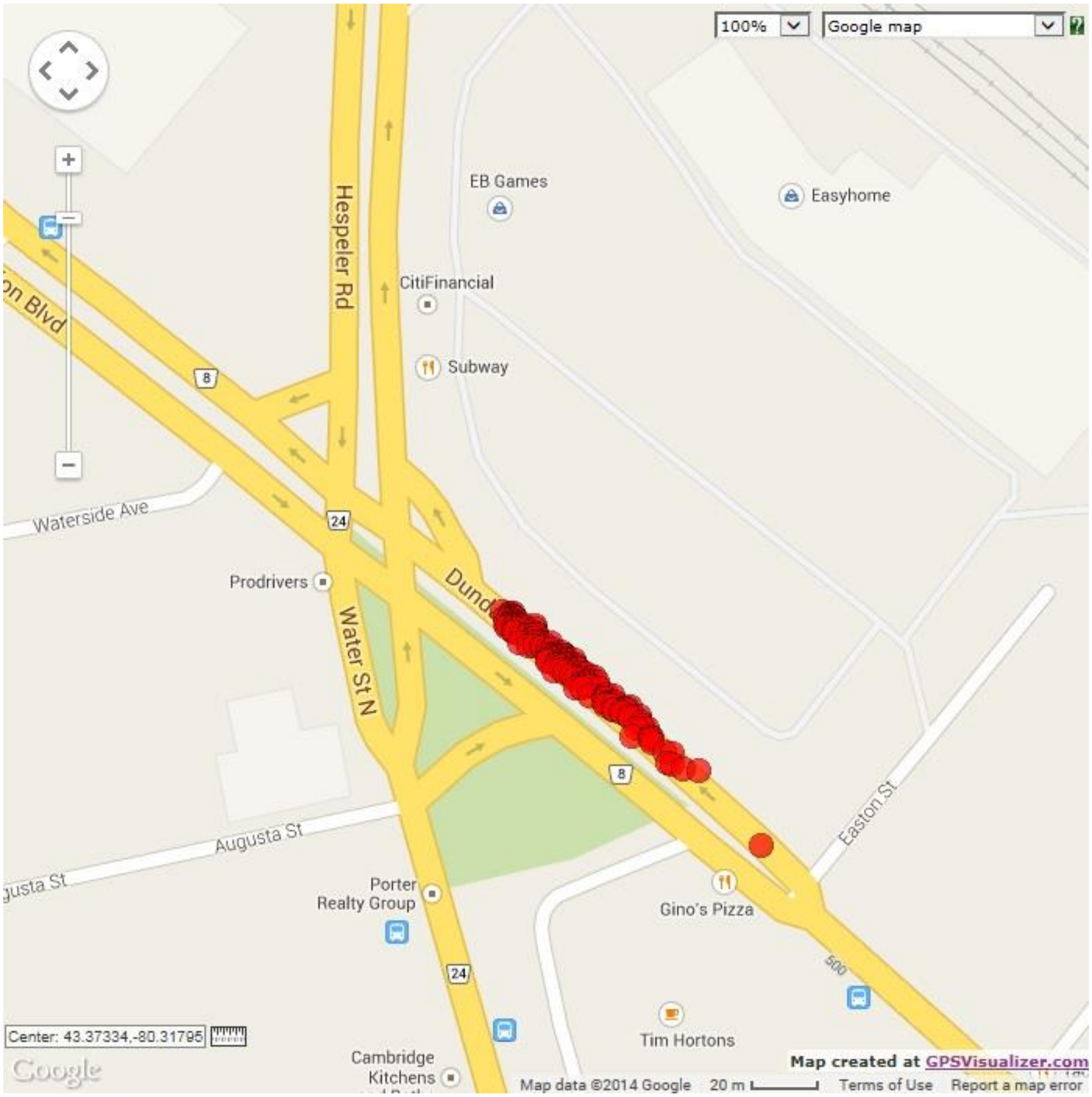
PINEBUSH @ Walmart and Home Depot to HESPELER @ Eagle and Pinebush

HESPELER @ Eagle and Pinebush

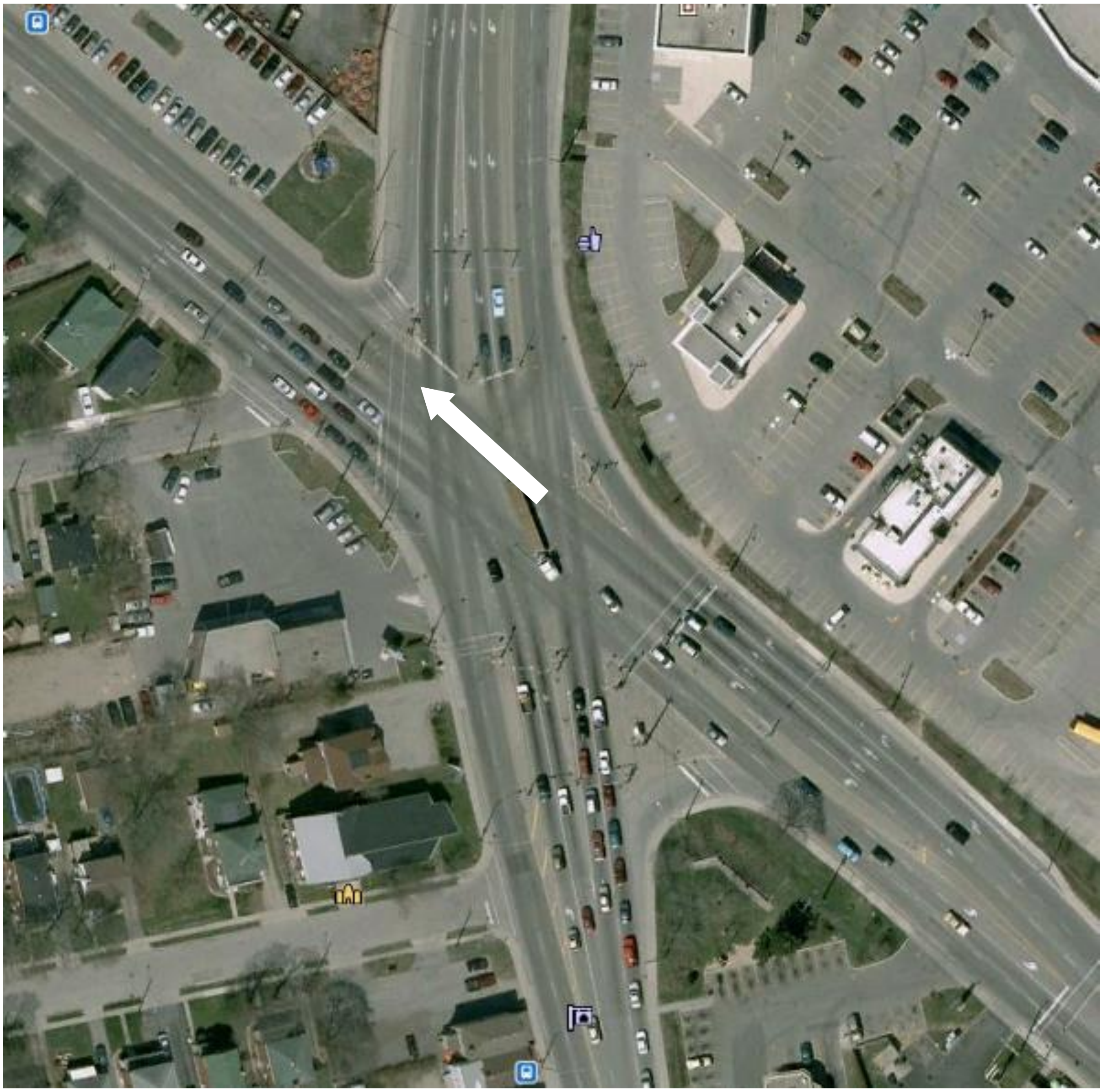


HOLIDAY INN @ Groh to QUEEN @ Goebel

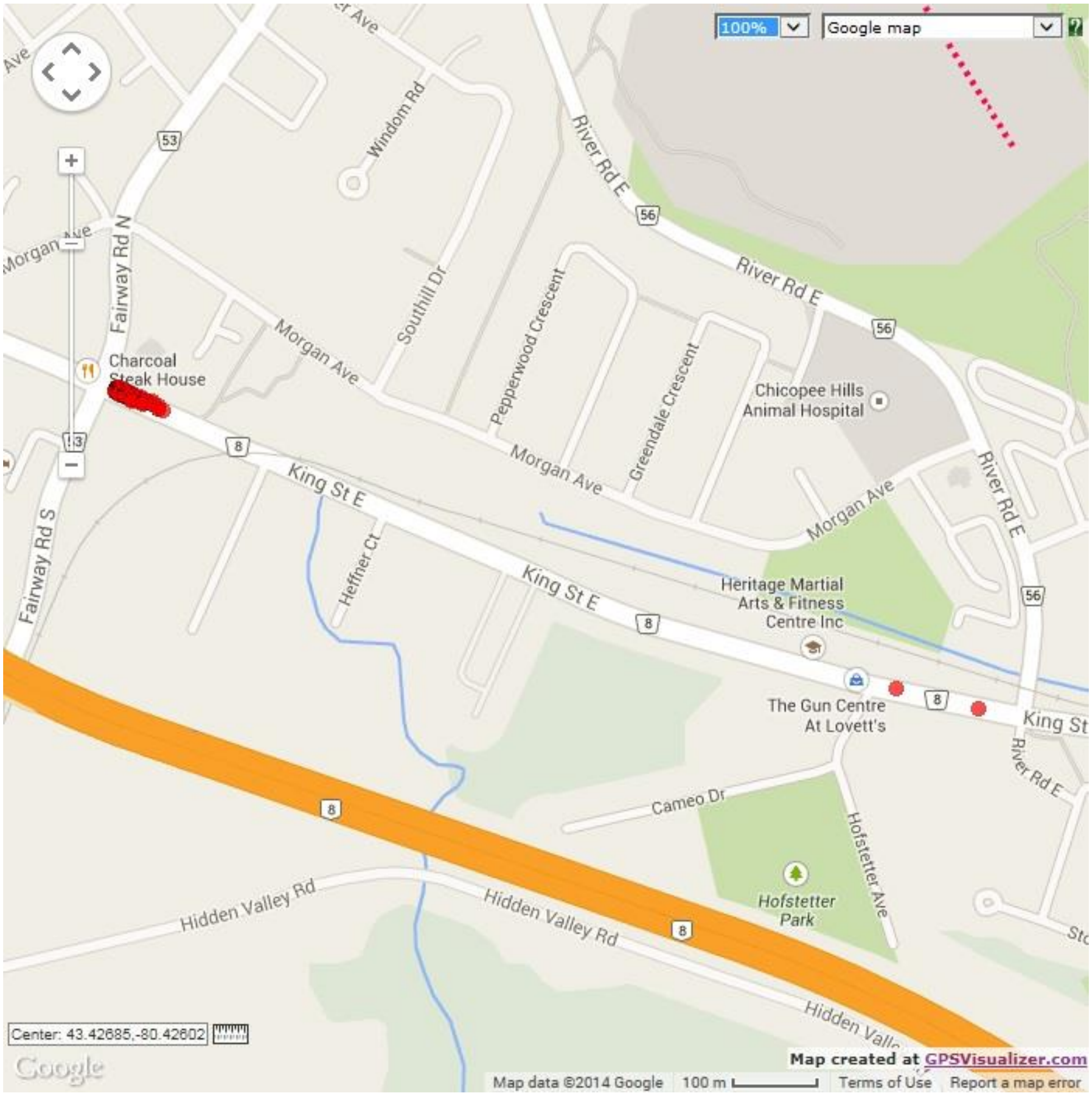
QUEEN @ Goebel



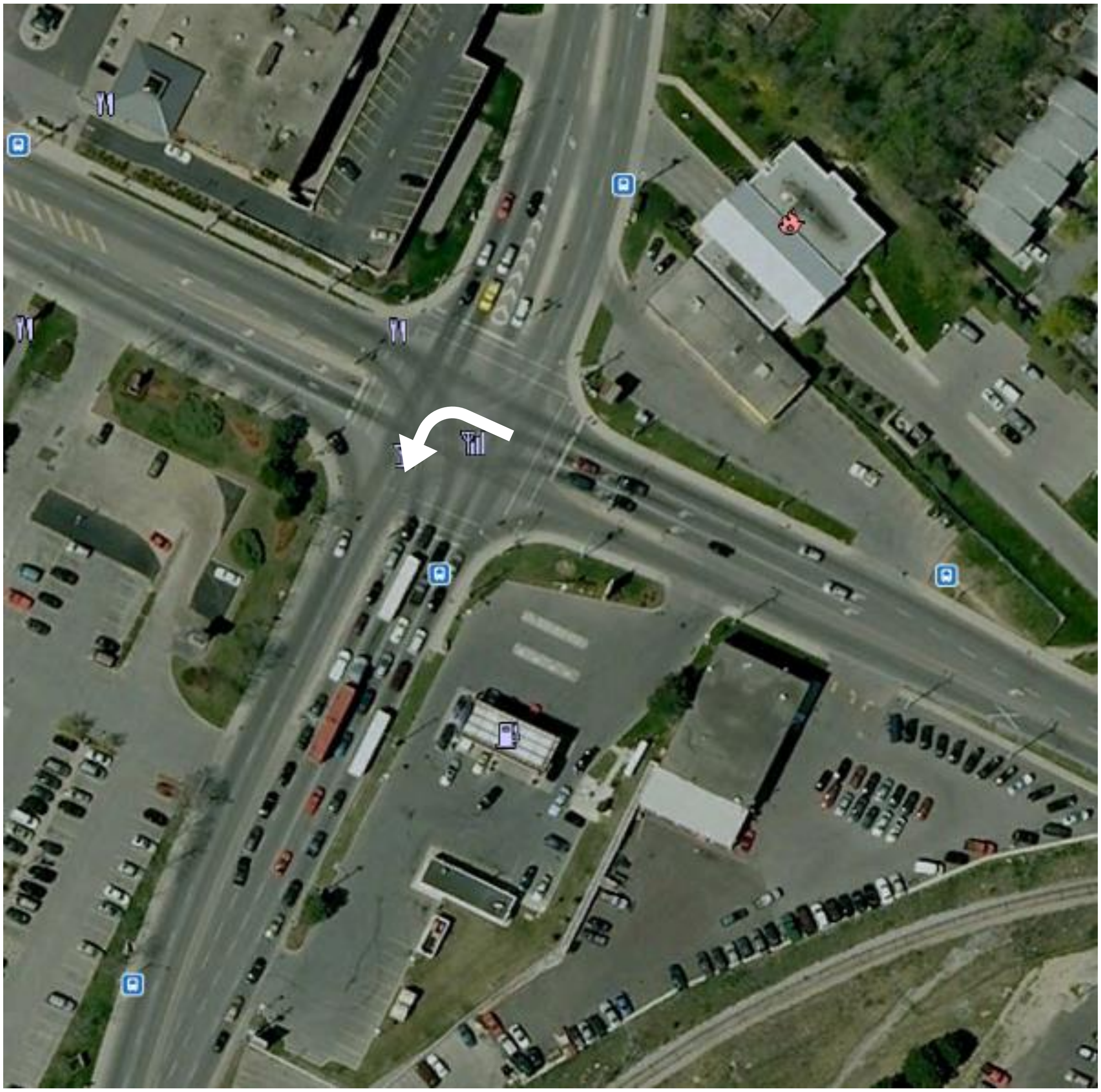
DUNDAS @ Easton to HESPELER and WATER @ Coronation and Dundas



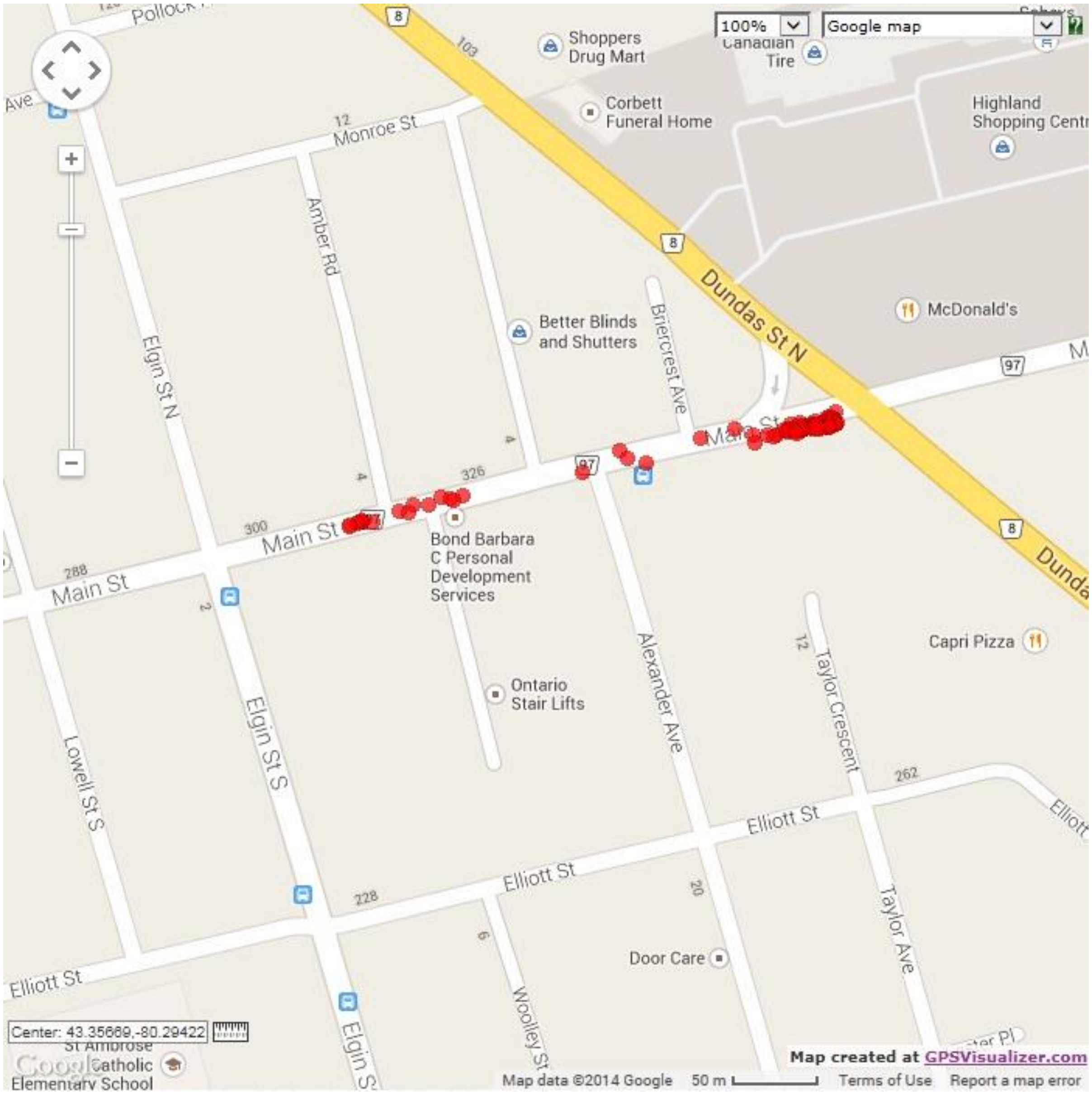
HESPELER and WATER @ Coronation and Dundas



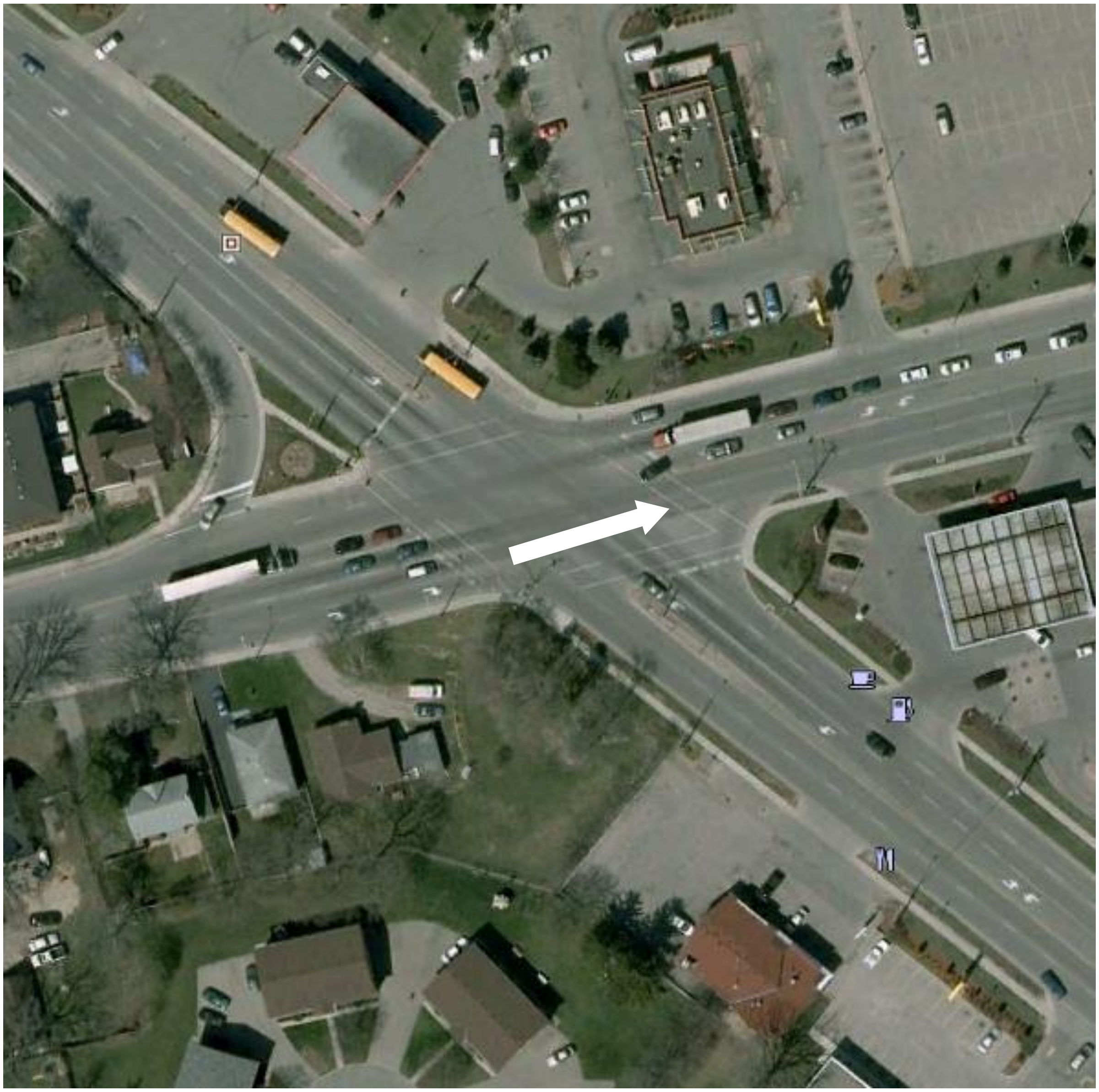
KING @ River to FAIRWAY @ King



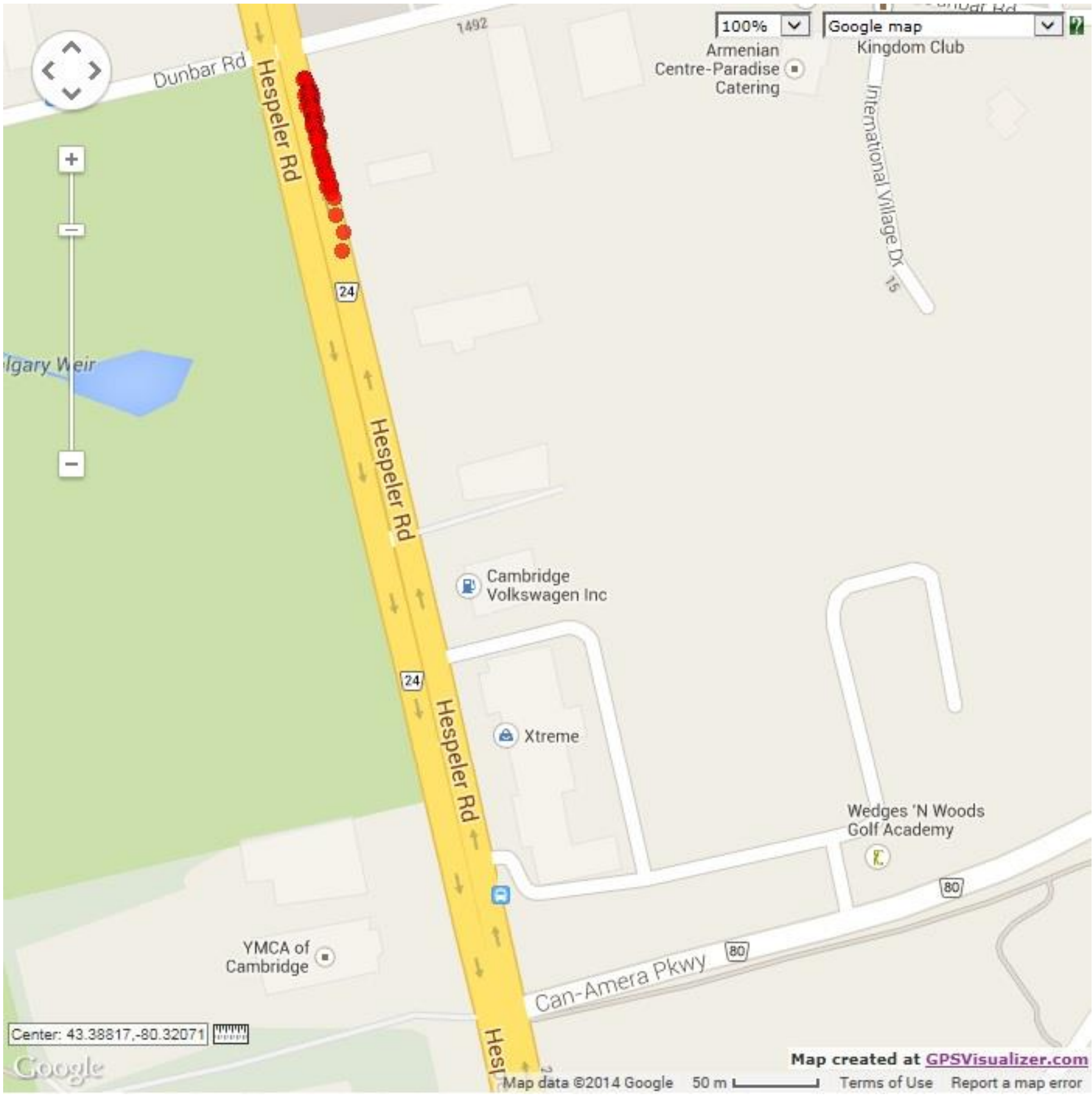
FAIRWAY @ King



MAIN @ Elgin to DUNDAS @ Main

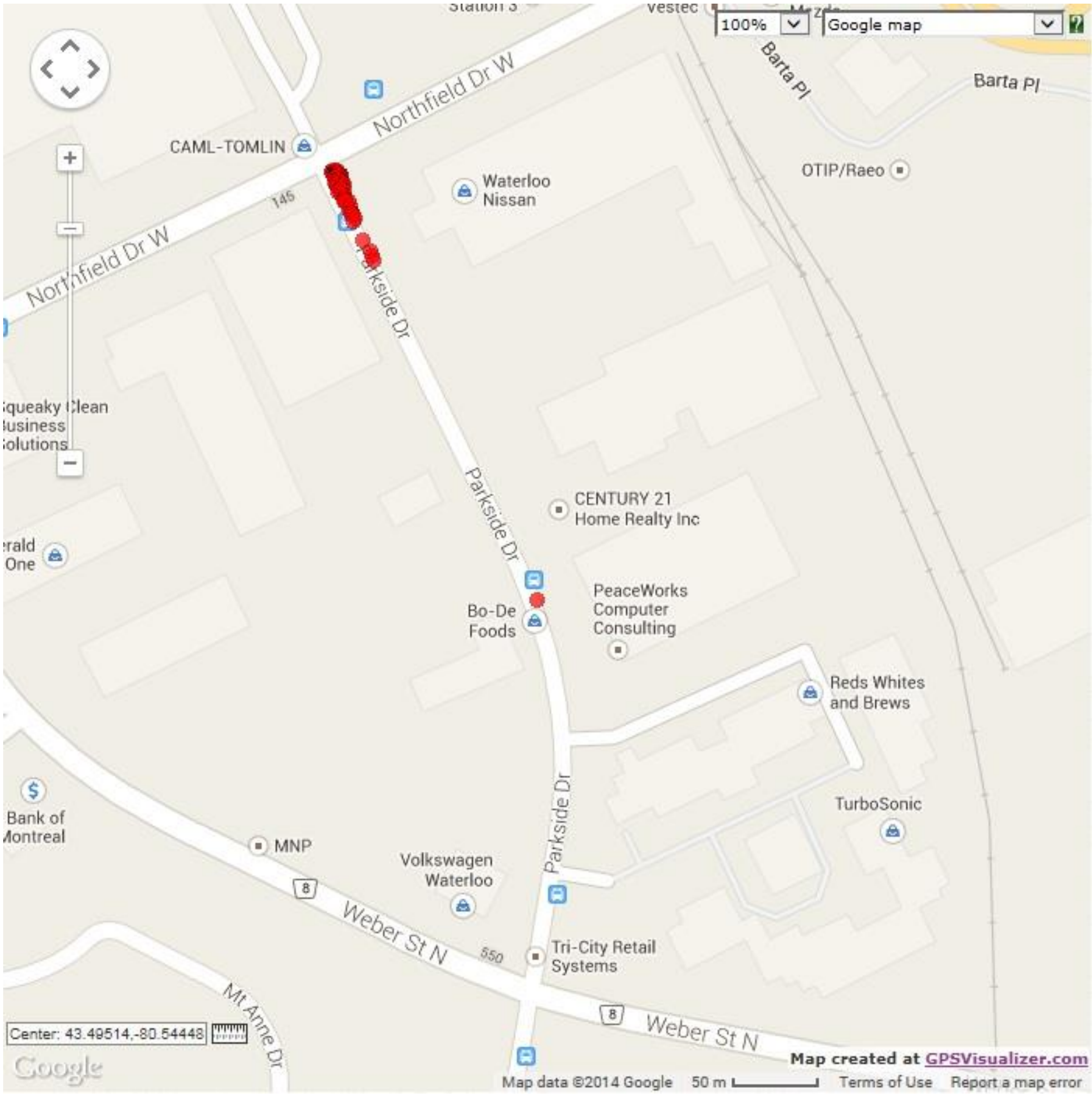


DUNDAS @ Main

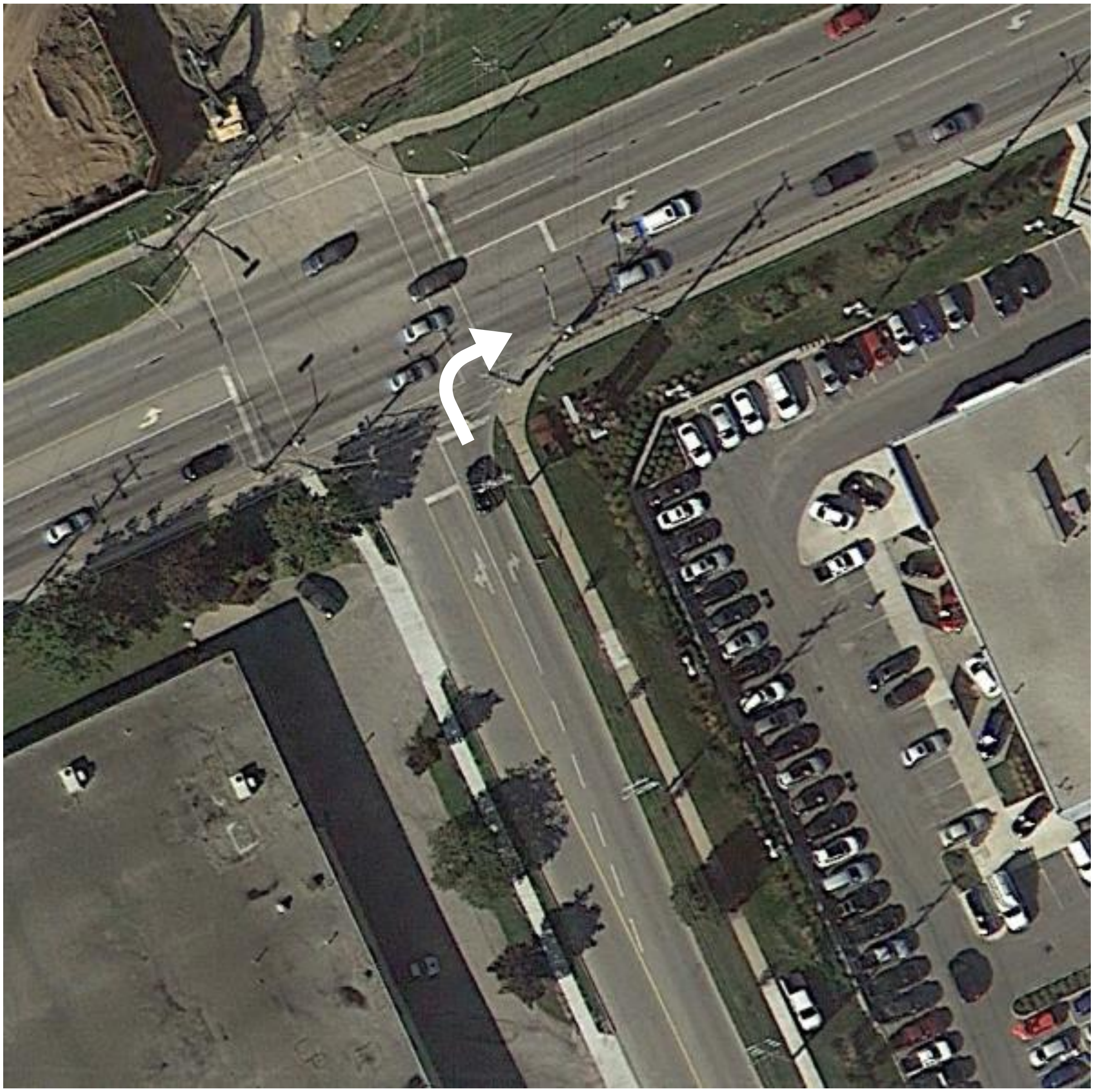


HESPELER @ Can Amera and YMCA Driveway to HESPELER @ Dunbar

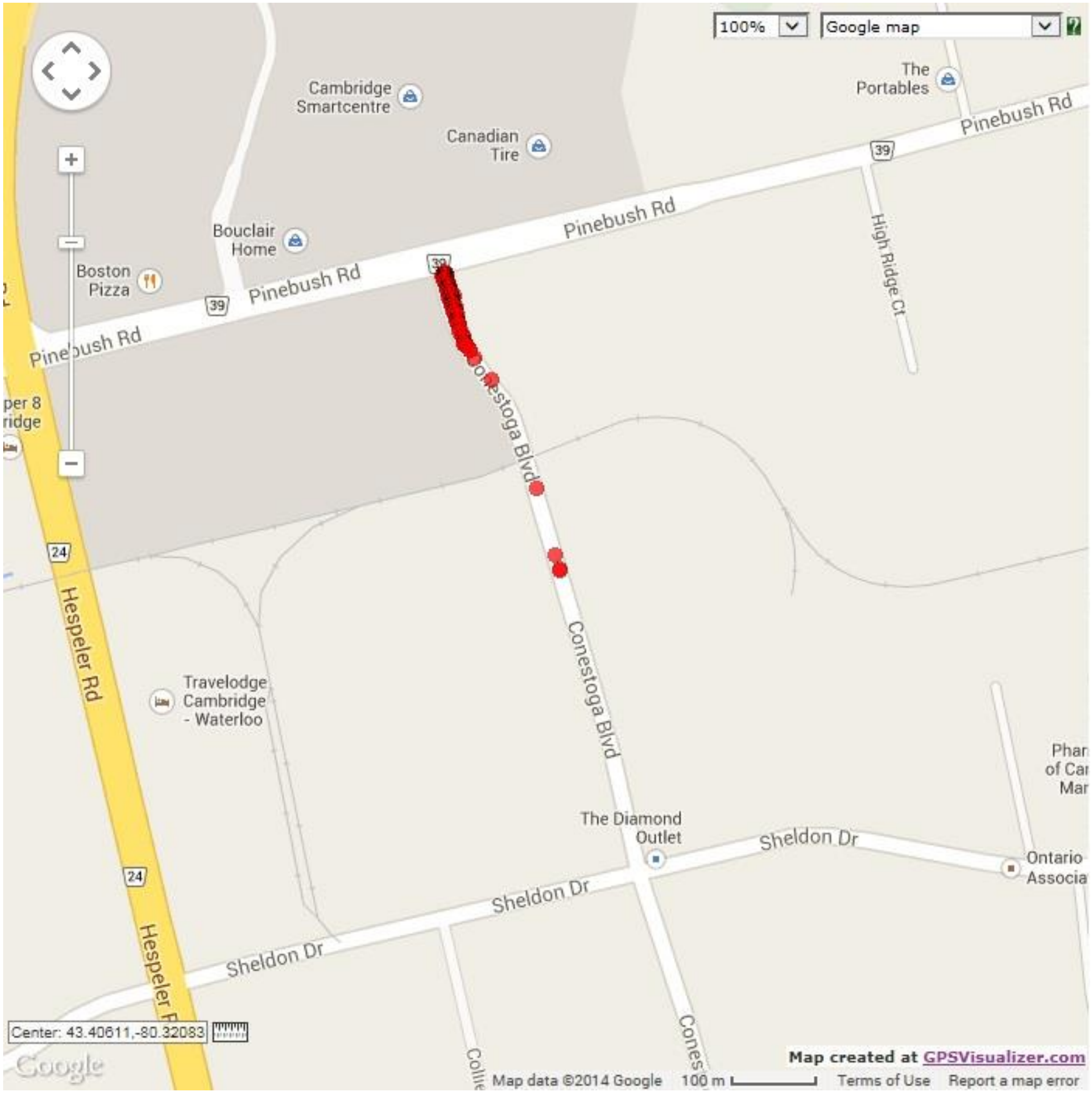
HESPELER @ Dunbar



WEBER @ Parkside to NORTHFIELD @ Parkside

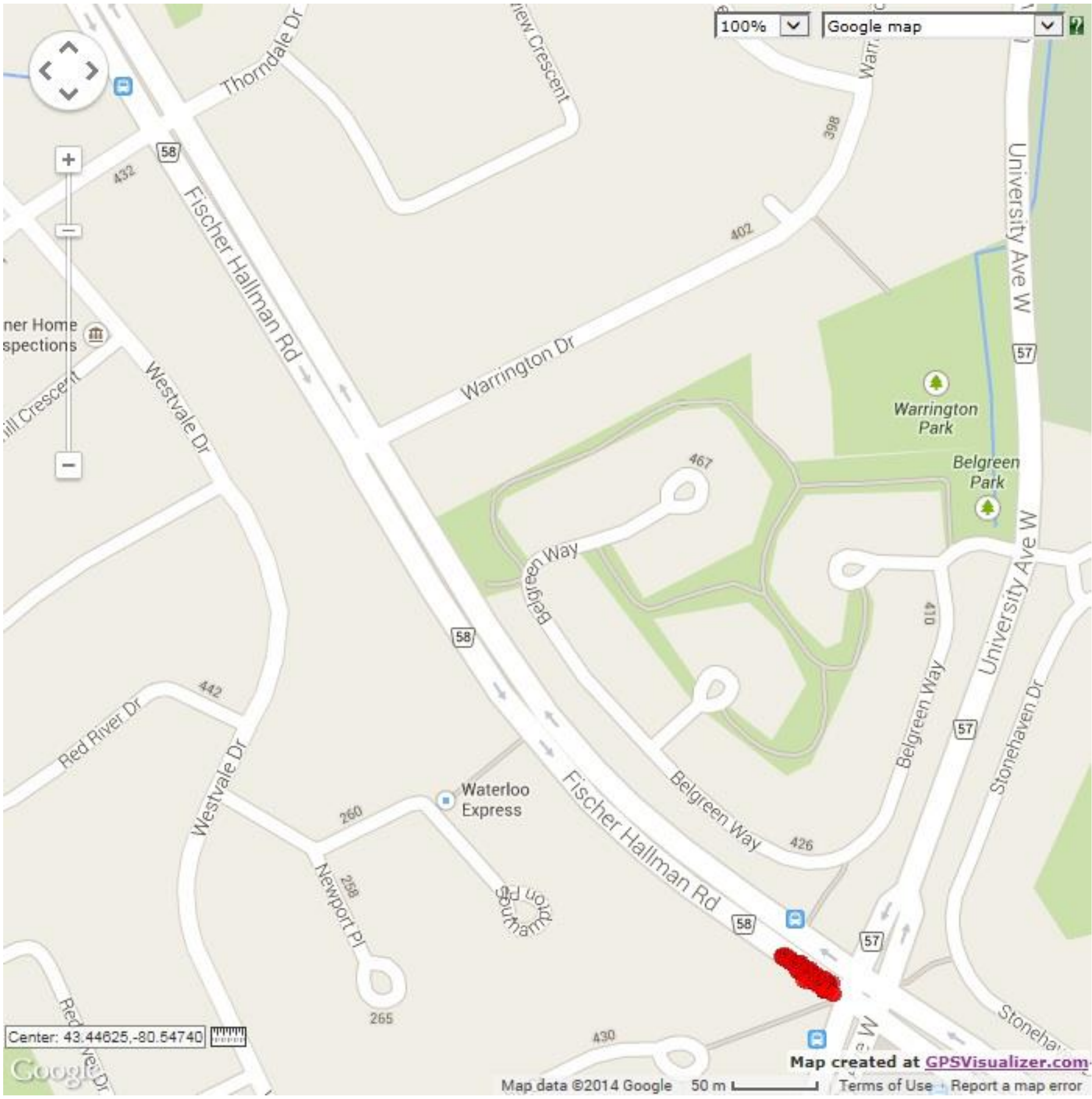


NORTHFIELD @ Parkside



SHELDON @ Conestoga to PINEBUSH @ Conestoga

PINEBUSH @ Conestoga



FISCHER-HALLMAN @ Thorndale to FISCHER-HALLMAN @ University

FISCHER-HALLMAN @ University