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Implementation of Proactive Traffic Signal Control System at Multiple Intersections at the Greater Houston Area: Final Report

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16. Abstract

This report summarizes the findings from implementing the proactive signal control system at 30 intersections at four testbeds in the Houston metropolitan area. The work started in April 2017 and the final installation was on December 17, 2018. The final data set was received on January 24, 2019. The data were used evaluate the performance of the implemented proactive signal control system.

This project is an extension of Project 0-6920, which proposed a new signal control logic aiming to help vehicles in a platoon more smoothly pass through an intersection at or close to the design speed. Vehicles in a platoon have smaller headways, so the capacity of an intersection could be increased if vehicles in a platoon could pass through the intersection without being interrupted. This logic relies on the existing detecting and control technologies used by TxDOT on Texas roads. Therefore, it is not necessary to test and install new hardware systems, which could be costly and time-consuming, and also may be subject to many uncertainties.

Our findings indicate that the proposed proactive signal control worked well when the traffic flow rates were more than 500 veh/hr/ln and the occupancy rates exceeded 0.4. If the flow became less than 350 veh/hr/lm and/or the occupancy rates dropped below 0.20, the new system had insignificant impact on the traffic patterns. When the flow and/or occupancy is low, vehicle movement is more independent, and the arrival pattern tends to be more random. Therefore, the proposed proactive signal control system is not so effective in impacting the traffic patterns at an intersection.

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TxDOT Project 5-6920-01

Implementation of Proactive Traffic Signal Control System at Multiple Intersections at the Greater Houston Area: FINAL REPORT

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Especially, I wish to thank Darrin Jensen, our Project Manager, for his great leading and coordinating efforts in this project. Due to Hurricane Harvey and the detector problems, the progress of this project was heavily delayed. He successfully solved the problems and helped us extended the schedule twice.

I also wish to show my appreciation to Steve Chiu, from the Houston District Office of TxDOT. It would be not possible for us to complete this project without his great help in the testbed selection, data collection, and field implementation. He also offered lots of valuable suggestions in model building and field implementation. Both students and PIs learnt a lot from him. He also introduced us to the latest developments in sensing technologies used on Texas roads. Roy Gonzalez from the Houston District Office of TxDOT also helped us in field implementations.

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Other students also helped in this project. Prakash Dahal, Prakash Koirala, MM Haque, Shahin Sajjadi, and Uttara Roy, who were partially sponsored by this project, made contributions in the data analysis, origin-destination matrix building, and onsite data collection. Kapil Dhital, Mdmasharul Kabir, Sital Shrestha, Prabal Tandukar, and Qian Wen, who voluntarily participated in this project, contributed to data cleaning, data analysis and report preparation.

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DISCLAIMER

EXECUTIVE SUMMARY

This report summarizes the findings from implementing the proactive signal control system to 30 intersections at four testbeds in the Houston metropolitan area. The work started in April 2017 and the final implementation was conducted on December 17, 2018. The final data set collected from the intersections was received on January 24, 2019. The data were used evaluate the performance of the implemented proactive signal control system.

This project is an extension of Project 0-6920, which proposed a new signal control logic aiming to help vehicles in a platoon more smoothly pass through an intersection at or close to the design speed. Vehicles in a platoon have smaller headways, so the capacity of an intersection could be increased if vehicles in a platoon could pass through the intersection without being interrupted. This logic relies on the existing detecting and control technologies used by the Receiving Agency in Texas roads. Therefore, it is not necessary to test and install new hardware systems, which could be costly and time-consuming, and also may be subject to many uncertainties.

The intersections selected for this project have more complicated conditions than the one tested in Project 0-6920, which is a single intersection far away from others and has small traffic volumes from minor road. In this project (5-5920), the research team had faced two challenges.

The first one is how to balance the existing coordinated signal control system and the new proactive signal control system. It was found that the coordinated signal control cannot be replaced if the through traffic on the major road has large volumes and the spacing between adjacent intersections is small. The reason is that the proactive signal control logic relies on volumes and occupancies, which are the only two available information on real traffic from the current detectors. These two parameters cannot be used to precisely determine the arrivals of vehicle platoons at an approach. Instead, an approximate method was employed, which was based on the simulation results. It was found that though it could not precisely capture the offset of flow between two closely spaced intersections, this proposed proactive control logic was particularly helpful in dealing with the left-turn traffic from the major road or traffic from the minor road.

The second challenge is how to deal with the traffic through a diamond intersection which combines two intersections together and have overlapping phases. This diamond intersection is much more complicated than others. Especially, some only have video detectors, which underestimate the traffic volumes largely. The research team developed two separate logics for diamond intersections: one for those with loop detectors, and the logic is based on volumes and occupancies; and another is for those with video detectors, and the logic relies on occupancies only. The data analysis show that these two new systems did work at diamond intersections. But generally, the one for loop detectors exhibited better performance, because both volumes and occupancies were used in the logic to capture the arrivals of vehicle platoons.

In this project, it was also found that the proposed proactive signal control worked well when the traffic flow rates were more than 500 veh/hr/ln and the occupancy rates exceeded 0.4. If the flow became less than 350 veh/hr/lm and/or the occupancy rates dropped below 0.20, the new system were found to have insignificant impact on the traffic patterns. It is because when the flow and/or occupancy is low, vehicles' movement is more independent, and the arrival pattern tends to be more random. Therefore, the proposed proactive signal control system is not so effective in impacting the traffic patterns at an intersection.

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CHAPTER 1

INTRODUCTION

1.1. BACKGROUND

As an extension of Project 0-6920 (January 2016–December 2016), Project 5-6920 focuses on the implementation of the signal control system developed and tested in Project 0-6920, which aimed to develop a *novel signal control system* to help vehicles in a platoon more smoothly passing through an intersection at speeds at or close to the design speed. Vehicles in a platoon have smaller headways than those not in a platoon, so the capacity of an intersection can be largely improved if vehicles in a platoon can more smoothly pass through this intersection without needing to slow down or being interrupted by traffic signal.

This new system shall utilize *the existing traffic detecting and controlling technologies used by the Performing Agency at the testbeds*, so that there is no need of purchasing and testing new hardware, which could be costly in acquisition and installation. The existing detectors used in this project are loop detectors (used in 28 intersections) and video detectors (2 intersections). In Project 0-6920, a *proactive signal control system* was developed and successfully tested at an intersection at NASA Road 1 (near Kemah, TX) in November 2016. The field observation and data analysis showed that this system helped more vehicles go through the intersection in a unit of time during peak hours, while the occupancy rates remained the same (or even less) at this intersection.

Due to the success of Project 0-6920, the Receiving Agency granted the research team Project 5-6920 to implement this proposed proactive signal control system to 30 intersections in the Houston Metropolitan Area, aiming to relieve the traffic congestions at these intersections by using the existing detecting and controlling technologies already applied to these intersections.

The 30 intersections are located in four arterial corridors in the Houston Metropolitan Area.

- 3 intersections at FM 528, located at the southeast suburb of Houston, near Friendswood,
- 8 intersections at SH 242, located at the north suburb near The Woodlands and Conroe,
- 11 intersections at FM 1464, located at the southwest suburb near Sugarland,
- 6 diamond intersections plus 2 intersections next to the diamond intersections along the frontage road of IH-10, in the Energy Corridor of Houston and Katy

Figure 1.1.1 gives the approximate locations of these four testbeds where these 30 intersections are located, and Table 1.1.1 shows the time when implementing the system to these 30 intersections. Please note that these 30 intersections are not exactly the same to those in the original work plan. The major changes are those along IH-10. Of the original eight diamond intersections, six were at the frontage road of IH-10, and two at the frontage road of I-610 (West Loop). Due to connection problems (the loop detectors could not report data), six were replaced with new ones. Only the intersections at Greenhouse Rd. and Westgreen Blvd. were kept. Table 1.1.2 shows the original and final lists of those eight intersections. Also, note that since such replacements were not finalized until the end of November 2018, just a month before the scheduled end time of the project, the implementation on these intersections were heavily delayed—the implementations to the final five intersections was conducted on December 17, 2018, just a week before the holiday and two weeks before the scheduled due day of the project final report. Such delay causes the delay of the completion of this final report.

Table 1.1.1 Implementation time for the intersections at different arterial corridors

Corridors	Implementation Time
3 intersections at FM 528	July 2017
8 intersections at SH 242	October–November 2017
11 intersections at FM 1464	February 2018 (Bissonnet), and April and May 2018 (other 10 intersections)
8 intersections along IH-10	November 2018 (Greenhouse, Westgreen, Katy Fort Bend) and December 2018 (other 5 intersections)

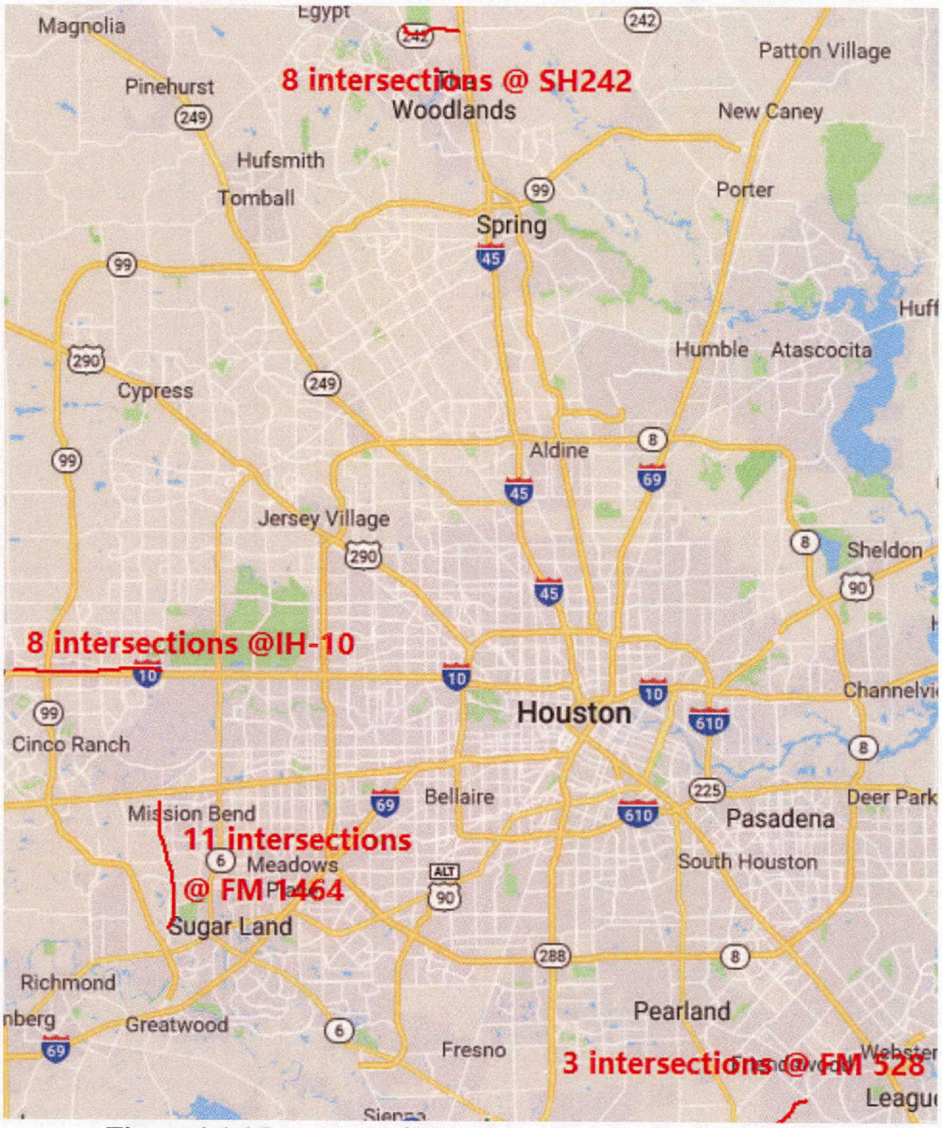


Figure 1.1.1 Locations of 30 intersections for implementations

Table 1.1.2 Intersections on the Frontage Road along IH-10

Original List	Final List
Greenhouse Rd. (D)	Greenhouse Rd. (D)
Westgreen Blvd. (D)	Westgreen Blvd. (D)
Sheldon Rd. (D)	Katy Fort Bend (D)
Bingle Rd./Voss Rd.	Barker Cypress (D)
Campbell Rd. (D)	Fry Road (D)
FM 1463 (D)	Fry Access Road (4-Way)
Fournace Place (D) @ IH-610	Mason Road (D)
Evergreen (D) @ IH-610	Mason access Road (4-Way)

Note: *D* stands for diamond intersection. If not specified, the intersections are along IH-10. The intersections in the shaded cell are those selected to replace the old ones that had connection problems. The last four from Fry Rd. to Mason Access Rd. were determined in the end of November 2018.

Meanwhile, among these 30 intersections, many are much more complicated (in terms of traffic control) than the one used for testing the control logic in Project 0-6920. Therefore, the original logic was largely revised to fit the properties of these intersections. Also, new control logics were proposed at some testbeds. Section 1.4 gives a brief introduction of these revisions, as well as the new control logics.

1.2. OVERVIEW OF PROACTIVE SIGNAL CONTROL

1.2.1 Literature Review

On arterial corridors, traffic streams are interrupted by traffic signals. Vehicles are forced to stop at signals on red, which increases their travel time, fuel consumption, and emission levels due to acceleration/deceleration maneuvers and idling required at the traffic signals. The fixed-time signal plan is widely used to operate traffic signals at intersections for its simple settings. It employs fixed green splits and cycle length which are based on historical traffic data. However, with the ever-growing demand of vehicles on the roads, fixed-time plan exacerbates traffic condition due to time-varying and fluctuating traffic demands. Regarding existing traffic control applications, the actuated signal control is another form of control of higher usability, which is able to actively cope the signal timing with the real-world demand-varying traffic volumes entering an intersection (Abbas et al., 2001; Chaudhary et al., 1993; Qi et al., 2013; Yin et al., 2007; TRB, 2015).

Operating transportation systems proactively with the advances of communications among vehicles, roadside infrastructures and traffic management centers is proved to be an effective approach on mitigating traffic congestions and improving network performance. Recently, optimization of traffic signal control incorporating advanced data collection methods has attracted intensive studies, such as cumulative time-responsive intersection control (Lee, 2010), vehicle-to-infrastructure assisted platoon-based signal control (He et al., 2012), a predictive microscopic simulation algorithm (Goodall, et al., 2013), and connected vehicle-enabled real-time adaptive signal control (Feng et al., 2015). In addition, many intelligent control algorithms, such as fuzzy control method (Khalid et al., 2004; Praneviˇcius and Kraujalis, 2012; Zhang et al., 2008), neural network control method (Srinivasan et al., 2006), reinforcement learning control method (Arel et al., 2010; Ma et al., 2002), etc., were introduced to optimize the performance of signals in the existing studies.

Though different types of signal optimizing techniques have been proposed, the practical application into the real-world is largely limited due to many reasons: the concern of shortage of personnel with required expertise and the concern for initial and maintenance cost (Lomax et al., 2013), unavailability of infield equipment, lack of communication and control devices, lower penetration rate of several advanced technologies, such as connected vehicle technologies (Hadi and Wallae, 1993), etc.

1.2.2 Description of Model

As mentioned above, the proactive signal control system aimed to design an optimal signal control system to minimize vehicle delay at signalized intersections with the help of *loop detectors* located in different place of an approach. Nowadays, various types of detectors are employed in traffic signal control systems, such as inductive loop detectors (see Figure 1.2.1), infrared sensors, video sensors (Basavaraju et al, 2014).

In this project, inductive loop detectors are the major reliable vehicle detecting tools at the testbeds for implementation. Video detectors were used at three *diamond intersections*: Mason, Fry and Baker Cypress along IH-10. These intersections were determined in 2018 to replace those with connection problems. These two types of detectors only report two parameters of traffic: volume and occupancy in each minute. Note that video detectors cannot detect the volume accurately—usually video detectors largely underestimate the volume, but they usually can detect the occupancy rate more accurately. Therefore, in general, the logic was designed based on loop detectors, and the discussion in the following is also based on loop detectors. As to the details of the logic based on video detectors, please refer to Chapter 6.

At a major approach of an intersection, loop detectors are usually placed at different location. Take Approach 1 of an intersection (see Figure 1.2.2) as an example. Two detectors are installed at the locations 'IN' and 'OUT' to detect traffic volumes entering and exiting the intersection through this approach, respectively. In the conventional actuated signal control, when a vehicle passes a detector associated with the controller, a vehicle 'call' is generated, and a phase gets started or extended. Therefore, typically, a 'call' represents the presence of one vehicle at a time. However, this new proposed proactive signal control system aims to utilizes loop detectors to detect the presence of upstream vehicle platoons moving toward an intersection. That is, the system looks at a platoon of vehicles, instead of individual vehicles. The only available information from detectors is traffic volumes and occupancies reported by loop detectors in every 1 minute. Simply based on the values of these two parameters, we cannot directly and precisely identify the arrivals of vehicle platoons at an approach. For this reason, it is necessary to find an approximate method.

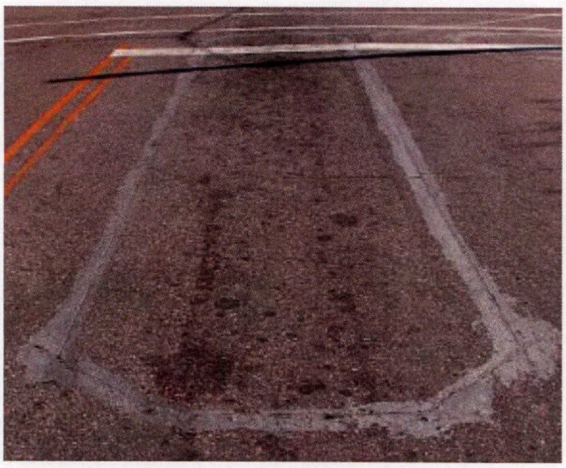


Figure 1.2.1 *An example of inductive loop detector*

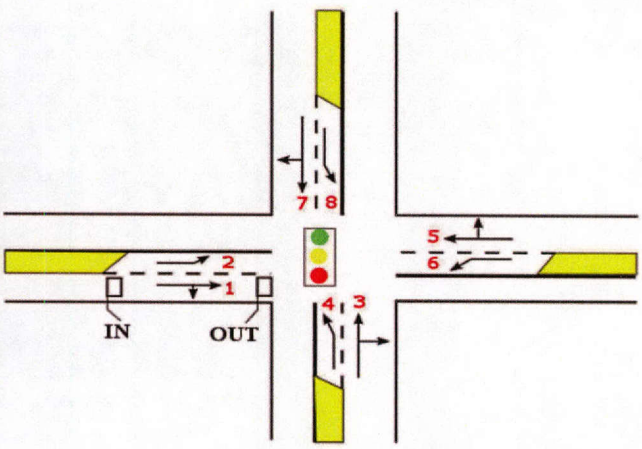


Figure 1.2.2 *An example of intersection with detectors in different locations*

For this purpose, the research team re-defined the traditional vehicle 'call' to identify vehicle platoons. Generally, a vehicle platoon is defined as a group of vehicles that can travel very closely together, safely at high speed. In that sense, the headways of vehicles inside one platoon are much smaller than those outside. Hence, once a platoon passes one loop detector, both occupancy and volume will be high. Therefore, it is critical to find *the proper values of volumes and occupancy, which can approximately reflect the arrivals of a platoon of vehicles at an approach.* Therefore, the information reported from upstream detectors (shown as "IN" detector in Figure 1.2.2) was focused.

Considering the intersection in Figure 1.2.2, it is assumed that the volume and occupancy collected by the 'IN' detector in phase n as $q_n(k)$ and $o_n(k)$ at time step k, $[t_k, t_{k+1})$, respectively, where $t_k = k\Delta t$ and Δt is the updating interval of the detector and also the green extension of each call. In the proposed system, a platoon is identified if $q_n(k) \geq q_n^c$ or $o_n(k) \geq o_n^c$, where q_n^c and o_n^c are the critical volume and occupancy for phase n.

Since the implementation is required, the design of the control algorithm has to be based on the available functions of the controller used in the testbed. For this reason, these available functions largely limit the design. Texas Department of Transportation (TxDOT) uses an Econolite Advanced System Controller, Series 3, 2100 Shelf Mount Models (ASC/3-2100) (see Figure 1.2.3) as the signal controller unit at the arterial intersection of the testbeds. It is placed in the NEMA traffic cabinets in the field. Therefore, all implementations to the arterial corridors monitored by TxDOT, are based on this controller. The available functions of the controller determine the way of logic development.

An ASC/3 Controller can work as a semi-actuated or fully-actuated traffic controller unit according to the National Electrical Manufacturers Association (NEMA) Standards Publication TS2-2003 (ECONOLITE, 2018). It can operate as a 16-phase controller with any combination of 16 vehicle phases, 16 pedestrian phases, 16 timed overlaps, and four timing rings (ASC, 2018). It can operate the actuated signal control system with the existing roadside facilities, including loop detectors, signals, and roadside control box.

Based on the available functions of ASC/3 controller, the research team developed the logic, as shown in Figure 1.2.4, based on volume and occupancy rates reported by upstream detectors (i.e., "IN" detectors shown in Figure 1.2.2). Please see the details in the Final Report of Project 0-6920 (Wu et al., 2016).

Note that the logic shown in Figure 1.2.4, is the basic logic applied to a single intersection with very small traffic from the minor road. In this project (5-6920), many intersections have much more complicated settings, such as the signal coordination, signal control for the traffic at diamond intersections, etc. More logic statements will be discussed in Chapters 4-6.

Figure 1.2.3 *Econolite ASC/3 2100 Controller*

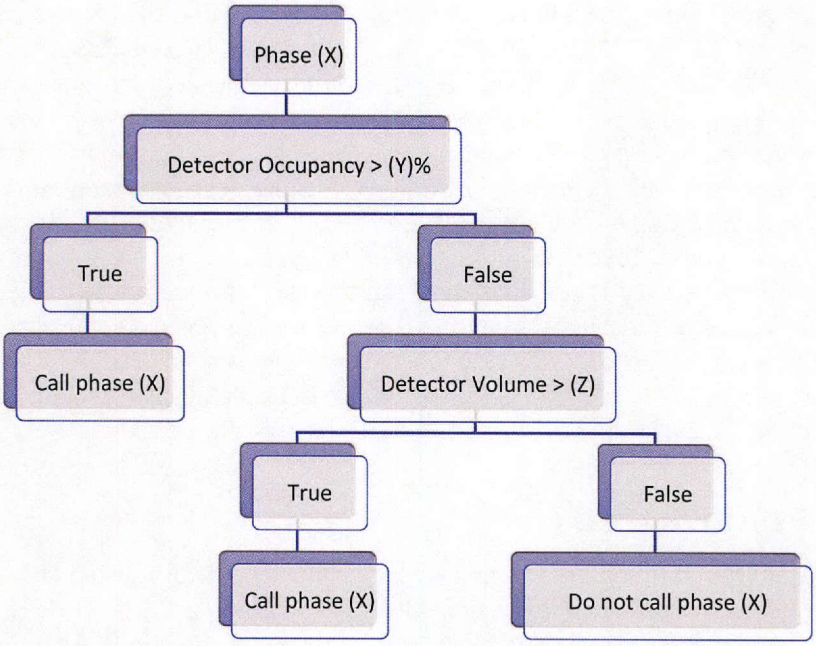


Figure 1.2.4 *Flow chart of the proposed logic for platoon-based actuated signal control (Wu et al., 2016)*

In an ASC/3 Controller, the detector can be set up with different functionalities (logic statement based on volume and occupancy) to re-define the 'call' for vehicle platoons. And then, each 'call' represents the arrival of one platoon to the intersection, not merely the presence of a single vehicle. Moreover, the updating interval for detectors can also be set through an ASC/3 controller to accommodate the change of traffic condition of the intersection and thus to make a more accurate estimation of volume and occupancy, respectively. In addition to replace the vehicle 'call' with the platoon 'call', the conventional actuated signal control system is also revised to accommodate these calls. In the proposed proactive signal control system, the minimum green time is set as its corresponding green extension, so that once there is no call in the phase during the green extension, the green light will be terminated and switched to the next phase. This is called GAP OUT. On the other hand, the maximum green time remains unchanged: when the green time reaches its maximum, the phase is over, and switched to the next. This is called MAX OUT. Note that in the proactive signal control system, the green extension is set large enough to make sure that all the vehicles detected during the updating interval can be released. This feature is particularly used in the implementation to the testbeds of SH 242 (see Chapter 4) and FM 1464 (see Chapter 5) where the signal coordination was considered.

1.3. TRAFFIC SIMULATION

In this project, the VISSIM microscopic simulation package was employed to simulate the traffic of the testbeds under the existing and proposed signal control systems, respectively. First, by simulating the traffic under the original control system at each intersection, the research team aims to check the accuracy of the simulation. Then, by running the simulation under the proposed signal control system, the research team could evaluate the performance of the proposed system, so as to adjust the parameters to find the best parameters for each intersection. Especially, note that the version of VISSIM used in this project, has a plug-in package of ASC/3 controller (which is used at all testbeds, as mentioned above), making it possible to simulate the traffic managed by this controller.

VISSIM was first developed at 1992 in Karlsruhe, Germany, and is being continuously developed. It is a multi-modal microscopic simulation. Multi-modal means that it can simulate vehicles of different categories: vehicles (cars, buses, and trucks), public transport (trams, buses), cycles (bicycles, motorcycles), pedestrians, and rickshaws.

A VISSIM simulation system consists of two basic elements: traffic flow model and signal control model (Fellendorf, 1994). VISSIM's traffic flow model is a discrete, stochastic, and time-step-oriented microscopic model (Ratrout and Rahman, 2009). This model considers the input of driver-vehicle-units individually and longitudinal movement of vehicles is based on psycho-physical car flowing model. The lateral movement generally follows the rule-based algorithm. The psycho-physical car flowing model is derived from the research of Wiedemann, known as Wiedemann car following model (Fritzsche and Ag, 1994; Leutzbach and Wiedemann, 1986; Wiedemann, 1974). This model is based on perceptual threshold, which is a function of individual vehicle's speed difference and spacing, i.e., a faster vehicle, approaching to a slower one, will deaccelerate at a point when it (faster vehicle) comes close to the slower vehicle based on the spacing and speed difference between them, while for the reverse condition (opposite threshold), acceleration will take place. This car following model also considers lane changes (lateral movement). The other major aspect of VISSIM simulation is signal control. In brief, VISSIM propagates the detector values to the signal program in each second to optimize the current signal aspects.

1.4. TASKS PERFORMED

The research team from the Performing Agency conducted the following tasks. The first three tasks were preparations for the Task 4 field implementations.

1.4.1 Project Preparation.

On April 21, 2017, the research team had kick-off meeting with Project Manager Darrin Jensen and two engineers from the Houston District Office of TxDOT: Steve Chiu and Roy Gonzales. In the meeting, the workplan was discussed, and after the meeting, the research team went through 22 intersections at FM 528, SH 242, and FM 1464 to collect the testbed geometric information, including the loop detector locations, the number of lanes, the speed limits, etc.

After Hurricane Harvey, the progress of the project was heavily delayed due to the damage of infrastructure. In Spring 2018, the research team revised the work plan, as well as the budget, to extend the project to the end of August 2018. However, due to the delay of the installations of loop detectors at eight diamond intersections, the research team revised the work plan and budget again in August 2018, and the project was extended to the end of December 2018.

The detector problems still occurred in late 2018. Finally four intersections were replaced with new ones at the end of November 2018, and the last data set was sent to the research team on January 24, 2019.

1.4.2 Data Collection

From April to June 2017, the research team collected the data of 22 intersections at the testbeds of FM 528, SH 242, and FM 1464. The data includes the geometric information of each intersection, the existing signal plan used at each intersection, and the traffic data (collected by loop detectors) at each intersection. Note that since the diamond intersections have very different properties from these 22 intersections, the analysis of these diamond intersections was left to the end of the project (see Chapter 6).

The data analysis of 22 intersections were conducted, and the results were summarized into TM 2.1 Data Analysis Report, submitted in the end of June 2017.

In this report, Chapter 2 reflects this task. Also, it also covers the geometric information analysis and existing signal plan review of the last eight intersections, including those selected in the end of November 2018 to replace the ones with detector connection problems.

1.4.3 Testbed Modeling and Simulation

Based on the data collected at 22 intersections belonging to the three testbeds: FM 528, SH 242 and FM 1464, respectively, the research team built the models of three testbeds in VISSIM, respectively, during the summer of 2017. For each testbed, by running the traffic simulation under the existing signal plans (provided by Steve Chiu from the Houston District Office) in VISSIM, the research team simulated the traffic flow and compared the simulated results with the observed results. The comparison was used to verify the accuracy of modeling and simulation settings. The results were summarized into TM 3.1 Network Performance Report, submitted in the end of July 2017.

Note that after Hurricane Harvey, the infrastructure at some testbeds has been renewed and updated. For example, the loop detectors were installed to all 11 intersections at FM 1464 gradually in Fall 2017 and Spring 2018. Therefore, the simulation was reconducted at SH 242, and FM 1464, respectively, based on the updated detector information. In this report, the simulation section at each testbed is placed in the chapter on the implementation to each testbed.

1.4.4 Field Implementation

This is the major task of this project. Since 30 selected intersections are located in four testbeds, the whole implementation was divided into three phases. Phase I is for the implementation on the three intersections at FM 528 only; Phase II covers the eight intersections at SH 242 and 11 intersections at FM 1464, respectively; and Phase III covers the eight intersections along IH-10. The following details the implementation at each testbed.

1.4.4.1 FM 528

The testbed of FM 528 is located in the southeast suburb of Houston (see Figure 1.1.1). FM 528 is the major road.

This testbed has three intersections at (1) Desota St., (2) Friendswood Blvd., and (3) Briar Creek Dr./Falcon Ridge Blvd. The data reported by loop detectors show that the average occupancy rates (for each 15-minute interval) for through traffic on the major road are quite low (< 0.2 during peak hours), but the average traffic flow rates are high (> 1000 veh/hr/ln even in the midday period), implying few or no congestions in most directions. Compared with the data reported from other testbeds, it implies that the through traffic on the major road might not be interrupted in most time and the headways between vehicles were large. Therefore, vehicles moved in more independently and randomly, entailing that no apparent platoons were formed on the major road.

According to the logic shown in Figure 1.2.4, if the occupancy is small, then the logic largely relies on volumes. The implementation was conducted to this testbed on August 3, 2017. After that, the research team traveled to this testbed several times to observe the traffic flow patterns when the implemented proactive signal control system was switched on or off, respectively. The data reported by loop detectors were also analyzed to evaluate the performance of the implemented system to this testbed.

Originally, the signal control system at three intersections were pre-timed coordinated signal control system. When conducting the onsite implementation, it was found that the coordination between adjacent intersections is critical to help vehicles smoothly pass through these adjacent intersections in peak hours in a weekday. Therefore, the proposed proactive system is only active in midday (9:30 am–2:00 pm) in a weekday and almost all day (7 am–11 pm) in a weekend day. Realizing the importance of the signal coordination, the research team developed a new logic to balance the coordinated signal control (for the through traffic on the major road) and proactive signal control (for the left-turn and minor traffic), when conducting the implementations to the testbed of SH 242.

The implementation results were summarized to TM 4.3 Phase-I Field Experiment Analysis Report, submitted in September 2017. Chapter 3 summarizes the implementation of the proactive signal control system to this testbed. Note that after the implementations to other testbeds, the research team got better understanding of the proposed proactive signal control system, so the summary of the field implementation to this testbed were significantly rewritten. Therefore, the contents of Chapter 3 now are quite different from those in TM 4.3 Report.

1.4.4.2 SH 242

The testbed of SH 242 is located at the far north suburb near the Woodlands and Conroe. SH 242 is the major road.

This testbed has eight intersections at (1) Green Bridge Dr., (2) Alden Woods, (3) Gosling Rd, (4) Windsor Hills Dr./Fellowship Dr., (5) W. Campus Dr./Honor Roll Dr., (6) Achievement Dr., (7) Maverick Dr., and (8) St. Lukes Way, respectively. This testbed experiences high traffic flows (>800 veh/hr/ln, and even >1000 veh/hr/ln at some intersections in peak hours) and high occupancy (>0.5 in daytime). More importantly, for many intersections, the spacings between two adjacent intersections are small, making the signal coordination critical to ensure that the through traffic flow can go through the intersections along the major road (SH 242) smoothly. However, on the other hand, at some intersections, especially the intersections (1-3), the left-turn movements from the major to the minor road and those from the minor to the major road is large. The coordination mode cannot well handle these left-turn and minor traffic, causing serious congestion and delay.

Such properties made the implementation more complicated and more difficult than the work on the testbed of FM 528. At this testbed, for the intersections (4-8), since they are spaced very closely, the signal coordination is necessary all the time, and thus a proactive control logic was just imbedded into the original signal plan at each intersection. On the other hand, for the intersections (1-3) which spacings are not as close as to those of five intersections (4-8) (especially, the intersection at Green Bridge Dr. is far away from others), a new control logic was proposed and implemented, which can adaptively switch the signal control between the coordination mode (or the free mode at the intersection of Green Bridge Dr.) and the new proactive control mode, according to the conditions of left-turn and minor traffic flows.

The implementation at this testbed was firstly conducted in October 2017. Due to the more complex situation, the model was revised several times, and it was not finalized until the end of November 2017. Through the field observation and data analysis, it was found that the congestions caused by the aforementioned left-turn and minor traffic flows were well solved after the implementation of the proposed logics.

The original report on this intersection was submitted in December 2017. However, since more traffic data at the intersections (1-3) were reported in Spring 2018, an updated report TM 4.5 Phase-II Field Experiment Analysis Report (Testbed of SH 242) was submitted in May 2018. In this report, Chapter 4 summarizes the implementation applied to this testbed.

1.4.4.3 FM 1464

The testbed of FM 1464 is located in the southwest suburb of Houston. The major road is FM 1464 between Westpark Tollway and TX-99 (Westloop).

This testbed is longest among all four testbeds, covering 11 intersections at (1) W. Oaks Village Dr. (2) Bellaire Blvd., (3) Highland Oak Ln, (4) Orchid Ridge Ln., (5) Beechnut St., (6) Bissonnet St., (7) W. Bellfort Blvd., (8) W. Airport Blvd., (9) Stephen F Austin High School, (10) Old Richmond Rd., and (11) Orchard Lake Estates Dr., respectively. They can be divided into two parts. Part I covers the first five intersections (1-5), which are closely spaced, so the coordination had to be considered; and the other six intersections (6-11) were in the free mode (not coordinated), so these six were put into Part II intersections.

For the six intersections in Part II, originally, the signal plans at these intersections are not coordinated, so the proactive signal plan (similar to the one shown in Figure 1.2.4) was directly implemented to these intersections, and the parameters for the proactive plan were designed based on the traffic condition at each intersection (verified via the simulations in VISSIM). However, for Part II intersections, except the intersection at Bissonnet St. (where the peak hour traffic flows are about 600 to 800 veh/hr/ln), the flow rates at others were not higher than 350 veh/hr/ln, implying vehicles may not move in platoons. As a result, similar to the case of FM 528, the performance of the implemented logic is not significant for the through traffic on the major road, except at the intersection of Bissonnet St., where performance was found good .

On the other hand, for the intersections in Part I, where the spacings between two adjacent intersections are close, the signal coordination is important. Also, the flow rates at these intersections were found higher (over 650 veh/hr/ln). For this reason, similar to the three intersections at SH 242, a new control logic was proposed and implemented to make the control plan adaptively switch between the coordination and proactive modes. For these five intersections, the major concern is the through traffic flow on the major road (FM 1464), so different from that implemented to the three intersections at SH 242 (where the logic is based on the left-turn and minor traffic flow), this new logic here is based on the conditions of the through traffic flow on the major road (FM 1464).

Due to the delay of loop detector installation at this testbed (they were not ready until April 2018), the implementations were conducted in April and May 2018, except the work at the intersection of Bissonnet St., which was conducted in February 2018. The findings were summarized into TM 4.5 Phase-II Field Experiment Analysis Report (Testbed of FM 1464), submitted in the end of May 2018. However, since the last implementation to the five intersections of Part I was conducted on May 11, 2018, some data were collected to the end of May. The Report of TM 4.5 Phase II Field Experiment does not reflect the results from these data. Actually, these data show that the implemented logic performs well. The final analysis on this testbed was not completed until the end of July. In this report, Chapter 5 summarizes the analysis for all intersections, including those based on the latest collected data not reflected in TM 4.5.

1.4.4.4 Frontage Road of IH-10

The last testbed is on the frontage road of IH-10 at Energy Corridor of Houston. Originally, these eight intersections were all diamond intersections along the frontage road of IH-10 or IH-610, i.e., an intersection that combines two intersections at two frontage roads on both sides of a freeway. As mentioned in Section 1.1, only two intersections in the original list were used for implementation (Greenhouse Rd. and Westgreen Blvd. at IH-10). The other six were all replaced with new ones (the last four were finalized in the end of November 2018), as shown in Table 1.1.2.

A diamond intersection is more complicated than an ordinary four-way or three-way intersection, because it combines two intersections together, and there exists some overlapping phases. In this case, the proactive signal control logic was redesigned to fit the properties of a diamond intersection. Also, note that since at three diamond intersections at Mason Rd., Fry Rd. and Baker Cypress Rd., only video detectors are available, which significantly underestimate the traffic volumes, the logic plan implemented to these three is different, only dependent on the occupancy rate. On the other hand, there are two four-way access intersections just around 500 ft south of the diamond intersections at Mason Rd. and Fry Rd., respectively, so the logic applied to these two are also different. Therefore, three types of logics were developed for these eight intersections. Please refer to Chapter 6 for the details.

The implementations to these eight intersections were conducted in November and December 2018. The implementations in November only covers three intersections at Greenhouse Rd., Westgreen Blvd. and Katy Fort Bend, so the Report of TM 4.6a Phase-III Field Experiment Analysis, submitted in December 1, 2018, only discusses the work at these three intersections and the results on two intersections at Greenhouse Rd. and Westgreen Blvd. (the data from the intersection at Katy Fort Bend was not available until the beginning of December).

The last implementation was conducted on December 17, 2018, to the last five intersections at Mason Rd., Fry Rd., Mason Access Rd., Fry Access Rd., and Baker Cypress Rd. This implementation was based on the site work on December 11, 2018. After the implementation, there were still some errors in data reporting from two intersections at Fry Rd. and Baker Cypress Rd. The data of these two intersections were finally collected in January 2019 (the last data package was received on January 24, 2019). For this reason, TM 4.6a is not complete. In this report, Chapter 6 presents a whole analysis of these eight intersections.

The data analysis shows that the proposed signal control systems work well when the flow rate exceeds 600 veh/hr/ln, because vehicle platoons become common at this level of flow.

1.5. ORGANIZATION OF REPORT

The remaining of this report is organized as follows. Chapter 2 summarizes the geometric properties and existing signal plans of the 30 intersections selected for implementation. Chapter 3 summarizes the implementation of the proposed proactive signal control system to the three intersections on the testbed of FM 528. Chapter 4 summarizes the implementation to the eight intersections on the testbed of SH 242. Chapter 5 shows the implementation to the 11 intersections on the testbed of FM 1464. Chapter 6 examines the implementation to eight intersections along IH-10 (six diamond intersections plus two four-way intersections), and Chapter 7 concludes this report.

As mentioned in Section 1.3, Chapters 5 and 6 include the analysis of data collected **after submitting the field implementation reports**. Therefore, the contents of these two chapters reflect the latest updates of the implementation efforts and the performance evaluations of the implemented proactive signal control systems to the testbeds of FM 1464 and intersections along IH-10. Also, the summary of the implementation to the testbed of FM 528 were also rewritten in Chapter 3. Therefore, the contents in these three chapters, especially Chapter 6, are different from the field experiment reports on these two testbeds, which were submitted in July 31, 2017; May 30, 2018; and December 1, 2018, respectively.

CHAPTER 2

GEOMETRIC INFORMATION AND EXISTING SIGNAL PLANS OF TESTBEDS

This chapter reviews the geometric properties and existing signal plan applied to the 30 intersections selected for implementation in the Houston Metropolitan Area: FM 528, SH 242, FM 1464 and other eight intersections along IH-10. Such information was used for the tasks of testbed modeling and traffic simulation in VISSIM.

Note that before the implementation, the proposed proactive signal control system was firstly tested in traffic simulation platform (VISSIM) in order to initially evaluate the performance of the proposed control system. On the other hand, it is necessary to check if VISSIM is able to simulate the traffic patterns at selected intersections. Therefore, the existing signal plans were imported into the simulation models built in VISSIM, and the simulation accuracy can be assessed by comparing the simulated results with the observed results. Also, the simulation results under the existing signal plans were employed as the baseline to evaluate the impact of simulated

2.1. ROAD GEOMETRIC PROPERTIES

The geometric properties of testbeds are necessary for building the testbed models in VISSIM for the purpose of simulation. Also, this information is also helpful for the research team to understand the traffic patterns in these testbeds. The geometric properties include the description of major and minor roads, and testbed modeling information, such as the length of road cut for each approach, the number of lanes, the purpose of lanes (such as left-turn, right-turn, etc.), and the design speed. This information was collected through Google Earth, as well as the onsite inspections. In this section, the geometric properties of all 30 intersections listed in the work plan is reviewed based on their locations on the testbeds.

2.1.1 Testbed of FM 528

2.1.1.1 Overview of Testbed

Three consecutive signalized intersections were selected in the testbed (FM 528). From west to east, they are Desota St. (A), Friendswood Lake Blvd. (B), and Falcon Ridge Blvd./Brian Creek Dr. (C) respectively, as shown in Figure 2.1.1. Although the testbed is extended from southwest to northeast, FM528 is simply regarded as "west-east" arterial road in this report for convenience.

Figure 2.1.1 *Testbed of FM528 with selected three intersections*

FM 528 is the major road for our study and the previously mentioned three roads are minor. In the testbed, the major road segment starts from 185 meters west of the center of the intersection at Desota St., and ends in 175 meters east of the intersection at Falcon ridge Blvd. Such two extra lengths extended respectively from east and west intersections along FM 528 were used to guarantee that vehicles queues generated ahead of the signals can be correctly captured and modeled in the traffic simulation models. The total length of selected testbed is 2070 meters. From west to east, the center to center distances between two consecutive intersections are 725 meters and 985 meters respectively. For the minor roads, a length of at most 100 meters was considered (calculated from the center of each intersection, extended to north and south, respectively). The length information of road segments, as well as the speed limits of the minor road segments associated with the three intersections, were summarized in Table 2.1.1.

All along the segment of FM 528, the lane width is 12 feet. For either westbound or eastbound traffic, there are two lanes and the speed limit is 45 miles per hour (mph). However, the speed limit is reduced in the segment between Desota St. and Friendswood Lake. Near each intersection, there is a central left turn lane found along FM 528, which provides an exclusive left-turn lane for both directions. On the other hand, near each intersection, the right-most lane of both directions accommodates both through and right-turn movements. This pattern of lane allocation is identical at the selected three consecutive intersections. No dedicated right turn lane is provided. Table 2.1.2 summarizes the lane features.

Table 2.1.1 Length information of the testbed of FM 528 (from west to east)

Direction	Intersection	Intersection Name	Speed Limit(mile/hour)	Center to Center Distance (meter) *	Cumulative Distance from Start point (meter)
West to East	Start				0
	A	Desota St.	20 (N)	185†	188
	B	Friendswood Lake	30(N)	725	913
	C	Falcon Ridge Blvd. Brian Creek Dr.	30(N) 30(S)	985	1911
	End			175	**2070**

Note: * Column 5 represents distance from previous column; † distance from the start point to the center of Intersection A.

Table 2.1.2 Major road segment properties of the testbed of FM 528

Section		Speed Limit (mile/hour)	Lane Width (feet)	Number of Lane
Eastbound	Start to Desota St.	45 School Time: 35	12	3 lanes with 1 central left turn lane, 1 through and one through-right turn lane
	Desota St. to Friendswood St.	45 School Time: 35	12	3 lanes with 1 central left turn lane, 1 through and one through-right turn lane
	Friendswood St. to Falcon Ridge Blvd.	45	12	3 lanes with 1 central left turn lane, 1 through and one through-right turn lane
	Falcon Ridge Blvd. to end	45	12	2 through lane
Westbound	End to Falcon Ridge Blvd.	45	12	3 lanes with 1 central left turn lane and 1 through and 1 through-right turn lane
	Falcon Ridge Blvd. to Friendswood St.	45	12	3 lanes with 1 central left turn lane and 2through lane
	Friendswood St. to Desota St.	45 School Time: 35	12	3 lanes with 1 central left turn lane and 2 through lane
	Desota St. to start	45 School Time: 35	12	2 lanes 1 through lane and 1 through-right lane

2.1.1.2 Minor Roads

Three consecutive minor roads are connected with the major road FM 528 at three signalized intersections: Desota St. at Intersection A, Friendswood Lake Blvd. at Intersection B, and Falcon Ridge Blvd./Brian Creek Dr. at Intersection C (from west to east along FM 528), as shown in Figure 2.1.1. The geometry and properties of these three minor roads are described in the following, respectively.

Desota St. (Intersection A)

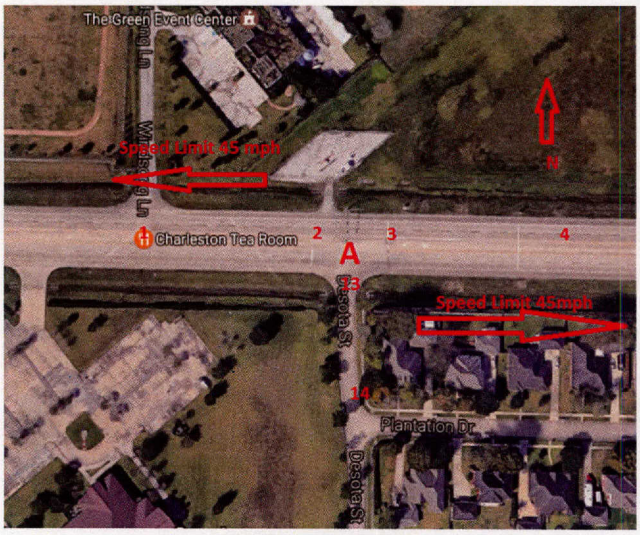
Desota St. is a T intersection that runs in a South direction and meets FM528, which is the first intersection on the testbed. The speed limit of traffic is 20 mph towards the intersection. In this project, a section of this road with 65 meters long from the intersection was considered into the testbed. It is a two-lane street with one lane used for vehicle moving towards the intersection and the other lane for vehicle moving from the intersection. The details of the intersections are shown in Figure 2.1.2 (a).

Friendswood Lake Blvd. (Intersection B)

Friendswood Lake Blvd. is the second minor road of this testbed. It meets FM 528 from the south side of FM 528. The speed limit is 30 mph towards the intersection. It has a raised median to divide the conflicting traffic. A section of this road with 80 meters long from the intersection was considered into the testbed. On the south side of the intersection, the northbound traffic towards the intersection has three lanes: right-turn, left-turn and through/left-turn. On the other hand, the southbound traffic from the intersection has two lanes. On the north side, there is small 24 feet gravel road with low traffic. The details of the intersection are shown in Figure 2.1.2 (b).

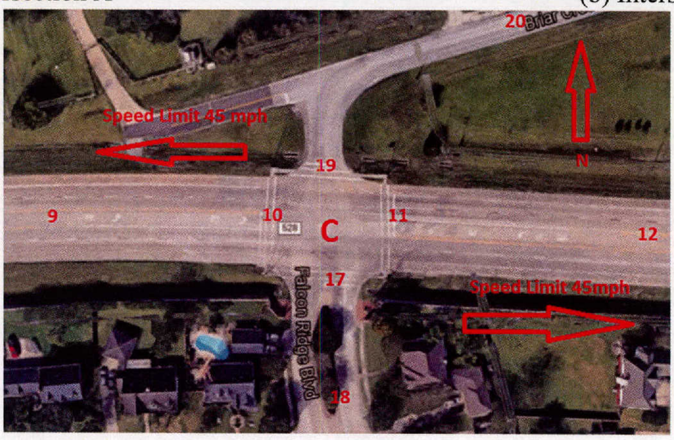
Falcon Ridge Blvd./Briar Creek Dr. (Intersection C)

As to the third minor road, Falcon Ridge Blvd. is at southern direction; and at the other side of the intersection (i.e., north) there is two-lane road, named Brian Creek Dr. with speed limit 20 mph towards the intersection. Falcon has one right lane and one left-through lane towards the intersection. It also has two lanes for the southbound traffic from the intersection. It has a raised median to divide the conflicting traffic. On the other hand, Brian Creek Dr. has two lanes: one lane for incoming traffic from the intersection and another one for the traffic towards the intersection. The details of the intersections are shown in Figure 2.1.2 (c).



(a) Intersection A

(b) Intersection B



(c) Intersection C

Figure 2.1.2 *Detailed map of Intersections A, B, and C at FM 528 with node numbering*

2.1.1.3 Network Modeling of Testbed

Once the geometric information is collected, the testbed model will be built in VISSIM. In the following, we will describe how this testbed is modeled. Left-turn movement is one of the most important factors for highway traffic analysis. It is also important to identify the location and length of left-turn lane to code a transportation network into the simulation software. Therefore, the research team conducted a comprehensive observation to divide the link into small segments. It provides the relative distance of different segments of the links. The segmented points were defined by node numbers (see Figure 2.1.2) and relative distance (see Table 2.1.3).

Table 2.1.3 summarizes the comprehensive geometric analysis of the testbed. Column 1 indicates the node number (the fragmented portion), corresponding to the segments shown in Figure 2.1.2. Column 2 lists the distance between two nodes mentioned in Column 1. For example, the distance is 168 meters between Nodes 1 and 2, where Node 1 presents the starting point of the testbed near the first intersection. The third and fourth columns indicate the west- and eastbound traffic direction, as well as the number of different types of lanes (such as left-turn lane, through lane, etc.). The fifth column indicates the speed limit. Similar geometric analysis for all three minor road segments is summarized in Table 2.1.4.

Table 2.1.3 Segment analysis of the major road (FM 528) from west to east

Node Number	Distance(meter)	Type/Number of Lane		Speed Limit (mph)
		West	East	
(Start) 1-2	170	2 T	1T,1T-R	45
2-3 (A)	30	-	-	
3-4	80	2 T,1**L**	2T	45
4-5	510	2T	2T,1**L**	45
5-6	95	2T	1**L**,1T,1T-R	45
6-7 (B)	50		-	
7-8	95	2 T,1**L**	2 T	45
8-9	745	2 T	2 T,1**L**	45
9-10	100	2 T	1**L**, 1T,1T-R	45
10-11 (C)	40	-	-	
11-12 (End)	155	1**L**,1T,1T-R	2T	45
Sum	**2070**			

Note: T: through movement, L: left-turn movement, T-R: through and right-turn movement, T-R: through and left-turn movement. For example, 3 T means there are three lanes for through movements. "L" bolded as the left-turn movement, as it is critical in modeling.

Table 2.1.4 Segment analysis of the minor roads on the testbed of FM 528

Intersection	Node Number	Distance (meter)	Type/Number of Lane		Speed Limit (mph)
			South	North	
A: Desota St.	13-14	70	1L	1R	20
B: Friendswood Lake Blvd.	15-16	100	2T	1L,1R,1T-L	30
C: Falcon Ridge Blvd.	17-18	70	2T	1T-L, 1R	20
Briar Creek Dr.	19-20	100	1T	1T	30

2.1.2 Testbed of SH 242

2.1.2.1 Overview of Testbed

Eight consecutive signalized intersections were selected in the testbed of SH 242, located at the north suburb of Houston. The intersections are Alden Woods (A), Gosling Rd. (B), Fellowship Dr. (C), Honor Roll Dr. (D), Achievement Dr. (E), Montgomery Dr. (F), Windsor Lakes Blvd. (G) and Greenbridge Dr. (H), respectively, as shown in Figure 2.1.3 below. SH 242 is regarded as a "west-east" arterial road in this report for convenience.

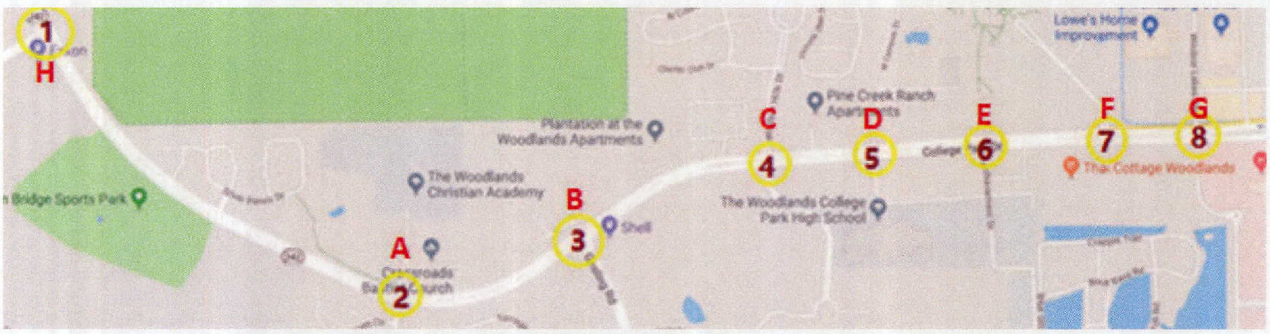


***Figure 2.1.3** Testbed of SH 242 with selected eight intersections*

Note that originally the intersection of SH 242 and North Fwy. (east of Intersection G) was listed in the work plan. However, since it is a diamond intersection, having different properties from other intersections. As discussed in the Kick-off Meeting with Project Manager and TxDOT Houston engineers, this one was excluded, and was replaced by the intersection at Green Bridge Dr. (H), as shown in Figure 2.1.3. Note that since it is relatively far away (1 mile) from the other intersections, the intersection at Green Bridge Dr. is treated as a separate testbed independently from others. Its properties are summarized in Table 2.1.5. At this intersection, SH 242 turns to be in the north-south direction, and Green Bridge Dr. runs in the east-west direction (see Figure 2.1.3).

Table 2.1.5 Properties of the intersection at Green Bridge Dr. (Intersection H)

Direction	**Speed Limit (MPH)**	**Lane Width(feet)**	**Number of Lane**
Northbound	45	12	2 lanes with 1 Left Turn Lane
Southbound	45	12	2 lanes with 1 Left Turn Lane

SH 242 is the major road for the study and the aforementioned eight north-south roads are minor. In the testbed, the major road segment starts from 150 meters west of the center of the intersection at Alden Woods, and ends in 150 meters east of the center of the intersection at Windsor Lakes Blvd. Such two extra lengths extended respectively from east- and west-most intersections along SH 242 were used to guarantee that vehicle queues generated ahead of the signals can be correctly captured and modeled in the traffic simulation models. The total length of the major road in the selected testbed is 3,040 meters (excluding the intersection of Green Bridge Dr.). For the minor roads, a length of at most 100 meters was considered (calculated from the center of each intersection, extended to north and south, respectively). The length of road segments, as well as the speed limits of the minor road segments, were summarized in Table 2.1.6. Note that an independent testbed will be built for the intersection at Green Bridge Dr.

All along the segment of SH 242, the lane width is 12 feet. For either westbound or eastbound traffic, there are two to four lanes and the speed limit ranges from 35 mph (school zone) to 50 mph. Near each intersection, there is a central left turn lane which provides an exclusive left-turn lane for both directions. On the other hand, near each intersection, the right-most lane of both directions accommodates right-turn movements. This pattern of lane allocation is identical at the selected eight consecutive intersections. Table 2.1.7 summarizes the aforementioned lane features. As mentioned above, since it is far away from other eight intersections, Intersection I will be treated independently (see Table 2.1.5 for its geometric properties).

Table 2.1.6 Length information of the testbed of SH 242 (from west to east)

Intersection	**Intersection Name**	**Speed Limit(mile/hour)**	**Center to Center Distance (meter) ***	**Cumulative Distance from Start point (meter)**
Start				0
A	Alden Woods	N/A (N), 25 (S)	150†	150
B	Gosling Road	N/A (N), 45 (S)	645	795
C	Windsor Hills Dr.	25 (N), 30 (S)	685	1480
D	Honor Roll Dr.	10 (N), 10 (S)	355	1835
E	Achievement Dr.	10 (N), 10 (S)	355	2190
F	Montgomery College Dr.	10 (N), 10 (S)	400	2590
G	Windsor Lakes Blvd.	25 (N), 30 (S)	300	2890
End			150	**3040**
Start				0
H‡	Green Bridge Dr.	45 (N), N/A (S)	185	185
End			185	**370**

Note: * Column 4 represents distance from previous column; † it is from the start point. ‡: Intersection H (at Green Bridge Dr.) is listed in separate row as it will be treated independently.

Table 2.1.7 Major road segment properties of the testbed of SH242

	Sections	Speed Limit (MPH)	Lane Width (feet)	Number of Lane
Eastbound	Start to Alden Wood	45, 35 (School Time)	12	2 lanes and 1 left turn lane and 1 right turn lane
	Alden Wood to Gosling Rd.	45, 35 (School Time)	12	2 lanes and 1 left turn lane
	Gosling Road to Windsor Hills Dr.	45, 35 (School Time)	12	2 lanes and 1 left turn lane and 1 right turn lane
	Windsor Hills Dr. to Honor Roll Dr.	45, 35 (School Time)	12	2 lanes and 1 left turn lane and 1 right turn lane
	Honor Roll Dr. to Achievement Dr.	45, 35 (School Time)	12	3 lanes and 1 left turn lane
	Achievement Dr. to Montgomery College Dr.	45, 35 (School Time)	12	3 lanes and 1 left turn lane
	Montgomery College Dr. to Windsor Lakes Blvd.	45, 35 (School Time)	12	3 lanes and 1 left turn lane
	Windsor Lakes Blvd. to End.	45	12	3 lanes and 1 left turn lane
Westbound	End to Windsor Lakes Blvd	45	12	3 lanes and 1 left turn lane
	Windsor Lakes Blvd to Montgomery College Dr.	45, 35 (School Time)	12	3 lanes and 1 left turn lane & ramp of 1 lane with 1 right turn lane
	Montgomery College Dr. to Achievement Dr.	45, 35 (School Time)	12	3 lanes and 1 left turn lane and 1 right turn lane
	Achievement Drive to Honor Roll Dr.	45, 35 (School Time)	12	3 lanes and 1 left turn lane
	Honor Roll Drive to Windsor Hills Dr.	45, 35 (School Time)	12	2 lanes and 1 left turn lane and 1 right turn lane
	Windsor Hills Drive to Gosling Rd.	45, 35 (School Time)	12	2 lanes and 1 left turn lane and 1 right turn lane
	Gosling Rd. to Alden Wood	50, 35 (School Time)	12	2 lanes and 1 left turn lane and 1 right turn lane
	Alden Wood to Start	50, 35 (School Time)	12	2 lanes

2.1.2.2 Minor Roads

Eight minor roads (including Green Bridge Dr.) are connected with the major road SH 242 at eight consecutive intersections. The geometry properties of these eight minor roads are described below, respectively.

Alden Woods (Intersection A)

Alden Woods runs in the north-south direction and meets SH 242 at Intersection A. There is a school on the north of the intersection, therefore, for northbound traffic, school sets speed limit. For southbound traffic, speed limit is 25 mph. The road is two-lane road in both ways where each lane is 12 feet wide. The road has a median strip to divide the traffic coming from opposing directions. For northbound traffic towards SH 242, the rightmost lane serves for both through and right-turn movements, while for southbound traffic towards SH 242, the rightmost lane serves for right-turn movement only. The details are shown in Figure 2.1.4 (a).

Gosling Road (Intersection B)

Gosling Rd. is a minor road that meets SH 242 at the second signalized intersection in our testbed. In the north side, there is a T-end which leads to apartments. Therefore, we only consider the southbound traffic speed limit. For southbound traffic, the speed limit is 45 mph. It has median strips to divide the conflicting traffic. This road has also two lanes in both directions, each of which is also 12 feet wide. For both north- and southbound traffic towards the intersection, the rightmost lane serves for right-turn movement only. The details are shown in Figure 2.1.4 (b).

Windsor Hills Dr. / Fellowship Dr. (Intersection C)

Following Gosling Rd., the next signalized intersection is at Windsor Hills Dr. and Fellowship Dr. running in the north-south direction. The lane width is 12 feet. Speed limit for north- and southbound traffic is 25 and 30 mph, respectively. There are two lanes for north- and southbound traffic, respectively. For both directions, the right-most lane serves for both right-turn and through movements. Median strips divide the traffic flowing in opposite directions. The details are shown in Figure 2.1.4 (c).

W. Campus Dr. / Honor Roll Dr. (Intersection D)

The next signalized intersection is at W. Campus Dr. and Honor Roll Dr., also running in the north-south direction. Both minor roads lead to a school, thus speed limit for both minor roads is 10 mph. The road towards the intersection from the south has two lanes, with rightmost lane serving right-turn movement only. After intersection, there is only one lane for northbound traffic. The road towards the intersection from the north has three lanes with left-most lane serving left-turn movement only. After the intersection, the road narrows to two lanes only. All the lanes are 12 feet wide. Honor Roll Dr. has a median strip and W Campus Dr. has a median paint. The details are shown in Figure 2.1.4 (d).

Achievement Dr. (Intersection E)

The next signalized intersection is at Achievement Dr. which is on both north and south sides of the intersection. The northbound and southbound ways are of two lanes, each 12 feet. Both ways lead to schools, so speed limit is 10 mph. The rightmost lane of the southbound road towards the intersection serves for right-turn movement only. The rightmost lane serves for northbound right-turn as well as through movement. There are median strips near the intersection for separation of traffic flow. The details are shown in Figure 2.1.4 (e).

Montgomery College Dr. (Intersection F)

This is the sixth signalized intersection of the testbed of SH 242. There is a school on north of the intersection. Therefore, the speed limit of northbound and southbound road towards the intersection is 10 mph. The northbound road towards the intersection is of two lanes with the right-most lane serving only right-turn movement. The southbound road towards intersection is of three lanes with right-most lane serving only right-turn movement and left-most lane serving only left-turn movement. A median strip divides the road traffic. The details are shown in Figure 2.1.4 (f).

Windsor Lakes Blvd. / St. Lukes Way (Intersection G)

The next signalized intersection is at Windsor Lakes Blvd. and St. Lukes Way, on the north and south of the intersection, respectively. The posted speed limit of the north- and southbound road towards the intersection are 25 and 30 mph, respectively. The roads in both directions has two lanes. The rightmost lane of northbound road towards the intersection serves as right-turn movement only. An addition lane is for the southbound left-turn movement. The rightmost lane serves for right-turn movement only. Median strip divides the traffic flow. The details are shown in Figure 2.1.4 (g).

Green Bridge Dr. (Intersection H)

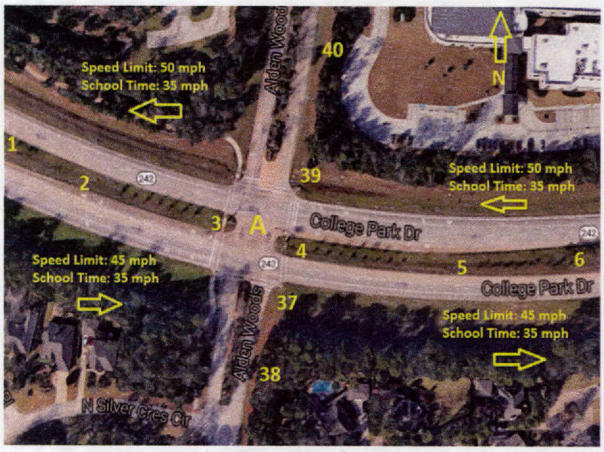
This is the only T intersection in the testbed SH 242. The posted speed limit of both east- and westbound traffic is 45 mph. The road in both directions has two lanes. For eastbound traffic at the intersection, there are two lanes for left-turn movement and a lane for right-turn movement. There is a median strip to divide the flow of the traffic. The details are shown in Figure 2.1.4 (h).

2.1.2.3 Network Modeling of the Testbed

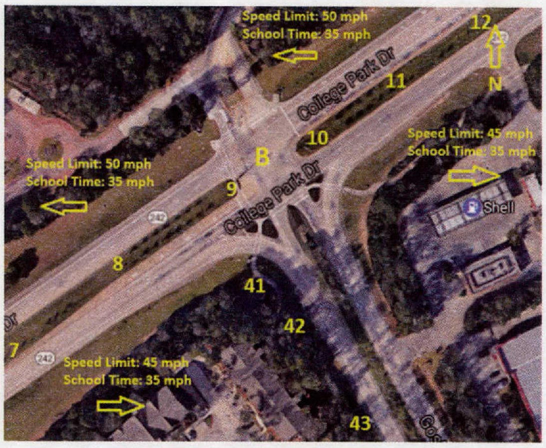
As the same process for the testbed of SH 242 mentioned above, once the geometric information is collected, we aim to build the testbed in VISSIM. In the first step, a network model will be built. In the network, the whole testbed is divided into a series of segments. The segmented points were defined by node numbers (see Figure 2.1.4) and relative distance (see Table 2.1.8). The network model of this testbed is summarized in Table 2.1.8, where presents the comprehensive geometric analysis of the testbed.

The table contains five columns. Column 1 indicates the node number (the fragmented portion), corresponding to the segments shown in Figure 2.1.4. Column 2 lists the distance between two nodes mentioned in Column 1. For example, the distance is 60 meters between Nodes 1 and 2, where Node 1 presents the starting point of the testbed near the first intersection. The third and fourth columns indicate the west- and eastbound traffic, as well as the number of different types of lanes (such as left-turn lane, through lane, etc.), respectively. Finally, the fifth column indicates the speed limit for this segment on the main road. Note that since Intersection I will be treated separately as a single testbed, it is not included in Table 2.1.8. Instead, the network modeling for this intersection is shown in Table 2.1.9.

Finally, the geometric analysis of all minor road segments is summarized in Table 2.1.10. This table has similar information to Table 2.1.8.



(a) Intersection A



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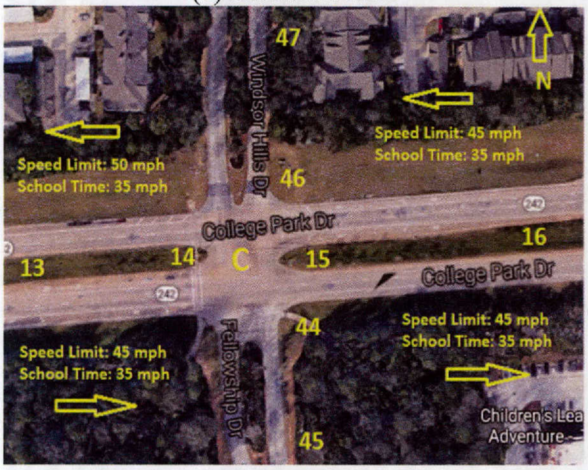
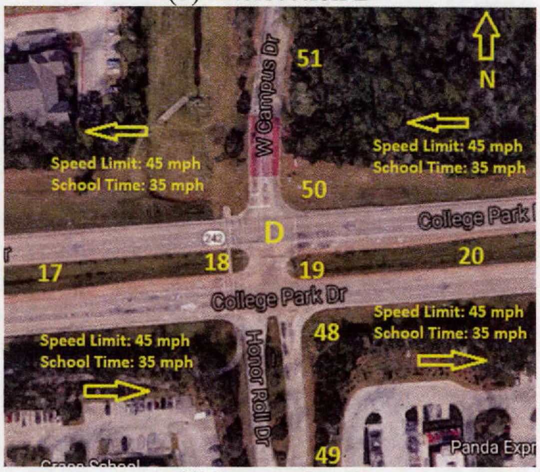
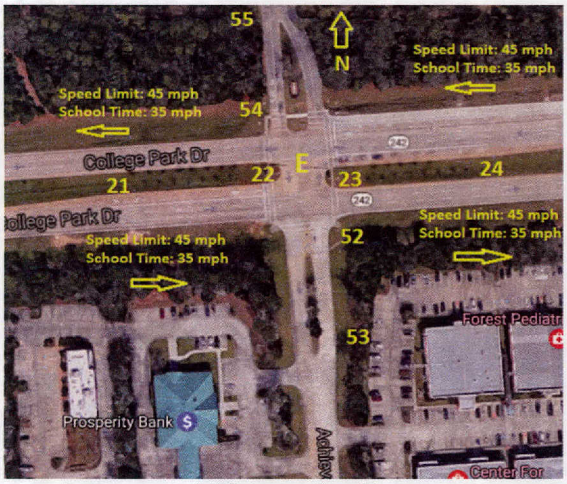


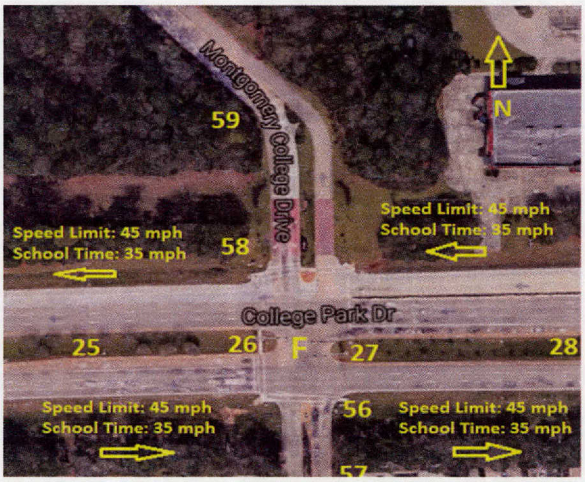
(c) Intersection C



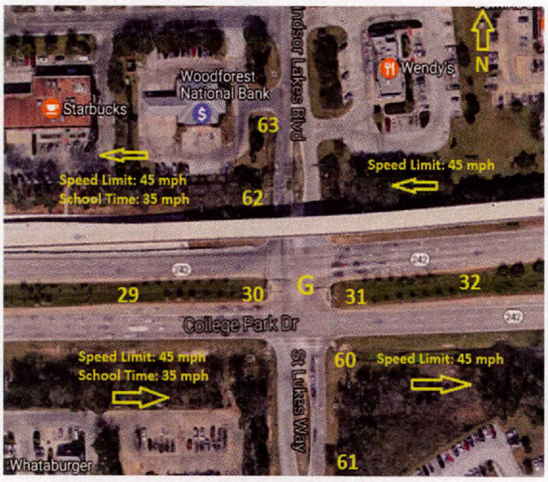
(d) Intersection D



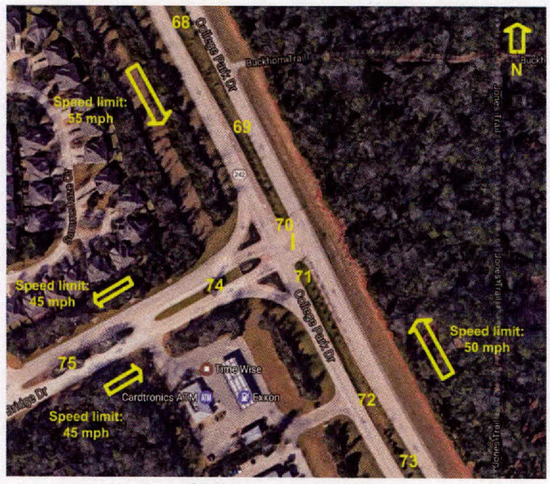
(e) Intersection E



(f) Intersection F



(g) Intersection G



(h) Intersection H

Figure 2.1.4 *Detailed map of Intersections at SH 242 with nodes numbering*

Table 2.1.8 Segment analysis of the major road (SH 242) from west to east

Node Number	Distance (m)	Type/Number of Lane		Speed Limit (mph)
		Eastbound	Westbound	
(start) 1-2	60	2T	2T	Westbound – 50 mph East bound – 45 mph (35 mph at School zone)
2-3	72	1L, 2T, 1R	2T	
3-4 (A)	35			
4-5	83	2T	1L, 2T, 1R	
5-6	100	2T	2T	
6-7	240	2T	2T	
7-8	100	2T	2T	
8-9	87	1L, 2T, lR	2T	
9-10 (B)	36			
10-11	77	2T	1L, 2T, 1R	
11-12	155	2T	2T	Westbound – 45 mph (35 mph at School zone) East bound – 45 mph (35 mph at School zone)
12-13	335	3 T	3 T	
13-14	81	1L, 2T, lR	2T	
14-15 (C)	38			
15-16	81	2T	1L, 2T, lR	
16-17	170	2T, 1R	3T	
17-18	72	1L, 2T, lR	3T	
18-19 (D)	25			
19-20	63	3T	1L, 3T	
20-21	180	3T	3T	
21-22	85	1L, 3T	3T	
22-23 (E)	29			Westbound – 45 mph East bound – 45 mph
23-24	76	3T	1L, 3T, 1R	
24-25	225	3T	4T	
25-26	69	1L, 3T	4T	
26-27 (F)	32			
27-28	84	3T	1L, 3T, Ramp (1T, 1R)	
28-29	100	3T	3T	
29-30	80	1L, 3T	3T	
30-31 (G)	34			
31-32 (end)	136	3T	1L, 3T	
Sum	**3040**			

Note: T: through movement, L: left-turn movement, T-R: through and right-turn movement. For example, 3 T means there are three lanes for through movements.

Table 2.1.9 Segment analysis of Green Bridge Dr. (Intersection H)

Node Number	Distance (m)	Type/Number of Lane		Speed Limit (mph)
		Eastbound	Westbound	
68-69	70	**1L**, 1T, 1TR	2T	Westbound
69-70	100	2T	2T	50 mph
70-71 (I)	30			
71-72	100	2T	**1L**, 2T	Eastbound
72-73	70	2T	2T	55 mph

Note: "L" bolded as the left-turn movement is critical in modeling

Table 2.1.10 Segment analysis of the minor roads on the testbed of SH 242

Intersection	Node Number	Distance (meter)	Type/Number of Lane		Speed Limit (mph)
			South	North	
A: Alden Woods	37-38	50	2T	1T, 1R	25
	39-40	70	2T	2T	25
B: Gosling Road	41-42	30	2T	1T, 1T-R	45
	42-43	39	1T, 1T-R	2T	45
C: Windsor Hills Dr.	44-45	50	2T	1T, 1T-R	30
	46-47	75	1T, 1R	2T	25
D: Honor Roll Dr.	48-49	75	2T	1T, 1T-R	10
	50-51	80	1T, 1R	1T	10
E: Achievement Roll Dr.	52-53	55	2T	1T, 1T-R	10
	54-55	85	1T, 1T-R	2T	10
F: Montgomery College Dr.	56-57	45	2T	1T, 1R	10
	58-59	95	1T, 1R	2T	10
G: Windsor Lakes Dr.	60-61	65	2T	1T, 1R	30
	62-63	45	1R, 1T-R, 1T	2T	25
H: Green Bridge Dr.	70-74	60	2L, 1R[1]	2T[2]	45
	74-75	100	2T[1]	2T[2]	45

Note: 1: Eastbound, 2: Westbound

2.1.3 Testbed of FM 1464

2.1.3.1 Overview of Testbed

The testbed of FM 1464 has 11 consecutive signalized intersections: W Oaks Village Dr.(A), Bellaire Blvd.(B), Highland Oak Ln/ Watering Oaks Ln(C), Orchid Ridge Ln(D), Beechnut St (E), Bissonnet Blvd (F), W Bellfort St.(G), W Airport Blvd(H), Stephen F Austin High School (I), Old Richmond Rd(J), and Old Orchard Dr./Orchard Lake Estates Dr. (K), listed from north to south, as shown in Figure 2.1.5.

Full Testbed **First Section (5 Intersections)** **Second Section (6 Intersections)**

Figure 2.1.5 *Testbed of FM 1464 with selected 11 intersections*

Note that in the work plan, the first intersection given by the Receiving Agency is George Bush High School. However, this high school exactly locates at the intersection of W Oaks Village Dr. (A); but the original work plan does not include the intersection at Bissonnet because it is a newly opened intersection. Therefore, the number of intersections at this testbed is still 11.

FM 1464 is the major road for the study and the previously mentioned 11 roads are minor. In the testbed, the major road segment starts from 150 meters north of the center of the intersection at W. Oaks Village Dr., and ends in 150 meters south of the center of the intersection at Old Orchard Dr. Similarly, such two extra lengths extended respectively from north and south intersections along FM 1464 are used to guarantee that vehicles queues generated ahead of the signals can be correctly captured and modeled in the traffic simulation models. The total length of selected testbed is 9500 meters. From west to east, the center to center distances between two consecutive intersections are shown in Table 2.1.11. Also, as we did for the previous two testbeds, a length of at most 200 meters was considered (calculated from the center of each intersection, extended to east and west, respectively) for the minor roads. The length information of road segments, as well as the speed limits of the minor road segments associated with the eleven intersections, are summarized in Table 2.1.14.

All along the segment of FM 1464, the lane width is 12 feet. For either westbound or eastbound traffic, there are two to four lanes and the speed limit ranges from 35 mph (school zone and pedestrian crossing) to 50 mph. Near each intersection, there is a central left turn lane which provides an exclusive left-turn lane for both directions. On the other hand, near each intersection, the right-most lane of both directions accommodates right-turn movements. This pattern of lane allocation is identical at the selected eleven consecutive intersections. Table 2.1.12 summarizes the aforementioned lane features.

Table 2.1.11 Length information of the testbed of FM 1464 (from north to south)

Direction	Intersection	Intersection Name	Speed Limit (mile/hour)	Center to Center Distance (meter) *	Cumulative Distance from Start point (meter)
North to South	Start				0
	A	W Oaks Village Dr.	35(E),10(W)	150†	150
	B	Bellaire Blvd.	20(E),20(W)	450	600
	C	Highland Oak Ln/ Watering Oaks Ln	30(E),20(W)	350	950
	D	Orchid Ridge Ln	20(E),20(W)	650	1600
	E	Beechnut St	35(W)	800	2400
	F	Bissonnet Blvd.	35(E),35(W)	1000	3400
	G	W Bellfort St.	40(E),40(W)	1500	4900
	H	W Airport Blvd.	35(E),20(W)	1700	6600
	I	Stephen F Austin High School	10(E),10(W)	750	7350
	J	Old Richmond Rd	35(W)	1100	8450
	K	Old Orchard Dr./Orchard Lake Estates	20(E),20(W)	900	9350
	End			150	9500

Note: * each column represents distance from previous column; † it is from the start point

Table 2.1.12 Major road segment properties of the testbed of FM 1464

Section		Speed Limit (mile/hour)	Lane Width (feet)	Number of Lanes
Southbound	FM 1464, Start to W Oaks Village Dr.	50 (School Time: 30)	12	4 lanes with 1 left turn lane 1 right turn lane and 2 through lane
	FM 1464 W Oaks Village Dr. to Bellaire Blvd.	50 (School Time: 30)	12	four lane with 1 left turn, 2 through and 1 right turn
	FM 1464,Bellaire Blvd. to Highland Oak Ln/ Watering Oaks Ln	50	12	three lane with 1 left turn, 1 through and 1 through-right
	FM 1464,Highland Oak Ln/ Watering Oaks Ln to Orchid Ridge Ln	50	12	four lane with 1 left turn two through and 1right turn
	FM 1464,Orchid Ridge Ln to Beechnut St	50	12	four lane with 1 left turn, 2 through and 2 right turn
	FM 1464, Beechnut St to Bissonnet St.	50	12	four lane with 1 left turn, 2 through and 1 right turn
	FM 1464, Bissonnet St to W Bellfort St.	50	12	four lane with 1 left turn, 2 through and 1 right turn

Section		Speed Limit (mile/hour)	Lane Width (feet)	Number of Lanes
	FM 1464, W Bellfort St. to W Airport Blvd.	50	12	four lane with 1 left, 2 through and 1 right
	FM 1464, W Airport Blvd. to Stephen F Austin High School to Old Richmond Rd	50	12	four lane with 2 left, 1 through and1 right
	FM 1464, Stephen F Austin High School to Old Richmond Rd	50	12	three lane with 1 left, 2 through
	Old Richmond Rd to Old Orchard Dr./Orchard Lake Estates Dr	50	12	three lane with 1 left, 1 through and 1 through-right
	FM 1464, Old Orchard Dr./Orchard Lake Estates Dr to End	50	12	3 lanes with 1 central left turn lane
Northbound	FM 1464, End to Old Orchard Dr./Orchard Lake Estates Dr	50	12	3 lane with 1 left, 1 through and 1 through-right
	FM 1464, Old Orchard Dr.to Old Richmond Rd	50	12	4 lane with 1 left, 2 through and 1 right
	FM 1464, Old Richmond Rd to Stephen F Austin High School	50	12	4 lane with 1 left, 2 through and 1 right
	FM 1464, Stephen F Austin High School to W Airport Blvd	50	12	4 lane with 1 left, 2 through and 1 right
	FM 1464, W Airport Blvd. to W Bellfort St.	50	12	4 lane with 1 left, 2 through and 1 right
	FM 1464, W Bellfort St. to Bissonnet St	50	12	4 lane with 1 left, 2 through and 1 right
	FM 1464, Bissonnet St. to Beechnut St	50	12	4 lane with 1 left, 2 through and 1 right
	FM 1464, Beechnut St to Orchid Ridge Ln	50 (School Time: 35)	12	4 lane with 1 left, 2 through and 1 right
	FM 1464, Orchid Ridge Ln to Highland Oak Ln/ Watering Oaks Ln	50 (School Time: 35)	12	4 lane with 1 left, 1 through and 1 through-right
	FM 1464, Highland Oak Ln/ Watering Oaks Ln to Bellaire Blvd	50	12	4 lane with 1 left, 2 through and 1 right
	FM 1464, Bellaire Blvd. to W Oaks Village Dr.	50	12	4 lane with 1 left, 2 through and 1 right
	FM 1464, W Oaks Village Dr to Start.	50	12	4 lane with 1 left, 2 through and 1 right

2.1.3.2 Minor Roads

Eleven east-west running minor roads are connected with the major road FM 1464 at 11 consecutive intersections. The geometry and properties of these minor roads are described below.

W. Oaks Village Dr. (Intersection A):

At this intersection, the minor road has two lanes for both east- and westbound traffic towards the intersection: one through-left and one through-right. The road has the raised median to divide the conflicting traffic. On the other hand, it has two through lanes for both east- and westbound traffic from the intersection. The speed limit of eastbound traffic towards the intersection is 35 mph. It has a school on the east side of the intersection, so the speed limit is 10 mph for the westbound traffic towards the intersection.

Bellaire Blvd. (Intersection B):

The second intersection is Bellaire Blvd at FM 1464. Both east- and westbound traffic towards the intersection has three lanes (12 feet wide): one through-right, one through, and one left-turn lane. On the other hand, it has two lanes for east- and westbound traffic from the intersection. The road has a raised median to divide the conflicting traffic. The speed limit for both east- and westbound traffic is 35 mph.

Highland Oak Ln./ Watering Oaks Ln. (Intersection C)

The third intersection is Highland Oak Ln./ Watering Oaks Ln. at FM 1464. The road has two lanes in each direction, and each lane is 12 feet wide. Both east- and westbound traffic towards the intersection has one through/right-turn lane and one through/left-turn lane. The road has a raised median to divide the conflicting traffic. The speed limits are 30 mph and 20 mph for east- and westbound traffic, respectively.

Orchid Ridge Ln. (Intersection D):

The fourth is Orchid Ridge Ln. The road has two lanes in both directions with each lane of width 12 feet. Both east- and westbound traffic has two lanes with one through-right and one through-left lane, towards the intersection. On the other hand, it has two lanes for both east- and westbound traffic moving from the intersection. The road has a raised median to divide the conflicting traffic. The posted speed limit for westbound traffic is 30 mph (20 mph at school time) and eastbound traffic is 20 mph, towards the intersection.

Beechnut St. (Intersection E):

Beechnut St meets FM 1464 at Intersection E. The road has two lanes in both directions with each lane of 12 feet. Both east- and westbound traffic has two lanes, one through/right-turn and one left-turn lane, respectively, towards the intersection. On the other hand, it also has two lanes for both east- and westbound traffic moving from the intersection. The road has a raised median to divide the conflicting traffic. The speed limit is 35 mph for westbound traffic across the intersection.

Bissonnet St. (Intersection F)

Bissonnet St. meets FM 1464 at Intersection F. The road has three lanes (one left-turn lane, one through lane and one through/right-turn lane) westbound direction towards the intersection. On the other hand, it also has four lanes (one left-turn lane, two through lanes, and one right-turn lane) for eastbound traffic moving from the intersection. The speed limit is 35 mph for westbound traffic across the intersection.

W. Belfort St. (Intersection G):

W. Belfort St. runs along in the east-west direction and meets FM 1464 at Intersection G. This intersection is far away from the previous intersections (almost 2.5 km). The road has three lanes (one left-turn lane, one through lane, and one through/right-turn lane) in both east- and westbound directions towards the intersection. On the other hand, it has two lanes for east- and westbound traffic moving from the intersection. The road has a raised median to divide the conflicting traffic. The speed limit is 40 mph for both east- and westbound traffic towards the intersection.

W. Airport Blvd. (Intersection H):

W. Airport Blvd. runs along East-West direction and meets FM 1464 at Intersection H. The road has three lanes (one exclusive left-turn lane, one through/left-turn lane and one exclusive right-turn lane) for westbound traffic towards the intersection. The eastbound traffic towards intersection also has three lanes: one left-turn lane, one through lane and one through/right-turn lane. After crossing the intersection, the road has two lanes for both east- and westbound traffic moving from the intersection. The road has a raised median to divide the conflicting traffic. The speed limit for eastbound traffic is 45 mph.

Stephen F. Austin High School (Intersection I):

Stephen F. Austin High School is located in eastern side from Intersection I. The road has two lanes (one left-turn lane and one through/right-turn lane) for westbound traffic towards the intersection. The speed limit for both east- and westbound traffic towards the intersection is assumed to be 10 mph. On the other side of the intersection, there is a 2-lane road from a Baptist church (one left-turn lane and one through/right-turn lane) for eastbound traffic towards intersection. After crossing the intersection, the road has only one lane for both east- and westbound traffic moving from the intersection.

Old Richmond Rd. (Intersection J):

Old Richmond Rd is a T intersection, and the minor road is on the east side of FM 1464. The road has two lanes: one left-turn lane and one right-turn lane for westbound traffic towards the intersection. On the other hand, it also has two through lanes for eastbound traffic moving from the intersection. The road has a raised median to divide the conflicting traffic. The speed limit towards the intersection is 35 mph (20 mph during school time).

Old Orchard Dr./Orchard Lake (Intersection K):

The last one is Old Orchard Dr./Orchard Lake at FM 1464. The road has two lanes (one through/left turn lane and one through/right-turn lane) in both directions towards the intersection. After crossing the intersection, the road has two lanes for both east- and westbound traffic moving from the intersection. The road has a raised median to divide the conflicting traffic. This minor road enters the community through a closed gate shortly from the intersection from each side. The speed limit is unknown, assumed to 20 mph.

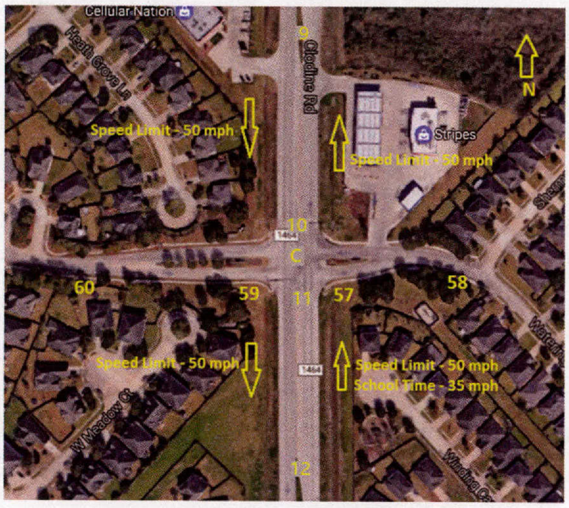
2.1.3.3 Network Modeling of Testbed

Similarly, after the geometric information is collected, we aim to build the network model of the testbed. The whole testbed is divided into a series of segments, labelled by node numbers (see Figure 2.1.6). The relative distance between two consecutive nodes is reported in Table 2.1.13.

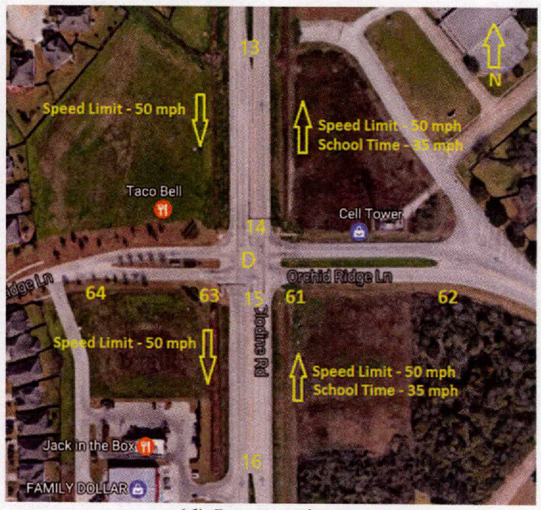
This table also summarizes the comprehensive geometric analysis of the testbed. Still, Column 1 indicates the node number (the fragmented portion), corresponding to the segments shown in Figure 2.1.6. Column 2 lists the distance between two nodes mentioned in Column 1. For example, the distance is 133 meters between Nodes 1 and 2, where Node 1 presents the starting point of the testbed near the first intersection. The third and fourth columns indicate the west- and eastbound traffic, as well as the number of different types of lanes (such as left-turn lane, through lane, etc.). The fifth column indicates the speed limit. Similar geometric analysis for all 11 minor road segments are summarized in the Tables 2.1.14.

(a) Intersection A

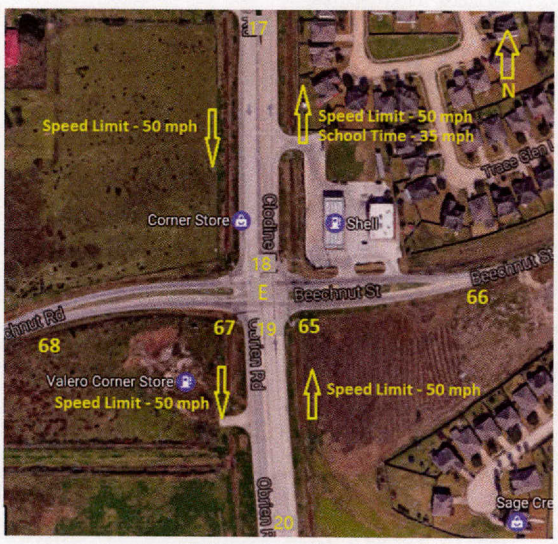
(b) Intersection B



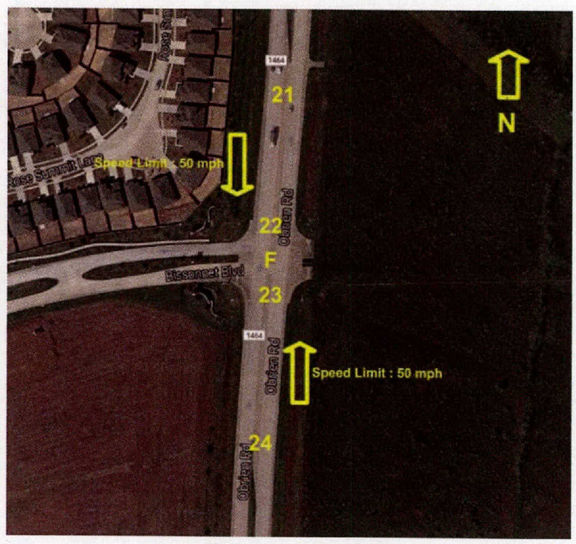
(c) Intersection C



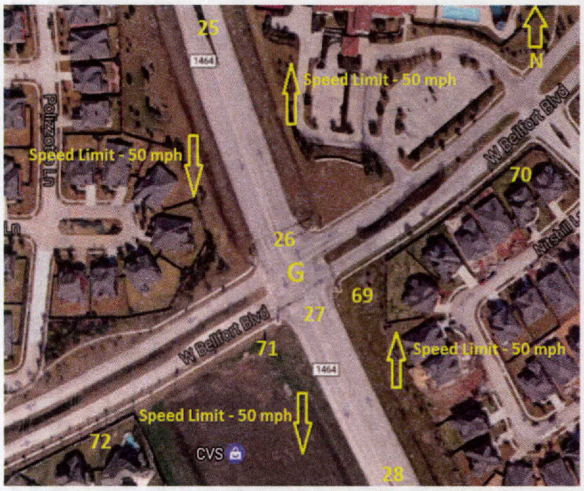
(d) Intersection D



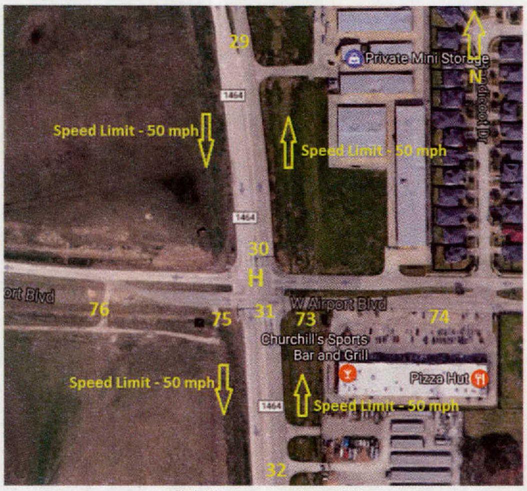
(e) Intersection E



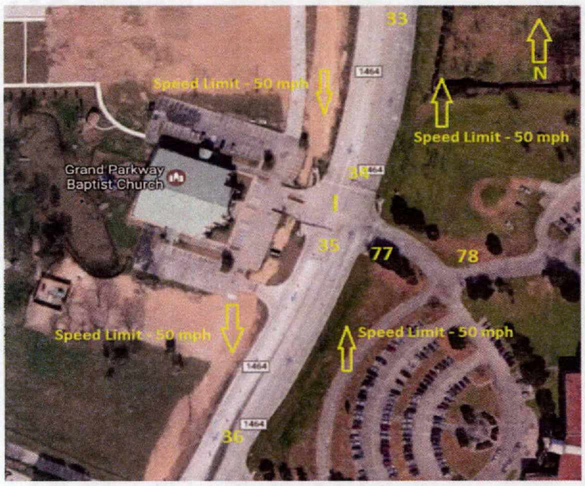
(f) Intersection F



(g) Intersection G

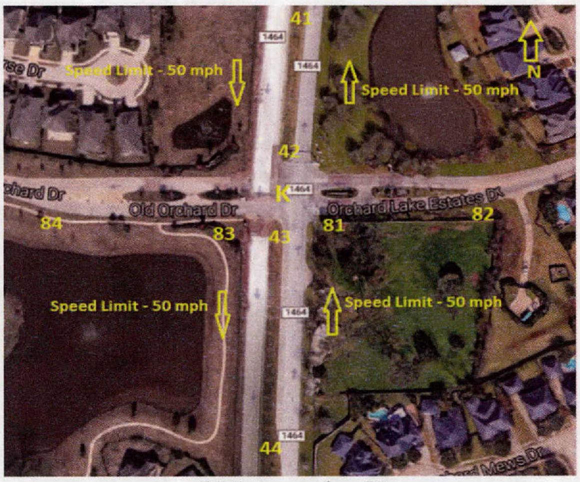


(h) Intersection H



(i) Intersection I

(j) Intersection J



(k) Intersection K

Figure 2.1.6 *Detailed map of Intersections (A-K) at FM 1464 with nodes numbering*

Note on Intersection F (at Bissonnet St.): the full four-way intersection was open at the end of 2017, and Bissonnet St. has been extended. However, the satellite image from Google is not updated yet.

Table 2.1.13 Segment analysis of the major road (FM 1464) from north to south

Node Number	Distance (m)	Type/Number of Lane		Speed Limit(mph)
		Northbound	Southbound	
1-2	133	2T	1L,2T,1R	Northbound-50mph Southbound-50mph (School time: 35 mph)
2-3(A)	35			
3-4	160	1L,2T,1R	2T	
4-5	86	2T	2T	Northbound-50mph Southbound-50mph
5-6	166	2T	1L,2T,1R	
6-7(B)	40			
7-8	170	1L,2T,1R	2T	
8-9	30	2T	2 T	Northbound-50mph Southbound-50mph
9-10	112	2T	1L,1T,1T-R	
10-11(C)	35			
11-12	133	1L, 1T, lT-R	2T	
12-13	345	2T	2T	Northbound-50mph Southbound-50mph (School time : 35 mph)
13-14	137	2T	1L,2T,1R	
14-15(D)	35			
15-16	127	1L,2T,1R	2T	
16-17	485	2T	2T	Northbound-50mph Southbound-50mph
17-18	150	2T	1L,2T,1R	
18-19 (E)	40			
19-20	180	1L,2T,1R	2T	
20-21	650	2T	2T	Northbound-50mph Southbound-50mph
21-22	130	2T	1L,1T,1T-R	
22-23(F)	40			
23-24	115	1L,1T,1T-R	2T	
24-25	1230	4T	2T	Northbound-50mph Southbound-50mph
25-26	115	2T	1L,2T,1R	
26-27(G)	40			
27-28	120	1L,2T,1R	2T	
28-29	1380	2T	2T	Northbound-50mph Southbound-50mph
29-30	160	2T	1L,2T,1R	
30-31 (H)	40			
31-32	180	1L,2T,1R	2T	
32-33	380	2T	2T	Northbound-50mph Southbound-50mph
33-34	155	2T	2L,1T,1R	
34-35(I)	30			
35-36	135	1L,2T,1R	2T	
36-37	750	2T	2T	Northbound-50mph Southbound-50mph
37-38	183	2T	1L,2T	
38-39(J)	35			
39-40	142	1L,2T,1T-R	2T	
40-41	595	2T	2T	
41-42	128	2T	1L,1T,1T-R	

Node Number	Distance (m)	Type/Number of Lane		Speed Limit(mph)
		Northbound	Southbound	
42-43(K)	35			
43-44	133	1**L**,1T,1T-R	2T	
Total	9500			

Note: T: through movement, L: left-turn movement, T-R: through and right-turn movement. For example, 3 T means there are three lanes for through movements. "L" was bolded as the left-turn movement is critical in modeling.

Table 2.1.14 Segment analysis of minor roads on the testbed of FM 1464

Node Number	Distance (meter)	Type/Number of Lane		Speed Limit (mph)
		East	West	
45-46	50	2T	1T, 1R	35
47-48	70	2T	2T	35
49-50	60	2T	1T-L,1T-R	10
51-52	90	1T-L,1T-R	2T	35
53-54	130	2T	1L,1T, 1T-R	20
55-56	170	1L,1T, 1T-R	2T	20
57-58	100	2T	1T-L,1T-R	30
59-60	160	1T-L,1T-R	2T	20
61-62	120	2T	1T-L,1T-R	20 30(School time 20)
63-64	110	1T-L,1T-R	2T	30(School time 20)
65-66	110	2T	1L,1T-R	30
67-68	160	1L,1T-R	2T	-
69-70	140	2T	1L,1T,1T-R	40
71-72	200	1L,1T,1T-R	2T	40
73-74	140	2T	1L,1T-L,1R	45
75-76	95	1T-L,1T,1T-R	2T	35
77-78	50	2T	1L,1T-R	10
79-80	140	2T	1L,1R	35(School Hour 20)
81-82	85	2T	1T-L, 1T-R	20
83-84	95	1T-L, 1T-R	2T	20

Note: Intersection I (Bissonnet Blvd at FM 1464) is not open yet, so the detailed minor road information is unknown. Therefore, it is not shown in this table.

2.1.4 Testbed of IH 10

2.1.4.1 Overview of Testbed

Originally, eight signalized diamond intersections (six at IH 10 and two at IH 610) were included in this project. However, due to the connection problems, many were replaced. The final intersections used for the implementations are Westgreen Blvd. (A), Greenhouse Rd. (B), Katy Fort Bend Rd. (C), Mason Rd. (D), Mason Access Rd (E), Fry Rd. (F), Fry Access Rd (G), and Baker Cypress (H), as shown in Figure 2.1.7. There are all along the frontage road of IH-10, except Mason Access Rd (E) and Fry Access Rd (G).

Among these eight intersections, only two—Westgreen Blvd. (A) and Greenhouse Rd. (B)—were determined in the original work plan. Katy Fort Bend Rd. (C) was determined in November 2017 to replace Campbell Rd. at IH-10, and Baker Cypress (H) was determined in June 2018 to replace the intersection of FM 1463 at IH-10. The rest four: Mason Rd. (D), Mason Access Rd (E), Fry Rd. (F), Fry Access Rd (G) were picked at the last minute before the implementation (late November 2018, a month before the ending time of the project), to replace four intersections: Katy Mills Blvd at IH-10 (which was determined in November 2017 to replace Bingle Road/Voss Road at IH-10), Sheldon Rd. at IH-10, Evergreen and Furnace Place at IH-610, due to the connection problems: no data from loop detectors can be reported from these intersections.

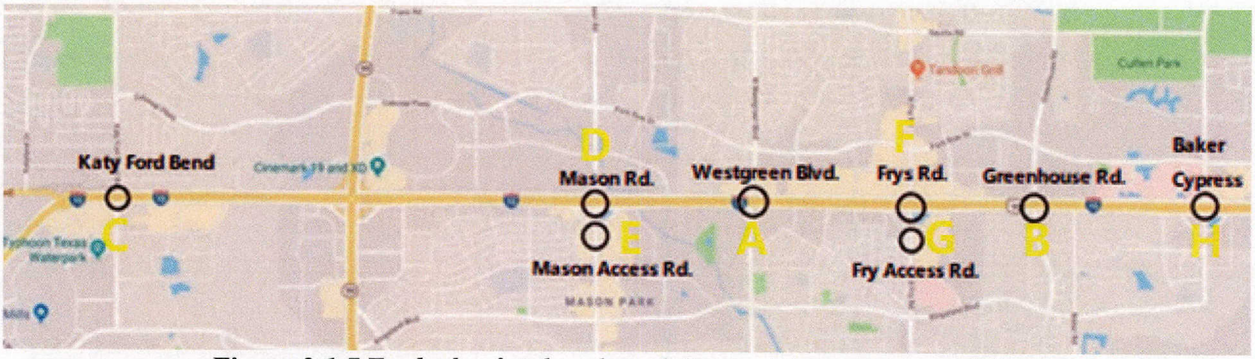


Figure 2.1.7 *Testbeds of eight selected diamond intersections at IH-10.*

Note that different from the intersections in the testbeds of FM 528, SH 242 and FM 1464, a diamond intersection is made up of two separate intersections, on each side of the freeway above it. Therefore, in one direction (across the freeway under the bridge), there are two sets of signals.

Westgreen Blvd (Intersection A)

The second one is Westgreen Blvd. at the frontage road (parallel to IH-10). Westgreen Blvd runs in the north-south direction. There are two lanes for northbound traffic toward the intersection (one through/left-turn lane and one through/right-turn lane), but three lanes for southbound traffic toward the intersection (one through/right-turn lane, one exclusive through lane and one exclusive left-turn lane). For both north- and southbound traffic on Westgreen Blvd., the speed limit is 40 mph. On the other hand, the speed limit for the east- and westbound traffic on the frontage road is 50 mph. For westbound traffic, the frontage road has four lanes: one exclusive U-turn, one exclusive left-turn, one through/left-turn and one through/right-turn lane. For eastbound traffic, it also has four lanes: one exclusive U-turn lane, one through/left-turn lane, one exclusive through lane, and one through/right-turn lane. The detailed segment analysis of Intersection A is reported in Table 2.1.15

Greenhouse Rd. (Intersection B)

Greenhouse Rd. runs in the north-south direction and meets with the frontage road (parallel to IH-10) at Intersection B. For northbound traffic, there are three lanes: one exclusive left-turn lane, one through lane and one exclusive right-turn lane. Similarly, for southbound traffic toward the intersection, there are also three lanes: one exclusive left-turn lane, one through/left-turn lane, and one through/right-turn lane. For both north- and southbound traffic on Greenhouse Rd, the posted speed limit is 45 mph. On the other hand, the speed limit for the east- and westbound traffic on the frontage road is 50 mph. For eastbound traffic, the frontage road has four lanes: one exclusive U-turn, one exclusive left-turn, one through/left-turn and one through/right-turn lane. For westbound traffic, it also has four lanes: one exclusive U-turn lane, one through/left-turn lane, one through lane, and one exclusive right-turn lane. The detailed segment analysis of Intersection B is reported in Table 2.1.15

Katy Fort Bend (Intersection C)

Katy Fort Bend also runs in the north-south direction and meets with Katy Freeway at Intersection C. For eastbound traffic, there are five lanes: one U-turn lane, one exclusively left-turn lane, one through/left-turn lane, one through lane and one exclusive right-turn lane. On the other hand, for westbound traffic, there are four lanes: one U-turn lane, one through/left-turn lane, one through lane and one exclusive right-turn lane. For both east- and westbound traffic, the speed limit is 50 mph. On the other hand, for both north- and southbound traffic, there are three lanes: one left-turn lane, one through/left-turn lane and one through lane. The posted speed limit for both north- and southbound traffic is 35 mph. The detailed segment analysis of this intersection is also reported in Table 2.1.15.

Mason Rd. and Mason Access Rd. (Intersections D and E)

Mason Rd. runs in in the north-south direction and meets with the frontage road (parallel to IH-10) at Intersection D. For the westbound and eastbound traffic on the frontage road, respectively, the lane distributions are the same: one U-turn lane, one exclusive left-turn lane, one lane for both through and left turn lane, two lanes for westbound or eastbound through traffic and one lane for right turn.

For the southbound traffic along the arterial road (Mason Rd.), there is one left-turn lane, one for both through and left-turn, one for through traffic, and one for right turn. On the other hand, for the northbound traffic, the through and left-turn lanes are the same, but there are two exclusive right-turn lanes. Therefore, it seems that the right-turn travel demand is large at this intersection.

Different from the diamond intersections A, B, and C, around 350 ft south of this intersection, there is another intersection made by Mason Access Rd and Mason Rd. Mason Access Rd. was built for vehicles to enter Mason Rd. more easily rather than having to use the intersection of Mason and the frontage road. It can also provide an access to the frontage road for northbound traffic. This access road has only one lane in each direction with a two-way-left-turn median. Please see Table 2.1.15 for the details at these two intersections.

Fry Rd. and Fry Access Rd. (Intersection F and G)

This intersection is very similar to the intersection of Mason Rd. The lane distributions on the frontage road is the same to the frontage road at the intersection of Mason Rd. However, for the arterial road (Fry Road), the lane distributions are a little bit different. For both southbound and northbound traffic, there are one exclusive left-turn lane, one lane for both left-turn and through traffic and one lane for both through and right-turn, and one for exclusive right-turn. Around 350 ft south of this intersection, an access road was also built for traffic from the frontage road to enter Fry Road. Please see Table 2.1.15 for the details at these two intersections.

Baker Cypress Rd. (Intersection H)

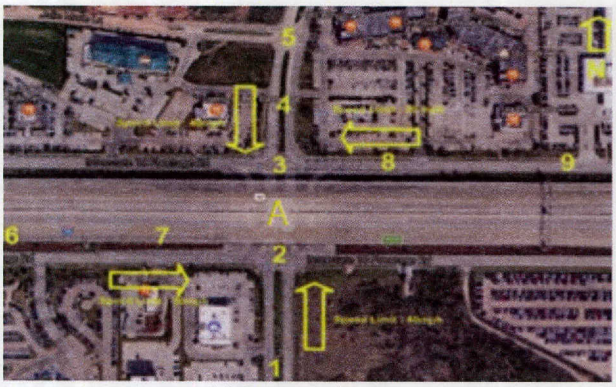
This intersection is also located at the frontage road of IH-10. It is east of the intersection of Greenhouse Rd. For the eastbound traffic along the frontage road, there one U-turn lane, one exclusive left-turn lane, one exclusive right-turn lane, one lane for left-turn and through traffic and two lane for exclusive through traffic; while for the westbound traffic, the lane distribution is similar: there are also six lanes: all are the same but one lane is for both right-turn and through traffic, so only one lane for exclusive through traffic.

On the other hand, for northbound traffic, there are one lane exclusively for right-turn, two lanes for left-turn, respectively, and one lane for both through and right-turn traffic; while for the southbound traffic, there are one lane exclusively for left-turn, right-turn traffic, and through traffic respectively, and one lane for both left-turn and through traffic. Please see Table 2.1.15 for the details at these two intersections.

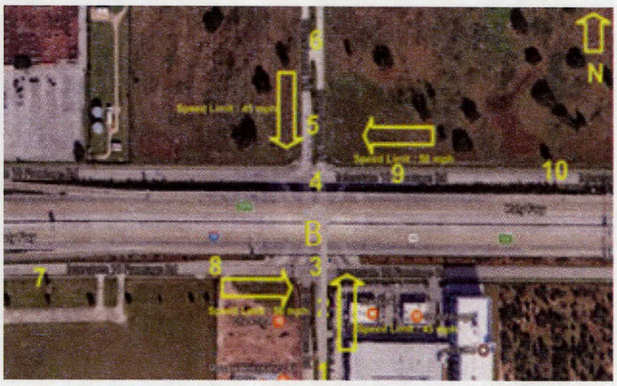
2.1.4.2 Network Modeling of the Testbed

Similar to the network modeling conducted for the testbeds of FM 528, SH 242, and FM 1464, the research team conducted a comprehensive observation to divide the link into small segments. It provides the relative distance of different segments of the links. The segmented points were defined by node numbers (see Figure 2.1.8) and relative distance (see Table 2.1.15).

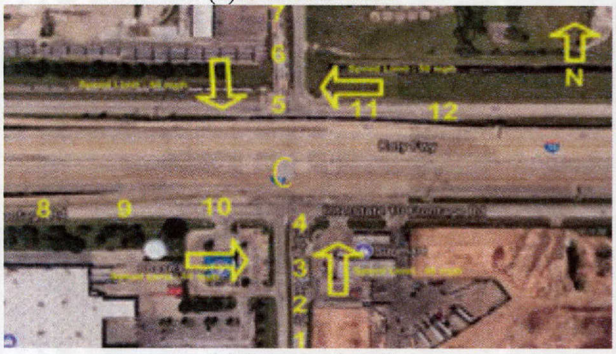
Table 2.1.15 summarizes the comprehensive geometric analysis of the testbed. Column 1 represents the name of intersection, Column 2 indicates the node number (the fragmented portion), corresponding to the segments shown in Figure 2.1.8. Column 3 lists the distance between two nodes mentioned in Column 2. The fourth, fifth, sixth and seventh columns indicate the number of different type of lanes (such as left-turn lane, through lane, etc.) of east-, west-, north- and southbound traffic, respectively. Finally, the eighth column indicates the speed limit of each section.



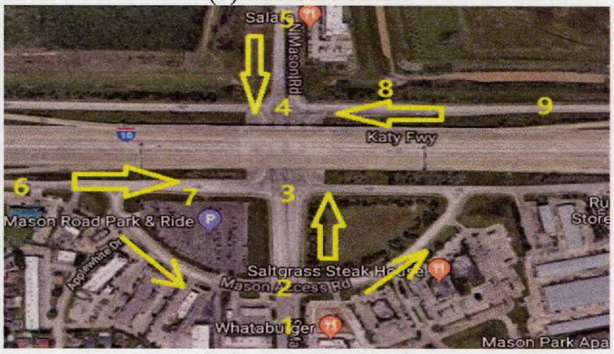
(a) Intersection A



(b) Intersection B

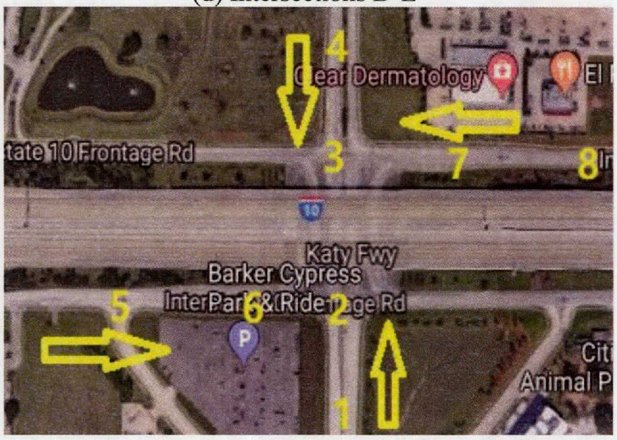


(c) Intersection C



(d) Intersections D-E

(e) Intersection F-G



(f) Intersection F

Figure 2.1.8 *Detailed map of Intersections (A-F) with nodes numbering*

Table 2.1.15 Segment analysis of diamond intersections at IH-10

Inter-section	Node Number	Distance (meter)	Type/Number of Lane				Speed Limit (mph)
			Eastbound	Westbound	Northbound	Southbound	
Westgreen Blvd.	1-2	200	-	-	1T, 1T-R	-	40 mph
	2-3	-	-	-	**1L**, 2T	-	
	3-2	-	-	-	-	**1L**, 2T	40 mph
	4-3	72	-	-	-	2T, 1T-R	
	5-4	128	-	-	-	2T	
	6-7	109	3T	-	-	-	50 mph
	7-2	91	1U, 1T-L, 1T, 1T-R	-	-	-	
	9-8	92	-	3T	-	-	50 mph
	8-3	108	-	1U, **1L**, 1T-L, 1T-R	-	-	
Green-house Rd.	1-2	144	-	-	2T		45 mph
	2-3	56	-	-	2T, 1R		
	3-4	-	-		**1L**, 1T-L		
	4-3	-	-	-	-	**1L**, 1T-L, 1T	45 mph
	5-4	70	-	-	-	2T, 1T-R	
	6-5	130	-	-	-	2T	
	8-3	90	1U, **1L**, 1T-L, 1T-R	-	-	-	50 mph
	7-8	110	3T	-	-	-	
	4-9	85	-	1U, 1T-L,1T-R, 1R	-	-	50 mph
	9-10	115	-	3T	-	-	
Katy Fort Bend	1-2	132	-	-	2T	-	35 mph
	2-3	28	-	-	**2T**, 1R	-	
	3-4	40	-	-	**1L,** 2T, 1R	-	
	4-5	-	-	-	**1L,** 1T-L,1T	-	
	5-4	-	-	-	-	**1L**, 1T-L, 1T	35 mph
	6-5	53	-	-	-	**1L**, 1T, 1T-R	
	7-6	147	-	-	-	2T	
	8-9	120	3T	-	-	-	50 mph
	9-10	25	1U, 3T	-	-	-	
	10-4	55	1U, 1L,1T-L 1T, 1R	-	-	-	
	11-5	65	-	1U, 1T-L, 1T, 1R	-	-	50 mph
	12-11	135	-	3T	-	-	

Inter-section	Node Number	Distance (meter)	Type/Number of Lane				Speed Limit (mph)
			Eastbound	Westbound	Northbound	Southbound	
Mason	1-2	50	--	--	1L, 2T, 1R		35 mph
	2-3	135	--	--	1L, 1T-L, 1T, 2R	--	35 mph
	4-5	135				1L, 1T-L, 1T, 1R	35mph
	6-7	135	1U, 1L,1T-L 2T, 1R				50 mph
	8-9	135		1U, 1L,1T-L 2T, 1R			50 mph
Mason Access	Westside	--	1 TR				35 mph
	Eastside	--		1R			35 mph
Fry Rd.	1-2	50	--	--	1L, 1T, 1T-R		35 mph
	2-3	135	--	--	2L, 1T, 1T-R, 1R	--	35 mph
	4-5	135				2L, 1T, 1T-R, 1R	35mph
	6-7	135	1U, 1L,1T-L 1T, 1T-R, 1R				50 mph
	8-9	135		1U, 1L,1T-L 1T, 1T-R, 1R			50 mph
Fry Access	Westside	--	1 TR				35 mph
	Eastside	--		1R			35 mph
Baker Cypress	1-2	135			1L, 1T-L, 1T, 1R		35 mph
	3-4	135				1L, 1T-L, 1T, 1R	40 mph
	5-6	135	1U, 1L,1T-L 2T, 1R				50 mph
	7-8	135		1U, 1L,1T-L 1T, 1T-R, 1R			50 mph

Note: T: through movement, L: left-turn movement, T-R: through, right-turn movement and U: U turn. For example, 3T refers there are 3 lanes for through movements. "L" was bolded as the left-turn movement is critical in modeling.

2.2. EXISTING TRAFFIC SIGNAL PLANS

The existing traffic signal plans used at these intersections were provided by Steve Chiu from the Receiving Agency (the Houston District Office). The plans include the sequences of phases, offsets, cycle lengths, minimum green times, maximum green times, yellow times, vehicle extensions, red clearance times, and phase split preferences. This information will be used in the simulation software (VISSIM) to simulate the traffic at each intersection for the purpose of (1) verifying the simulation results, and (2) evaluating the performance of the proposed proactive signal control system at each intersection.

To code the signal plans into VISSIM correctly, approaches at these intersections have been specified with all phases. From the raw detector data provided by TxDOT Houston Office, the research team from the Performing Agency also identified the loop detectors associated with each phase. The existing signal plans of these intersections, together with the collected traffic data will be imported into VISSIM for the purpose of simulation verification, i.e., to verify whether VISSIM can simulate the traffic on these intersections. Please see the following chapters for the details of simulation verifications.

2.2.1 Testbed of FM 528

Three consecutive signalized intersections are in the testbed (FM 528) (see Figure 2.1.1). From west to east, they are Desota St.(A), Friendswood Lake Blvd.(B), and Falcon Ridge Blvd./ Brian Creek Dr.(C) respectively. In the following, the signal plan at each intersection will be reviewed.

2.2.1.1 Signal plan at Intersection A (Desota St.)

At Intersection A, a dual-ring six-phase signal plan is applied to control vehicles approaching the intersection. Figure 2.2.1 shows the phase numbers associated with vehicle movements. In addition, loop detectors associated with each phase are also listed in Figure 2.2.1. The loop detectors are marked as 'D' associated with a number, e.g., D4 refers to the fourth loop detector installed at this intersection. From Figure 2.2.1, it can be seen that, Phase 2 is set for Approach 1. Phase 2 represents the through traffic from west to east, and four detectors (Detectors 1, 2, 3, and 10 denoted by 'D1', 'D2', 'D3' and 'D10', respectively) were installed to detect vehicle arrivals during this phase. Phase 5 represents left-turn from FM 528 to Desota St. There is only one detector (D5) installed to detect vehicle arrivals for Phase 5. The detailed settings of phases and detectors at Intersection A is reported in Table 2.2.1.

Table 2.2.2 shows the signal timing plan applied to this intersection (note that the time is measured in seconds). It provides the values of minimum and maximum green times, vehicle extension, yellow and red clearances of each phase and pedestrian clearance time. In this intersection, Phases 2 and 6 represent the through traffic on the major road (westbound and eastbound, respectively). As the traffic volumes on FM 528 are much larger than other approaches, the maximum green times for these two phases is much larger than others. This intersection does not allow pedestrian crossing from any approach. Regarding to the split preference and the coordination of this intersection (see Table 2.2.3), Phases 2 and 6 are allocated with the highest splits (67/90 and 52/90, respectively) due to their high volumes (with an offset of 40 seconds).

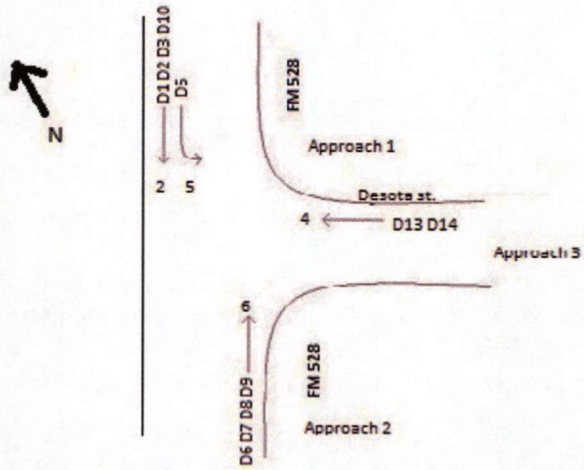


Figure 2.2.1 *Specification of approaches, phases, and detectors of the intersection at Desota St.*

Table 2.2.1 Approach, phase, and detector number distributions of the intersection at Desota St.

Approach Number	Phase Number	Detector Number
1	2	1, 2, 3, 10
	5	5
2	6	6, 7, 8, 9
3	4	13, 14

Table 2.2.2 Signal timing plan of the intersection at Desota St. (in seconds)

Phase	1	2	3	4	5	6
Minimum green time	0	15	0	7	5	15
Maximum green time	0	35	0	20	15	35
Vehicle Extension time	2.0	2.0	2.0	2.0	3.0	2.0
Yellow time	3.0	4.5	3	3.5	4.5	4.5
Red clearance time	0	1.5	0	2.5	1.5	1.5
Pedestrian clearance time	0	0	0	0	0	0

Table 2.2.3 Split pattern of the intersection at Desota St. (in seconds)

Phase	1	2	3	4	5	6
Split	0	67	0	23	15	52

2.2.1.2 Signal plan at Intersection B (Friendswood Lake Blvd.)

Friendswood Lake Blvd. at FM 528 is the second intersection, where a dual-ring eight-phase plan is applied to control traffic. Figure 2.2.2 shows the signal plan with phase numbers and their associated loop detectors, and Table 2.2.4 reports the detailed settings of phases and detectors at this intersection.

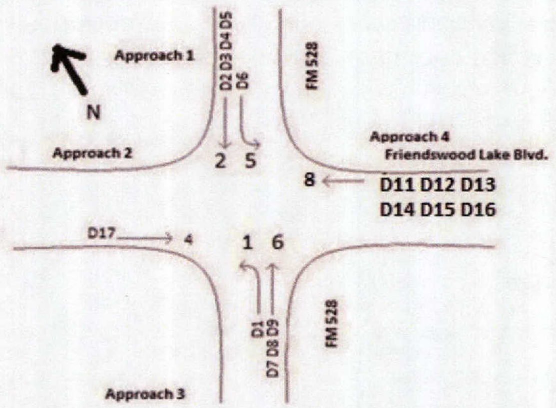


Figure 2.2.2 *Specification of approaches, phases, and detectors of the intersection at Friendswood Lake*

Table 2.2.5 provides the detailed information of minimum and maximum green times, vehicle extension, yellow, red clearance, walk and pedestrian clearance of each phase. Phases 2 and 6, associated with through traffic on the major road, are given the highest values of the maximum green times of 35 seconds. Intersection B allows pedestrian crossing from only three approaches.

Table 2.2.4 Approach, phase, and detector number distributions of the intersection at Friendswood Lake

Approach Number	Phase Number	Detector Number
1	2	2, 3, 4, 5
	5	6
2	4	17
3	1	1
	6	7, 8, 9, 10
4	**8**	11, 12, 13, 14, 15, 16

Table 2.2.5 Signal timing plan of the intersection at Friendswood Lake Blvd. (in seconds)

Phase	1	2	4	5	6	8
Minimum green time	5	15	7	5	15	7
Maximum green time	15	35	20	15	35	20
Vehicle Extension time	2.0	2.0	2.0	2.0	2.0	2.0
Yellow time	4.5	4.5	3.5	4.5	4.5	3.5
Red clearance time	2.5	2.5	2.5	2.5	2.5	2.5
Pedestrian clearance time	0	5	0	0	45	24

Table 2.2.6 Split pattern of the intersection at Friendswood Lake Blvd. (in seconds)

Phase	1	2	4	5	6	8
Split	15	52	23	15	52	23

Regarding to the split preference and the signal coordination, Phases 2 and 6 are given the largest splits to release volumes on the major road, and the offset is set to be 80 seconds. Table 2.2.6 reports the detailed split pattern information for each phase at this intersection.

2.2.1.3 Signal plans at Intersection C (Falcon Ridge Blvd./Briar Creek Dr.)

At Intersection C, a dual-ring six-phase plan is applied to control traffic. Figure 2.2.3 shows the signal plan with phase numbers and their associated loop detectors at this intersection, and Table 2.2.4 reports the detailed settings of phases and detectors at Intersection B.

Table 2.2.8 provides the detailed information of minimum and maximum green times, vehicle extension, yellow, red clearance, walk and pedestrian clearance of each phase. For Phases 2 and 6, which are associated with through traffic on the major road, the highest values of the maximum green times are allocated. Intersection C allows pedestrian crossing from all four approaches.

Regarding to the split preference and the signal coordination, Phases 2 and 6 are allocated the largest splits to release volumes on the major road, and the offset is set to be 45 seconds. Table 2.2.9 reports the detailed split pattern information for each phase at this intersection.

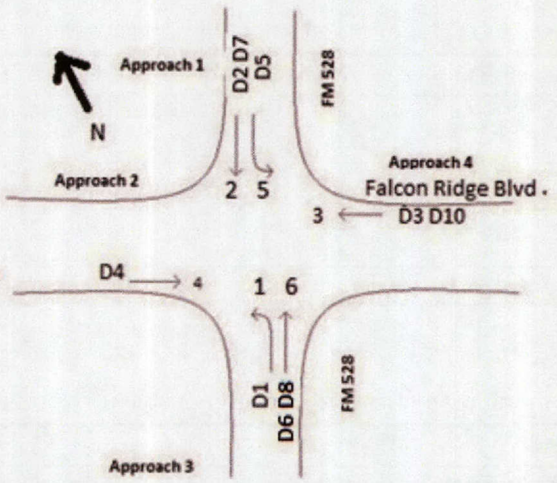


Figure 2.2.3 *Specification of approaches, phases, and detectors of the intersection at Falcon Ridge Blvd.*

Table 2.2.7 Approach, phase, and detector number distributions of the intersection at Falcon Ridge Blvd.

Approach Number	Phase Number	Detector Number
1	2	2, 7
	5	5
2	4	4
3	1	1
	6	6, 8
4	3	3, 10

Table 2.2.8 Signal timing plan of the intersection at Falcon Ridge Blvd. (in seconds)

Phase	1	2	3	4	5	6
Minimum green time	5	15	7	7	5	15
Maximum green time	15	35	20	20	15	35
Vehicle Extension time	2.0	2.0	3.5	3.5	3.0	2.0
Yellow time	4.5	4.5	3.5	4.5	4.5	3
Red clearance time	2	2	2.5	2.5	2	2
Pedestrian clearance time	0	13	22	23	0	23

Table 2.2.9 Split pattern of the intersection at Falcon Ridge Blvd. (in seconds)

Phase	1	2	3	4	5	6
Split	15	33	21	21	15	33

2.2.2 Testbed of SH 242

Eight consecutive signalized intersections are located in the testbed of SH 242. From west to east, they are Greenbridge Dr. (I), Alden Woods (A), Gosling Road (B), Fellowship Dr. (C), Honor Roll Dr. (D), Achievement Dr. (E), Montgomery Dr. (F) and Windsor Lakes Blvd. (G), respectively (see Figure 2.1.3). Note that Greenbridge Dr. is used to replace Intersection at North Freeway. The detailed explanation is given in Section 2.2.1. In the following, the signal plan at each intersection will be reviewed.

2.2.2.1 SH 242 Signal plan at Intersection A (Alden Woods)

For Intersection A, a dual-ring six-phase signal plan is applied to control vehicles approaching the intersection. Figure 2.2.4 shows the phase numbers associated with vehicle movements. In addition, loop detectors associated with each phase are also listed in Figure 2.2.4 Similarly, the loop detectors are marked as 'D' associated with a number, e.g., D4 refers to the fourth loop detector installed at this intersection.

From Figure 2.2.4, it can be seen that two phases, Phases 1 and 6, are set for Approach 2. Phase 2 represents the through traffic from east to west, and two detectors (Detectors 1 and 6, denoted by 'D1' and 'D6'. respectively) were installed to detect vehicle arrivals during this phase. Phase 1 represents left-turn from SH 242 to Alden Woods. There is only one detector (D1) installed to detect vehicle arrivals for Phase 1. The detailed settings of phases and detectors at Intersection A is reported in Table 2.2.10.

Table 2.2.11 shows the signal timing plan applied to Intersection A (note that the time is measured in seconds). It provides the values of minimum and maximum green times, vehicle extension, yellow and red clearances of each phase and pedestrian clearance time. In this intersection, Phases 2 and 6 represent the through traffic on the major road (westbound and eastbound, respectively). As the traffic volumes on the major road are much larger than other approaches, the maximum green times set for these two phases is much larger than others. In addition, since the speed limits on the major road are larger than the minor roads, the green extension times of the two phases are smaller than others. This intersection allows pedestrian crossing from all four approaches.

Regarding to the split preference and the coordination of this intersection (see Table 2.2.12), Phases 2 and 6 are allocated with the highest splits (77/135 and 77/135, respectively) due to their high volumes. The offset of the signal is set to 0 second.

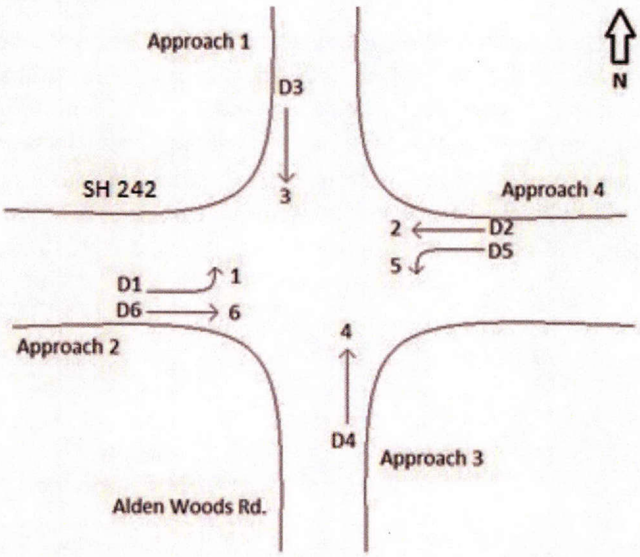


Figure 2.2.4 *Specification of approaches, phases, and detectors of the intersection at Alden Woods*

Table 2.2.10 Approach, phase, and detector number distributions of the intersection at Alden Woods

Approach Number	Phase Number	Detector Number
1	3	3
2	1	1
	6	6
3	4	4
4	2	2
	5	5

Table 2.2.11 Signal timing plan of the intersection at Alden Woods (in seconds)

Phase	1	2	3	4	5	6
Minimum green time	5	15	8	8	5	15
Maximum green time	30	40	25	25	20	40
Vehicle Extension time	2.0	2.0	2.0	2.0	2.0	2.0
Yellow time	5.0	5.0	4.0	3.5	5.0	5.0
Red clearance time	1.5	1.5	3.0	3.5	1.5	1.5
Pedestrian clearance time	0	21	17	17	0	24

Table 2.2.12 Split pattern of the intersection at Alden Woods (in seconds)

Phase	1	2	3	4	5	6
Split	18	77	20	20	18	77

2.2.2.2 Signal plan at Intersection B (Gosling Road)

Similar to Intersection A, a dual-ring six-phase signal plan is applied to Intersection B, where Gosling Road meets SH 242. Figure 2.2.5 shows the phase numbers at different approaches, as well as the loop detectors associated with each phase. The detailed information of phases and detectors at each approach is shown in Table 2.2.13.

Table 2.2.14 reports the detailed signal timing plan at Intersection B. Similarly, as Phases 2 and 6 represent the through traffic on the major road, their maximum green times are much larger than others. Intersection B allows pedestrian crossing from three approaches—1, 2, and 4. Based on the widths of the roads, the pedestrian clearance is 25 seconds on the major road, and up to 17 seconds on the minor road.

Regarding to the split preference and the signal coordination, Phases 2 and 6 are assigned with the highest splits to clear high volumes on the major road, and the offset is set to be 75 seconds. Table 2.2.15 reports more details about the split pattern information for each phase at this intersection.

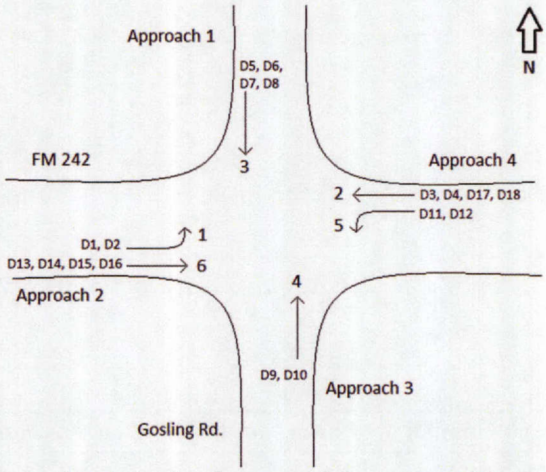


Figure 2.2.5 *Specification of approach, phase, and detectors of the intersection at Gosling Rd.*

Table 2.2.13 Approach, phase, and detector number distributions of the intersection at Gosling Rd.

Approach Number	Phase Number	Detector Number
1	3	5, 6, 7, 8
2	1 6	1, 2 13, 14, 15 ,16
3	4	9, 10
4	2 5	3, 4, 17, 18 11, 12

Table 2.2.14 Signal timing plan of the intersection at Gosling Rd. (in seconds)

Phase	1	2	3	4	5	6
Minimum green time	5	15	8	8	10	15
Maximum green time	20	40	25	25	25	40
Vehicle Extension time	2.0	2.0	2.0	2.0	2.0	1.7
Yellow time	5.0	5.0	3.5	4.5	5.0	5.0
Red clearance time	1.5	1.5	3.5	3.0	1.5	1.5
Pedestrian clearance time	0	25	13	0	0	17

Table 2.2.15 Split pattern of the intersection at Gosling Rd. (in seconds)

Phase	1	2	3	4	5	6
Split	18	60	20	27	53	35

2.2.2.3 Signal plan at Intersection C (Fellowship Dr.)

At Intersection C, where Fellowship Dr. meets SH 242, a dual-ring six-phase signal plan is applied to control traffic. Figure 2.2.6 shows the signal plan with phase numbers and their associated loop detectors at this intersection, and Table 2.2.16 reports the detailed information of phases and detectors at each approach.

Table 2.2.17 provides the detailed information of minimum and maximum green times, vehicle extension, yellow, red clearance, walk and pedestrian clearance of each phase. For the two phases associated with through traffic on the major road, the highest values of the maximum green times are assigned. Intersection C allows pedestrian crossing from only one approach—Approach 3.

Regarding to the split preference and the signal coordination, Phases 2 and 6 are allocated the largest splits to release volumes on the major road, and the offset is set to be 80 seconds. Table 2.2.18 reports the detailed split pattern information for each phase at this intersection.

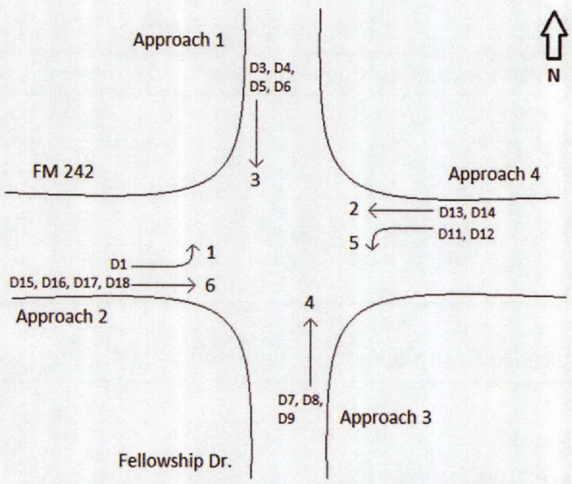


Figure 2.2.6 *Specification of approach, phase, and detectors of the intersection at Fellowship Dr.*

Table 2.2.16 Approach, phase, and detector number distributions of the intersection at Fellowship Dr.

Approach Number	Phase Number	Detector Number
1	3	3, 4, 5, 6
2	1	1
	6	15, 16, 17, 18
3	4	7, 8, 9
4	2	13, 14
	5	11, 12

Table 2.2.17 Signal timing plan of the intersection at Fellowship Dr. (in seconds)

Phase	1	2	3	4	5	6
Minimum green time	5	15	8	8	5	15
Maximum green time	20	40	25	25	20	40
Vehicle Extension time	2.0	2.0	2.0	2.0	2.0	2.0
Yellow time	5.0	5.0	3.5	3.5	5.0	5.0
Red clearance time	1.5	1.5	3.5	3.5	1.5	1.5
Pedestrian clearance time	0	0	17	0	0	0

Table 2.2.18 Split pattern of the intersection at Fellowship Dr. (in seconds)

Phase	1	2	3	4	5	6
Split	18	72	20	25	18	72

2.2.2.4 Signal plan at Intersection D (Honor Roll Dr.)

Intersection D, where Honor Roll Dr. meets SH 242, also has a dual-ring six-phase signal plan implemented. Figure 2.2.7 shows the phase numbers associated with loop detectors at this intersection, and Table 2.2.19 reports the detailed information of phases and detectors of each approach.

Table 2.2.20 provides the detailed information of minimum and maximum green times, vehicle extension, yellow, red clearance of each phase. Similarly, Phases 2 and 6 are allocated with the maximum amount of minimum and maximum green times, because they cover the traffic movement on the major road (SH 242). Table 2.2.21 reports the split preference and the signal coordination at Intersection D. Still, Phases 2 and 6 are assigned with the highest splits. The cycle length and offset value are 135 seconds and 70 seconds, respectively at this intersection.

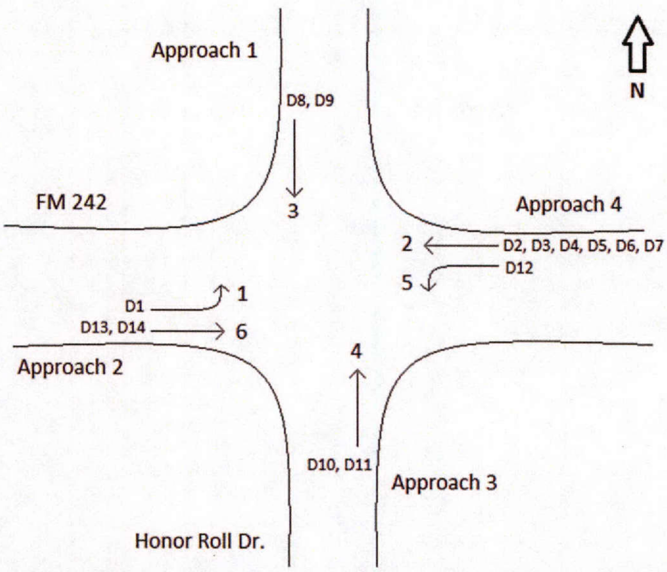


Figure 2.2.7 *Specification of approach, phase, and detectors of the intersection at Honor Roll Dr.*

Table 2.2.19 Approach, phase, and detector number distributions of the intersection at Honor Roll Dr.

Approach Number	Phase Number	Detector Number
1	3	8, 9
2	1	1
	6	13, 14
3	4	10, 11
4	2	2, 3, 4, 5, 6, 7
	5	12

Table 2.2.20 Signal timing plan of the intersection at Honor Roll Dr. (in seconds)

Phase	1	2	3	4	5	6
Minimum green time	5	15	8	8	5	15
Maximum green time	20	40	25	25	20	40
Vehicle Extension time	2.0	2.0	2.0	2.0	2.0	2.0
Yellow time	5.0	5.0	3.5	3.5	5.0	5.0
Red clearance time	1.5	1.5	3.5	3.5	1.5	1.5
Pedestrian clearance time	0	0	17	0	0	0

Table 2.2.21 Split pattern of the intersection at Honor Roll Dr. (in seconds)

Phase	1	2	3	4	5	6
Split	20	60	35	20	20	55

2.2.2.5 Signal plan at Intersection E (Achievement Dr.)

Intersection E at Achievement Dr. also has a dual-ring six-phase signal plan applied. Figure 2.2.8 shows the signal plan with phase numbers and their associated loop detectors at this intersection.

Table 2.2.22 provides the detailed information of minimum and maximum green times, vehicle extension, yellow, red clearance, walk and pedestrian clearance of each phase. For Phases 2 and 6, associated with through traffic on the major road, the highest values of the maximum green times of 40 seconds are assigned. This intersection allows pedestrian crossing from all four approaches.

Regarding to the split preference and the signal coordination, Phases 2 and 6 are allocated the largest splits to release volumes on the major road, and the cycle length is 135 seconds. Table 2.2.24 reports the detailed split pattern information for each phase at this intersection.

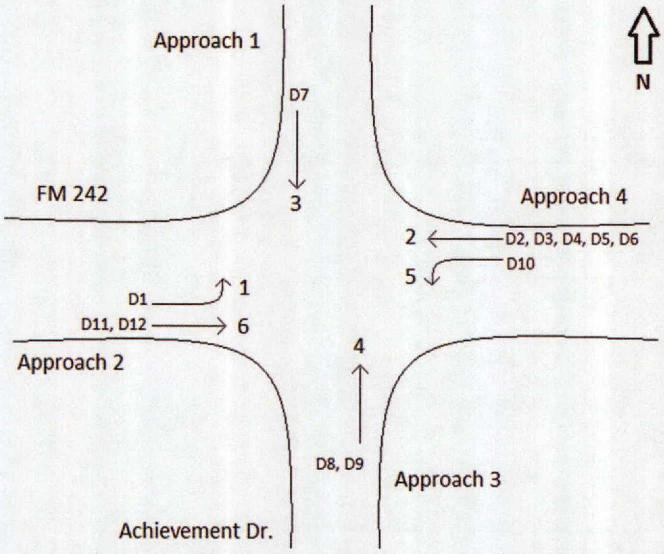


Figure 2.2.8 Specification of approach, phase, and detectors of the intersection at Achievement Dr.

Table 2.2.22 Approach, phase, and detector number distributions of the intersection at Achievement Dr.

Approach Number	Phase Number	Detector Number
1	3	7
2	1 6	1 11, 12
3	4	8, 9
4	2 5	2, 3, 4, 5, 6 10

Table 2.2.23 Signal timing plan of the intersection at Achievement Dr. (in seconds)

Phase	1	2	3	4	5	6
Minimum green time	5	15	8	8	5	15
Maximum green time	20	40	25	25	20	40
Vehicle Extension time	2.0	2.0	3.5	2.0	2.0	2.0
Yellow time	5.0	5.0	3.5	3.5	5.0	5.0
Red clearance time	1.5	1.5	3.5	3.5	1.5	1.5
Pedestrian clearance time	0	19	16	20	0	20

Table 2.2.24 Split pattern of the intersection at Achievement Dr. (in seconds)

Phase	1	2	3	4	5	6
Split	25	60	25	20	25	65

2.2.2.6 Signal plan at Intersection F (Montgomery Dr.)

At Intersection F, where Montgomery Dr. meets SH 242, a dual-ring six-phase signal plan is applied to control traffic. Figure 2.2.9 shows the phase numbers at different approaches, as well as the loop detectors associated with each phase, and Table 2.2.25 reports the detailed information of phases and detectors at each approach. Intersection F allows pedestrian crossing from three approaches: 1, 2, and 4.

Table 2.2.26 provides the detailed information of minimum and maximum green times, vehicle extension, yellow, red clearance, walk and pedestrian clearance of each phase. For two phases associated with through traffic on the major road, the highest values of the maximum green times are assigned.

Similar to previous intersections, Phases 2 and 6 are allocated the largest splits to release volumes on the major road. The cycle length and offset value are 135 seconds and 5 seconds, respectively at this intersection. Table 2.2.27 reports the detailed split pattern information for each phase at this intersection.

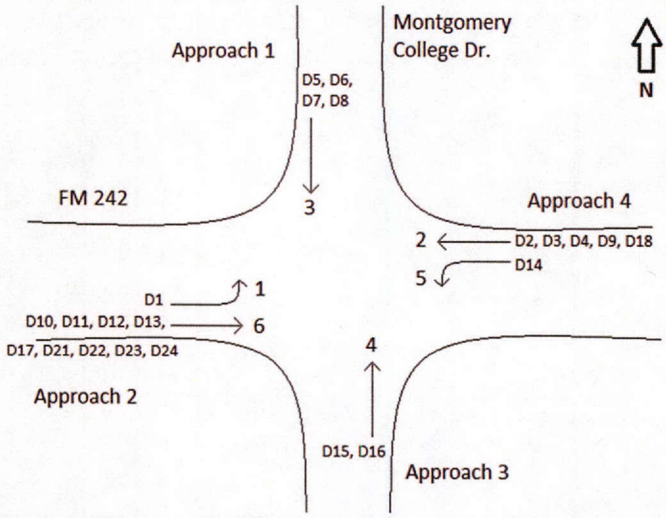


Figure 2.2.9 *Specification of approach, phase, and detectors of the intersection at Montgomery Dr.*

Table 2.2.25 Approach, phase, and detector number distributions of the intersection at Montgomery Dr.

Approach Number	Phase Number	Detector Number
1	3	5, 6, 7, 8
2	1	1
	6	10,11,12,13,17, 21,22,23,24
3	4	15, 16
4	2	2, 3, 4, 9, 18
	5	14

Table 2.2.26 Signal timing plan of the intersection at Montgomery Dr. (in seconds)

Phase	1	2	3	4	5	6
Minimum green time	5	15	8	8	5	15
Maximum green time	20	40	25	25	20	40
Vehicle Extension time	2.0	2.0	2.5	2.0	2.0	2.0
Yellow time	5.0	5.0	3.5	3.5	5.0	5.0
Red clearance time	1.5	1.5	3.5	3.5	1.5	1.5
Pedestrian clearance time	0	24	20	0	0	18

Table 2.2.27 Split pattern of the intersection at Montgomery Dr. (in seconds)

Phase	1	2	3	4	5	6
Split	18	70	22	25	25	63

2.2.2.7 Signal plan at Intersection G (St. Lukes Way)

The east-most intersection of interest is where St. Lukes Way meets SH 242. At this intersection, a dual-ring six-phase signal plan is applied to control traffic. Figure 2.2.10 shows the phase numbers at different approaches, as well as the loop detectors associated with each phase, and Table 2.2.28 reports the detailed information of phases and detectors at each approach. Intersection F allows pedestrian crossing from all four approaches.

Table 2.2.29 provides the detailed information of minimum and maximum green times, vehicle extension, yellow, red clearance, walk and pedestrian clearance of each phase. Phases 2 and 6 that are associated with through traffic on the major road, are given the highest values of the maximum green times of 40 seconds. Similar to previous intersections, Phases 2 and 6 are allocated the largest splits to release volumes on the major road. The cycle length and offset value are 135 seconds and 5 seconds, respectively at this intersection. Table 2.2.30 reports the detailed split pattern information for each phase.

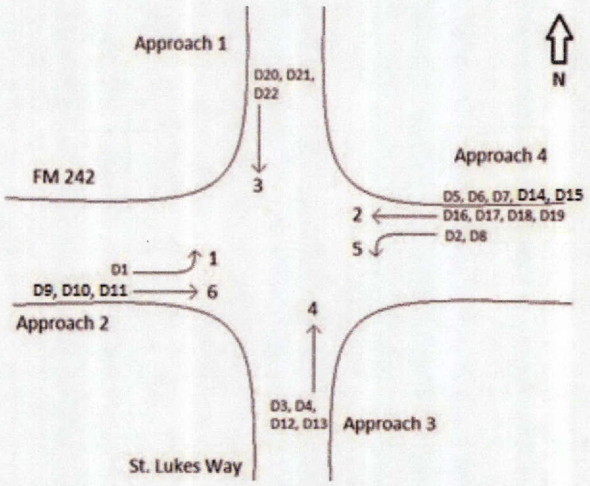


Figure 2.2.10 *Specification of approach, phase, and detectors of the intersection at St. Lukes Way*

Table 2.2.28 Approach, phase, and detector number distributions of the intersection at St. Lukes Way

Approach Number	Phase Number	Detector Number
1	3	20, 21, 22
2	1	1
	6	9, 10, 11
3	4	3, 4, 12, 13
4	2	5, 6, 7,14, 15, 16, 17, 18, 19
	5	2, 8

Table 2.2.29 Signal timing plan of the intersection at St. Lukes Way (in seconds)

Phase	1	2	3	4	5	6
Minimum green time	5	15	8	8	5	15
Maximum green time	20	40	25	25	20	40
Vehicle Extension time	2.0	2.0	2.0	2.0	2.0	2.0
Yellow time	5.0	5.0	3.5	3.5	5.0	5.0
Red clearance time	1.5	1.5	3.5	3.5	1.5	1.5
Pedestrian clearance time	0	25	20	22	0	28

Table 2.2.30 Split pattern of the intersection at St. Lukes Way (in seconds)

Phase	1	2	3	4	5	6
Split	35	50	25	25	32	53

2.2.2.8 Signal plan at Intersection I (Green Bridge Dr.)

This is the only T intersection in the testbed of SH 242. This intersection is added after the discussion in the Kick-off meeting, because the originally selected intersection at IH45 has different properties from other intersections at SH 242. At Intersection I, a dual-ring five-phase signal plan is applied to control traffic. Figure 2.2.11 shows the signal plan with phase numbers and their associated loop detectors at this intersection, and Table 2.2.31 reports the detailed information of phases and detectors at each approach.

Table 2.2.32 provides the detailed information of minimum and maximum green times, vehicle extension, yellow, red clearance, walk and pedestrian clearance of each phase.

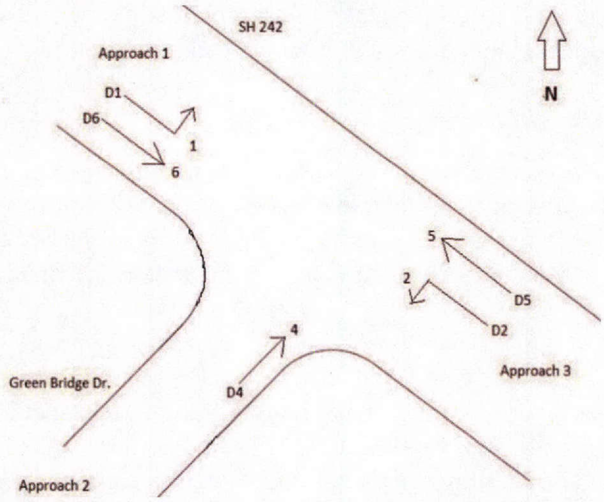


Figure 2.2.11 *Specification of approach, phase, and detectors of the intersection at Green Bridge Dr.*

For three phases—Phases 2, 5, and 6, associated with through traffic on the major road—the highest values of the maximum green times of 60 seconds are assigned. However, this intersection does not allow pedestrian crossing from any approach. On the other hand, the cycle length and offset value are 135 seconds and 5 seconds, respectively. Table 2.2.33 reports the detailed split pattern information for each phase at this intersection.

Table 2.2.31 Approach, phase, and detector number distributions of the intersection at Green Bridge Dr.

Approach Number	Phase Number	Detector Number
1	1	1
	6	6
2	4	4
3	2	2
	5	5

Table 2.2.32 Signal timing plan of the intersection at Green Bridge Dr. (in seconds)

Phase	1	2	4	5	6
Minimum green time	5	20	0	5	20
Maximum green time	10	60	25	60	60
Vehicle Extension time	2.0	3.0	2.0	3.0	3.0
Yellow time	5.1	5.1	4.3	5.1	5.1
Red clearance time	1.5	1.5	2.1	1.5	1.5
Pedestrian clearance time	0	0	0	0	0

Table 2.2.33 Split pattern of the intersection at Green Bridge Dr. (in seconds)

Phase	1	2	4	5	6
Split	15	35	20	20	30

2.2.3 Testbed of FM 1464

The testbed of FM 1464 has 11 consecutive signalized intersections: W. Oaks Village Dr.(A), Bellaire Blvd.(B), Highland Oak Ln/ Watering Oaks Ln(C), Orchid Ridge Ln(D), Beechnut St (E), Bissonnet Blvd (F), W. Bellfort St.(G), W. Airport Blvd(H), Stephen F Austin High School (I), Old Richmond Rd(J), and Old Orchard Dr./Orchard Lake Estates Dr. (K), listed from north to south (see Figure 2.1.5).

As discussed in Section 2.3.1, Intersection F is not ready yet, and its signal plan is unknown. Therefore, only 10 intersections' signal plans will be reviewed in the following.

2.2.3.1 Signal plan at Intersection A (W. Oaks Village Dr.)

At Intersection A, a dual-ring six-phase signal plan is applied to control vehicles. Figure 2.2.12 shows the phase numbers associated with vehicle movements. In addition, loop detectors associated with each phase are also listed in Figure 2.2.12. Similar to the setting in the previous two testbeds, the loop detectors are marked as 'D' associated with a number, e.g., D1 means the first loop detector installed at this intersection.

From Figure 2.2.12, it is seen that Phases 2 and 6 represent the through traffic from south to north or from north to south, and Detectors 1 and 2 (denoted by 'D1' and 'D2') were installed to detect vehicle arrivals during these two phases. On the other hand, Phases 1 and 5 represent left-turn from the major road (FM 1464) towards the minor road, and two detectors, D9 and D10, were installed to detect vehicle arrivals for these two phases. The detailed settings of phases and detectors are reported in Table 2.2.34.

Table 2.2.35 shows the signal timing plan applied to this intersection. It provides the values of minimum and maximum green times, vehicle extension, yellow and red clearances of each phase and pedestrian clearance time. As the traffic volumes on the major road are much larger than other approaches, the maximum green times for these two phases are much larger than others.

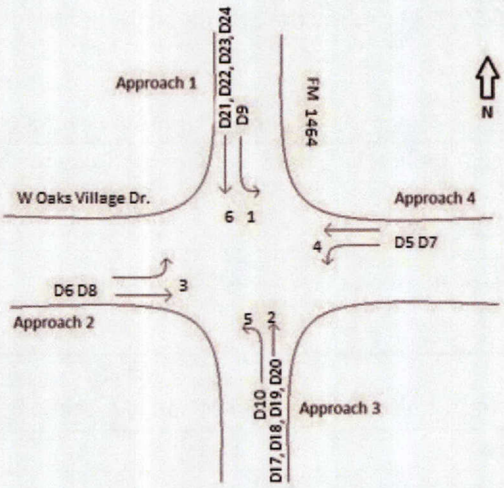


Figure 2.2.12 *Specification of approaches, phases, and detectors of the intersection at W. Oaks Village*

Table 2.2.34 Approach, phase, and detector number distributions of the intersection at W. Oaks Village

Approach Number	Phase Number	Detector Number
1	1	9
	6	2122,23,24
2	3	6, 8
3	2	17,1819,20
	5	10
4	4	5, 7

Table 2.2.35 Signal timing plan of the intersection at W. Oaks Village Dr. (in seconds)

Phase	1	2	3	4	5	6
Minimum green time	5	15	7	7	5	15
Maximum green time	15	35	25	25	15	35
Vehicle Extension time	2.0	2.0	2.0	2.0	2.0	2.0
Yellow time	5.0	5.0	4.0	4.0	5.0	5.0
Red clearance time	1.5	1.5	3.0	2.5	1.5	1.5
Pedestrian clearance time	0	24	0	31	0	25

Regarding to the split preference and the coordination of this intersection (see Table 2.2.36), Phases 2 and 6 are allocated with the highest splits (52/105 and 52/105, respectively) due to their high volumes. The offset of the signal is set to 0 second.

Table 2.2.36 Split pattern of the intersection at W. Oaks Village Dr. (in seconds)

Phase	1	2	3	4	5	6
Split	15	52	20	18	15	52

2.2.3.2 Signal plan at Intersection B (Bellaire Blvd.)

The second intersection (Bellaire Blvd. at FM 1464) has a dual-ring eight-phase signal plan. Figure 2.2.13 shows the phase numbers at different approaches, as well as the loop detectors associated with each phase.

Table 2.2.37 reports the detailed information of phases and detectors at each approach. Compared with Intersection A, it seems that the minor road (Bellaire Blvd.) at this intersection has more traffic, as it has separate phases for left-turn traffic (Phases 3 and 7). Table 2.2.38 reports the detailed signal timing plan at Intersection B. Similar to Intersection A, Phases 2 and 6 represent the through traffic on the major road, and both these phases are assigned with a maximum green time of 35 seconds. This intersection allows pedestrian crossings from all four approaches. Based on the widths of road, the pedestrian clearance is 30 seconds on the major road, and up to 29 seconds on the minor road.

Regarding to the split preference and the signal coordination, Phases 2 and 6 are assigned with the highest splits to clear high volumes on the major road, and the offset is set to be 0 seconds. Table 2.2.39 reports more details about the split pattern information for each phase at this intersection.

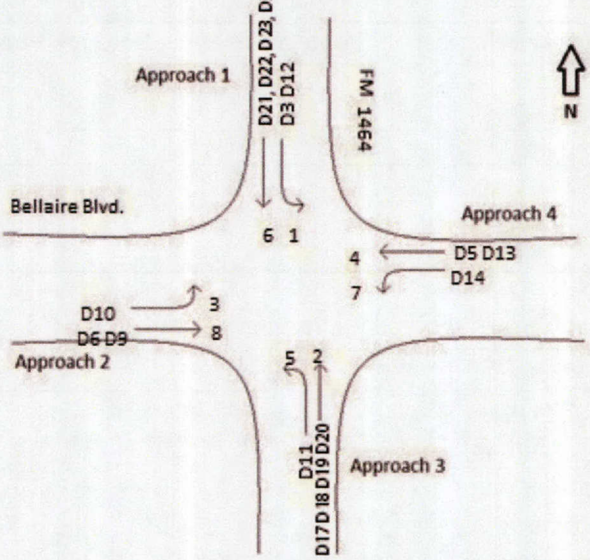


Figure 2.2.13 *Specification of approaches, phases, and detectors of the intersection at Bellaire Blvd.*

Table 2.2.37 Approach, phase, and detector number distributions of the intersection at Bellaire Blvd.

Approach Number	Phase Number	Detector Number
1	1	3, 12
	6	21,22,23,24
2	3	10
	8	6, 9
3	2	17,1819,20
	5	11
4	4	5, 13
	7	14

Table 2.2.38 Signal timing plan of the intersection at Bellaire Blvd. (in seconds)

Phase	1	2	3	4	5	6	7	8
Minimum green time	5	15	5	7	5	15	5	7
Maximum green time	15	35	15	25	15	35	15	25
Vehicle Extension time	2.0	2.0	2.0	2.0	2.0	2.0	2	2
Yellow time	5	5	4	4	5	5	4	4
Red clearance time	1.5	1.5	3	3	1.5	1.5	3	3
Pedestrian clearance time	0	30	0	29	0	24	0	29

Table 2.2.39 Split pattern of the intersection at Bellaire Blvd. (in seconds)

Phase	1	2	3	4	5	6	7	8
Split	15	55	15	20	15	55	15	20

2.2.3.3 Signal plan at Intersection C (Highland Oak Ln/ Watering Oaks Ln.)

Intersection C also has a dual-ring six-phase signal plan applied. Figure 2.2.14 shows the phase numbers at different approaches, as well as the loop detectors associated with each phase, and Table 2.2.40 reports the detailed information of phases and detectors at each approach.

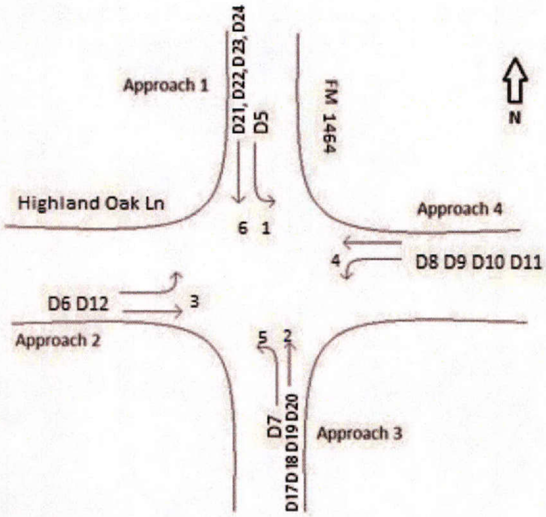


Figure 2.2.14 *Specification of approaches, phases and detectors of the intersection at Highland Oak Ln.*

Table 2.2.41 reports the detailed signal timing plan at Intersection C. Similar to Intersection B, as Phases 2 and 6 represent the through traffic on the major road, their maximum green times are much larger than others. This intersection allows pedestrian crossings from all four approaches.

Regarding to the split preference and the signal coordination, Phases 2 and 6 are assigned with the highest splits to clear high volumes on the major road, and the offset is set to be 55 seconds. Table 2.2.42 reports more details about the split pattern information for each phase at this intersection.

Table 2.2.40 Approach, phase, and detector number distributions of the intersection at Highland Oak Ln.

Approach Number	Phase Number	Detector Number
1	1	5
	6	21,22,23,24
2	3	6,12
3	2	1
	5	17,18,19,20
4	4	8, 9, 10, 11

Table 2.2.41 Signal timing plan of the intersection at Highland Oak Ln. (in seconds)

Phase	1	2	3	4	5	6
Minimum green time	5	15	7	7	5	15
Maximum green time	15	35	25	25	15	35
Vehicle Extension time	2.0	2.0	2.0	2.0	2.0	2.0
Yellow time	5	5	3.5	3.5	5	5
Red clearance time	1.5	1.5	3.0	3.0	1.5	1.5
Pedestrian clearance time	0	24	31	31	0	24

Table 2.2.42 Split pattern of the intersection at Highland Oak Ln. (in seconds)

Phase	1	2	3	4	5	6
Split	20	45	20	20	20	45

2.2.3.4 Signal plan at Intersection D (Orchid Ridge Ln)

Intersection D, where Orchid Ridge Ln meets FM 1464, also has a dual-ring six-phase signal plan applied. Figure 2.2.15 shows the signal plan with phase numbers and their associated loop detectors at this intersection, and Table 2.2.43 reports the detailed information of phases and detectors at each approach. It seems that the phase setting is exactly the same to that of Intersection C.

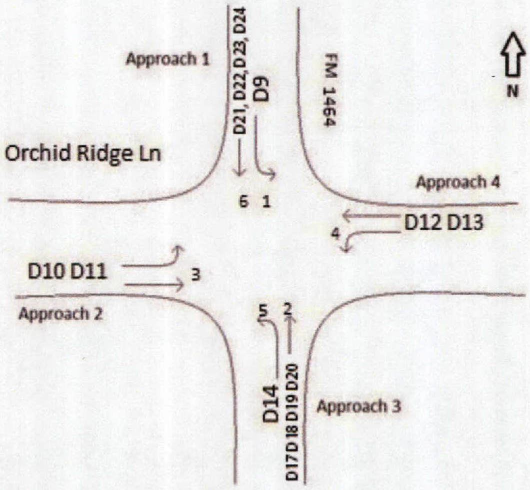


Figure 2.2.15 *Specification of approaches, phases, and detectors of the intersection at Orchid Ridge Ln.*

Table 2.2.43 Approach, phase, and detector number distributions of the intersection at Orchid Ridge Ln.

Approach Number	Phase Number	Detector Number
1	1	9
	6	21,22,23,24
2	3	10, 11
3	2	17,18,19,20
	5	14
4	4	12, 13

Table 2.2.44 provides the detailed information of minimum and maximum green times, vehicle extension, yellow, red clearance, walk and pedestrian clearance of each phase. Similarly, Phases 2 and 6, associated with through traffic on the major road, are given the highest values of the maximum green times of 35 seconds.

Regarding to the split preference and the signal coordination, Phases 2 and 6 are allocated the largest splits to release volumes on the major road, and the offset is set at 55 seconds. Table 2.2.45 reports the detailed split pattern information for each phase at this intersection.

Table 2.2.44 Signal timing plan of the intersection at Orchid Ridge Ln. (in seconds)

Phase	1	2	3	4	5	6
Minimum green time	5	15	7	7	5	15
Maximum green time	15	35	25	25	15	35
Vehicle Extension time	2.0	2.0	2.0	2.0	2.0	2.0
Yellow time	5	5	3.5	3.5	5	5
Red clearance time	1.5	1.5	3	3	1.5	1.5
Pedestrian clearance time	0	27	0	27	0	23

Table 2.2.45 Split pattern of the intersection at Orchid Ridge Ln. (in seconds)

Phase	1	2	3	4	5	6
Split	20	40	23	22	20	40

2.2.3.5 Signal plan at Intersection E (Beechnut St.)

Intersection E is located where Beechnut St. meets FM 1464. It has a dual-ring eight-phase signal plan applied. Figure 2.2.16 shows the phase numbers associated with loop detectors at this intersection, and Table 2.2.46 reports the detailed information of phases and detectors of each approach. It seems that this intersection is like Intersection B, where the left-turns at minor road are given separate phase.

Table 2.2.47 provides the detailed information of minimum and maximum green times, vehicle extension, yellow, red clearance of each phase. Similarly, Phases 2 and 6 are assigned with the maximum amount of minimum and maximum green times, as they are major through traffic. Table 2.2.48 reports the split preference and the signal coordination at Intersection F. Still, Phases 2 and 6 have been assigned with the highest splits. The cycle length and offset value are 105 seconds and 5 seconds, respectively.

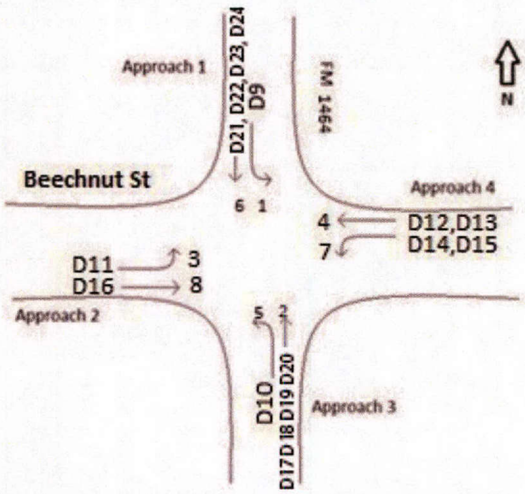


***Figure 2.2.16** Specification of approaches, phases, and detectors of the intersection at Beechnut St.*

Table 2.2.46 Approach, phase, and detector number distributions of the intersection at Beechnut St.

Approach Number	Phase Number	Detector Number
1	1	9
	6	21,22,23,24
2	3	11
	8	16
3	2	17,18,19,20
	5	10
4	4	12, 13
	7	14, 15

Table 2.2.47 Signal timing plan of at the intersection of Beechnut St. (in seconds)

Phase	1	2	3	4	5	6	7	8
Minimum green time	5	15	5	7	5	15	5	7
Maximum green time	15	35	15	25	15	35	15	25
Vehicle Extension time	2.0	2.0	2.0	2.0	2.0	2.0	2	3
Yellow time	5	5	4.5	4	5	5	4	4.5
Red clearance time	1.5	1.5	2	2.5	1.5	1.5	2.5	2
Pedestrian clearance time	0	28	0	28	0	28	0	27

Table 2.2.48 Split pattern of the intersection at Beechnut St. (in seconds)

Phase	1	2	3	4	5	6	7	8
Split	20	37	23	25	20	37	23	25

2.2.3.6 Signal plan at Intersection F (Bissonet St.)

Intersection F is located where Bissonet St. meets FM 1464. It has a dual-ring eight-phase signal plan applied. Figure 2.2.17 shows the phase numbers associated with loop detectors at this intersection, and Table 2.2.49 reports the detailed information of phases and detectors of each approach. It seems that this intersection is like Intersection B, where the left-turns at minor road are given separate phase.

Table 2.2.50 provides the detailed information of minimum and maximum green times, vehicle extension, yellow, red clearance of each phase. Similarly, Phases 2 and 6 are assigned with the maximum amount of minimum and maximum green times, because they are for through traffic on the major road. Finally, Table 2.2.51 reports the split preference and the signal coordination at this intersection.

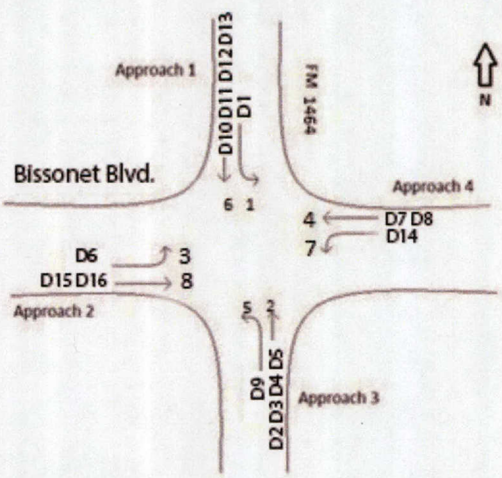


Figure 2.2.17 *Specification of approaches, phases, and detectors of the intersection at Bissonet St.*

Table 2.2.49 Approach, phase, and detector number distributions of the intersection at Bissonet St.

Approach Number	Phase Number	Detector Number
1	1	1
	6	10,11,12,13
2	3	6
	8	15,16
3	2	2,3,4,5
	5	9
4	4	7,8
	7	14

Table 2.2.50 Signal timing plan of the intersection at Bissonet St. (in seconds)

Phase	1	2	3	4	5	6	7	8
Minimum green time	5	15	5	7	5	15	5	7
Maximum green time	20	50	20	35	20	50	20	35
Vehicle Extension time	1.5	2.0	1.5	2.0	1.5	2.0	1.5	2.0
Yellow time	5	5	4	4	5	5	4	4
Red clearance time	2	2	2.5	2.5	2	2	2.5	3
Pedestrian clearance time	0	31	0	38	0	32	0	36

Table 2.2.51 Split pattern of the intersection at Bissonet St. (in seconds)

Phase	1	2	3	4	5	6	7	8
Split	0	0	0	0	0	0	0	0

2.2.3.7 Signal plan at Intersection G (W. Bellfort St.)

Intersection G is located where W. Bellfort St. meets FM 1464. A dual-ring eight-phase signal plan is also applied. Figure 2.2.18 shows the phase numbers associated with loop detectors at this intersection, and Table 2.2.52 reports the detailed information of phases and detectors of each approach. Like Intersections B and E, the intersection gives separate left-turn signal for the minor road, because the traffic on the minor road (W. Bellfort St) is relatively heavy.

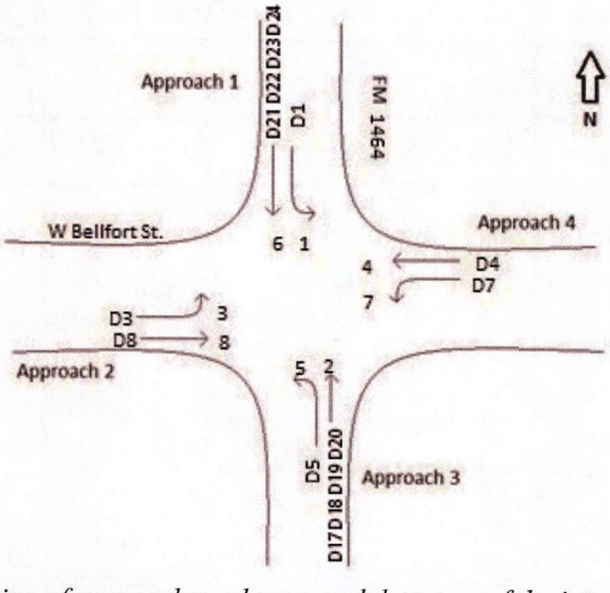


Figure 2.2.18 *Specification of approaches, phases, and detectors of the intersection at W. Bellfort St.*

Table 2.2.53 provides the detailed information of minimum and maximum green times, vehicle extension, yellow, red clearance of each phase. As to the same phase setting on the previous intersections, Phases 2 and 6 are given the highest values of maximum green times (50 seconds, respectively), much larger than others, because they are for through traffic on the major road (FM 1464).

Compared with previous intersections, we can see that the maximum green time for through traffic on the major road (FM 1464) is much larger than those for other phases. Note that at the previous intersections, the maximum green times are just 35 seconds for through traffic on the major road. It implies that the volumes of left-turn traffic on the major road toward the minor road (W. Bellfort St.), as well as those of the traffic from the minor road is much smaller than those of through traffic on the major road.

Table 2.2.54 reports the split preference at Intersection G. Interestingly, it is seen that the split values are all 0, i.e., no any split is assigned to any phase at this intersection. The reason may be due to that big difference of volumes between through traffic on the major road and other traffic movements.

Table 2.2.52 Approach, phase, and detector number distributions of the intersection at W. Bellfort St.

Approach Number	Phase Number	Detector Number
1	1	1
	6	21,22,23,24
2	3	3
	8	8
3	2	17,18,19,20
	5	5
4	4	4
	7	7

Table 2.2.53 Signal timing plan of the intersection at W. Bellfort St. (in seconds)

Phase	1	2	3	4	5	6	7	8
Minimum green time	5	15	5	7	5	15	5	7
Maximum green time	20	50	20	35	20	50	20	35
Vehicle Extension time	1.5	2.0	1.5	2.0	1.5	2.0	1.5	2.0
Yellow time	5.0	5.0	4.0	4.0	5.0	5.0	4.0	4.0
Red clearance time	2.0	2.0	2.5	2.5	2.0	2.0	2.5	2.5
Pedestrian clearance time	0	25	0	29	0	25	0	29

Table 2.2.54 Split pattern of the intersection at W. Bellfort St. (in seconds)

Phase	1	2	3	4	5	6	7	8
Split	0	0	0	0	0	0	0	0

2.2.3.8 Signal plan at Intersection H (W. Airport Blvd.)

The next intersection, Intersection H, is located at W. Airport Blvd. and FM 1464. It has a dual-ring six-phase signal plan applied. Figure 2.2.19 shows the phase numbers associated with loop detectors.

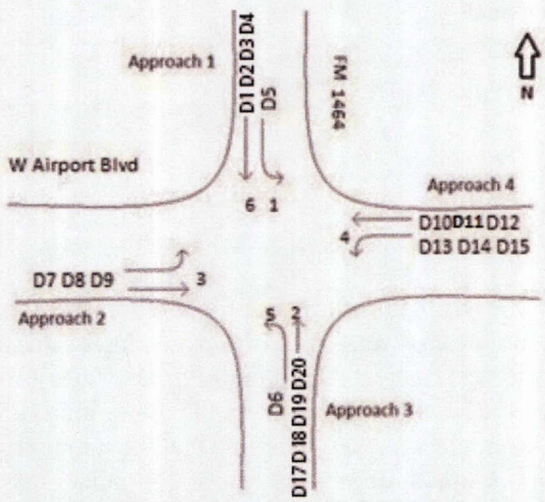


***Figure 2.2.19** Specification of approaches, phases, and detectors of the intersection at W. Airport Blvd.*

Table 2.2.55 reports the detailed information of phases and detectors of each approach, and Table 2.2.56 provides the detailed information of minimum and maximum green times, vehicle extension, yellow, red clearance of each phase. Again, Phases 2 and 6 are allocated with the maximum amount of minimum and maximum green times (50 seconds, respectively), because they are for the traffic movement on the major road (FM 1464). Like Intersection G, we can see that the difference between the maximum green times given to through traffic on the major road and to other traffic movement is large, implying the big difference between the volumes of these movements. Note that at Intersection G, left-turn traffic from the minor road to the major road is protected. However, at this intersection, such left-turn traffic is not protected, i.e., no separate phase is given.

Table 2.2.55 Approach, phase, and detector number distributions of the intersection at W. Airport Blvd.

Approach Number	Phase Number	Detector Number
1	1	5
	6	1,2,3,4
2	3	7, 8, 9
3	2	17,18,19,20
	5	6
4	4	10, 11,12, 13, 14, 15

Table 2.2.56 Signal timing plan of the intersection at W. Airport Blvd. (in seconds)

Phase	1	2	3	4	5	6
Minimum green time	5	15	8	5	5	15
Maximum green time	20	50	30	30	20	50
Vehicle Extension time	1.5	2.0	2.0	2.0	2.0	2.0
Yellow time	4.5	4.5	4	4	4.5	4.5
Red clearance time	2	2	3	3	2	2
Pedestrian clearance time	0	28	28	28	0	28

Furthermore, Table 2.2.57 reports the split preference and the signal coordination. Like Intersection G, all split values 0, i.e., no any split is assigned to any phase at this intersection. Again, the reason may be due to that big difference of volumes between through traffic on the major road and other traffic movements.

Table 2.2.57 Split pattern of the intersection at W. Airport Blvd. (in seconds)

Phase	1	2	3	4	5	6
Split	0	0	0	0	0	0

2.2.3.9 Signal plan at Intersection I (Stephen F. Austin High School.)

The next one, Intersection I, where Stephen F. Austin High School meets FM 1464, has a dual-ring six-phase signal plan applied to control traffic. Figure 2.2.20 shows the phase numbers at different approaches, as well as the loop detectors associated with each phase.

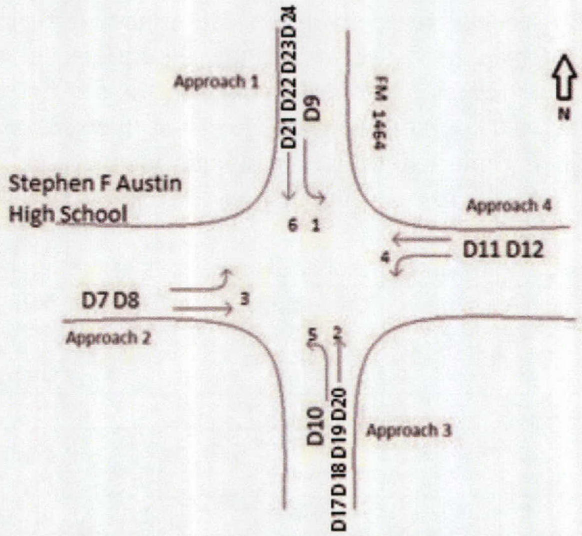


Figure 2.2.20 *Specification of approaches, phases, and detectors of the intersection at Stephen F. Austin High School*

Table 2.2.58 Approach, phase, and detector number distributions of the intersection at Stephen F. Austin High School

Approach Number	Phase Number	Detector Number
1	1	9
	6	21,22,23,24
2	3	7, 8
3	2	17,18,19,20
	5	10
4	4	11, 12

Table 2.2.59 Signal timing plan of the intersection at Stephen F. Austin High School (in seconds)

Phase	1	2	3	4	5	6
Minimum green time	5	15	7	7	5	15
Maximum green time	35	50	20	20	20	50
Vehicle Extension time	2.0	2.0	2.0	3.5	2.0	2.0
Yellow time	5	5	3.5	3.5	5	5
Red clearance time	1.5	1.5	3	2.5	1.5	1.5
Pedestrian clearance time	0	17	28	0	0	18

Table 2.2.58 reports the detailed information of phases and detectors at each approach. Table 2.2.59 provides the detailed information of minimum and maximum green times, vehicle extension, yellow, red clearance, walk and pedestrian clearance of each phase. Again, Phases 2 and 6 are given the highest values of the maximum green times (50 seconds, respectively), as they are for through traffic on the major road (FM 1464).

Finally, Table 2.2.60 reports the split preference and the signal coordination at this intersection. Still, like Intersections G and H, no any split is assigned to any phase at this intersection.

Table 2.2.60 Split pattern of the intersection at Stephen F. Austin High School (in seconds)

Phase	1	2	3	4	5	6
Split	0	0	0	0	0	0

2.2.3.10 Signal plan at Intersection J (Old Richmond Rd)

Next is a T-intersection, located at Old Richmond Rd and FM 1464. Since it is a T-intersection, it has only five phases (also dual-ring). Figure 2.2.21 shows the phase numbers at each approach, as well as the loop detectors associated with each phase.

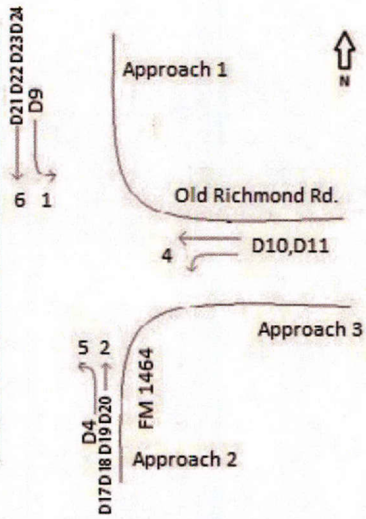


Figure 2.2.21 *Specification of approaches, phases, and detectors of the intersection at Old Richmond Rd.*

Table 2.2.61 Approach, phase, and detector number distributions of the intersection at Old Richmond Rd.

Approach Number	Phase Number	Detector Number
1	1	9
	6	21,22,23,24
2	2	17,18,19,20
	5	4
3	4	10, 11

Table 2.2.61 reports the detailed information of phases and detectors at each approach, and Table 2.2.62 provides the detailed information of minimum and maximum green times, vehicle extension, yellow, red clearance, walk and pedestrian clearance of each phase. Again, Phases 2 and 6 (for through traffic on the major road) are given the highest values of the maximum green times (50 seconds, respectively). Finally, Table 2.2.63 reports the split preference and the signal coordination at this intersection. Still, like Intersections G, H and I, no any split is assigned to any phase at this intersection.

Table 2.2.62 Signal timing plan of the intersection at Old Richmond Rd. (in seconds)

Phase	1	2	4	5	6
Minimum green time	5	15	7	5	15
Maximum green time	30	50	30	20	50
Vehicle Extension time	1.5	2	3	1.5	2.0
Yellow time	5	5	4	5	5
Red clearance time	1.5	1.5	2	1.5	1.5
Pedestrian clearance time	0	21	25	0	0

Table 2.2.63 Split pattern of the intersection at Old Richmond Rd. (in seconds)

Phase	1	2	3	4	5	6
Split	0	0	0	0	0	0

2.2.3.11 Signal plan at Intersection K (Old Orchard Dr./Orchard Lake Estates)

The last one on this testbed is the intersection where Orchard Lake Estates meets FM 1464. As usual, it has a dual-ring six-phase signal plan applied to control traffic. Figure 2.2.22 shows the phase numbers at each approach, as well as the loop detectors associated with each phase.

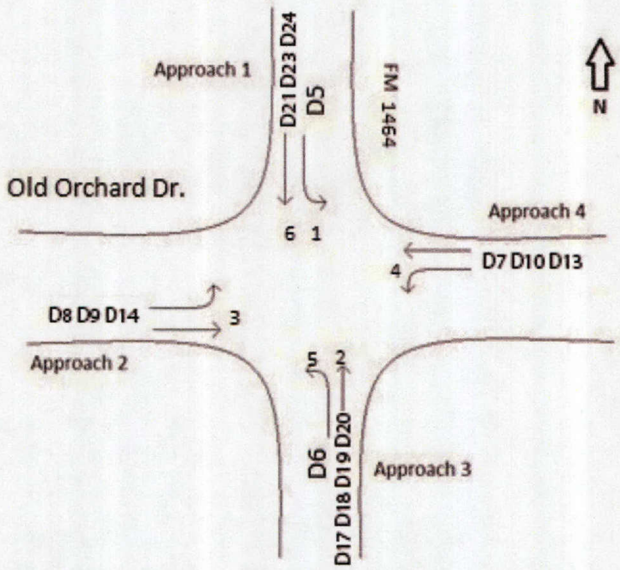


Figure 2.2.22 *Specification of approaches, phases, and detectors of the intersection at Orchard Lake Estate*

Table 2.2.64 reports the detailed information of phases and detectors at each approach. The phase setting is like Intersections A, C, D, and H, where the traffic on the minor road is relatively low, so the left-turn traffic from the minor road to the major road is not protected.

Table 2.2.65 provides the detailed information of minimum and maximum green times, vehicle extension, yellow, red clearance, walk and pedestrian clearance of each phase. Again, Phases 2 and 6, associated with through traffic on the major road, are given the highest values of the maximum green times (50 seconds, respectively). Finally, Table 2.2.66 reports the split preference and the signal coordination at Intersection K. Still, like Intersections G, H, I and J, no any split is assigned to any phase at this intersection.

Table 2.2.64 Approach, phase, and detector number distributions of the intersection at Orchard Lake Estate

Approach Number	Phase Number	Detector Number
1	1	5
	6	21,23,24
2	3	8,9,14
3	2	17,18,19,20
	5	6
4	4	7,10,13

Table 2.2.65 Signal timing plan of the intersection at Orchard Lake Estate (in seconds)

Phase	1	2	3	4	5	6
Minimum green time	5	15	7	7	5	15
Maximum green time	20	50	25	25	20	50
Vehicle Extension time	1.5	2	2	2	1.5	2.0
Yellow time	4	4	4	4	4	4
Red clearance time	2	2	2.5	2.5	2	2
Pedestrian clearance time	0	23	0	27	0	20

Table 2.2.66 Split pattern of the intersection at Orchard Lake Estate (in seconds)

Phase	1	2	3	4	5	6
Split	0	0	0	0	0	0

2.2.4 Testbeds of IH 10

As mentioned in Section 1.4, for the last eight intersections for implementation, five were substituted due to the connection problems. The final intersections used for the implementations are: Westgreen Blvd. (A), Greenhouse Rd. (B), Katy Fort Bend Rd. (C), Mason Rd. (D), Mason Access Rd (E), Fry Rd. (F), Fry Access Rd (G), and Baker Cypress (H), as shown in Figure 2.1.7. There are all along the frontage road of IH-10, except Mason Access Rd (E) and Fry Access Rd (G).

As mentioned earlier, six are diamond intersections; the intersections at Mason Access Rd. (E) and Fry Access Rd.(G) are four-way intersections. Loop detectors are only available at the intersections of Westgreen Blvd. (A), Greenhouse Rd. (B), and Katy Fort Bend Rd. (C). The other three—Mason Rd. (D), Fry Rd. (F), and Baker Cypress (H)—have video detectors only.

2.2.4.1 Signal plan at Intersection A (Westgreen Blvd. at IH-10)

The first diamond intersection is Westgreen Blvd. at the frontage road of IH-10. Figure 2.2.23 shows the signal plan with phase numbers and their associated detectors at this intersection.

The detailed settings of phases and detectors at this intersection is reported in Table 2.2.67, and Table 2.2.68 reports the detailed information of minimum and maximum green times, vehicle extension, yellow, red clearance and pedestrian clearance times of each phase. At this intersection, Phases 4, 6, and 8 are given the maximum green time (55 seconds). On the other hand, Phases 1 and 2 are in the overlapping of A (OLA) and they share the same green time. Similarly, Phases 5 and 6 are in the overlapping of B (OLB) and they share the same green time.

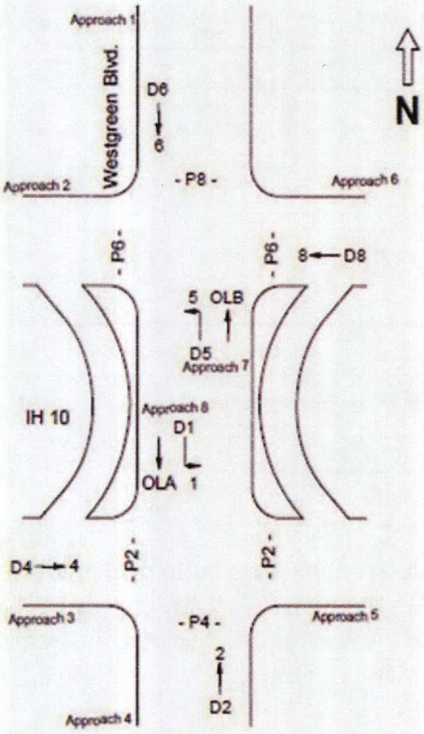


Figure 2.2.23 *Specification of approaches, phases, and detectors of the intersection at Westgreen Blvd.*

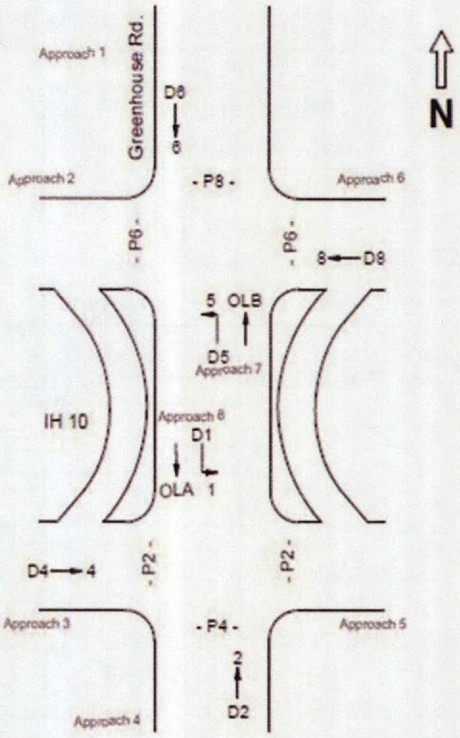


Figure 2.2.24 *Specification of approaches, phases, and detectors of the intersection at Greenhouse Rd.*

Table 2.2.67 Approach, phase, and detector number distributions of the intersection at Westgreen Blvd.

Approach Number	Phase Number	Detector Number
1	6	14,15
3	4	11,12,13
4	2	9,10
6	8	16,17,18
7	5	-
8	1	-

Table 2.2.68 Signal timing plan of the intersection at Westgreen Blvd.

Phase	2	4	6	8
Minimum green time	7	5	7	5
Maximum green time	35	55	55	55
Vehicle Extension time	2.0	2.2	2.0	2.0
Yellow time	4.0	5.0	4.0	5.0
Red clearance time	1.5	2.0	1.5	2.0
Pedestrian clearance time	15	15	15	15

2.2.4.2 Signal plan at Intersection B (Greenhouse Rd. at IH-10)

The second intersection is located at Greenhouse Rd. and the frontage road of I-10. Figure 2.2.24 depicts the signal plan with phase numbers and their associated detectors at this intersection.

This intersection's detailed settings of phases and detectors is reported in Table 2.2.69. Table 2.2.70 reports the detailed signal plan information of each phase. At this intersection, Phase 8 is given the maximum green time (50 seconds). On the other hand, Phases 1 and 2 are in the OLA and they share the same green time. Similarly, Phases 5 and 6 are in the OLB and they share the same green time.

2.2.4.3 Signal plan at Intersection C (Katy Fort Bend Rd. at IH-10)

The next intersection considered is the one located at Katy Fort Bend Rd. It is the west-most intersection at this testbed. Figure 2.2.25 depicts the signal plan with phase numbers and their associated detectors at this intersection. This intersection's detailed settings of phases and detectors are reported in Table 2.2.71. Further, Table 2.2.72 reports the detailed information of minimum and maximum green times, vehicle extension, yellow, red clearance and pedestrian clearance times of each phase.

Table 2.2.69 Approach, phase, and detector number distributions of the intersection at Greenhouse Rd.

Approach Number	Phase Number	Detector Number
1	6	14,15,16
3	4	11,12,13
4	2	9,10
6	8	17,18
7	5	-
8	1	-

Table 2.2.70 Signal timing plan of the intersection at Greenhouse Rd. (in seconds)

Phase	2	4	6	8
Minimum green time	7	5	7	5
Maximum green time	35	35	35	50
Vehicle Extension time	2.0	2.0	2.0	2.0
Yellow time	4.0	4.5	4.0	4.5
Red clearance time	1.5	2.0	1.5	2.0
Pedestrian clearance time	16	29	16	29

Table 2.2.71 Approach, phase, and detector number distributions of the intersection at Katy Fort Bend Rd.

Approach Number	Phase Number	Detector Number
1	6	15,16,17
3	4	12,13,14
4	2	9,10,11
6	8	18,19,20
7	5	-
8	1	-

Table 2.2.72 Signal timing plan of at the intersection of Katy Fort Bend Rd. (in seconds)

Phase	2	4	6	8
Minimum green time	5	8	5	8
Maximum green time	30	30	30	30
Vehicle Extension time	2.0	1.5	2.0	1.5
Yellow time	4.5	5.0	4.5	5.0
Red clearance time	1.5	2.0	1.5	2.0
Pedestrian clearance time	13	21	13	22

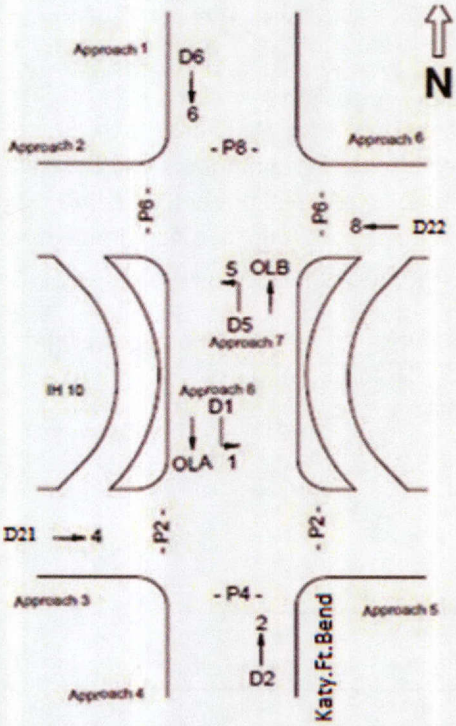


Figure 2.2.25 Specification of approaches, phases, and detectors of the intersection at Katy Fort Bend Rd.

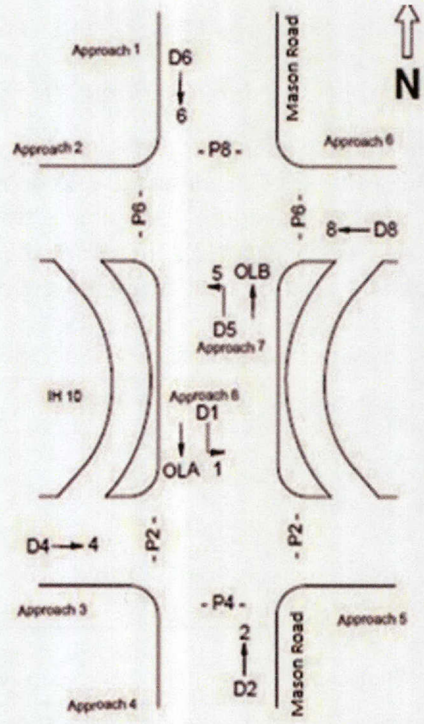


Figure 2.2.26 Specification of approaches, phases, and detectors of the intersection at Mason Rd.

2.2.4.4 Signal plan at Intersection D (Mason Rd. at IH-10)

The intersection at Mason Rd. and the frontage road of IH-10 was determined in late November 2018 to replace the one with connection problems. Figure 2.2.26 shows the signal plan with phase numbers and their associated detectors at this intersection. This intersection has only video detectors.

This intersection's detailed settings of phases and detectors is reported in Table 2.2.73, and Table 2.2.74 reports the detailed information of minimum and maximum green times, vehicle extension, yellow, red clearance and pedestrian clearance of each phase.

At this intersection, Phase 9 is given the maximum green time (80 seconds). On the other hand, Phases 1 and 2 are in the OLA and they share the same green time; and Phases 5 and 6 are in the OLB and they share the same green time.

Table 2.2.73 Approach, phase, and detector number distributions of the intersection at Mason Rd.

Approach Number	Phase Number	Detector Number
1	2	2
2	3	-
3	8	22
4	6	6
6	4	21
7	1	-
8	5	-

Table 2.2.74 Signal timing plan of the intersection at Mason Rd. (in seconds)

Phase	2	4	6	8
Minimum green time	10	7	10	10
Maximum green time	45	50	50	80
Vehicle Extension time	1.7	1.7	1.7	1.7
Yellow time	4.5	4.0	4.5	4.0
Red clearance time	1.5	2.0	1.5	2.0
Pedestrian clearance time	14	25	20	25

2.2.4.5 Signal plan at Intersection E (Mason Access Rd.)

Two four-way intersections were selected in November 2018, to replace the ones with connection problems. They are located around 500 ft south of the diamond intersections along IH-10. One of them is the intersection at Mason Access Rd. (E) and another is at Fry Access Rd. (G). This section looks at the former. Figure 2.2.27 shows the phase numbers associated with each approach at this intersection.

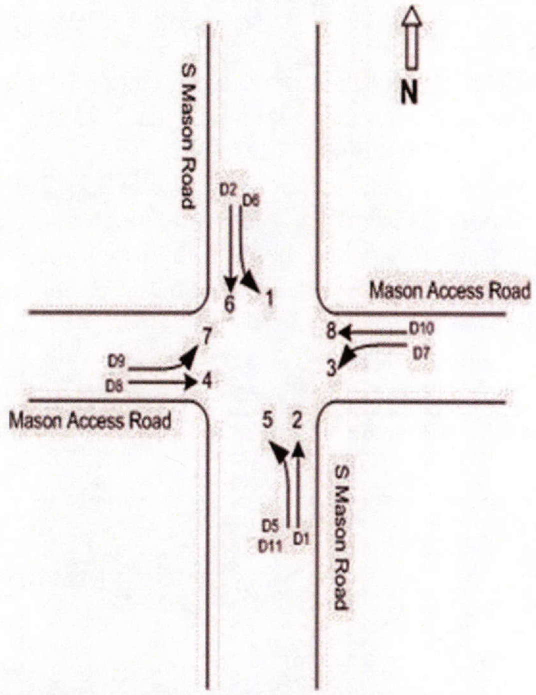


Figure 2.2.27 *Specification of approaches, phases, and detectors of the intersection at Mason Access Rd.*

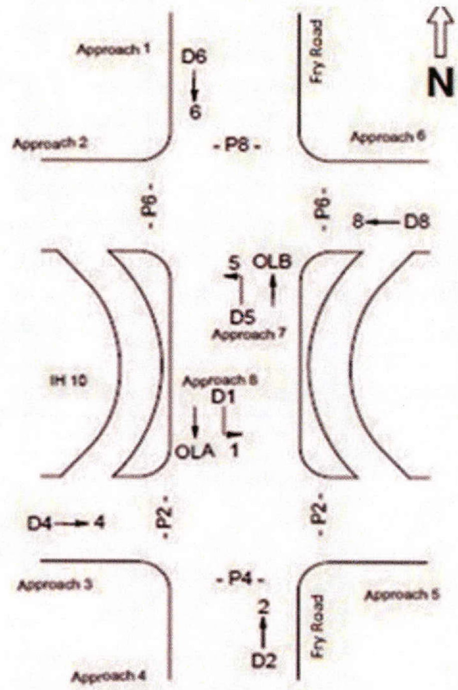


Figure 2.2.28 *Specification of approach, phase, and detectors of the intersection at Fry Rd.*

Table 2.2.75 Approach, phase, and detector number distribution at the intersection of Mason Access Rd.

Approach Number	Phase Number	Detector Number
1	2	1
3	8	4
4	6	2
6	4	8
7	1	6
8	5	5,11

Detectors associated with each phase are also listed in Figure 2.2.27. From this figure, it can be seen that Phases 2, 4, 6, and 8 are for through traffic; Phase 5 is for left-turn westbound traffic towards north; and Phase 1 is for left-turn eastbound traffic to the south. The detailed settings of phases and detectors at this intersection is reported in Table 2.2.75.

Table 2.2.76 reports the signal plan applied to this intersection. It provides the values of minimum and maximum green times, vehicle extension, yellow and red clearances of each phase and pedestrian clearance time.

Table 2.2.76 Signal timing plan of the intersection at Mason Access Rd. (in seconds)

Phase	1	2	3	4	5	6	7	8
Minimum green time	7	10	7	7	7	10	7	7
Maximum green time	35	80	35	45	40	80	35	45
Vehicle Extension time	2.0	2.5	1.8	1.8	1.7	2.5	1.8	1.8
Yellow time	3.9	3.9	3.2	3.2	3.9	3.9	3.2	3.2
Red clearance time	2.3	1.7	3.1	3.7	2.7	2.0	3.2	3.9
Pedestrian clearance time	0	25	0	37	0	22	0	0

2.2.4.6 Signal plan at Intersection F (Fry Rd. at IH-10)

This intersection was also selected in November 2018 to replace the one with connection problems. Figure 2.2.28 shows each phase's numbers, as well as the detectors associated with each phase. This intersection has only video detectors. Its phase setting is very similar to that at Mason St.

The detailed information of phases and detectors at each approach is reported in Table 2.2.77. Table 2.2.78 reports the detailed signal timing plan at this intersection. Phases 2 and 6 are assigned a maximum value of green time of 75 seconds. Note that this intersection allows for the pedestrian crossing from all approaches. On the other hand, Phases 1 and 2 are in the OLA and they share the same green time; and similarly, Phases 5 and 6 are in the OLB, and they share the same green time.

Table 2.2.77 Approach, phase, and detector number distributions of the intersection at Fry Rd.

Approach Number	**Phase Number**	**Detector Number**
1	2	2
3	8	8
4	6	6
6	4	4
7	1	1
8	5	5

Table 2.2.78 Signal timing plan of the intersection at Fry Rd. (in seconds)

Phase	2	4	6	8
Minimum green time	15	5	15	5
Maximum green time	75	35	75	35
Vehicle Extension time	2.0	2.0	2.0	2.0
Yellow time	3.9	4.7	3.9	4.9
Red clearance time	2.0	2.4	2.6	2.5
Pedestrian clearance time	18	23	13	24

2.4.7 Signal plan at Intersection G (Fry Access Rd. at IH-10)

Another four-way intersection is the one 500 ft south of the diamond intersection at Fry St. This one was also determined in November 2018 to replace the one with connection problems. Figure 2.2.29 shows the signal plan with phase numbers and their associated detectors at this intersection. Apparently, this one is very close to the one at Mason Access Rd. This intersection's detailed settings of phases and detectors are reported in Table 2.2.79. Further, the detailed settings in the currently implemented signal plan, including the information of minimum and maximum green times, vehicle extension, yellow, red clearance and pedestrian clearance times of each phase are reported in Table 2.2.80.

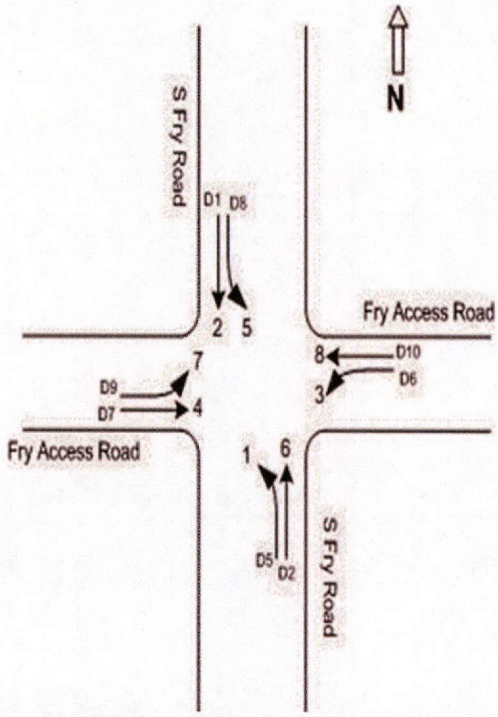


Figure 2.2.29 Specification of approach, phase, and detectors of the intersection at Fry Access Rd.

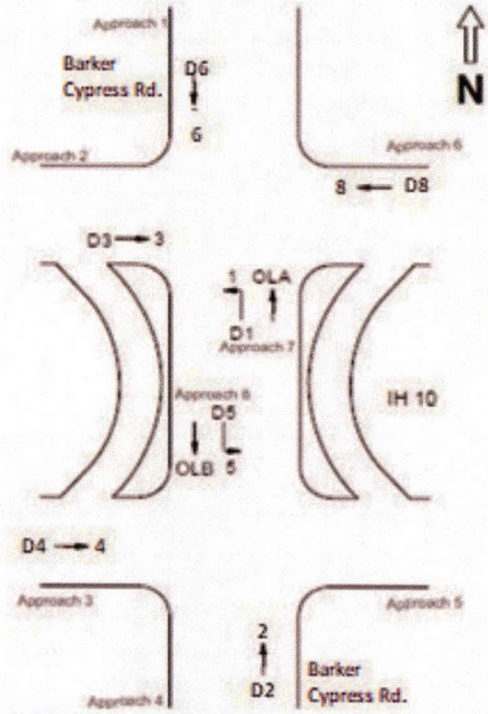


Figure 2.2.30 Specification of approach, phase, and detectors of the intersection at Barker Cypress Rd.

Table 2.2.79 Approach, phase, and detector number distributions of the intersection at Fry Access Rd.

Approach Number	Phase Number	Detector Number
1	6	6
3	4	21
4	2	2
6	8	22
7	5	5
8	1	1

Table 2.2.80 Signal timing plan of the intersection at Fry Access Rd. (in seconds)

Phase	1	2	3	4	5	6	7	8
Minimum green time	7	10	7	7	7	10	7	7
Maximum green time	35	80	35	45	40	80	35	45
Vehicle Extension time	2.0	4.0	2.0	2.0	2.0	4.0	2.0	2.0
Yellow time	3.9	3.9	3.2	3.2	4.0	3.9	3.2	3.2
Red clearance time	2.2	1.9	3.1	3.1	2.3	1.7	3.1	3.1
Pedestrian clearance time	0	17	0	28	0	17	0	0

2.2.4.8 *Signal plan at Intersection H (Barker Cypress Rd. at IH-10)*

The last intersection is located at Barker Cypress Rd. This is the east-most intersection of this testbed along IH-10, selected in June 2018 to replace one with detector problems. Figure 2.2.30 depicts the signal plan with phase numbers and their associated detectors at this intersection. This intersection also has only video detectors, similar to the intersections at Mason St. and Fry St.

This intersection's detailed settings of phases and detectors are reported in Table 2.2.88, and Table 2.2.89 reports the detailed information of minimum and maximum green times, vehicle extension, yellow, red clearance and pedestrian clearance times of each phase.

Table 2.2.81 Approach, phase, and detector number distribution at the intersection of Barker Cypress Rd.

Approach Number	Phase Number	Detector Number
1	6	6
3	4	21
4	2	2
6	8	22
7	5	-
8	1	-

Table 2.2.82 Signal timing plan of the intersection at Barker Cypress Rd. (in seconds)

Phase	2	4	6	8
Minimum green time	13	7	12	10
Maximum green time	45	60	50	80
Vehicle Extension time	1.7	1.3	1.7	1.3
Yellow time	3.9	4.7	3.9	4.7
Red clearance time	2.0	2.0	2.0	2.0
Pedestrian clearance time	18	22	18	24

2.3. SUMMARY

This chapter comprehensively reviews the geometric properties and the detailed existing signal plan settings of 30 intersections selected for the implementation of the proactive signal control system in this project. The geometric information was collected in Google Earth, as well as the onsite inspections. The existing signal plan setting of each intersection was provided by Steve Chiu from the Houston District Office of the Receiving Agency.

These two types of information provide the base for building the testbed models in VISSIM for the purpose of simulation, as described in Chapters 3 through 6.

CHAPTER 3

IMPLEMENTATION AT TESTBED OF FM 528

3.1. INTRODUCTION

The testbed on FM 528 is the first one the research team used to implement the proposed proactive signal control system, chosen for its simple conditions. This testbed has three intersections at (1) Desota St., (2) Friendswood St., and (3) Briar Creek Dr./Falcon Ridge Blvd. The major road is FM 528. Originally, at this testbed, in weekday peak hours, the traffic at these three intersections was controlled by a coordinated signal control system, and during off-peak hours (9:30 am–2 pm), it was under the actuated mode based on individual vehicles' arrival at an approach; however, on weekends (Saturday and Sunday), the coordination mode was applied from 7 am to 11 pm. Since it was the first testbed used in implementation of this project, both the Performing and Receiving Agencies were conservative and cautious in the implementation.

Since these three intersections have similar conditions to the one tested in Project 0-6920, the similar control logic used in that intersection (see Figure 1.2.4) were implemented here. The implementation was conducted on August 3, 2017. Note that not only the control logic was updated, the basic settings of signal plan were also revised to make the new control logic more effective. Section 3.2 details such revisions on the signal settings, and Section 3.3 describes the logic implemented to the testbed. The optimal parameters in the control logic were estimated via traffic simulations. Section 3.4 gives the details of the simulation.

Because the detectors cannot directly reflect the queue length and delay due to the signal, after the implementation, the research team traveled to the testbed several times in August 2017 to observe the traffic flows (record the queue length and maximum delay) when the implemented proactive signal control system was switched on or off serval time. When it was off, the working control system is the original actuated signal control system. Based on the observations over five cycles, the average queue length and delay were used to evaluate the performance of the proposed proactive signal control. Moreover, the Performing Agency requested the Receiving Agency to collect the data from this testbed for a period of 8/3/2017–8/18/2017 by switching the applied proactive system on and off, respectively. These data were summarized to evaluate the performance of the proposed proactive signal control system implemented to the testbed. Such data analysis under two plans were reported in Sections 5 and 6, respectively.

***Figure 3.1.1** Testbed of FM528 with selected three intersections*

The reminder of this chapter is organized as follows. Section 3.2 describes the signal timing setting for the proposed proactive signal control system; Section 3.3 introduces the logic statement settings for this proposed system; Section 3.4 summarizes the simulation testing; Section 3.5 outlines the findings from the field observation; Section 3.6 analyzes the data reported by detectors on the testbed; and Section 3.7 summarizes this chapter.

3.2. SIGNAL TIMING SETTING

The signal timing plans provided by the Receiving Agency was modified to make the proposed control logic more effective, which is supposed to be activated by vehicle platoons. Also, all settings were tested in VISSIM simulations. These changes were applied to the ASC/3 Controller.

Tables 3.2.1, 3.2.2 and 3.2.3 show the changes made in the ASC/3 Controller for three intersections, respectively. Term "old" in these three tables refer to the parameter values of the existing actuated signal control settings implemented by the Receiving Agency. All of these parameters can be edited through the function of MM (2-1) in the ASC/3 Controller, and MM refers to "Main Menu" of the Controller.

The detector volume and occupancy data are updated for a defined interval to fetch the latest information of the intersection. Therefore, it is recommended to set the vehicle extension time equivalent to the updating interval. The updating interval of 5.0 seconds were found to be the best to optimize the intersection performance. Therefore, 5.0 seconds were set as the vehicle extension period for the through movement on the major road (Phases 2 and 6). On the other hand, for other approaches (mainly minor roads), a smaller value of 3.0 seconds was set to minimize the overall delay. The minimum green time was changed to be 0 for all phases except for Phases 2 and 6 (for the through movement on the major road) to expedite the skipping or switching a phase. Furthermore, the maximum green time was also revised to relatively larger values than that of the original settings by the TxDOT.

Table 3.2.1 Revised signal timing parameters for the intersection at Desota St.

Phase	**2 (old)**	**2**	**4 (old)**	**4**	**5 (old)**	**5**	**6(old)**	**6**
Minimum green time	15	**10**	7	**0**	5	**0**	15	**10**
Maximum green time	35	**40**	20	**25**	15	15	35	**40**
Vehicle extension time	2	**5**	2	**3**	3	3	2	**5**

Table 3.2.2 Revised signal timing parameters for the intersection at Friendswood Blvd.

Phase	**1 (old)**	**1**	**2 (old)**	**2**	**4 (old)**	**4**	**5 (old)**	**5**	**6 (old)**	**6**	**8 (old)**	**8**
Minimum green time	5	**0**	15	**10**	7	**0**	5	**0**	15	**10**	7	**0**
Maximum green time	15	**20**	35	**50**	20	**25**	15	**10**	35	**30**	20	20
Vehicle extension time	2	**3**	2	**5**	2	**3**	2	**3**	2	**5**	2	**3**

Table 3.2.3 Revised signal timing parameters for the intersection at Briar Creek Dr./Falcon Ridge Blvd.

Phase	**1** (old)	**1**	**2** (old)	**2**	**3** (old)	**3**	**4** (old)	**4**	**5** (old)	**5**	**6** (old)	**6**
Minimum green time	5	**0**	15	**10**	7	**0**	7	**0**	5	**0**	15	**10**
Maximum green time	15	**20**	35	**50**	20	**25**	20	**25**	15	**20**	35	**45**
Vehicle extension time	2	**3**	2	**5**	3.5	**3**	3.5	**3**	2	**3**	2	**5**

Note: "old" refers to the existing parameter values used by TxDOT in the actuated signal control on the testbed.

3.3. DEVELOPMENT OF LOGICAL STATEMENT

3.3.1 General Logical Statement

In the original actuated signal control, when a vehicle passed a detector, a 'call' was generated to start a phase or extend an existing phase. Therefore, typically a 'call' represented the presence of one vehicle at a time. In the new proactive signal control, the definition of traditional 'call' was re-defined as follows.

This new logic aims to the call from "a platoon of vehicles", so as to help such a platoon more smoothly pass through an intersection.

Vehicles in a platoon has smaller headways, compared with those not in a platoon, so each 'call' represents a level of congestion in the intersection, not merely the presence of a vehicle. However, a loop detector can only report two parameters: volume and occupancy. Therefore, it is important to find an approximate setting of volume and occupancy to reflect the arrival of a platoon of vehicles at the approach. Since it is for a platoon of vehicles, such values must be from upstream detectors (i.e., "IN" detectors shown in Figure 1.2.2).

As demonstrated in Figure 3.3.1 (the same as the one shown in Figure 1.2.4), the general logic used at this testbed is the same tested at the single intersection at NASA Road 1 in Project 0-6920.

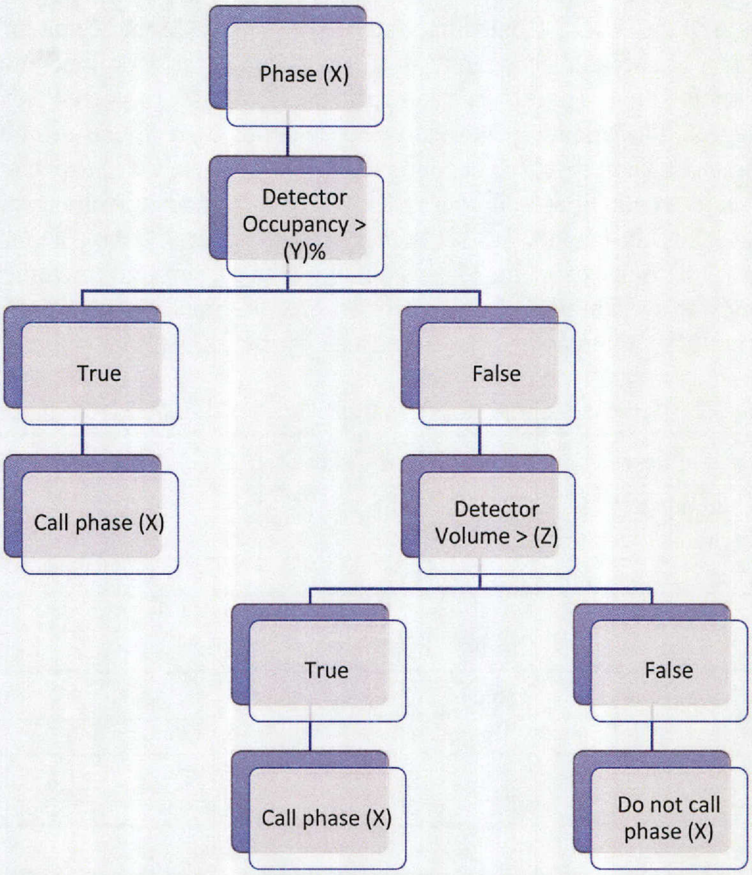


Figure 3.3.1 *Flow chart of the proposed algorithm for the proactive signal control*

To illustrate, according to Figure 3.3.1, suppose that a "call" is to be decided for Phase X. At first, a particular detector occupancy, say Y percent is assigned. If the occupancy exceeds Y percent, a "call" for Phase X will be initiated. Otherwise, it will be further checked whether the detector volume is larger than a certain number, say Z. If the detector volume is more than Z, a call will be initiated; otherwise, no call will be placed for Phase X. Based on this logic, each of the phases will generate a call based on its settings of Vehicle Detector and Occupancy. The detailed settings of such function are described as follows.

3.3.2 Coding in Econolite ASC/3 Controller

As mentioned in Chapter 1 (Section 1.2.2), MM refers to the Main Menu of the Econolite ASC/3 Controller. The current detector assignment with respect to their phases can be seen at MM 6-1. The function of each detector can be modified at MM 6-2. In the original settings, the detector type is set as "Standard" mode. For our studies, Mode NTCIP was chosen for the detectors. Meanwhile, the NTCIP

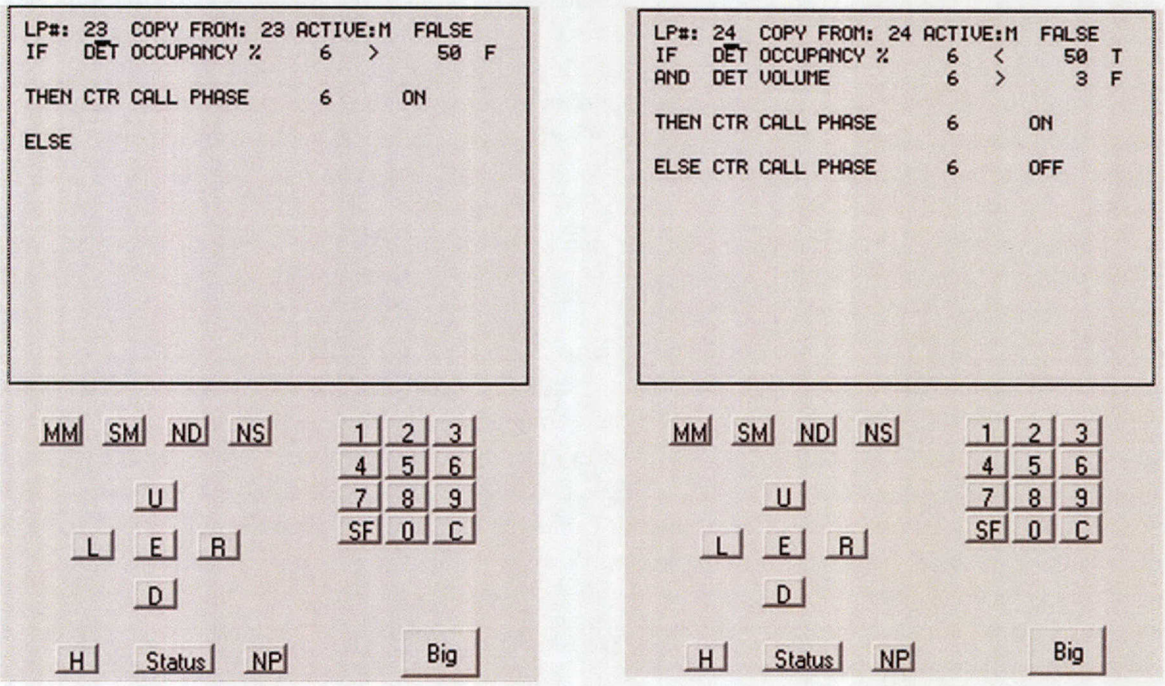
Volume and NTCIP Occupancy log are enabled. To set the updating interval, the NTCIP log interval is to be 5 seconds via MM 6-4.

Using the Logic Statement by MM 1-8-2, the algorithm described in Figure 3.3.1 was uploaded to the ASC/3 Controller. MM 1-8-1 can be used to activate or de-active the logic statements as demonstrated in Figure 3.3.1. The algorithm covers all the phases. For brevity, this report only covers the coding of Phase 6 for the intersection at Briar Creek Dr./Falcon Ridge Blvd., as shown in Figures 3.3.2 and 3.3.3, respectively.

From Figure 3.3.2(a), it is seen that for the stop bar at Detector 6, the occupancy percentage margin is set as 50 percent. Therefore, if the occupancy percentage at Detector 1 exceeds 50, then Phase 6 is called. Figure 3.3.2(b) captures the scenario if the occupancy is less than 50. Detector volumes are taken into consideration for such case. For this particular case of Phase 6, if the volume is more than 3, Phase 6 is called; otherwise, no call is made. On the other hand, for upstream detector at this intersection, it's occupancy percentage is generally low because the vehicles pass through it with high speeds. Thus, the 'Volume' is used to define the call. Figure 3.3.3 shows this case, where logic statements for Phase 6 with upstream detector 8 are showed. From Figure 3.3.3, it is seen if the detector has a volume more than 3, Phase 6 will be called; otherwise no call is initiated. Table 3.3.1 lists the identified best input values (volume and occupancy) for each phase at this intersection. Similarly, for the phases of other two intersections, detector volume and occupancy values are set to define the call. The value of the optimum detectors' volume and occupancy are identified by a series of simulation runs in VISSIM. They are reported in Tables 3.3.2 and 3.3.3, respectively.

Table 3.3.1 Selected occupancy and volume margins for the intersection at Briar Creek Dr.

Phase Number	Detector	Occupancy (%)	Volume
1	1 (SD)	70	2
2	2 (UD)	50	3
	7 (UD)	-	6
3	3 (SD)	70	2
	10 (SD)	70	2
4	4 (SD)	70	2
5	5 (SD)	70	2
6	6 (UD)	50	3
	8 (UD)	-	6



(a) (b)

Figure 3.3.2 *Logic statements for Phase 1 (Stop bar Detector (SD)) of the intersection at Briar Creek Dr./Falcon Ridge Blvd.*

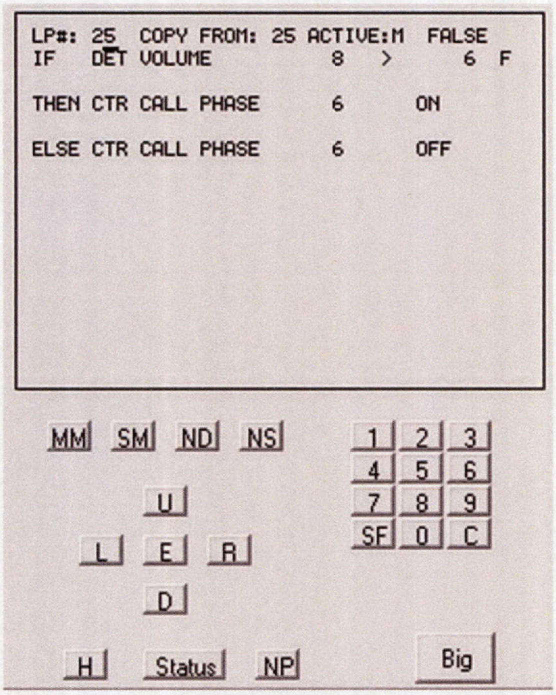


Figure 3.3.3 *Logic Statements for Phase 1 (Volume Detector (VD)) of the intersection at Briar Creek Dr.*

Table 3.3.2 Selected occupancy and volume margins for the intersection at Desota St.

Phase Number	Detector	Occupancy (%)	Volume
2	1 (SD)	70	4
	10 (SD)	70	4
	2 (UD)	-	6
	3 (UD)	-	6
4	13 (SD)	70	2
	14 (SD)	70	2
5	5 (SD)	70	1
6	6 (SD)	70	4
	7 (SD)	70	4
	9 (UD)	-	6

Table 3.3.3 Selected occupancy and volume margins for the intersection at Friendswood Blvd.

Phase Number	Detector	Occupancy (%)	Volume
1	1 (SD)	70	2
2	3 (SD)	70	2
	4 (SD)	70	2
	5 (UD)	-	4
4	17 (SD)	70	2
5	6 (SD)	70	2
6	7 (SD)	70	2
	8 (SD)	70	2
	9 (UD)	-	6
	10 (UD)	-	6
8	11 (SD)	70	2
	12 (SD)	70	2
	13 (SD)	70	2
	14 (SD)	70	2
	15 (SD)	70	2
	16 (SD)	70	2

3.4. TEST VIA SIMULATIONS

3.4.1 Simulation Verification

Before the field experiment, VISSIM was used to simulate the traffic on the testbed under the proposed proactive signal control (as described in Section 3.3), in order to evaluate the performance, and to determine the best parameter settings. A testbed model was built in VISSIM (see Figure 3.4.1). However, before testing the proposed control logic, it is necessary to show that VISSIM is able to correctly simulate the real traffic at the testbed. This called simulation verification. This section details the simulation work to this testbed, and this work has been reported in TM 3.1 Report submitted in July 31, 2017.

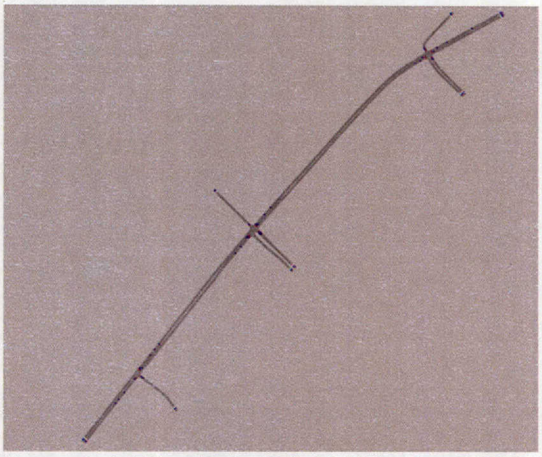


Figure 3.4.1 *Testbed of FM 528 built in VISSIM*

The data collected from 5/1/2017 6:04:01 am to 5/25/2017 7:45:00 am were used for the simulation verification. Note that the data reported by loop detectors are the volume and occupancy per minute. The research team aggregated the volume for each 15 minutes, starting from 00:01:00 am of each day, i.e., for a particular day, the first 15-minute section is from 00:01:00 am to 00:15:00 am, and the second is from 00:15:01 am to 00:30:00 am. If no enough data for one 15-minute section, no data exists for this section. Finally, the multi-day average of volume for each 15-minute section was calculated respectively (as the hourly volume, i.e., 15-minute volume multiplied by 4) for weekday and weekend, respectively. These data were firstly used to estimate the origin-destination matrix of each testbed (such demand data serve as the input parameters for simulations in VISSIM), and then they were used as the base for comparing the simulated flows at this testbed.

Figures 3.4.1 through 3.4.3 report the simulated and observed eastbound traffic flows on the major road (FM 528) of the intersection at Desota St. on weekday, as well as the flow on Desota St. toward FM 528. It is seen that on the major road, the simulated flows are quite close to the observed ones. However, the gap between them on the minor road at this intersection is large. This is because the flow on the minor road is very small (only about 10% of those on the major road)—meaning that this minor road sees a large uncertainty in flows. Given this feature, it is difficult to accurate simulate the flow there. Since the flow on the minor road is very small, it is not important to have them accurately simulated.

Figures 3.4.4 through 3.4.7 report the simulated and observed flows on FM 528 at other two intersections. The results are also generally good—some gaps exist but are relatively small. It shows that VISSIM simulation can be employed to evaluate the performance of the proposed proactive signal control system at this testbed by simulating the traffic flow under this proposed control system. The report of Task 3, TM-3.1 Network Performance Analysis Report, gives the details of simulation verification for each intersection of this testbed (flow on major or minor road on weekday or weekend). Please read that report for the details.

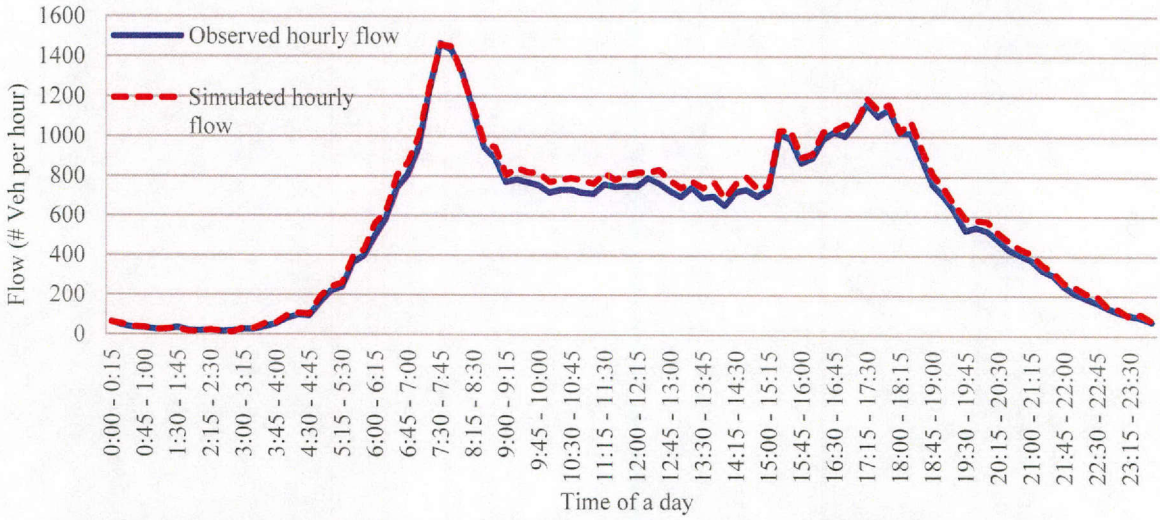


Figure 3.4.2 *Weekday's eastbound through traffic on FM 528 of the intersection at Desota St.*

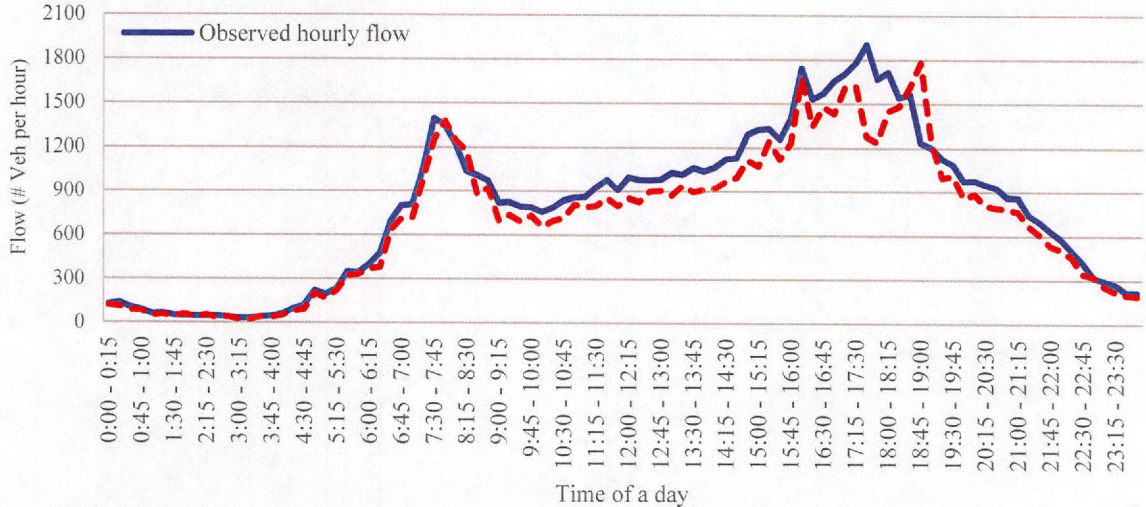


Figure 3.4.3 *Weekday's westbound through traffic on FM 528 of the intersection at Desota St.*

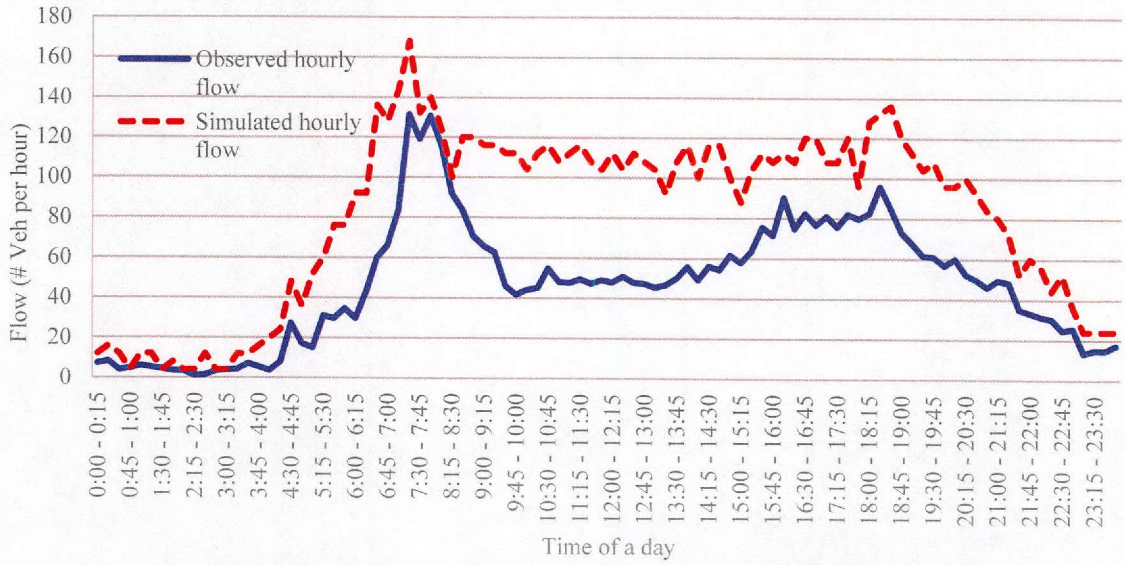


Figure 3.4.4 *Weekday's traffic on Desota St. toward to FM 528*

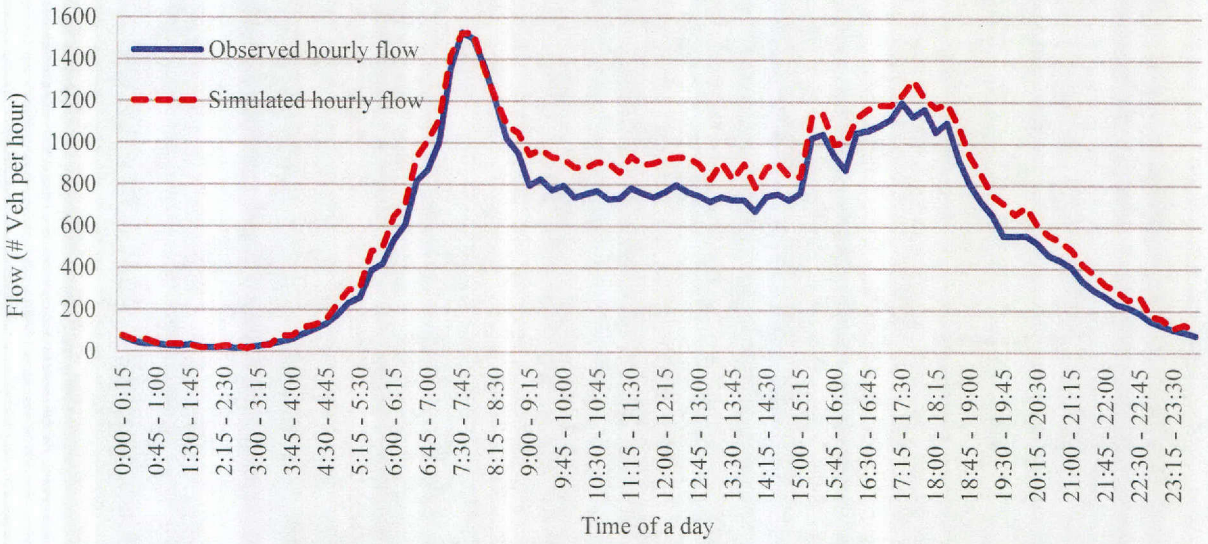


Figure 3.4.5 *Weekday's eastbound traffic on FM 528 of the intersection at Friendswood St.*

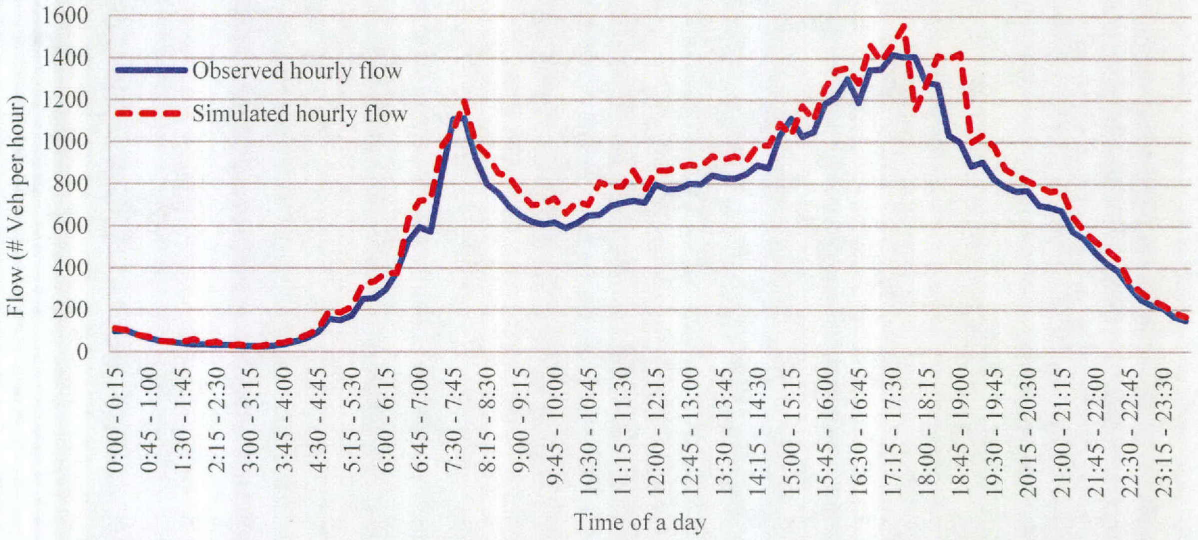


Figure 3.4.6 *Weekday's westbound traffic on FM 528 of the intersection at Friendswood St.*

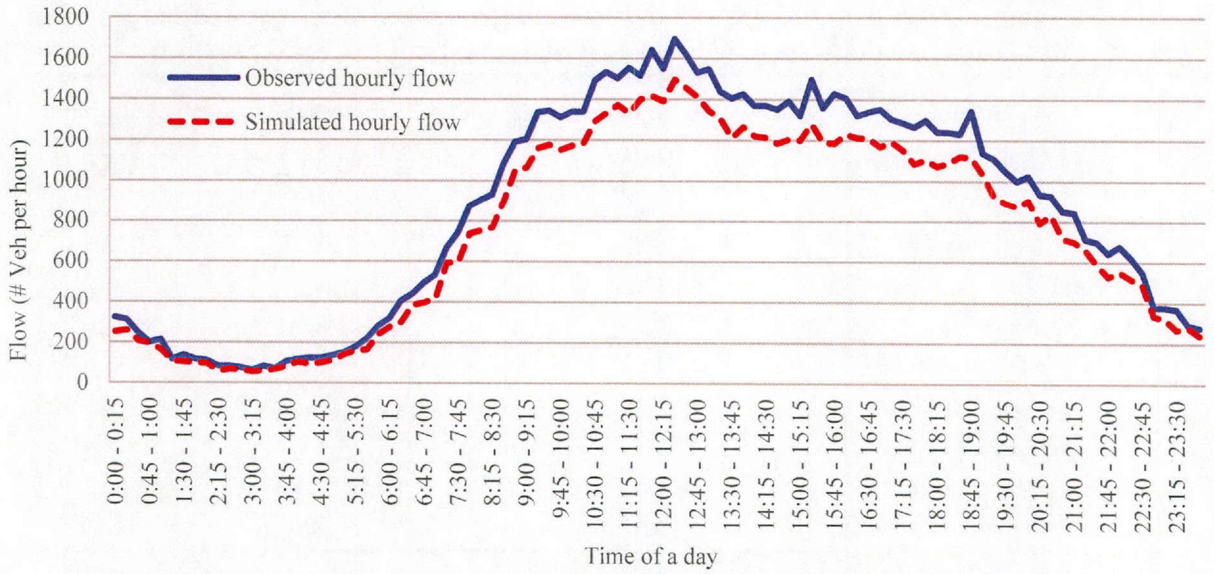


Figure 3.4.7 *Weekend's eastbound traffic on FM 528 of the intersection at Briar Creek Dr.*

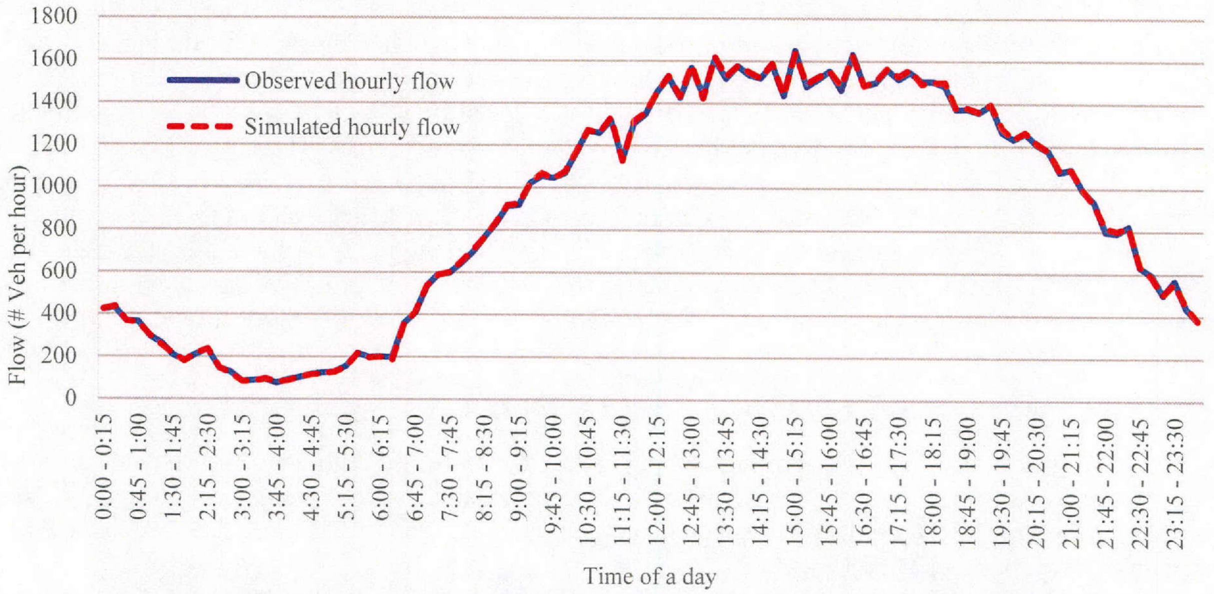


Figure 3.4.8 *Weekend's westbound traffic on FM 528 of the intersection at Briar Creek Dr.*

3.4.2 Test of Proactive Signal Control via Simulations

Given the volume and occupancy pattern of peak hours from the detector data, the Performing Agency sets different values of the critical volumes, the critical occupancies, and the extension time, to define the proposed logic for each detector through the simulator of ASC/3 signal Controller plugged in VISSIM. The settings with the highest savings in vehicle delay are recommended in the field experiment. Table 3.4.1 shows a weekday/weekend 30-minute's simulation (12:00 pm to 12:30 pm) results. The benefit of the proactive signal control algorithm was evaluated, using the results under the actuated signal control as the baseline.

Table 3.4.1 Comparison of the simulated mid-day traffic on the testbed under actuated and proactive signal control systems on a weekday and a weekend, respectively

	Parameters	Actuated Signal Settings	Proactive Signal Settings	Difference
Weekday	Average Delay (seconds)	39.35	24.80	-37.0%
	Average Stop	1.27	1.00	-21.3%
	Average Stop delay (seconds)	19.72	9.19	-53.4%
	Total Delay (seconds)	54,742.47	34,494.63	-37.0%
	Speed Average (mile/hour)	29.05	32.69	+12.5%
Weekend	Average Delay (seconds)	41.51	33.42	-19.5%
	Average Stop	1.34	1.14	-14.9%
	Average Stop delay (seconds)	19.29	14.40	-25.4%
	Total Delay (seconds)	66,590.03	53,601.17	-19.5%
	Speed Average (mile/hour)	26.47	30.58	+15.5%

The delay is one of the most important parameters to measure the efficiency of a signal control system. It shows that in a weekday, the total delay reduces from 54, 742 to 34,495 seconds after implementing the proposed proactive control, resulting in a 37.0% reduction in delay during the 1800-second simulation period. Consequently, the average speed of the vehicle on the network also increases. Moreover, the average delay and average stop delay also reduced up to 37.0% and 53.4%, respectively. Due to less delay, the number of stops experienced by vehicles also decreases. Therefore, it is expected to bring satisfactory driving experience on the testbed. Due to the improved performance of the testbed, environmental hazard and monetary loss are also reduced. On the other hand, it shows that the savings on a weekend are not as significant as those on a weekday. The average delay and average stop delay are reduced by 19.5% and 25.4%, respectively. Therefore, we can see that the proactive signal control system does help reduce the traffic congestions on these intersections on both weekday and weekend.

3.5. ANALYSIS OF FIELD OBSERVATION DATA

On August 3, 2017, the Performing Agency worked with Steve Chiu from the Houston District Office of the Receiving Agency, to implement the aforementioned proactive signal control logic (see Figures 3.3.1 through 3.3.3) and the revised signal plan settings (see Tables 3.2.1 through 3.2.3) to the three intersections at testbed of FM 528.

Originally, in a weekday, the traffic at these three intersections was controlled by the pre-timed coordinated traffic control system in morning and evening peak hours, and by the actuated signal control (based on the arrival of individual vehicles) in non-peak hours. On the other hand, in a weekend day (Saturday or Sunday), the coordinated signal control mode was applied from 7:00 am to 11:00 pm on weekend days.

In theory, if the detectors are perfectly accurate in capturing the arrival of vehicle platoon, the proactive signal controls at two adjacent intersections can realize the signal coordination automatically. However, since loop detectors can only reflect two values: volume and occupancy, which cannot directly reflect the arrivals of vehicle platoons, the settings of these two parameters shown in Tables 3.3.2 through 3.3.3 reflect an approximation. Therefore, the proposed proactive signal control cannot completely replace the coordinated signal control mode, especially if the through traffic on the major road is large. For this reason, **in a weekday, the proactive signal control system is only applied to a midday period (9:30 am to 2:00 pm); but it was applied from 7 am to 11 pm of a weekend day (Saturday and Sunday)** given the weekend flow volumes are smaller than those in weekday peak hours.

To evaluate the performance of the implemented proactive signal control system, after the implementation, the research team drove to the testbed several times to observe the traffic condition under either the original actuated or the new proactive signal control system for both weekday and weekend, respectively. The onsite observations focused on collecting two parameters: *maximum queue length* and *maximum delay* due to red light for a phase in one cycle, which are the keys to evaluate the performance of the implemented signal control system. These two parameters cannot be reported from loop detectors. Note that in the following, for brevity, we may just call them queue length and delay, respectively.

Steve Chiu switched the signal control system upon the request of the research team during these field observations. One observation was on August 6-7, 2017 (Sunday, under the original actuated signal control system, and Monday, under the new proactive signal control system), and another was on August 13-14, 2017 (Sunday, under the mew proactive system and Monday, under the old actuated system). Additionally, for collecting more real traffic data, a supplementary observation was made on August 25, 2017 (Friday, under the new proactive system).

Since the proactive signal control system is only applied to the midday period of a weekday: 9:30 am to 2:00 pm (originally, the actuated signal control was applied during this period), and on the other hand, though the system is effective from 7am to 11pm on a weekend day (Saturday or Sunday), the peak hours usually incur in the middle of weekend day. Therefore, the observation was made around 11:00 am to 1:00 pm for both weekday and weekend.

During these filed observations, queue lengths (in number of vehicles) and delay (in seconds) were recorded for several cycles at all approaches of all three intersections on the testbed. The average values of queue lengths and delay times are reported in Tables 3.5.1 through 3.5.3, respectively for three intersections.

It is seen that for weekdays, after implementing the proactive signal settings, both vehicle queue length and vehicle delay time drop at all three intersections. In the best case, the average queue length can be dropped up to 71%, and the average delay can drop up to 33%. However, for weekends, the observation did not reveal excellent results: the queue length and delay time dropped at some approaches, but some increased instead at some approaches. For example, in the weekdays at the first intersection Desota St., the average queue length for eastbound traffic reduced to 2.6 vehicles from 9.0 vehicles after implementing the proactive signal settings. This shows an improvement of 71%.

On the other hand, for weekend, on average, the original coordination mode seemed to perform better than the proposed proactive signal control for the traffic in some direction. It implies that when through traffic flow is high along the major road, the coordination sometimes is more important. For this reason, in the next testbeds (SH 242 and FM 1464), where the through traffic is even higher and the spacing between adjacent intersections are even closer, the research team particularly developed a logic to deal with the problem of signal coordination at intersection. The details are reported in Chapters 4 and 5, respectively.

Table 3.5.1 Field observation of the intersection at Desota St.

	Traffic flow	Average queue length (# vehicles)			Average delay (seconds)		
		Actuated	**Proactive**	**Difference**	**Actuated**	**Proactive**	**Difference**
Weekdays	Eastbound	9.0	2.6	-71%	18.7	13.4	-28%
	Westbound	5.3	3.4	-36%	17.3	11.6	-33%
	Northbound	1.7	1.0	-41%	43.7	38.5	-12%
Weekends	Eastbound	1.6	2.0	+25%	9.4	18.6	+98%
	Westbound	2.0	2.3	+15%	14.0	12.3	-12%
	Northbound	1.0	1.0	0%	32.5	44.5	+37%

Table 3.5.2 Field observation of the intersection at Friendswood Blvd.

	Traffic flow	Average queue length (# vehicles)			Average delay (seconds)		
		Actuated	Proactive	Difference	Actuated	Proactive	Difference
Weekdays	Eastbound	5.7	3.8	-33%	15.9	14.0	-12%
	Westbound	3.5	1.8	-49%	15.0	12.0	-20%
	Northbound	1.3	1.0	-23%	40.0	30.3	-24%
Weekends	Eastbound	3.8	2.4	-37%	8.3	13.4	+61%
	Westbound	5.0	1.0	-80%	10.5	4.3	-59%
	Northbound	1.0	1.0	0%	33.0	29.8	-10%

Table 3.5.3 Field observation of the intersection at Briar Creek Dr./Falcon Ridge Blvd.

	Traffic flow	Average queue length (# vehicles)			Average delay (seconds)		
		Actuated	Proactive	Difference	Actuated	Proactive	Difference
Weekdays	Eastbound	7.3	5.5	-25%	31.6	23.0	-27%
	Westbound	8.3	3.3	-60%	32.4	14.3	-56%
	Northbound	3.0	2.0	-33%	46.1	28.3	-39%
	Southbound	2.3	1.3	-43%	53.3	31.0	-42%
Weekends	Eastbound	5.8	4.7	-19%	19.0	15.3	-19%
	Westbound	1.8	4.7	+161%	16.2	22.5	+39%
	Northbound	2.0	1.3	-35%	32.3	24.0	-26%
	Southbound	3.0	2.5	-17%	48.3	40.2	-17%

3.6. ANALYSIS OF FIELD DATA FROM DETECTORS

The volume and occupancy data recorded by each detector on the testbed were provided by the Houston District Office from 8/3/2017 to 8/18/2017. Among them, the data collected during the following weekdays: 8/3/2017 to 8/4/2017 and 8/7/2017 to 8/11/2017 were recorded under the new proactive signal control system; and those collected from 8/14/2017 to 8/18/2017 are under the original actuated signal control system. On the other hand, the data collected from 8/5/2017 to 8/6/2017 are the weekends' data under the original coordinated signal control and those from 8/12/2017 to 8/13/2017 are the weekend's data under the proactive signal control.

The original data (volume and occupancy) were reported for every 1 minute by detectors. The volumes were aggregated for every 15 minutes from 00:00:00 am to 23:59:00 in each day, i.e., a total of 96 15-minute volumes were obtained if the data covers all day. Otherwise, for example, if the data was collected from 10:40:00 in a day, then in that day, the first 15-minute interval is 10:45:00–10:59:00. For occupancy, on the other hand, the average of each 15-minute interval was calculated, so there are also 96 15-min average occupancies in a day if the data covers all day.

Then, the multi-day averages were calculated for each 15-minute volume and occupancy on a weekday and weekend, respectively. Finally, the hourly traffic flow (15-min volume X 4) and occupancy for each 15-min interval, collected under the original and new control systems, respectively, were used to compare the performance of the new proactive signal control system.

As noted in the beginning of Section 3.5, the proactive signal control is only applied to the period from 9:30 am to 2:00 pm of a weekday; on the other hand, for a weekend, the proactive signal control is applied to the period from 7:00 am to 11:00 pm. Therefore, this report only compares the data between 9:30 am and 2:00 pm in weekday, but in a weekday, one period: 10 am to 4 pm, was considered in comparison.

By reviewing the data reported from detectors, it was surprised to see that the occupancy is very low: in most cases, the values are below 0.2, especially for the traffic on major road. Given such small occupancy rate, according to the model described in Section 3.3 and the parameter settings described in Section 3.4, the logic will simply rely on traffic volumes. On the other hand, the volumes of through traffic on major road were found not low: even during midday (9:30 am–2 pm), the traffic flow could be over 1000 veh/hr/ln. This level indicates a level of service (LOC) of C. With a high level of flow rate, such low level of occupancy indicates that the vehicle platoons are not clearly formed. Therefore, the impact of the proposed proactive signal control system on the through traffic on major road may not be clear.

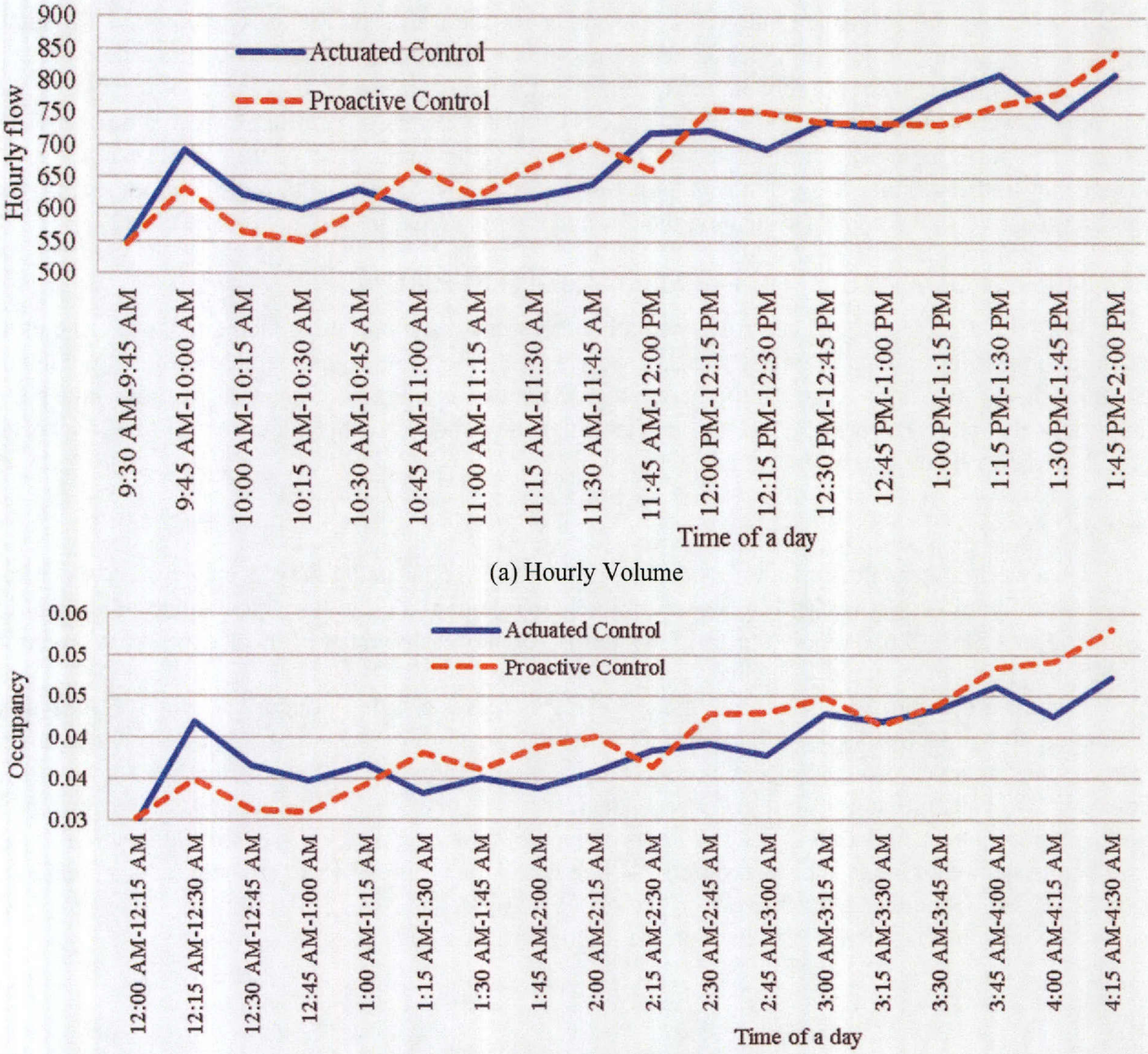
3.6.1 Comparison of Traffic Flows on Major Road (FM 528)

With the proactive signal control system, the intersection is supposed to facilitate more vehicles to pass through the intersection. Therefore, the data were expected to show higher vehicle volumes and lower occupancy during the time when the proactive signal control is applied, compared to those under the actuated signal control. For brevity, this report selects the one direction on the major road (FM 528) at each intersection for weekday and weekend, respectively.

Weekday

On a weekday, the traffic volumes on the major road (FM 528) under the old and new signal control systems were found very close, and the occupancy levels was kept a quite low level under either control system. Figures 3.6.1–3.6.3 report the traffic patterns for the eastbound traffic in a weekday at three intersections.

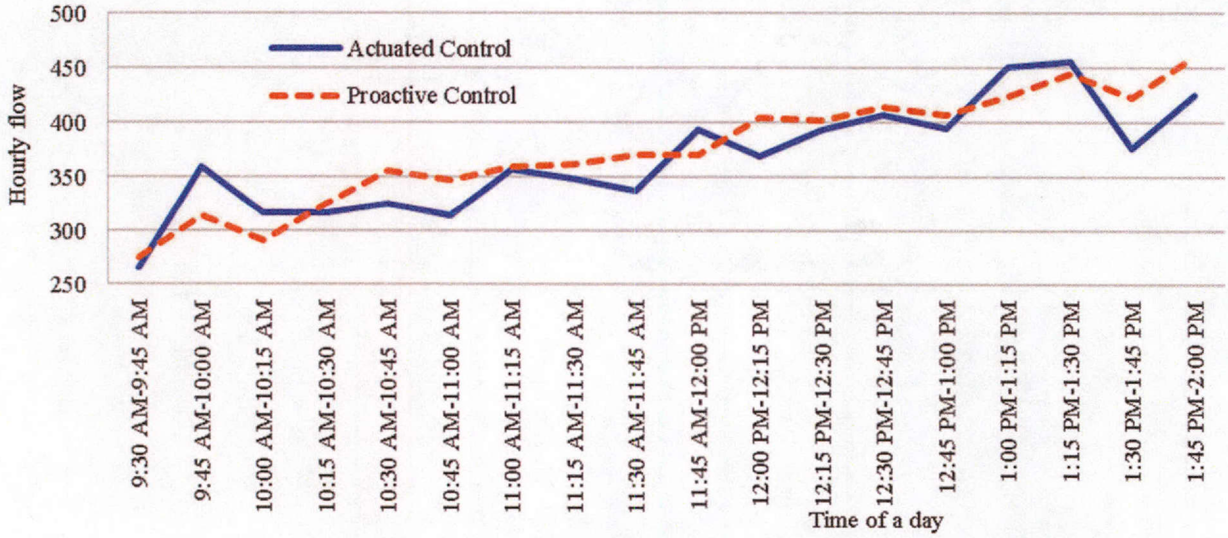
Figures 3.6.1 and 3.6.2 show that the flow rate increases a little (around 2.5% on average) after implementing the proactive signal control system at the intersections of Desota St. and Friendswood Blvd. However, the flow even dropped of the intersection at Falcon Ridge Blvd. It seems that the traffic of the intersection at Falcon Ridge Blvd. is larger than that at other two intersections. However, after implementation, it was surprised to see the flow dropped but the occupancy increased. Interestingly, it is not well consistent with the onsite observation—which found the proposed logic did help reduce the queue length and delay. Generally speaking, given the occupancy rates are small, the impact of the proposed proactive signal control system is not clear on the through traffic on FM 528.



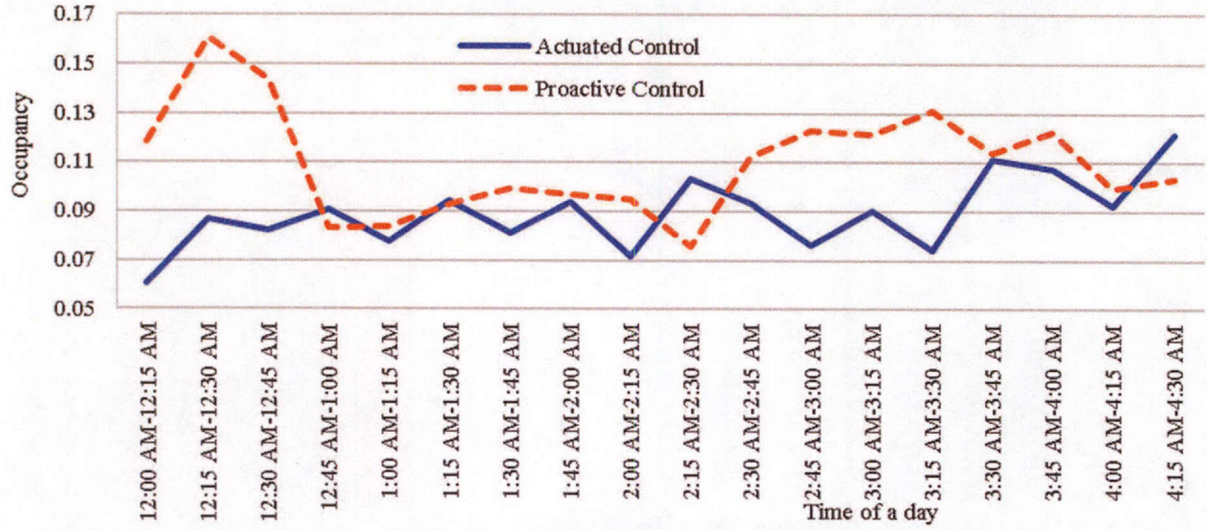
(a) Hourly Volume

(b) Average Occupancy (in percentage)

Figure 3.6.1 *Multi-day average eastbound traffic of the intersection at Desota St. on weekday*

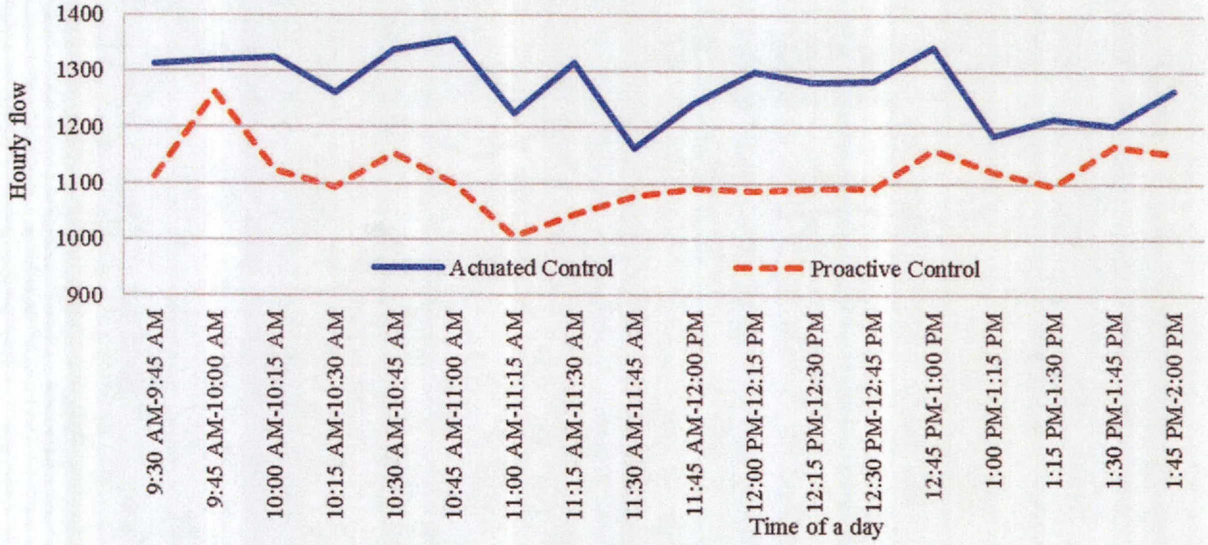


(a) Hourly Volume

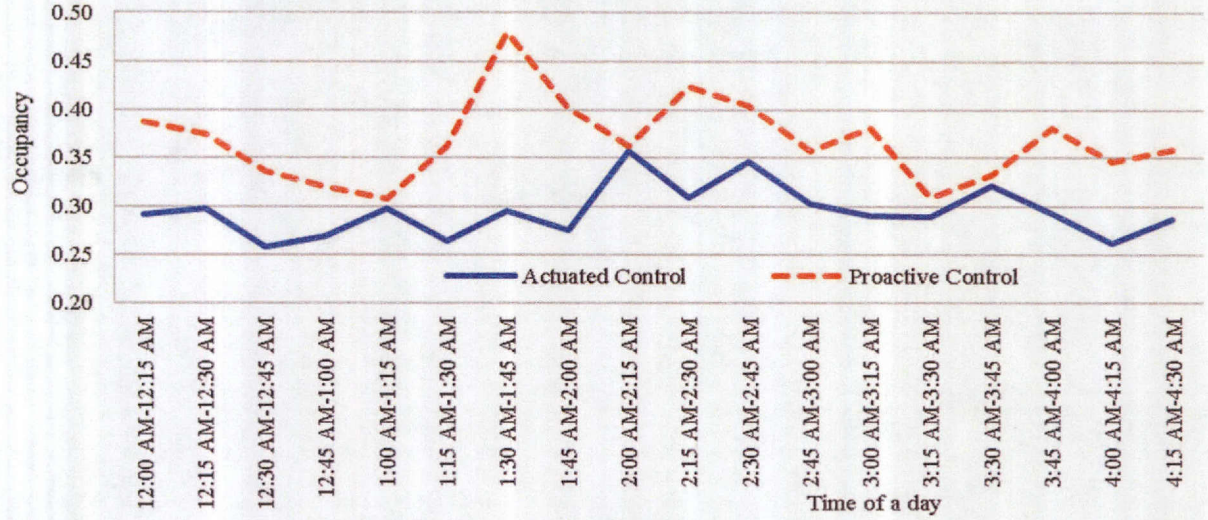


(b) Average Occupancy (in percentage)

Figure 3.6.2 *Multi-day average eastbound traffic of the intersection at Friendswood Blvd. on weekday*



(a) Hourly Volume



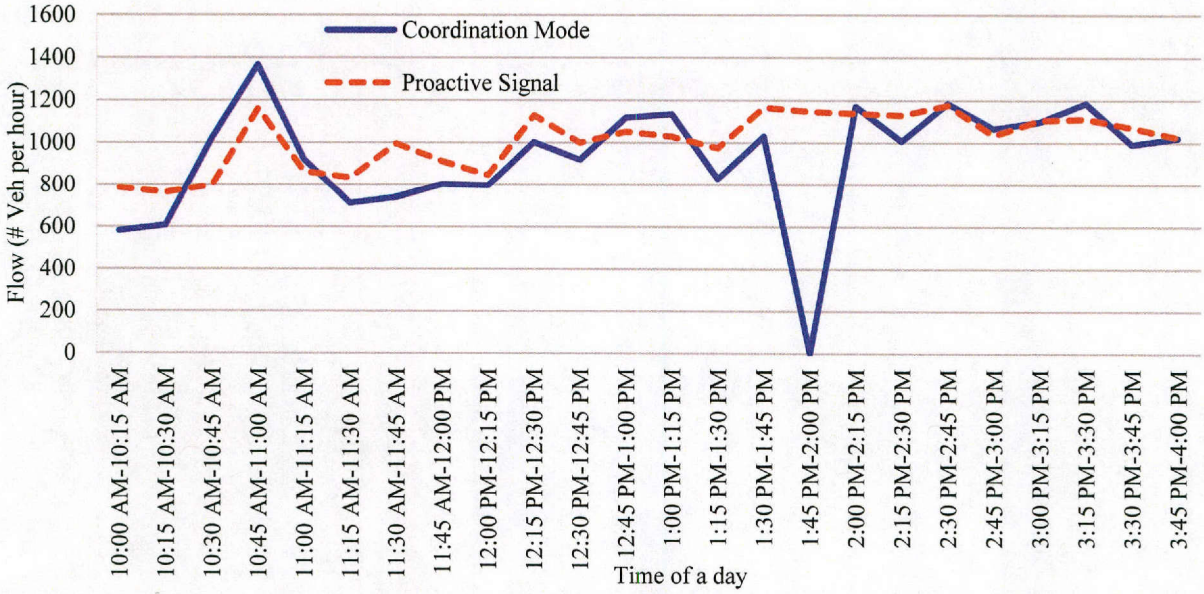
(b) Average Occupancy (in percentage)

Figure 3.6.3 *Multi-day average eastbound traffic of the intersection at Falcon Ridge Blvd. on weekday*

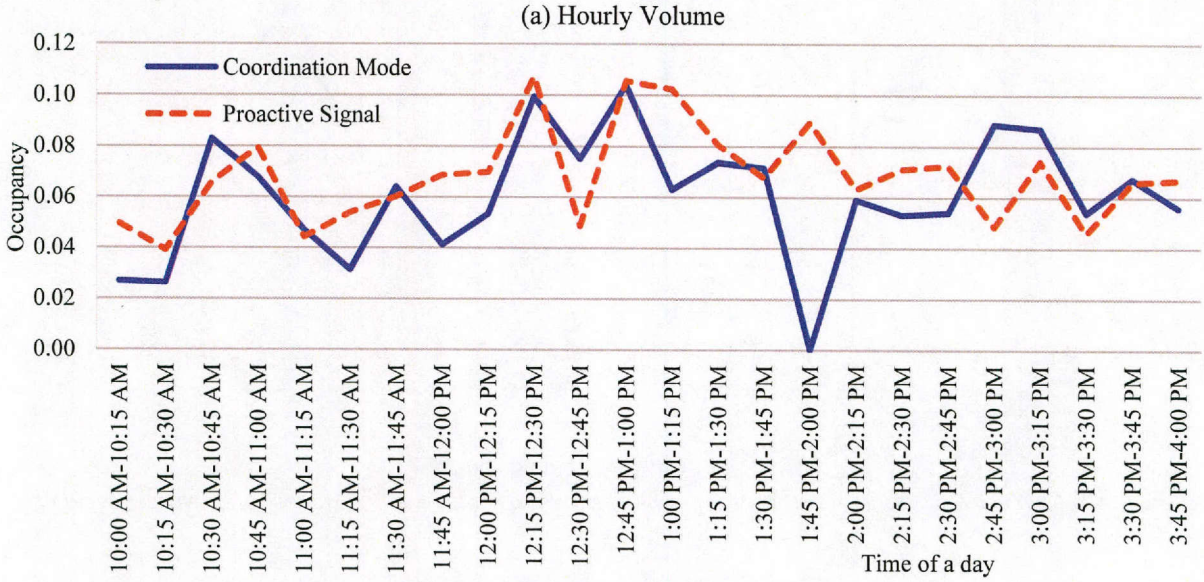
Weekend

On a weekend, a similar pattern can be seen: the occupancy rates are still very low even when the flow rates are as high as 1000 veh/hr/ln. As shown in Figures 3.6.4 through 3.6.6, the flow rate increased after applying the new logic. However, the occupancy rates also increased. As mentioned before, the occupancy rates were small, indicating that vehicles were moving in the road more independently and randomly. We use flow to evaluate the performance.

The increase of the flow rates after implementing the new proactive system, indicates that the traffic flow was improved for through traffic on the major road at these intersections during a weekend.

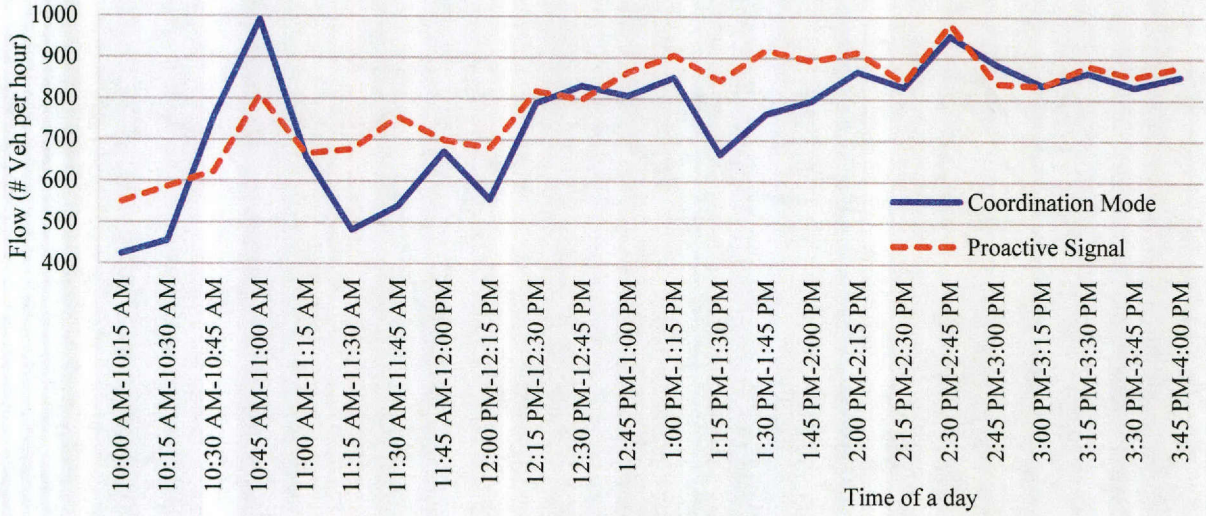


(a) Hourly Volume

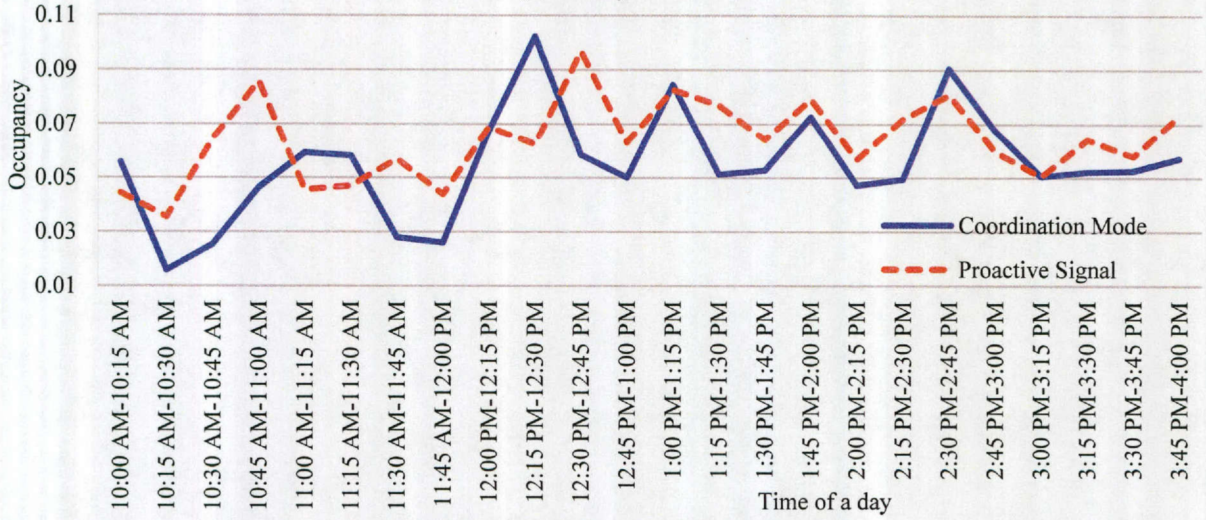


(b) Average Occupancy (in percentage)

Figure 3.6.4 *Multi-day average westbound traffic of the intersection at Desota St. on weekend*



(a) Hourly Volume



(b)Average Occupancy (in percentage)

Figure 3.6.5 *Multi-day average westbound traffic of the intersection at Friendswood Blvd. on weekend*

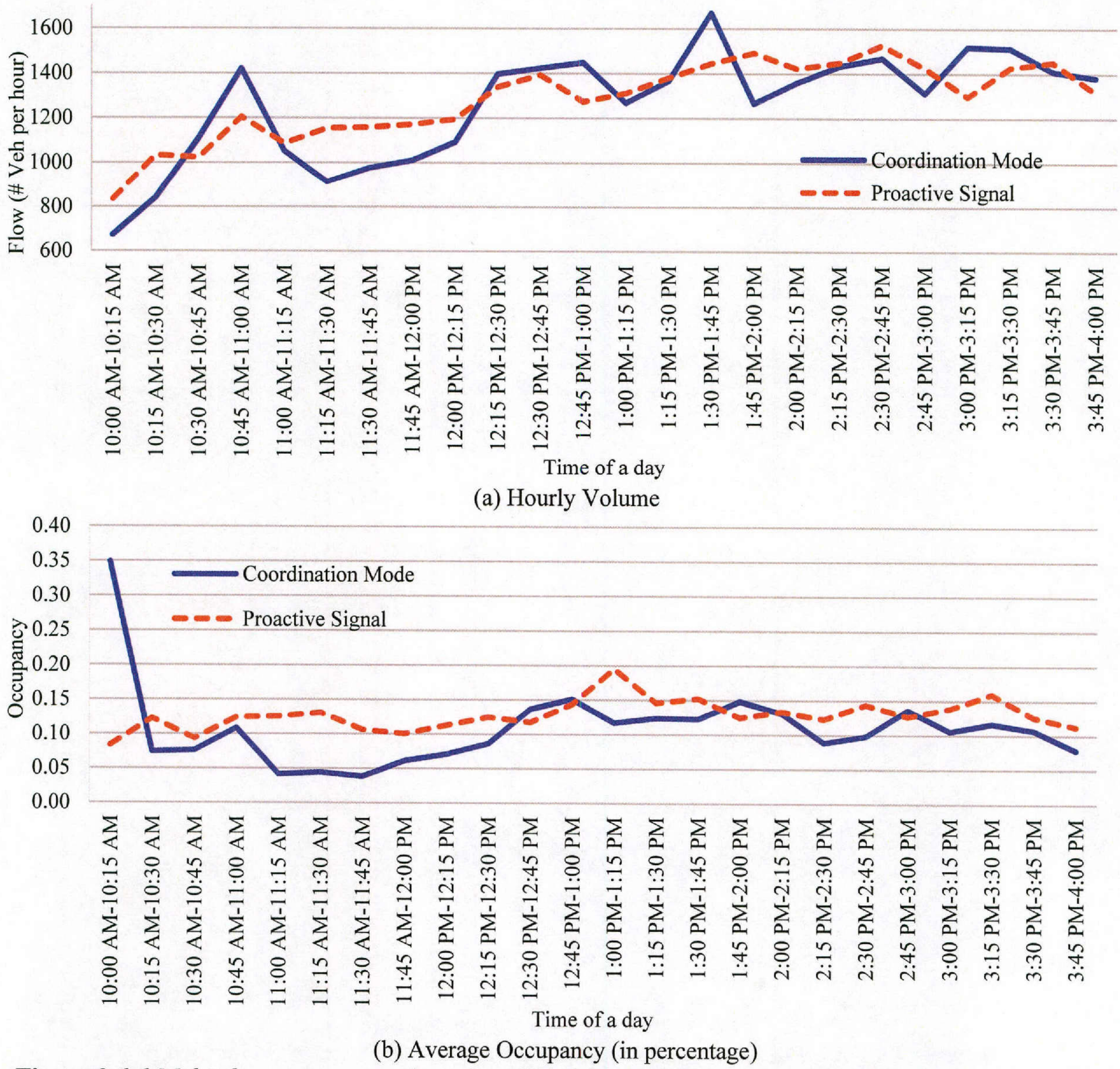


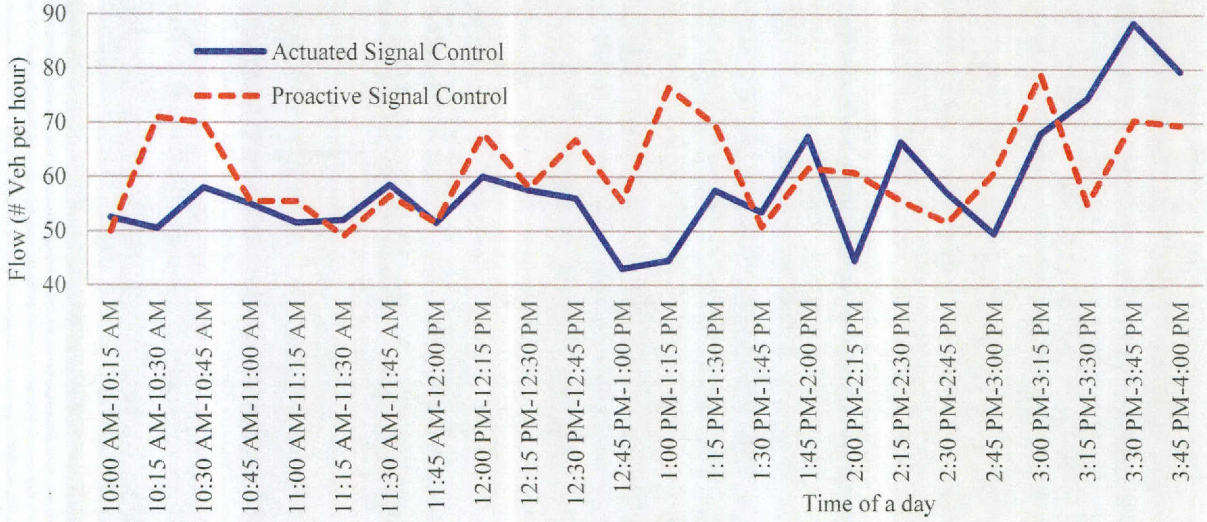
Figure 3.6.6 *Multi-day average westbound traffic of the intersection at Falcon Ridge Blvd. on weekend*

3.6.2 Comparison of Traffic Flows on Minor Roads

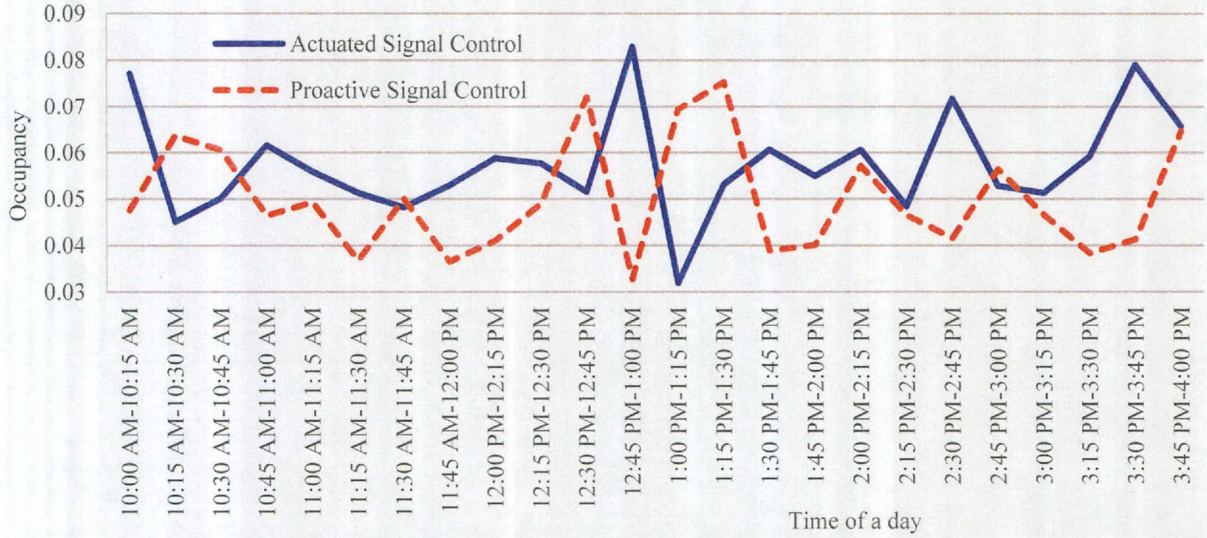
The impact of the proposed proactive signal control system on the traffic of minor roads were also studied. Actually, at this testbed, a major problem found during onsite inspections is the left-turn traffic from a minor road—Briar Creek Dr. (the other side of FM 528 is called Falcon Ridge Blvd.)—to FM 528. A long queue could be seen waiting for the signal on this minor road.

Desota St.

Figures 3.6.7 and 3.6.8 report the average flow and occupancy for northbound traffic on Desota St. (minor road) toward the intersection on weekday and weekend, respectively. It is seen that after applying the new proactive signal control system, larger flows were observed. On average, it was increased by 13% in weekday, and even 40% in weekend. On the other hand, however, the recorded occupancy rates are still quite small, even less than 0.1. At this low level of occupancy, it was still found that the occupancy rate dropped on average, especially in weekday. But it should be noted that given such a small occupancy rate, the change is not significant.

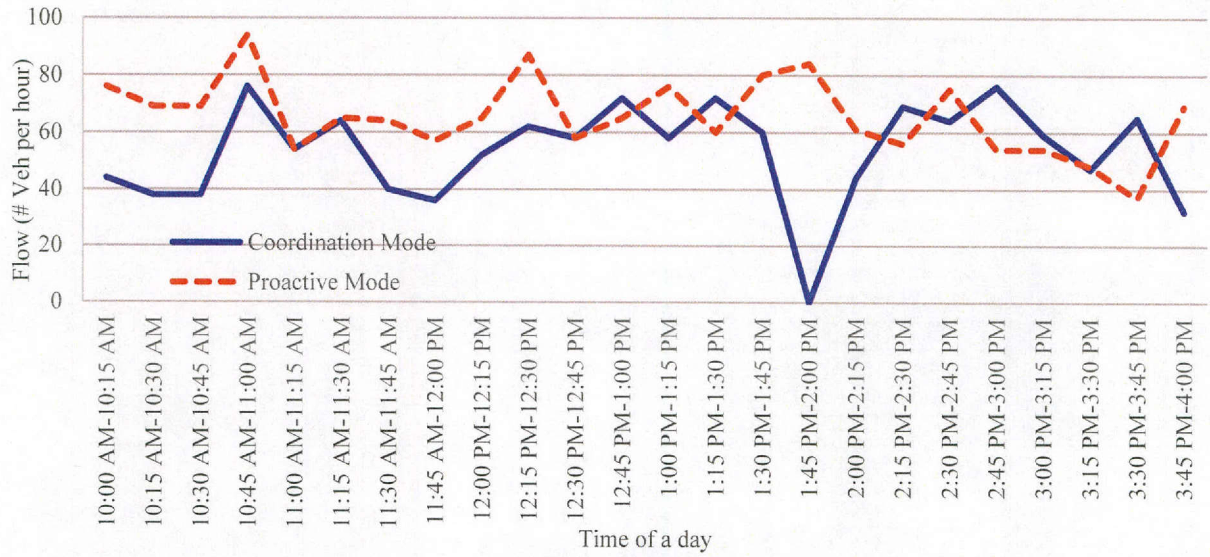


(a) Hourly Volume

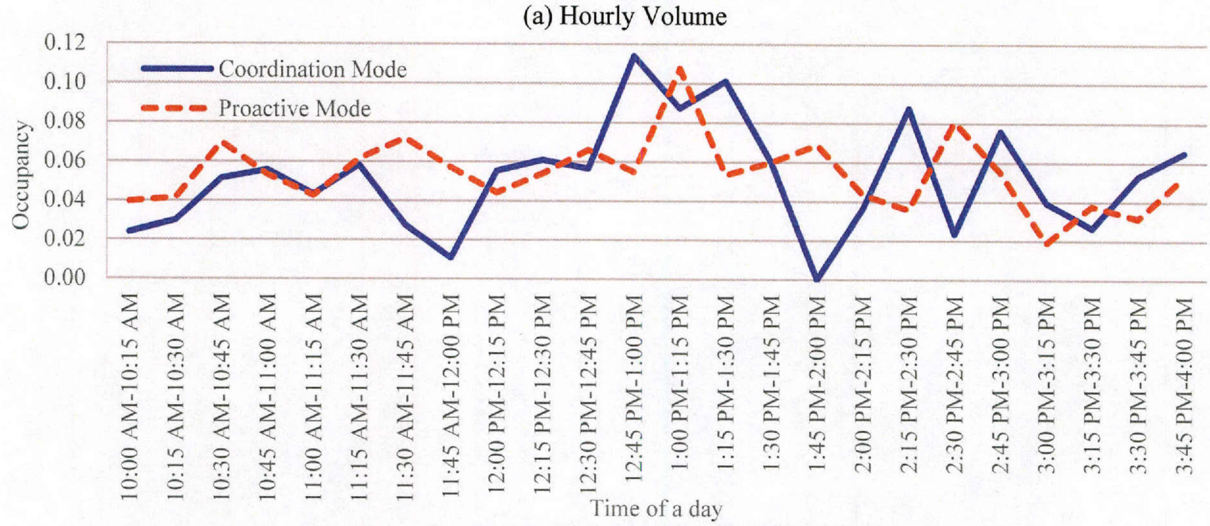


(b) Average Occupancy (in percentage)

Figure 3.6.7 *Multi-day average northbound traffic from Desota St. on weekday*



(a) Hourly Volume

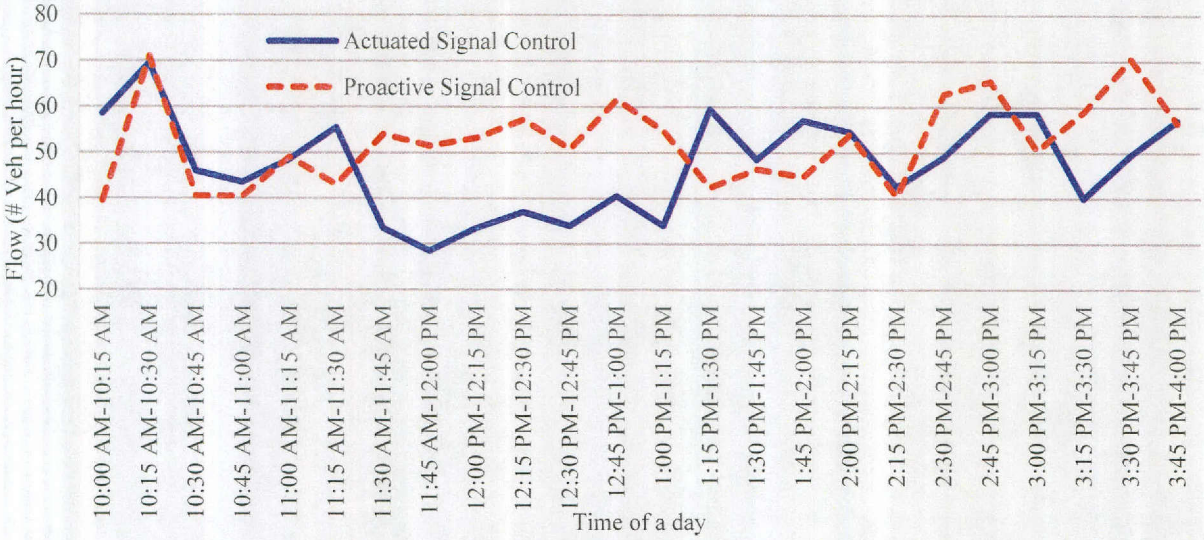


(b) Average Occupancy (in percentage)

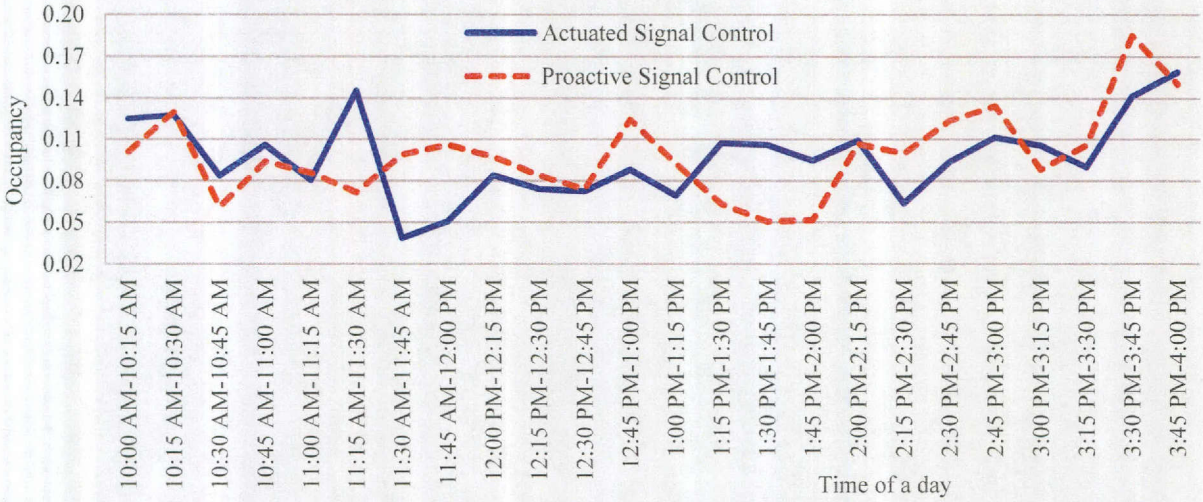
Figure 3.6.8 *Multi-day average northbound traffic from the intersection of Desota St. on weekend*

Friendswood Blvd.

Figures 3.6.9 and 3.6.10 reports the comparison of the traffic flows and occupancies for the northbound traffic on Friendswood Blvd toward the intersection on weekday and weekend, respectively. Similar to the traffic patterns seen at Desota St., it was found that the flow rates increased largely after implementing the proposed proactive signal control system. Still, the occupancy rates are small—in most cases, they are no more than 0.15.

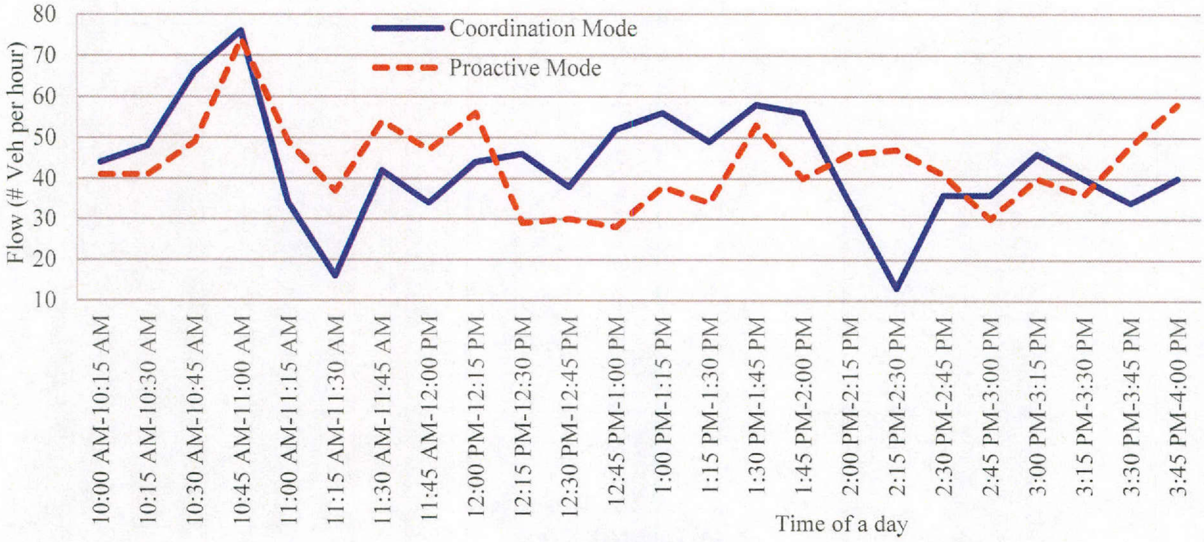


(a) Hourly Volume

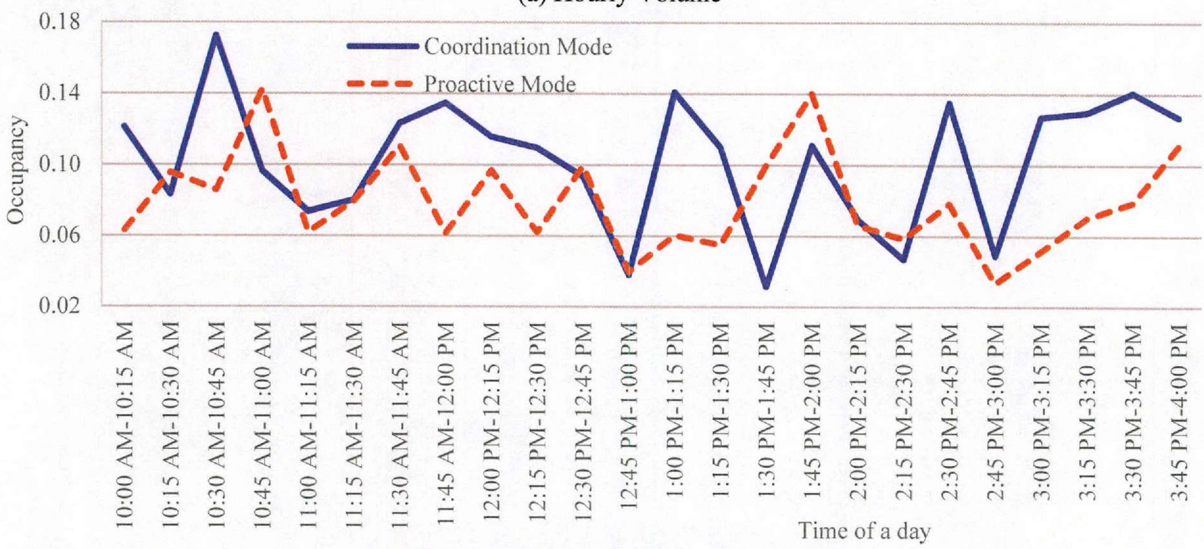


(b) Average Occupancy (in percentage)

Figure 3.6.9 *Multi-day average northbound traffic from Friendswood Blvd. on weekday*



(a) Hourly Volume

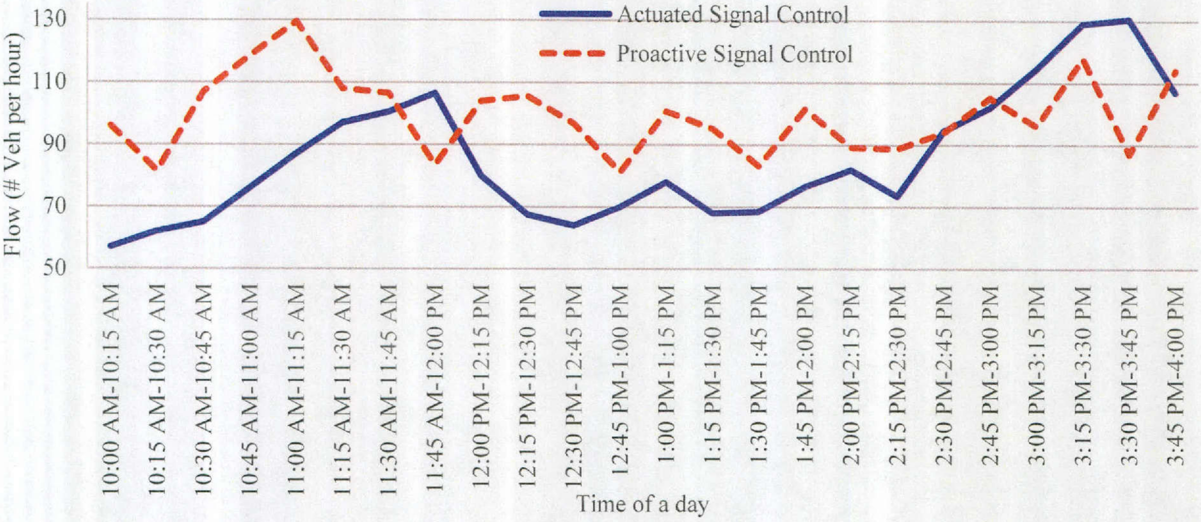


(b) Average Occupancy (in percentage)

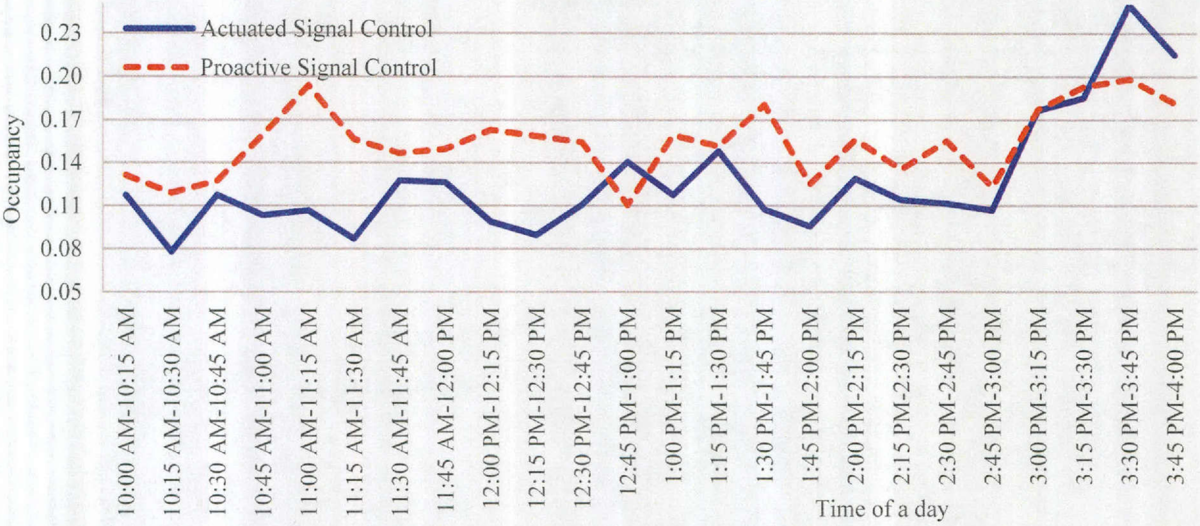
Figure 3.6.10 *Multi-day average northbound traffic from Friendswood Blvd. on weekend*

Briar Creek Dr./Falcon Ridge Blvd.

Finally, the traffic patterns on the minor road—Briar Creek Dr./Falcon Ridge Blvd.—were investigated. This minor street was found to the busiest one among all three intersections. For brevity, we look at the southbound traffic on Briar Creek Dr. toward the intersection. It was found that this is the major problem at this testbed—a long queue can be seen even during the non-peak midday hours. The comparisons were reported in Figures 3.6.11 and 3.6.12, for weekday and weekend, respectively. It is seen that the proactive signal control system also performs well for the traffic in this direction. More vehicles were allowed to go through this intersection on both weekday and weekend. The occupancy also increased but the increase is small in most of time.

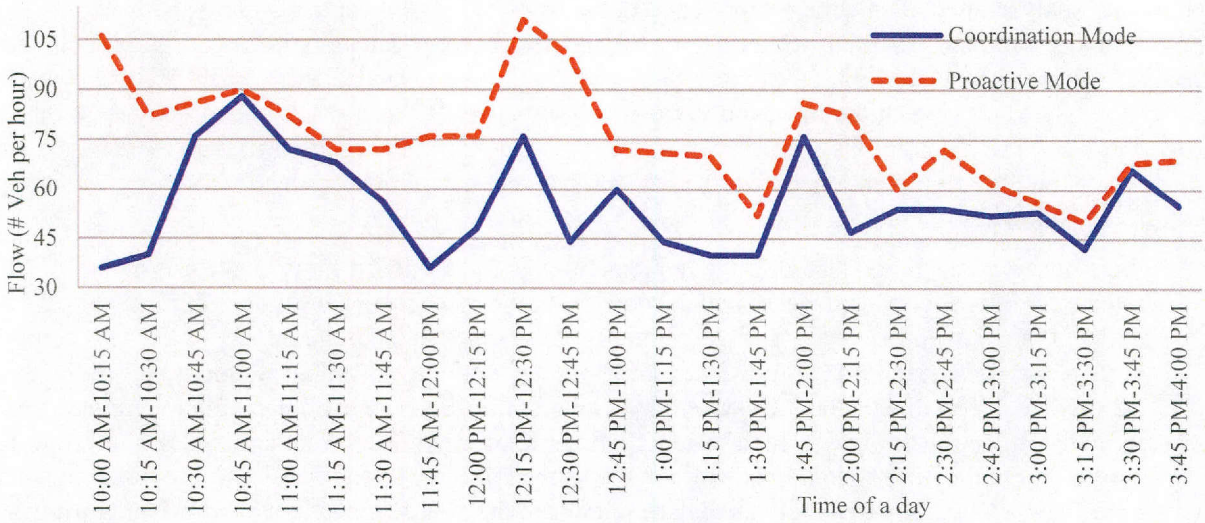


(a) Hourly Volume

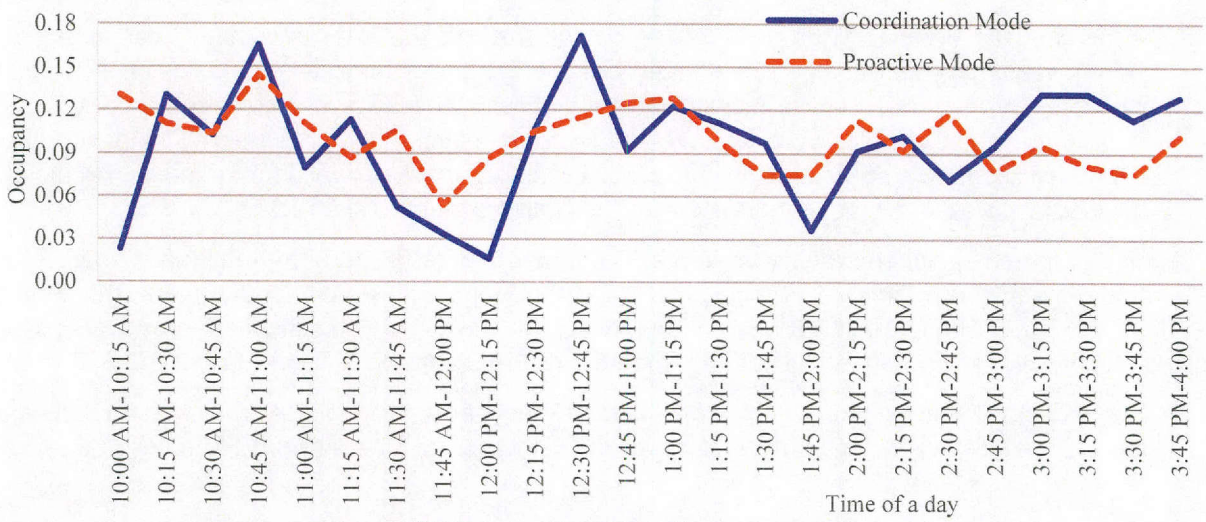


(b) Average Occupancy (in percentage)

***Figure 3.6.11** Average southbound traffic on Briar Creek Dr. toward the intersection on weekday*



(a) Hourly Volume



(b) Average Occupancy (in percentage)

***Figure 3.6.12** Multi-day average southbound traffic from Briar Creek Dr. on weekend*

3.7. SUMMARY

Based on the data collected in April and May 2017, the research team developed the proactive signal control system for the testbed of FM 528 in July 2017, and then implemented it on August 3, 2017. In weekdays, the proactive signal control is applied to the midday period (9:30 am–2:00 pm) to replace the original actuated signal control; in weekends (Saturday and Sunday), the proactive signal control is applied to a much longer period from 7:00 am to 11:00 pm to replace the original coordination signal control mode. Note that in weekdays, the original pretimed coordinated signal control system is still effective during morning and evening peak hours, because the onsite observation when conducting the implementation revealed that it is critical to maintain the signal coordinated when the through traffic volumes on the major road are high.

Before implementation, the proactive system was first tested via simulations in VISSIM. The simulation showed that this system can well reduce the average delay and average number of stops, as well as increase the average speed of vehicles that go through the intersection.

The traffic data were collected by the detectors on the testbed for about one week under the original and new signal control systems, respectively. The average of traffic flows and occupancies over multi-day were used to analyzed to evaluate the performance of the proactive signal control system.

A major problem was that occupancy rates reported by loop detectors at three intersections are quite small on major road (FM 528)—in most cases, they were less than 0.2. Such small occupancy rates imply that vehicles' movement are more independently and randomly. As a result, the effectiveness of the proposed proactive signal control system may not be clearly seen for the through traffic on the major road.

Based on the aforementioned comparison of the real data from onsite observations and loop detectors, the followings were found.

1. Field observations found that after the implementation, the proposed signal control system did help reduce the queue length and delay in weekdays in most approaches in weekdays during the midday hours (11am–1pm). However, during the midday hours in weekend, the impact of the new control system was not so significant. In weekend, the midday hours are peak hours, and it was found that the pre-timed coordinated control system may be more effective to handle the high volumes of the through traffic on the major road. Such findings related with coordinated signal control were considered in the implementations work on the testbed of SH 242 and FM 1464: the research team developed a new logic to handle the balance between the coordinated signal control and proactive signal control. Please see Chapter 4 and 5 for the details.
2. Generally, the benefit of the new control on the through traffic on the major road is not so significant based on the comparison of multi-day average hourly flow and occupancy rates. In most cases, the through traffic on the major road saw quite small occupancy rates, even for peak hours, indicating that the vehicles were not largely impacted by others. More importantly, it indicates that in most cases, there is no platoon of vehicles. Since this logic was designed for a platoon of vehicles, the effectiveness of the proposed proactive signal control is not so significant.
3. The proposed signal control system did help improve the traffic flow from the minor road. This finding is important. It seems that the original control logic focuses on the through traffic. When the flow on the minor road is large, the original control may not well handle this minor traffic. This finding was also used in model building for other testbeds, especially the testbed of SH 242.
4. As reported by Steve Chiu, the Houston District Office of the Receiving Agency has not received any compliant since the proactive system was implemented on the testbed of FM 528.

CHAPTER 4

IMPLEMENTATION AT TESTBED OF SH 242

4.1. INTRODUCTION

The testbed of State Highway 242 (i.e., SH 242) is located at north suburb of Houston, near The Woodlands. The section of SH 242 used as the testbed is on the west side of IH-45. This testbed includes an independent intersection at (1) Green Bridge Dr., and seven others at (2) Alden Woods, (3) Gosling Rd, (4) Fellowship Dr., (5) Honor Roll Dr., (6) Achievement Dr., (7) Maverick Dr. and (8) St. Lukes Way, as shown in Figure 4.1.1. The detailed road geometry (including the locations of loop detectors) and original signal plans of these intersections were reported in Chapter 2.

Among those seven intersections, five—Fellowship Dr., Honor Roll Dr., Achievement Dr., Maverick Dr. and St. Lukes Way (on the right side of the testbed)—are closely spaced and are interconnected with a fully coordinated signal system. Therefore, for brevity, only one of these five intersections, Achievement Dr., is selected for the analysis in this report (please see the detailed explanation below). On the other hand, the leftmost intersection at Green Bridge Dr., is far away from other intersections, so the traffic signal of this intersection runs in a completely free mode without needing to consider coordination with other intersections. The signal controls at other two at Alden Wood and Gosling Rd., respectively, can be treated in a mode between a free and fully coordinated mode: when the traffic is low (non-peak hours), a fixed time plan with coordination with other intersections would be good enough; while if the traffic is heavy, it would be more important to adjust the signal plan accordingly based on the incoming traffic flow (please see Section 4.3.1 for the details).

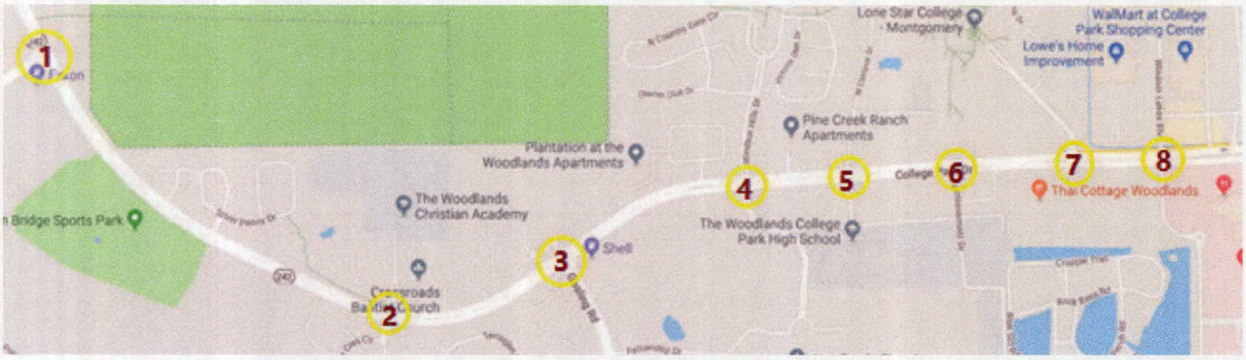


Figure 4.1.1 *Intersections at the testbed of SH 242*

The first implementation to this testbed was conducted on October 2, 2017. In this implementation, the basic logic/algorithm of the proactive signal control applied to the testbed of FM 528, was applied on the testbed of SH 242 in all intersections. After the implementation of the proactive settings, the research team from the Performing Agency, together with the engineers from the Houston District Office, observed the traffic at the testbed. Unlike the testbed of FM 528, the testbed of SH 242 experiences serious congestion on major road. Especially, as mentioned above, five intersections (from Fellowship Dr. to St. Lukes Way) are very closely located each other. Therefore, it is very important to have them coordinated in the signal control.

For this reason, during the implementation (testing while observing the traffic), their basic original coordination mode was still kept, but the logic on the flow and occupancy used in the proactive signal control was imbedded into the original coordination mode. On the other hand, the intersection at Gosling Rd., suffers from heavy left-turn traffic from the major road. After the first implementation, the observation found that the results were not so effective due to the heavy traffic. Moreover, it was found that the algorithm was not well compatible with the original coordination mode implemented on the testbed, which is important as intersections are closely spaced and traffic is heavy. For this reason, based on the results of the first field experiment, the research team discussed with Steve Chiu from the Houston District Office about adding some new logic statements in the proactive signal system to tackle the heavy congestions at this testbed, especially for three intersections: Green Bridge Dr., Alden Woods, and Gosling Rd.

On November 20, 2017, the research team from the Performing Agency implemented the improved proactive signal control system at the two intersections: Green Bridge Dr. and Alden Woods. The data collected after the implementation showed positive results. Then, in December 2017, based on the logic for these two intersections, the research team from the Performing Agency developed a logic for the intersection at Gosling Rd., which experiences heavy left-turn traffic. Due to the holiday, the research team did not go to the testbed. Instead, the implementation on that intersection was finally conducted remotely from by Steve in early January 2018.

The Performing Agency had requested the Receiving Agency to monitor this testbed before and after the implementation. The Receiving Agency provided the real detector data of the intersections in the testbed covering from October 2017 to March 2018 (different time periods were used for different intersections due to the implementation was not completed for all intersections). The data were used to evaluate the performance of the proposed signal control logic/algorithm applied to different intersections at this testbed.

The reminder of this chapter is organized as follows. Section 4.2 describes the signal timing setting for the proactive signal control system; Section 4.3 describes the latest logic statement settings for the proactive system; Section 4.4 describes the simulation testing; Section 4.5 provides analysis of field observation data; Section 4.6 summarizes and analyzes the data recorded by the testbed detectors; and Section 4.7 summarizes this chapter.

4.2. SIGNAL TIMING SETTING

At first, the signal timing plans provided by the Receiving Agency was modified for the purpose of simulation. Tables 4.2.1–4.2.4 show the changes made in the ASC/3 Controller for the intersections at Green Bridge Dr., Alden Woods, Gosling Rd. and Achievement Dr. respectively. Term "old" in these two tables refer to the parameter values of the existing signal control settings implemented by the Receiving Agency. All of these parameters can be edited through the function of MM (2-1) in the ASC/3 Controller.

The volume and occupancy data (recorded by loop detectors) were updated for a defined interval to fetch the latest information of the intersection. Therefore, it is recommended to set the vehicle extension time equivalent to the updating interval. The updating interval of 5.0 seconds for the ASC/3 controller were found to be the best to optimize the intersection performance. For the through movement on the major road (Phases 2 and 6), 4.0 seconds were set as the vehicle extension period. On the other hand, for other approaches (mainly on minor roads), a smaller value of 3.0 seconds was set to minimize the overall delay. The minimum green time was changed to 4 seconds for Phases 2 and 6 (for the through movement on the major road) and 3 seconds for other phases to expedite the skipping or switching a phase. Furthermore, the maximum green time was also revised to relatively larger values than that of the original settings by the TxDOT to flush the heavy traffic during the peak hours.

4.2.1 Intersections not in Coordination all the Time

The first three intersections, Green Bridge Dr., Alden Woods, and Gosling Rd. originally run on time-of-day (TOD) plan, i.e., the hours for the signal controller to run at actuated mode or semi-actuated mode are predetermined.

As mentioned in the first section, the signal control at the intersection at Green Bridge Dr. runs in a free mode—not coordinated with other intersections at any time; while the other two at Alden Wood and Gosling Rd., respectively, are coordinated with the other five intersections in the east, when the traffic from those five intersections is low. At this testbed, these three intersections were focused on, because they are not too close to other intersections, so that the research team would have more freedom to revise their signal plan. Please see Section 4.3.1 for the details.

4.2.2 Intersections on Coordination all the Time

The remaining five intersections, from Fellowship Dr. to St. Luke's Way run on coordination all the time because they are very closely spaced (see Figure 4.1.1). Table 4.2.4 reports the revised signal timings of one of the five closely-spaced intersections: Achievement Dr.

Table 4.2.1 Revised signal timing parameters for the intersection at Green Bridge Dr.

Phase	1 (old)	**1**	2 (old)	**2**	4 (old)	**4**	5 (old)	**5**	6 (old)	**6**
Minimum green time	5	**3**	20	**4**	5	**3**	5	**3**	20	**4**
Maximum green time	10	10	60	**80**	25	**55**	60	**80**	60	**80**
Vehicle extension time	2.0	**3.0**	3.0	**4.0**	2.0	**3.0**	3.0	**3.0**	3.0	**4.0**

Note: "old" refers to the existing parameter values used by TxDOT in the actuated signal control on the testbed.

Table 4.2.2 Revised signal timing parameters for the intersection at Alden Woods

Phase	**1** (old)	**1**	**2** (old)	**2**	**3** (old)	**3**	**4** (old)	**4**	**5** (old)	**5**	**6** (old)	**6**
Minimum green time	5	**3**	15	**4**	8	**3**	8	**3**	5	**3**	15	**4**
Maximum green time	30	30	40	**55**	25	**25**	25	**25**	20	20	40	**55**
Vehicle extension time	2.0	**3.0**	2.0	**4.0**	2.0	**3.0**	2.0	**3.0**	2.0	**3.0**	2.0	**4.0**

Table 4.2.3 Revised signal timing parameters for the intersection at Gosling Rd.

Phase	**1** (old)	**1**	**2** (old)	**2**	**3** (old)	**3**	**4** (old)	**4**	**5** (old)	**5**	**6** (old)	**6**
Minimum green time	5	**3**	15	**4**	8	**3**	8	**3**	10	**3**	15	**4**
Maximum green time	20	20	40	**50**	25	25	25	25	25	**45**	40	**50**
Vehicle extension time	2.0	**3.0**	2.0	**4.0**	2.0	**3.0**	2.0	**3.0**	2.0	**3.0**	1.7	**4.0**

Table 4.2.4 Revised signal timing parameters for the intersection at Achievement Dr.

Phase	**1** (old)	**1**	**2** (old)	**2**	**3** (old)	**3**	**4** (old)	**4**	**5** (old)	**5**	**6** (old)	**6**
Minimum green time	5	**3**	15	**4**	8	**3**	8	**3**	5	**3**	15	**4**
Maximum green time	20	20	40	**50**	25	25	25	25	20	20	40	**50**
Vehicle extension time	2.0	**3.0**	2.0	**4.0**	3.5	**3.0**	2.0	**3.0**	2.0	**3.0**	2.0	**4.0**

Note: "old" refers to the existing parameter values used by TxDOT in the actuated signal control on the testbed.

4.3. DEVELOPMENT OF LOGICAL STATEMENTS

4.3.1 General Logical Statements

Basic Logic/Algorithm

The basic logic/algorithm of the proactive control system is already described in Chapter 1 and 3, respectively. Note that when applying the basic proactive logic (see Figure 4.3.1) to the testbed of FM 528, it has already been found that this logic may not be effectively help the traffic if the through traffic volumes are high and the spacing between adjacent intersections are close. Therefore, for the testbed of FM 528, this proactive signal control system was only applied to the off-peak hours from 9:30 am to 2 pm.

New Logic Statements for Improved Proactive Control System

As mentioned in Section 4.1, the testbed of SH 242 suffers from heavy congestions, and intersections are spaced closely. Originally, except the intersection at Green Bridge Dr, the intersections of testbed of SH 242 were run under the coordination mode with semi-actuated system, i.e., major phases were fixed timed while the left-turn phases and minor phases were actuated.

On October 2, 2017, the basic logic as shown in Figure 4.3.1 (the same to the one shown in Figures 1.2.4 and 3.3.1) was applied to the intersections. However, it was found that the system could not well handle the through traffic on the major road, which were found to have high volumes, and the occupancy rates were high too (the occupancy rates were low on the major road of the testbed of FM 528). Therefore, it was necessary to consider the signal coordination between adjacent intersections, except the intersection of Green Bridge Dr., which is far away from others. On the other hand, different from the traffic on the testbed of FM 528, the left-turn traffic from the major road to minor roads, and the traffic from the minor roads see higher volumes at three intersections at Greenhouse Rd., Alden Wood and Gosling Rd. Especially, the left-turn traffic to Gosling Rd. in midday hours (11 am–2pm) were found particularly large.

The implementation at the testbed of FM 528 has shown that the proposed proactive signal control is particularly effective in handling the left-turn and minor traffic. Therefore, to balance the coordinated signal control (for the through traffic on the major road) and proactive signal control (mainly for the left-turn and minor traffic), the research team from the Performing Agency revised the basic logic by adding new logic statements, especially focusing on the *left-turn traffic* and *traffic on minor roads*.

The purpose of these new logic statement is to provide an automatic switch between coordination mode and actuated mode, so that the high volume of vehicles in these directions in peak hours can be released in the actuated control mode and the controller switches back to the coordination mode (with fixed-time plan) in non-peak hours. Figure 4.3.2 illustrates the flowchart of the algorithm of the new logic statements. Note that when the flows in these directions is low, the system runs in the coordination mode to enable the through traffic passing these intersections smoothly.

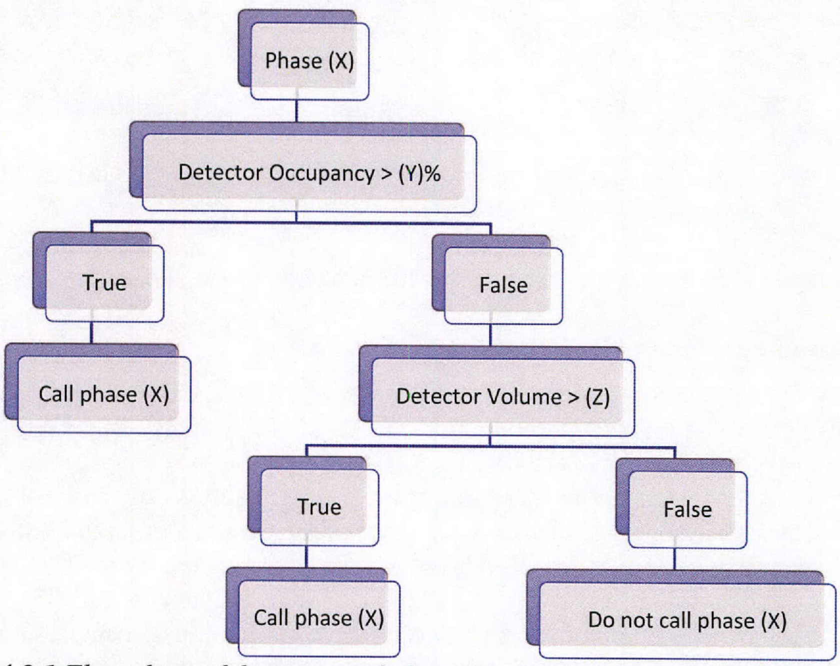


Figure 4.3.1 *Flow chart of the proposed algorithm for the proactive signal control*

The ASC/3 controller allows us to build logic statements based on GAP OUT and MAX OUT, respectively. GAP OUT refers to phase termination due to a lack of vehicle calls within a specific period of time; and MAX OUT refers to phase termination after reaching the designated maximum green time because of high volume of vehicles. The MAX OUT is used only when the controller is in the actuated mode; while a GAP OUT is identified when the controller is at either the coordination or actuated mode. Therefore, GAP OUT feature was used to build the logic statements that provides the switch between the coordination and actuated mode. Note that here the "actuated mode" is different from the original actuated mode implemented at some intersections. Originally, the actuation is simply based on the arrivals of individual vehicles. In this implementation, however, if upstream detectors are available at an intersection, the "actuated mode" is more like a "proactive mode", i.e., a short-term prediction is used to control the signal accordingly for the arrival of a platoon of vehicles.

As demonstrated in Figure 4.3.2, when operating in the actuated mode at one intersection, if simultaneous GAP OUT occurs in all of the left-turn and minor phases for a certain number of consecutive cycles (N_x, where x is for phase x), the operation mode switches to the coordination mode (fixed time plan) from the actuated mode. Note that if GAP OUT keeps occurring (i.e., phase termination due to a lack of vehicle calls), it implies the traffic flow is small. On the other hand, if there are no simultaneous GAP OUT (i.e., MAX OUT keeps occurring) in one of the phases during a certain number of cycles, it indicates high traffic flow. Therefore, the signal operation switches back to the actuated mode. In the following, the values of N_x for phase x will be given.

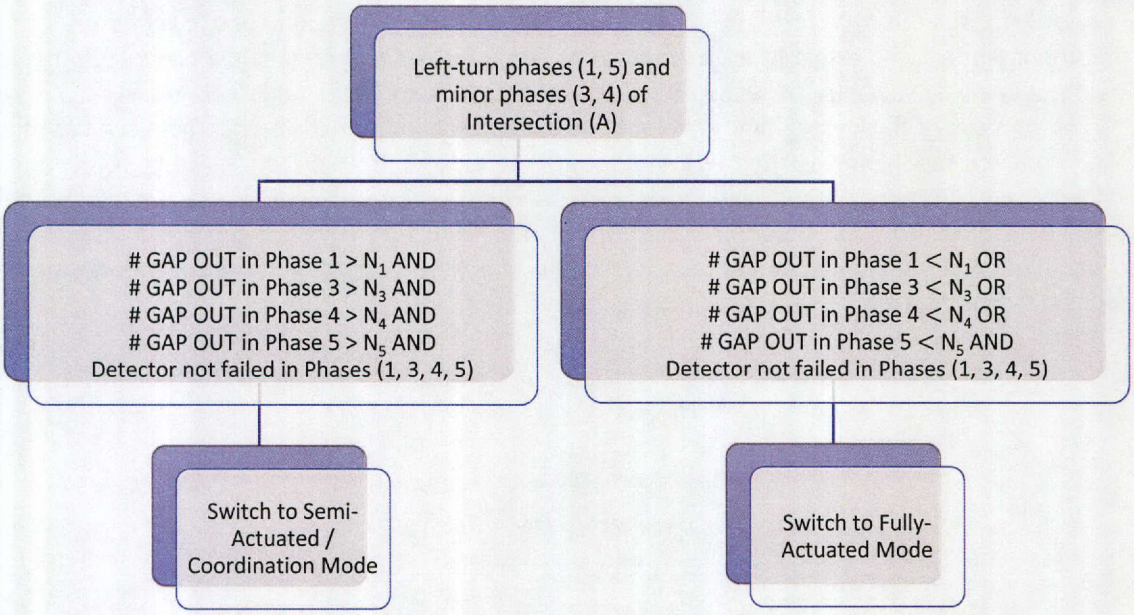


Figure 4.3.2 *Flow chart of the new logic statements for the proactive signal control*

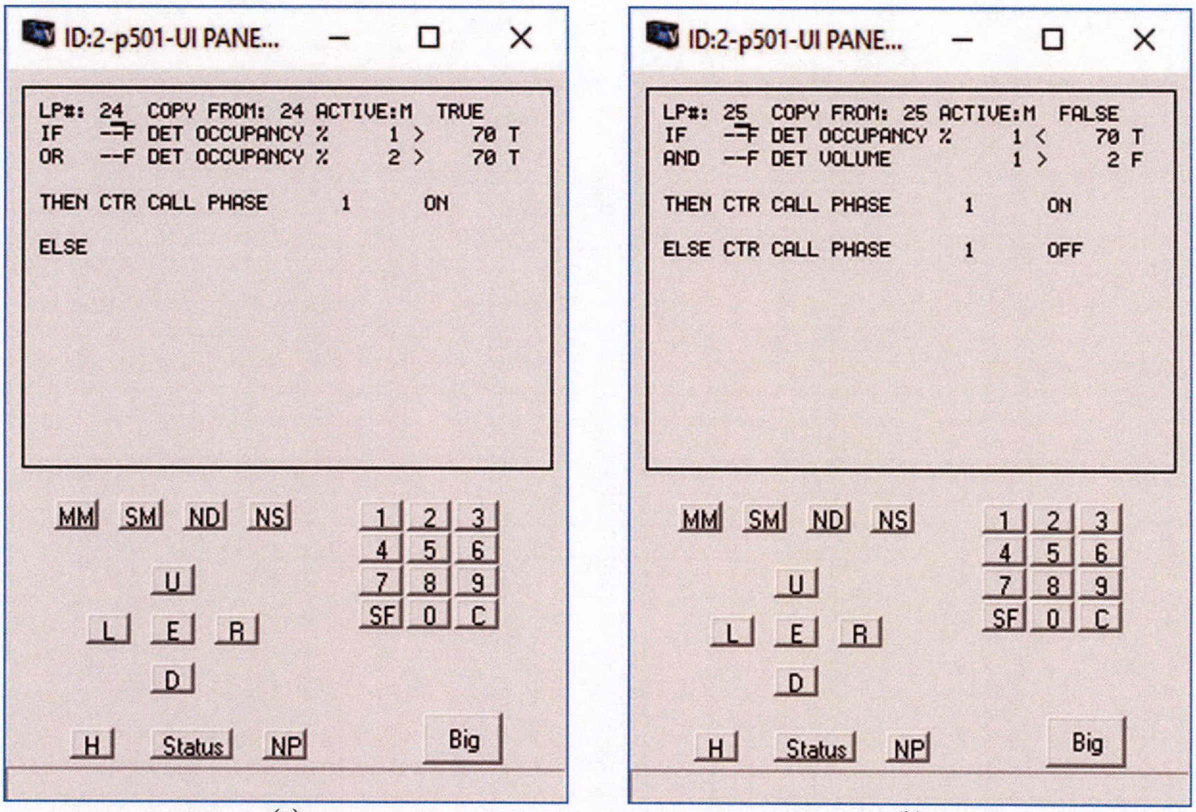
4.3.2 Logic Statements in ASC/3 Econolite Controller

The proactive system uses the parameters such as VOLUME, OCCUPANCY and GAP OUT in its logic statements to control the signal operation. For the details of how to use VOLUME and OCCUPANCY in the logic statements, please refer to the TM-4.3 Report.

Figures 4.3.3 and 4.3.4 show the application of the logic statements in the ASC/3 Econolite emulator in VISSIM. Tables 4.3.1–4.3.3 shows the value of the parameters set in VISSIM for simulation at intersections at Green Bridge Dr., Alden Woods and Gosling Rd., respectively.

Figures 4.3.5 (a) and (b) show the application of logic statements in VISSIM to automatically switch the ASC/3 controller's operation mode between the coordination and actuated modes, respectively. This logic uses GAP OUT and MAX OUT in semi-actuated phases, applied to left-turn phases (1 and 5) and minor phases (3 and 4), in order to switch between the coordination and actuated modes. If any detector in any phase fails, then the controller cannot identify any GAP OUT. Therefore, both logics work only when all detectors of selected phases are functioning properly.

If simultaneous GAP OUTs do occur for two consecutive cycles in all of these four phases (i.e., none one of the phase has simultaneous MAX OUT during two consecutive cycles), then the controller's operation switches to the coordination mode due to low traffic flow for all directions. If simultaneous GAP OUTs occur for more than **two** consecutive cycles (i.e., $N_x = 2$) in any of these four phases (Phases 1, 3, 4, and 5), then the ASC/3 controller switches to the actuated mode. Table 4.3.5 shows the values of GAP OUT and MAX OUT used in logic statements for each intersection.



(a)

(b)

Figure 4.3.3 *Logic statements for Phase 1 (Stop bar Detector (SD)) at the intersection at Gosling Rd.*

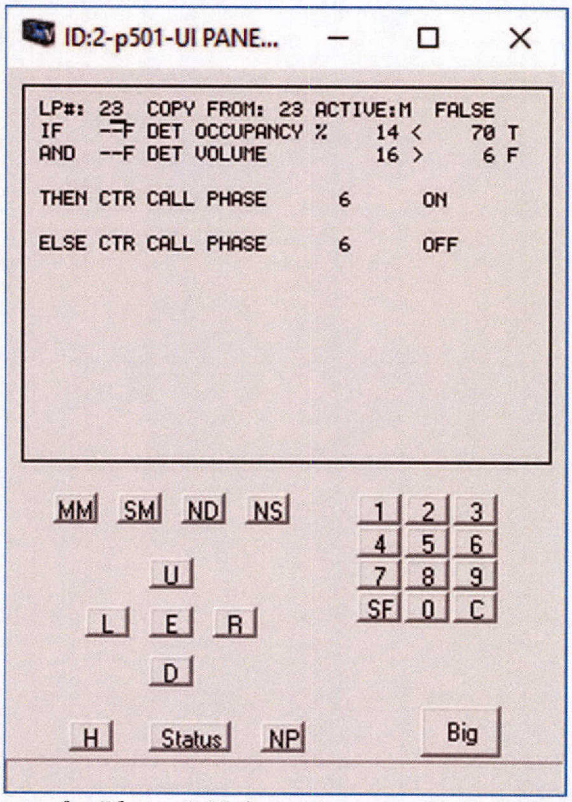


Figure 4.3.4 *Logic Statements for Phase 6 (Volume Detector (VD)) at the intersection at Gosling Rd.*

Table 4.3.1 Selected occupancy and volume margins for the intersection at Green Bridge Dr.

Phase Number	Detector	Occupancy (%)	Volume
1	1 (SD)	70	2
2	2 (SD)	70	2
4	4 (SD)	70	2
5	5 (SD)	70	2
6	6 (SD)	70	2

Table 4.3.2 Selected occupancy and volume margins for the intersection at Alden Woods

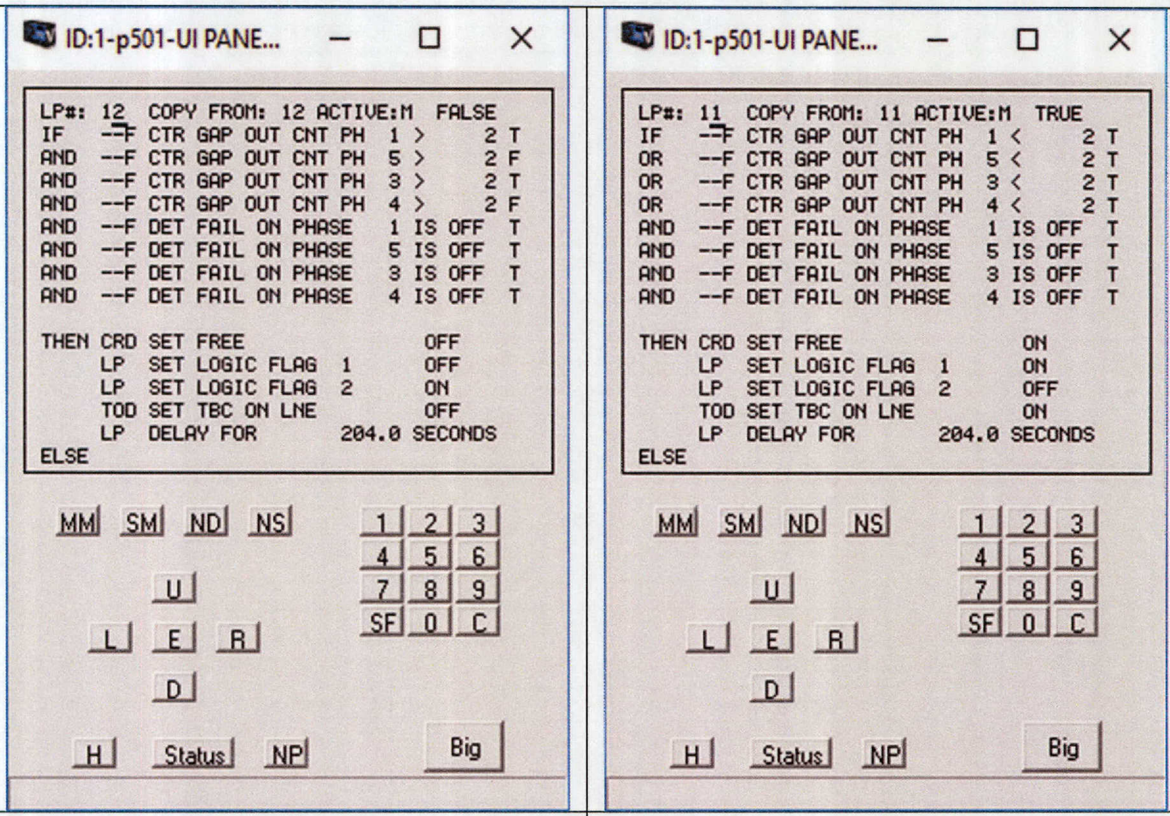
Phase Number	Detector	Occupancy (%)	Volume
1	1 (SD)	70	2
2	2 (SD)	70	2
3	3 (SD)	70	2
4	4 (SD)	70	2
5	5 (SD)	70	2
6	6 (SD)	70	2

Table 4.3.3 Selected occupancy and volume margins for the intersection at Gosling Rd.

Phase Number	Detector	Occupancy (%)	Volume
1	1 (SD)	70	2
	2 (SD)	70	2
2	3 (SD)	70	-
	4 (SD)	70	-
	17 (VD)	-	6
	18 (VD)	-	6
3	5 (SD)	70	2
	7 (SD)	70	2
4	9 (SD)	70	2
	10 (SD)	70	2
5	11 (SD)	70	2
	12 (SD)	70	2
6	13 (SD)	70	-
	14 (SD)	70	-
	15 (VD)	-	6
	16 (VD)	-	6

Table 4.3.4 Selected occupancy and volume margins for the intersection at Achievement Dr.

Phase Number	Detector	Occupancy (%)	Volume
1	1 (SD)	70	2
2	2 (SD)	70	-
	4 (SD)	70	-
	5 (SD)	70	-
	3 (VD)	-	4
	6 (VD)	-	4
3	7 (SD)	70	2
4	8 (SD)	70	2
	9 (SD)	70	2
5	10 (SD)	70	2
6	11 (SD)	70	2
	12 (SD)	70	2



(a) (b)

Figure 4.3.5 *Logic Statements to switch between Coordination control and Actuated Control*

Table 4.3.5 Selected values of simultaneous GAP OUTs and MAX OUTs in logic statements

Intersection	Phase	Number of simultaneous GAP OUTS to switch to Coordination Mode	Number of simultaneous GAP OUTS to switch to Actuated Mode	Number of simultaneous MAX OUTS to revert to Coordination Mode
Green Bridge Dr.	5	> 2 AND	< 2 OR	-
	4	> 2 AND	< 2 OR	-
Alden Wood	1	> 2 AND	< 2 OR	-
	5	> 2 AND	< 2 OR	-
	3	> 2 AND	< 2 OR	-
	4	> 2 AND	< 2 OR	-
Gosling Rd.	1	> 2 AND	< 2 OR	-
	5	> 2 AND	< 3 OR	> 5
	3	> 2 AND	< 2 OR	-
	4	> 2 AND	< 2 OR	-

Note: the above conditions are those for a switch from a current model to another mode.

In the intersection at Gosling Rd., Phase 5 (left-turn lane) is very congested and this phase sees MAX OUTs continuously for many cycles in peak hours. This causes the controller to remain in the actuated mode to release the high volume of vehicles coming from Phase 5. The vehicles at the left-turn lane benefit from this. However, this also increases the waiting time for the vehicles under other phases. To address issue, the logic statements for this intersection was modified, so that the controller would revert to the coordination mode if Phase 5 sees MAX OUTs for over five consecutive times. This helps to maintain a better balance of waiting time for the stopped vehicles at the intersection heading to different directions.

4.4. TEST VIA SIMULATIONS

4.4.1 Simulation Verification

Before field experiments, the simulation package VISSIM was used to simulate the traffic flow on the testbed with the proposed proactive signal control (as described in Section 4.3), so that we can evaluate the performance of the proposed system, and then by adjusting the parameters to find the best parameter settings. However, it is needed to first verify if the VISSIM simulation can correctly represent the real flow on the testbed. Therefore, the first step is simulation verification, by comparing the simulated traffic flow and observed one (which was summarized from data collected from loop detectors), using the same original signal plan.

Figure 4.4.1 shows the testbed models built in VISSIM: one is for the intersection of Green Bridge Dr., and another is for the remaining seven intersections (called the SH 242 testbed). As mentioned at the beginning, the intersection of Green Bridge Dr. is far away from others, so it was treated as a single testbed. Note that even though of the intersection at Green Bridge Dr., the major road (SH 242) runs in the direction from northwest to southeast, in the following, we still call the traffic on SH 242 at this intersection eastbound or westbound, for the consistency to other intersection at this testbed.

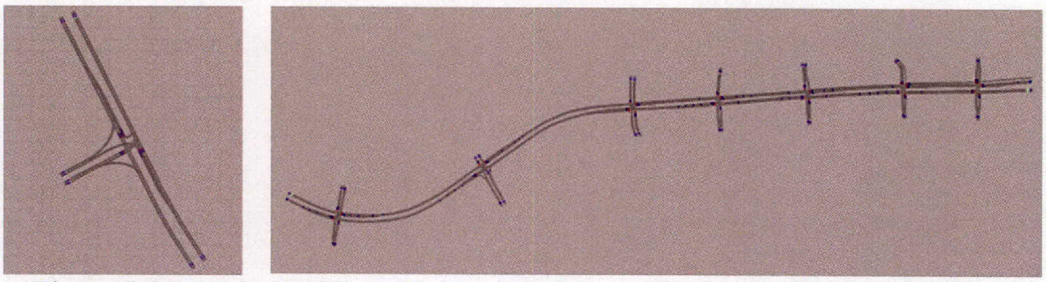


Figure 4.4.1 *Testbeds of Green Bridge Dr. (left) and SH 242 (right) built in VISSIM*

In the simulation, the traffic volume and occupancy data collected by the detectors of the testbed were utilized to estimate the demands for all origin-destination (OD) pairs. The data collected from 5/22/2017 6:32:00 am (or 5/23/2017 8:43:00 am for some detectors) to 6/8/2017 7:06:00 am were used for the simulation verification. Note that the data reported by loop detectors are the volume and occupancy per minute. The research team aggregated the volume for each 15 minutes starting from 00:01:00 am of each day. That is, for a particular day, the first 15-minute section is from 00:01:00 am to 00:15:00 am, and the second is from 00:15:01 am to 00:30:00 am. If no enough data for one 15-minute section, no data exists for this section. And finally, the multi-day average of volume for each 15-minute section was calculated respectively (as the hourly volume, i.e., 15-minute volume multiplied by 4) for weekday and weekend, respectively. These data were firstly used to estimate the origin-destination matrix of each testbed (such demand data serve as the input parameters for simulations in VISSIM), and then they were used as the base for comparing the simulated flows at this testbed.

Figures 4.4.2 and 4.4.3 report the simulated and observed eastbound and westbound though traffic flow (in weekday) on SH 242 (major road) of the intersection at Green Bridge Dr.; and Figure 4.4.4 reports the simulated and observed flow on the minor road in weekday. It is seen that the simulated flows are quite close to the observed one in each case, implying VISSIM simulation is able to well reflect the flow at this intersection.

Figures 4.4.5 through 4.4.10 report the comparison of simulated and observed traffic flows (eastbound or westbound in weekday or weekend) for three intersections: Alden Wood, Gosling Rd. and Achievement Dr., respectively, under the original signal plans implemented at these intersections. It is seen that generally, the simulated results are close to the observed ones. At Achievement Dr., the simulated results are found a little bit higher than the observed one, it may be because the estimate of the travel demand at this intersection is not accurate. However, such gap is small, indicating that the simulation results are good enough to reflect the flow on the testbed.

In the report summarizing Task 3 (TM 3.1 Network Performance Analysis, which was provided to the Receiving Agency) the research team reported the detailed simulation verification analysis at this testbed (for each intersection). That report contains the details of the simulation verification of each intersection at this testbed.

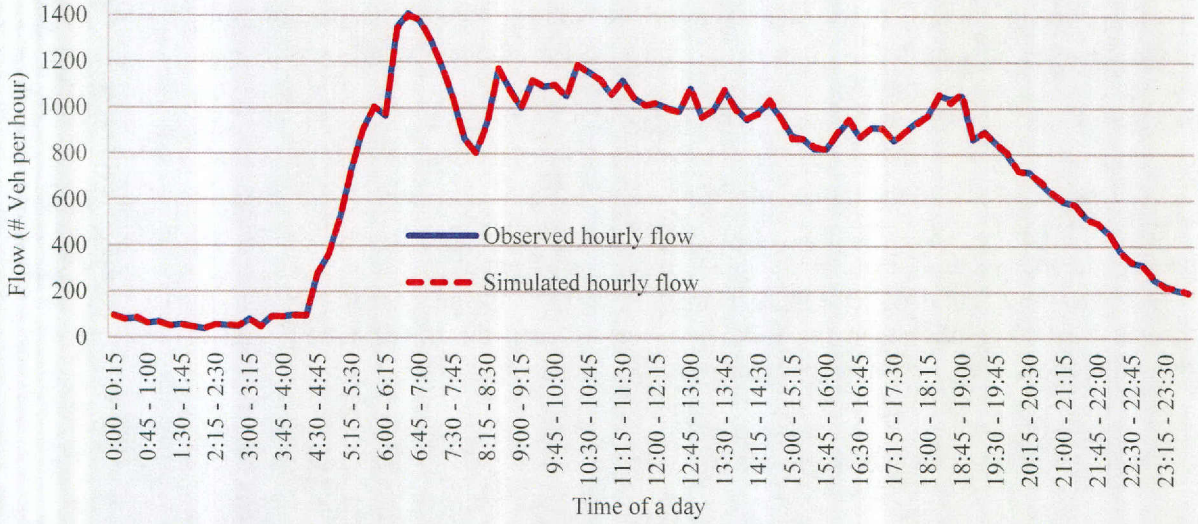


***Figure 4.4.2** Weekday's eastbound through traffic on SH 242 of the intersection at Green Bridge Dr.*

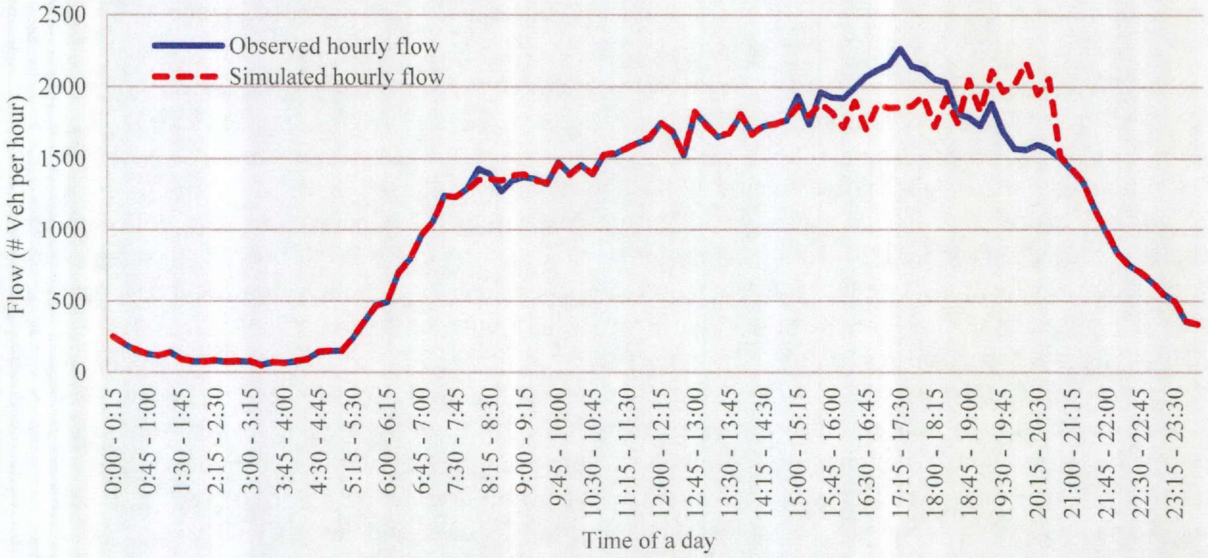


***Figure 4.4.3** Weekday's westbound through traffic on SH 242 of the intersection at Green Bridge Dr.*

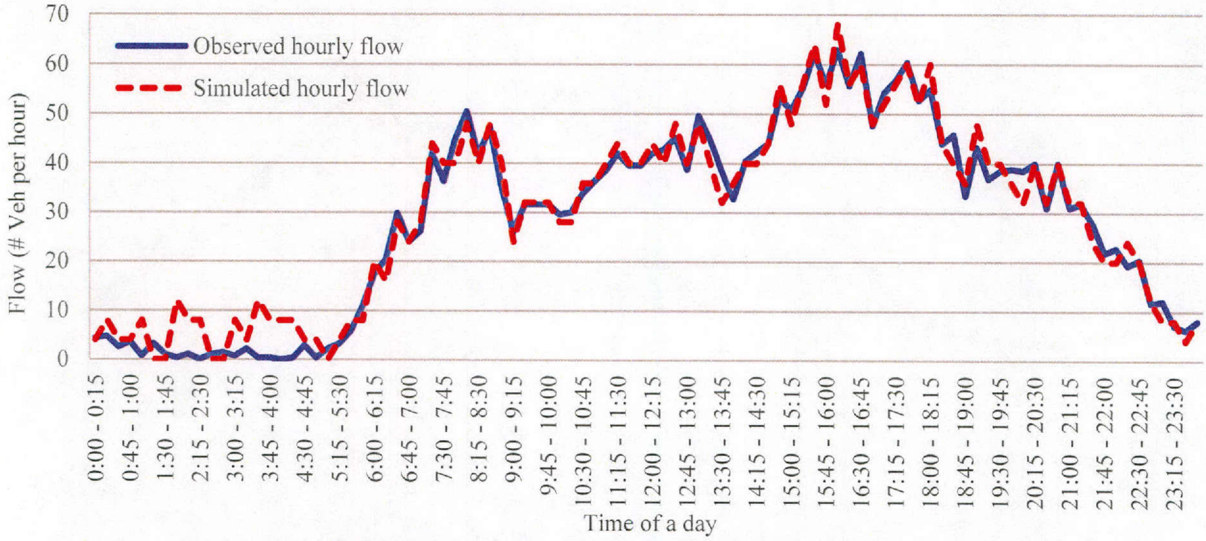


***Figure 4.4.4** Weekday's traffic volume on Green Bridge Dr. toward SH 242*

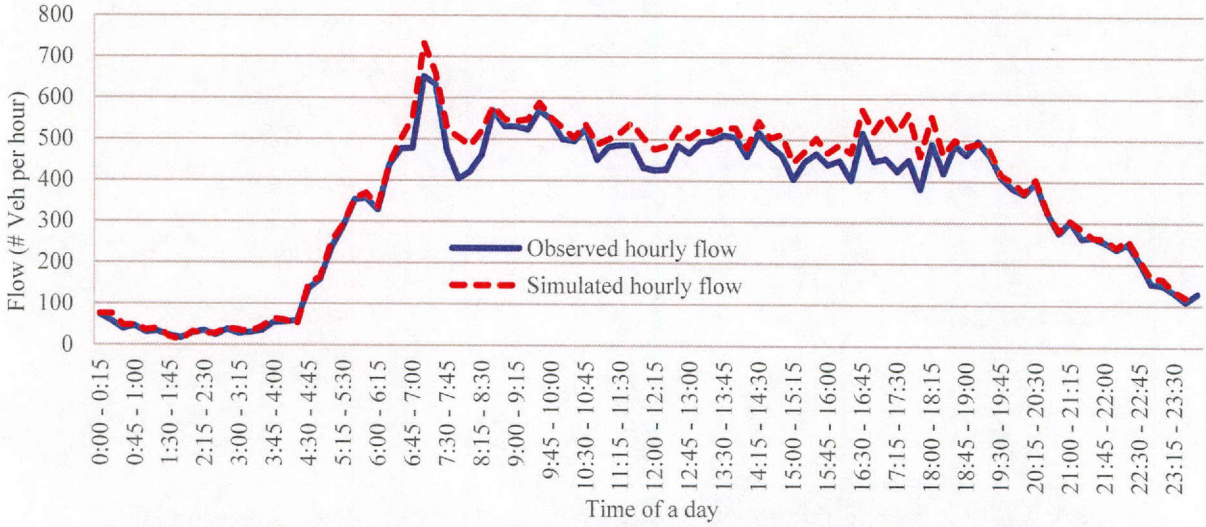


***Figure 4.4.5** Weekday's eastbound traffic on SH 242 of the intersection at Alden Wood*

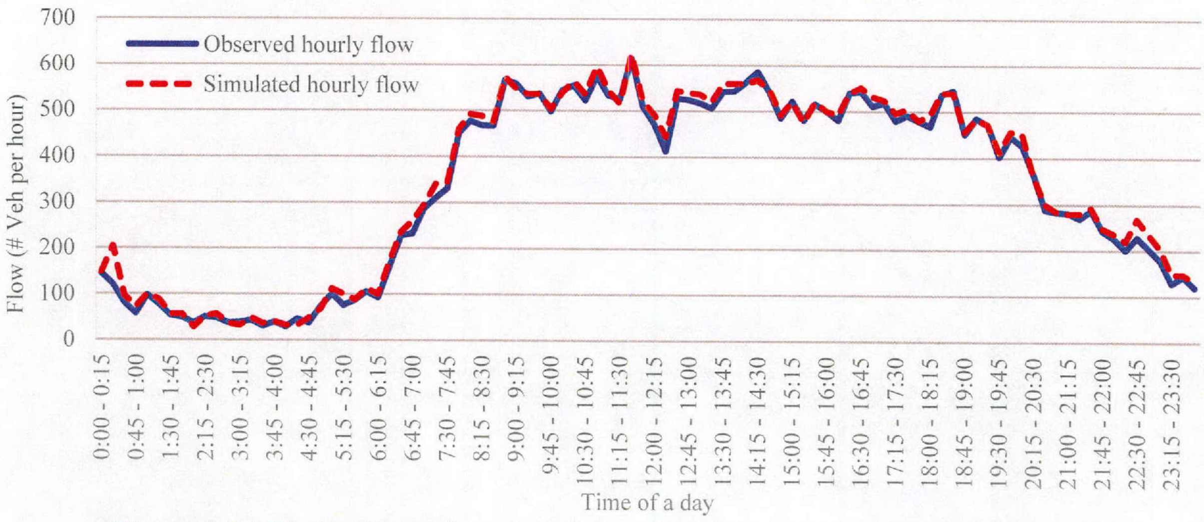


***Figure 4.4.6** Weekend's eastbound traffic on SH 242 of the intersection at Alden Wood*

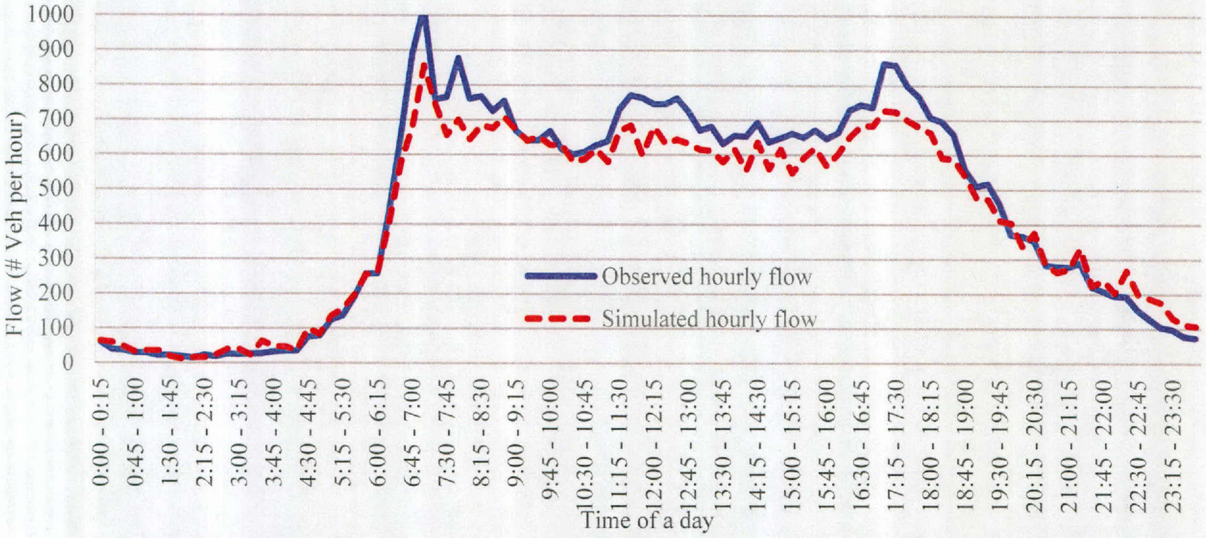


Figure 4.4.7 *Weekday's eastbound traffic on SH 242 of the intersection at Gosling Rd.*

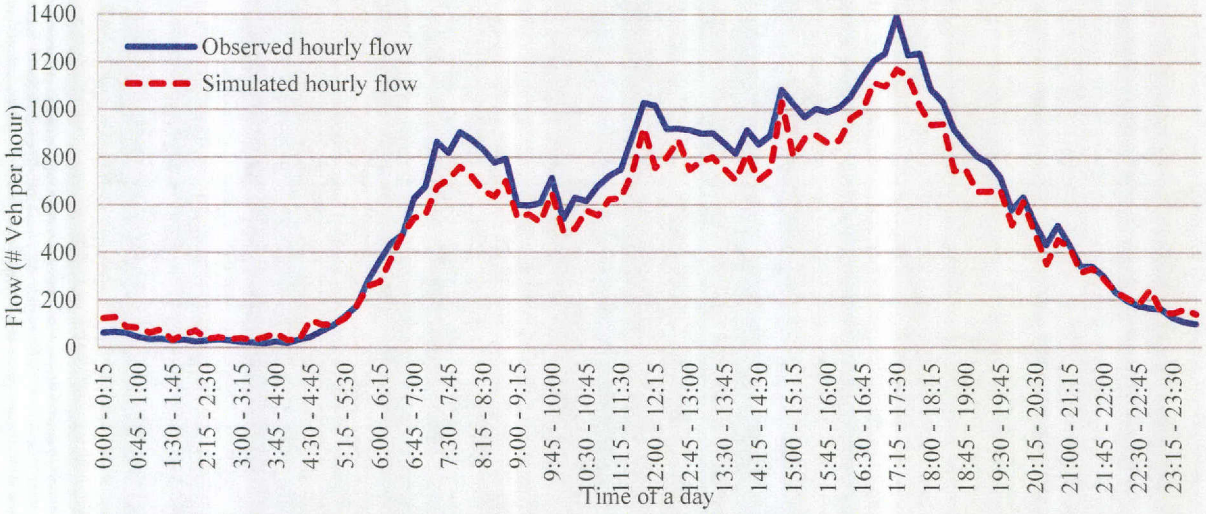


Figure 4.4.8 *Weekday's westbound traffic on SH 242 of the intersection at Gosling Rd.*

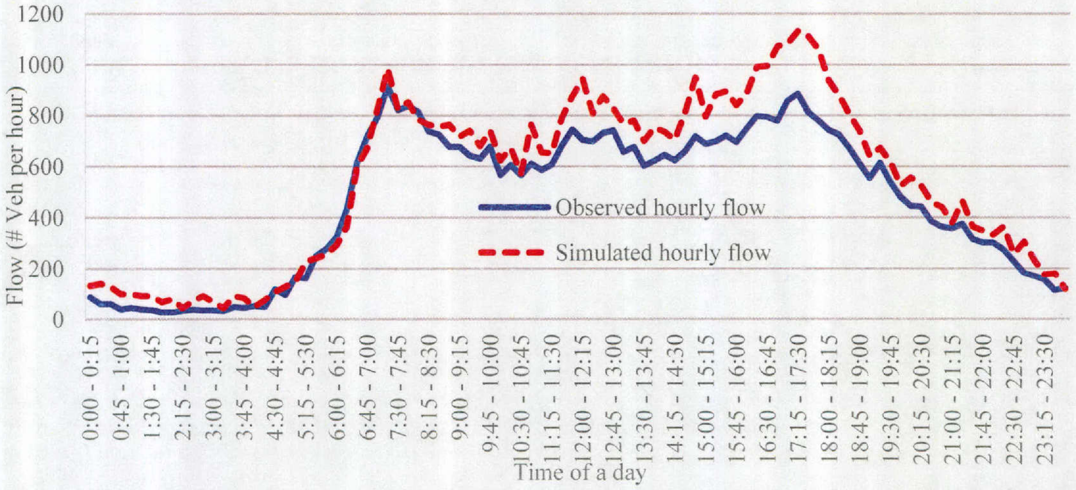


Figure 4.4.9 *Weekday's eastbound traffic on SH 242 of the intersection at Achievement Dr.*

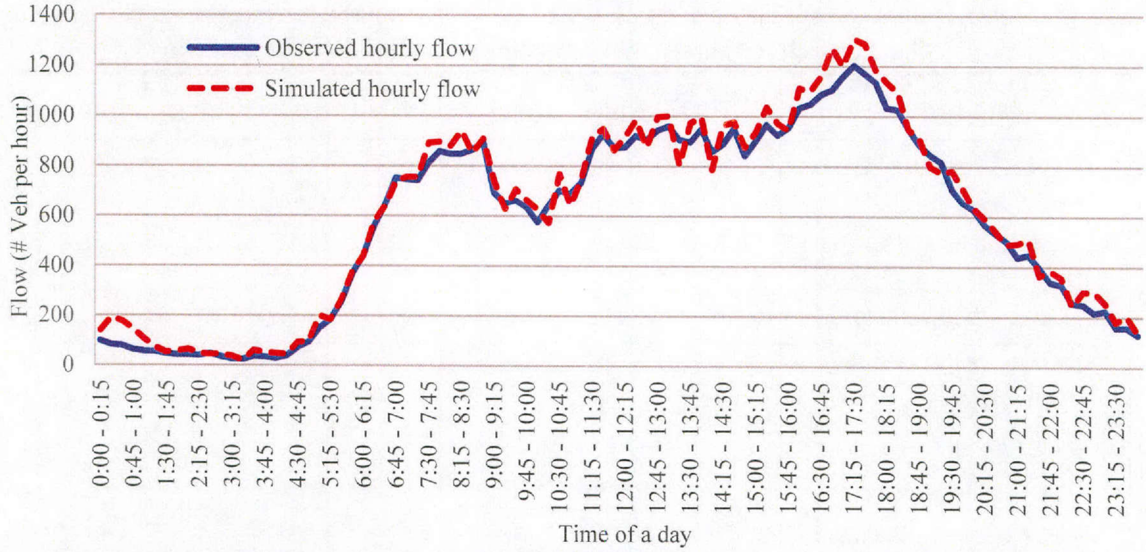


Figure 4.4.10 *Weekday's westbound traffic on SH 242 of the intersection at Achievement Dr.*

4.4.2 Test of Proactive Signal Control via Simulations

Based on the volume and occupancy pattern in peak hours collected from detector data, for each detector the research team sets different values of the critical volumes and occupancies, and the extension time in the proposed logic through the simulator of the ASC/3 controller imbedded in VISSIM. Though intensive tests via simulations, the settings of various parameters that produce the highest savings in vehicle delay via simulations were recommended for the field experiment. Please see the details in Section 4.3.2, and the values of these parameters in Tables 4.3.1–4.3.4.

Table 4.4.1 repots the results from a 30-minute's simulation (5:00 pm to 5:30 pm) for the intersection at Green Bridge Dr. for a weekday and weekend, respectively. The benefit of the proposed proactive signal control algorithm was evaluated in terms of average delay, average number of stops, total delay, etc., based on the results under the original signal control.

The delay is one of the most important parameters to measure the efficiency of a signal control system. Table 4.4.1 shows that in a weekday for the intersection at Green Bridge Dr., the total delay is reduced from 59,743 to 43,377 seconds after implementing the proposed proactive control, resulting in a 43.7% reduction in delay during an 1800-second simulation period. Consequently, the average speed of the vehicle in the network also increases. Moreover, the average delay and average stop delay is also reduced up to 43.70% and 31.74%, respectively. Due to less delay, the number of stops experienced by vehicles drops too. Therefore, it is expected to bring better driving experience on the testbed. Due to the improved performance of the testbed, the environmental hazard and monetary loss are also reduced. On the other hand, it shows that the savings on a weekend are not as significant as those on a weekday. The average delay and average stop delay is reduced by 33.94% and 47.99%, respectively.

Similar to Table 4.4.1, Table 4.4.2 reports the difference before and after implementing the proactive system in the simulation for the rest seven intersections at the testbed of SH 242. From this table, we can also see from the simulations that the proactive signal control system does help reduce the traffic congestions on this testbed on both weekday and weekend, respectively.

Since the results from simulations look good, these values of parameters were employed the field tests at this testbed, and the field test results are reported in Sections 5 and 6.

Table 4.4.1 Comparison of the simulated evening peak-hour traffic on the intersection at Green Bridge Dr. under semi-actuated and proactive signal control systems on a weekday and a weekend, respectively

	Parameters	TxDOT Signal Settings	Proactive Signal Settings	Difference
Weekday	Average Delay (seconds)	43.83	28.11	-43.70%
	Average Stop	0.98	0.62	-45.00%
	Average Stop delay (seconds)	24.64	13.15	-60.81%
	Total Delay (seconds)	59743	43377	-31.74%
	Speed Average (mile/hour)	22.93	30.46	+28.21%
Weekend	Average Delay (seconds)	21.78	15.46	-33.94%
	Average Stop	0.43	0.37	-15.00%
	Average Stop delay (seconds)	12.04	7.38	-47.99%
	Total Delay (seconds)	29300	20797	-33.95%
	Speed Average (mile/hour)	35.18	41.35	+16.12%

Table 4.4.2 Comparison of the simulated evening peak-hour traffic on the testbed of SH 242 under coordinated and proactive signal control systems on a weekday and a weekend, respectively

	Parameters	TxDOT Signal Settings	Proactive Signal Settings	Difference
Weekday	Average Delay (seconds)	85.98	81.74	-5.06%
	Average Stop	1.75	1.85	+5.56%
	Average Stop delay (seconds)	59.31	54.45	-8.54%
	Total Delay (seconds)	163620	156699	-4.32%
	Speed Average (mile/hour)	32.44	33.32	+2.68%
Weekend	Average Delay (seconds)	67.17	63.33	-5.88%
	Average Stop	1.57	1.55	-1.28%
	Average Stop delay (seconds)	48.35	44.70	-7.85%
	Total Delay (seconds)	74965	74790	-0.24%
	Speed Average (mile/hour)	35.04	36.08	+2.93%

Note: The new logic statements are implemented at three intersections—Green Bridge Dr., Alden Woods, and Gosling Rd.—in the simulation.

4.5. ANALYSIS OF FIELD OBSERVATION DATA

On October 2, 2017, the research team from the Performing Agency worked with the Receiving Agency in the first implementation. After that, the research team drove to the testbed several times to observe the traffic conditions under either the original or the new proactive signal control system for both weekday and weekend, respectively. After the field observation and analysis, some problems were found, and thus the research team made some revisions, and the second implementation was on November 20, 2017. Steve Chiu from the Houston District Office switched the signal control system upon the request of the Performing Agency during these field observations for the purpose of comparison. As mentioned in Chapter 3, due to the limitation of technology, the loop detectors currently can only report two measures: volume and occupancy. From loop detectors, we cannot know the queuing and delay information directly. Therefore, the research team relies on the on-site observations to record the queuing and delay information of vehicles at the testbed. Photos and videos were taken at these intersections for record.

The first observation was on October 3, 2017 (Tuesday) after the original proactive signal control was implemented on the last day. As requested, on October 13, 2017, TxDOT Houston Office switched the system back to the original actuated control for all eight intersections at SH 242. The second observation was on October 15-16, 2017 (Sunday-Monday). At the beginning of November, the control system was then switched to the proactive system control that was implemented on October 2, 2017. The third field observation was on November 19-20, 2017 (Sunday-Monday), just before and after the second field test on November 20, 2017, when the improved proactive control system was implemented.

In this section, the field observation covers four intersections only: (1) Green Bridge Dr., (2) Alden Wood, (3) Gosling Rd., and (4) Achievement Dr. As mentioned before, five intersections from Fellowship Dr. to St. Lukes Way, are closely spaced, and the traffic through them are heavy. The traffic conditions at these five are similar. Therefore, the research team made the observations at one only—Achievement Dr., which is located in the middle of them.

During the field observations, queue lengths and delay times were recorded for several cycles at all approaches for each intersection observed. The average values of queue lengths and delay times were reported in Tables 4.5.1 through 4.5.4, respectively for four intersections. It is seen that for weekdays, after the implementation in the second field experiment (on November 20, 2017), both vehicle queue length and vehicle delay time dropped at the intersections at Green Bridge Dr. and Alden Woods. In the best case, the average queue length dropped up to 54%, and the average delay dropped up to 35%. It is also seen that on the minor roads, the results were not so good. The average delay was even increased. Note that at these two intersections, the traffic on the minor road is quite small (see the traffic data reported in Section 4.6), so the uncertainty of the flow on the minor roads is expected to be large.

Table 4.5.1 Weekday field observation at the intersection at Green Bridge Dr. (4:30 pm–4:45 pm)

	Traffic flow	**Average queue length (# vehicles)**			**Average delay (seconds)**		
		Semi-actuated	**Proactive**	**Difference**	**Semi-actuated**	**Proactive**	**Difference**
Weekdays	Eastbound	4.8	2.2	-54.17%	54.0	37.6	-30.37%
	Westbound	5.0	3.0	-40.00%	19.2	12.4	-35.42%
	Northbound (minor)	3.6	2.8	-22.22%	44.7	84.4	+88.81%
	Left-turn (Westbound)	12.0	10.2	-15.00%	50.5	58.6	+16.04%

Note: the traffic condition under semi-actuated control was observed on October 16, 2017 and those under the improved proactive signal control was observed on November 20, 2017.

Table 4.5.2 Weekday field observation at the intersection at Alden Woods (4:45 pm–5:00 pm)

	Traffic flow	**Average queue length (# vehicles)**			**Average delay (seconds)**		
		Coordination	**Proactive**	**Difference**	**Coordination**	**Proactive**	**Difference**
Weekdays	Eastbound	5.0	2.8	-44.00%	21.0	19.2	-8.57%
	Westbound	7.2	6.8	-5.56%	30.5	20.2	-33.77%
	Northbound (minor)	3.0	3.8	+26.67%	96.0	111.0	+15.63%
	Southbound (minor)	1.3	0.0	-	44.0	0.0	-
	Left-turn (Eastbound)	2.0	1.5	-25.00%	53.0	51.5	-2.83%
	Left-turn (Westbound)	2.0	1.5	-25.00%	125.0	109.7	-12.24%

Note: the traffic condition under semi-actuated control was observed on October 16, 2017 and those under the improved proactive signal control was observed on November 20, 2017.

Table 4.5.3 Field observation at the intersection at Gosling Rd. (10:20 am–10:40 am)

	Traffic flow	Average queue length (# vehicles)			Average delay (seconds)		
		Coordination	Proactive	Difference	Coordination	Proactive	Difference
Weekdays	Eastbound	8.2	19.0	+131.71%	74.6	109.0	+46.11%
	Westbound	7.2	6.5	-9.72%	47.0	33.0	-29.79%
	Northbound	4.2	3.4	-19.05%	88.4	75.0	-15.16%
	Southbound	1.8	1.0	-44.44%	61.4	23.0	-62.54%
	Left-turn (Eastbound)	1.0	0.0	-	83.5	0.0	-
	Left-turn (Westbound)	11.0	10.5	-4.55%	77.3	62.4	-19.27%
Weekends	Eastbound	6.0	7.0	+16.67%	79.3	77.2	-2.65%
	Westbound	1.0	1.5	+50.00%	20.7	46.5	+124.64%
	Northbound	3.0	4.8	+60.00%	82.5	82.5	0.00%
	Southbound	2.2	1.3	-40.91%	60.6	66.3	+9.41%
	Left-turn (Eastbound)	1.5	1.8	+20.00%	100.0	59.8	-40.20%
	Left-turn (Westbound)	4.8	3.4	-29.17%	73.4	79.0	+7.63%

Note: the traffic conditions under semi-actuated control was observed on October 15 and16 (Sunday and Monday), 2017, and those under the proactive control was observed on October 3 (Tuesday) and November 19 (Sunday), 2017.

Table 4.5.4 Field observation at the intersection at Achievement Dr. (10:50 am–11:20 am)

	Traffic flow	Average queue length (# vehicles)			Average delay (seconds)		
		Coordination	Proactive	Difference	Coordination	Proactive	Difference
Weekdays	Eastbound	11.3	11.8	+4.43%	59.8	69.8	16.72%
	Westbound	3.8	6.6	+73.68%	35.5	72.2	+103.38%
	Northbound	1.0	1.3	+30.00%	63.8	58.5	-8.31%
	Southbound	4.8	5.0	+4.17%	83.2	91.8	+11.68%
	Left-turn (Eastbound)	1.5	2.0	+33.33%	103.0	84.5	-17.96%
	Left-turn (Westbound)	3.7	2.8	-24.32%	82.7	64.8	-21.65%
Weekends	Eastbound	5.0	7.8	+56.00%	31.6	42.0	+32.91%
	Westbound	1.0	1.0	0.00%	13.0	14.0	+7.69%
	Northbound	1.0	1.0	0.00%	13.0	54.7	+320.77%
	Southbound	0.0	0.0	-	0.0	0.0	-
	Left-turn (Eastbound)	0.0	1.0	-	0.0	95.0	-
	Left-turn (Westbound)	2.7	2.0	-25.93%	51.0	70.8	+38.82%

Note: the traffic conditions under semi-actuated control was observed on October 15 and16 (Sunday and Monday), 2017, and those under the proactive control was observed on October 3 (Tuesday) and November 19 (Sunday), 2017.

As to the intersection of Gosling Rd., the research team made some observations at October and November. However, some problems were still found. Therefore, the algorithm was further revised (see the end of Section 4.3.2) to balance the waiting times of different phases since the volumes of left-turn traffic and through traffic are both large. The revised algorithm was finally implemented remotely by Steve Chiu in early January 2018. To save the travel cost, the new algorithm's performance was evaluated solely based on the traffic data collected by loop detectors only (see Section 4.6).

For the intersection of Achievement Dr., all results on both weekday and weekend seem to be not so good. Note that the traffic of the intersection at Achievement Dr. is quite heavy, making the accuracy of vehicle counting by naked eyes is low. Also, at this intersection, since the proposed logic was just embedded into the existing coordination mode, the impact of this logic is limited.

4.6. ANALYSIS OF FIELD DATA FROM DETECTORS

The field observations only cover several cycles, and the accuracy of vehicle counting by naked eyes might be low, so the results based on field observation could be bias. For this reason, the performance of the proposed algorithm is also evaluated using the data collected by loop detectors for a couple of weeks before and after the implementation. These data are believed to much more accurate (compared with manually counting by humans) in reflecting the real traffic condition, compared with the data observed in field (just covering several cycles). Therefore, the results are regarded to be more reliable. As mentioned before, loop detectors can only report two measures: volume and occupancy. In this section, the average volume and occupancy (covering 1-2 weeks) were used to evaluate the performance of the proposed proactive control system.

4.6.1 Data Coverage of Time

The volume and occupancy data recorded by each detector at the intersections of Green Bridge Dr., Alden Woods and Gosling Rd., respectively were provided by the Receiving Agency during October 13, 2017–March 23, 2018, because the implementations were conducted several times at these intersections.

- The first implementation on October 2, 2017 was applied to all intersections. However, excepting those five closed spaced intersections (represented by Achievement Dr.), all others needed some revisions to fit the specific traffic conditions;
- The second implementation was applied to the intersections of Alden Wood and Green Bridge Dr. on November 20, 2017; and
- It was in early January, 2018 when the final implementation was completed for the intersection of Gosling Rd. (remote implementation by Steve Chiu).

Table 4.6.1 reports the periods when the data were collected, for the performance evaluation for different intersections at the testbed of SH242.

Table 4.6.1 Time interval of detector data collection based on controller's operation mode

Intersection	**Period when the data were collected**	
	Under original semi-actuated control system	**Under the proposed proactive control system**
Green Bridge Dr.	11/09/2017 – 11/20/2017	01/09/2018 – 01/26/2018
Alden Wood	01/26/2018 – 02/09/2018	01/09/2018 – 01/26/2018
Gosling Rd.	02/23/2018 – 03/02/2018	03/05/2018 – 03/23/2018
Achievement Dr.	10/13/2017 – 10/27/2017	10/02/2017 – 10/12/2017

As described in TM-3.2 Data Analysis Report, the traffic volume (i.e., the number of vehicles passing) and occupancy (in percentage) are reported in every minute by each loop detector on the testbed. In occasional cases (no more than 1%), some data are missing. Then if the gap without data is less than 5 minutes, the missing data can be estimated by interpolation. After making up missing data, the research team calculated the aggregated volumes (i.e., the sum of total 15 volumes) and the average occupancies (average of all 15 occupancies) for every 15 minutes starting from 12:00:00 am to 11:59:50 pm of each day, so there are 96 time-sections (i.e., 15 × 24) in each day. For one detector, if data are not enough for a

15-minute session, this session was left blank. For example, the starting time of data recording is 12:10:00 am at 11/01/2017, then the time section from 12:00:00 am to 12:15:00 am of that day is left blank. Finally, multi-days'15-minute aggregated volumes and averaged occupancies were averaged for a weekday and a weekend, respectively, under either control system (original semi-actuated system or proposed proactive signal control system). For example, if the data covers a week's data, then there are five weekdays and two weekend days. Note that 15-minute volumes were all scaled to hourly volumes by a multiplier of 4. In the following, *please note that both "flow" and "occupancy" are actually the average of multi-days' values.*

Still, four intersections' results were reported: (1) Green Bridge Dr., (2) Alden Wood, (3) Gosling Rd., and (4) Achievement Dr. The last one at Achivement Dr. is used to represent five closely spaced intersections: Fellowship Dr., Honor Roll Dr., Achievement Dr., Maverick Dr. and St. Lukes Way.

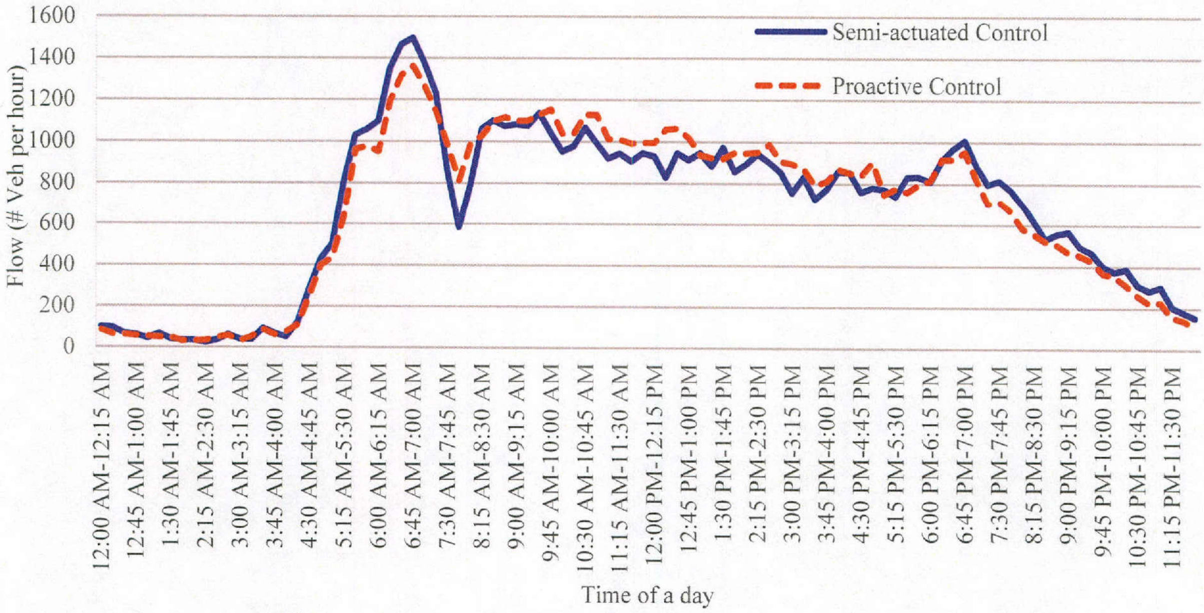
4.6.2 Weekday Traffic Flows on Major Road (SH 242)

With the proactive signal control system, the intersection is supposed to facilitate more vehicles to pass through the intersection. Therefore, it is expected to see the higher vehicle volumes and lower occupancy in a unit time after the proactive signal control is applied, compared to those under the original semi-actuated signal control. For brevity, we focus on the weekday's traffic conditions on the major road (SH242)'s eastbound approach at each intersection, together with those of Phase 5 (left-turn lane) of the intersection at Gosling Rd., since this phase see one of the most congested lanes in the entire testbed.

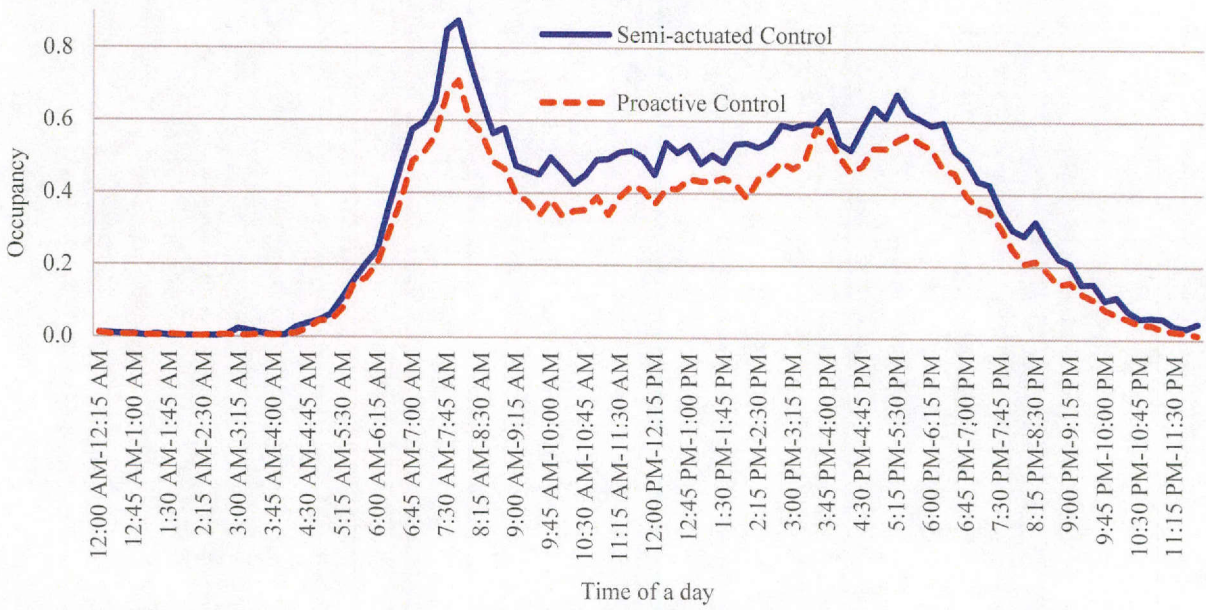
As reported in Figures 4.6.1, 4.6.2, and 4.6.3, on weekdays, after the implementation of the improved proactive system, the traffic volume appears to increase at three intersections (Green Bridge Dr., Alden Wood and Gosling Rd.). On the other hand, along with the increase in volume, the occupancy has also dropped at these three intersections (including the left-turn phase of the intersection at Gosling Rd.). Especially, it seems that the proactive system performs most effectively for the eastbound through traffic of the intersection at Gosling Rd: the traffic flow increased by almost 100%; while the occupancy dropped remarkably, implying that at this phase, the proposed proactive signal control may be able to largely increase the capacity, and also help release the congestion significantly.

As reported in Figure 4.6.4, the proactive system also seems to perform well for the westbound left-turn traffic of the intersection at Gosling Rd., which is one of the most congested approach in the entire testbed (especially, it is most congested among these three intersections) with uncountable queue of stopped vehicles. The proposed logic (see Table 4.3.5) well solved the problem of balancing the waiting time for different phase at this intersection.

Figures 4.6.5 reports the comparison of the traffic flows and occupancies for the eastbound traffic on Achievement Rd. toward the intersection on a weekday. Note that at this intersection, as described in the first section, the proactive signal control system is not fully used. Instead, only the logic was imbedded into the existing coordination mode, because this intersection is very close to other intersections. It is seen that the flow and occupancy before and after the implementation were quite close, indicating that the logic imbedded into the existing coordination mode probably does not have significant impact on the flow—the original coordination plan still dominates the control.

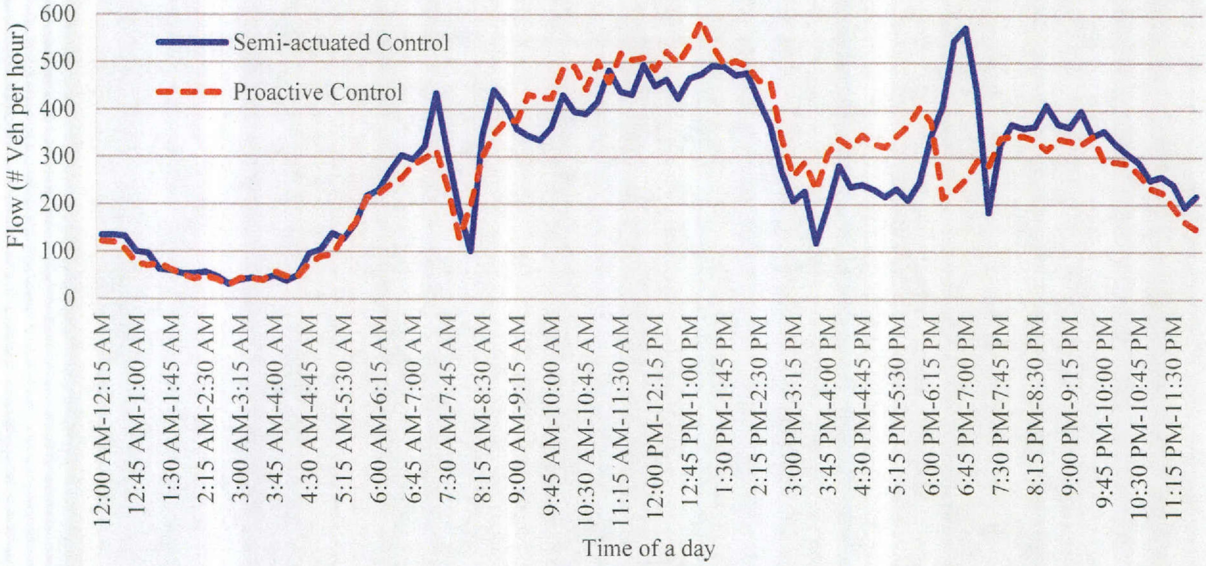


(a) Hourly volume

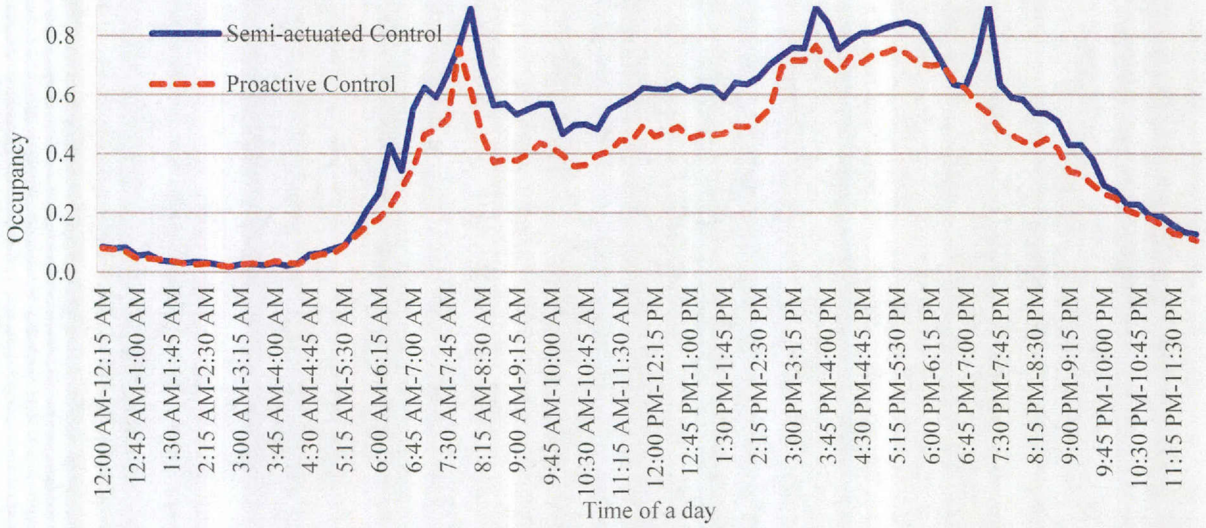


(b) Average Occupancy

***Figure 4.6.1** Multi-day average eastbound traffic of the intersection at Green Bridge Dr. on weekday*

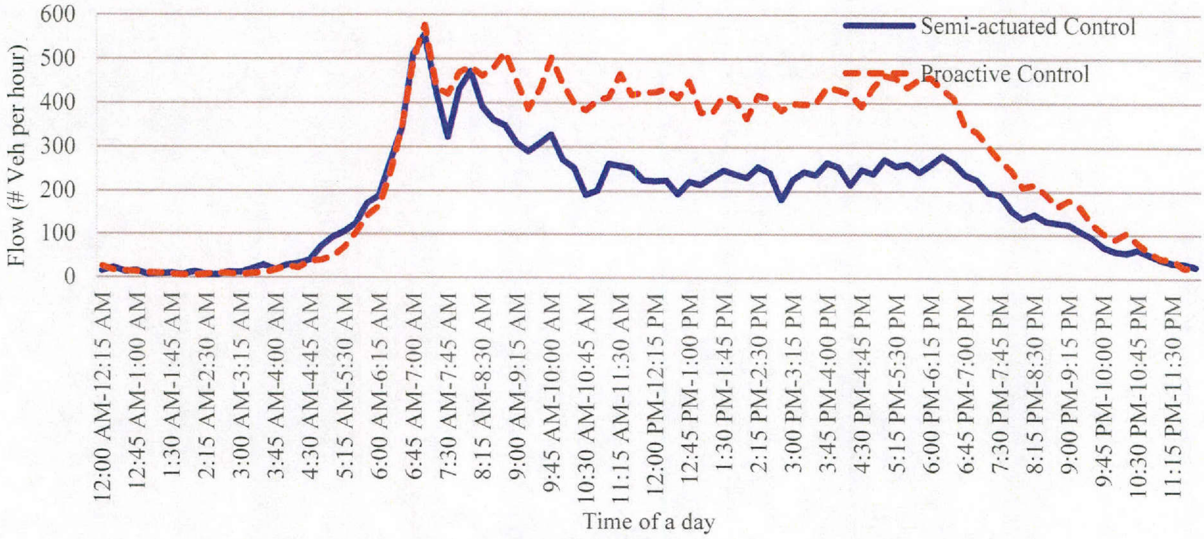


(a) Hourly Volume

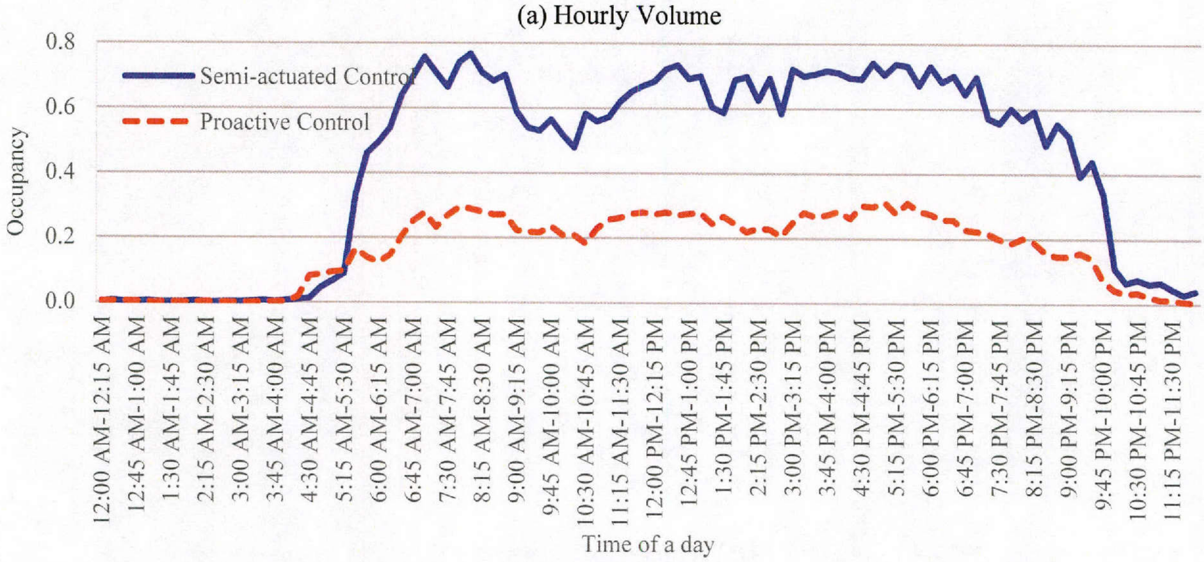


(b) Average Occupancy

Figure 4.6.2 *Multi-day average eastbound traffic of the intersection at Alden Woods on weekday*



(a) Hourly Volume



(b) Average Occupancy

Figure 4.6.3 *Multi-day average eastbound traffic of the intersection at Gosling Rd. on weekday*

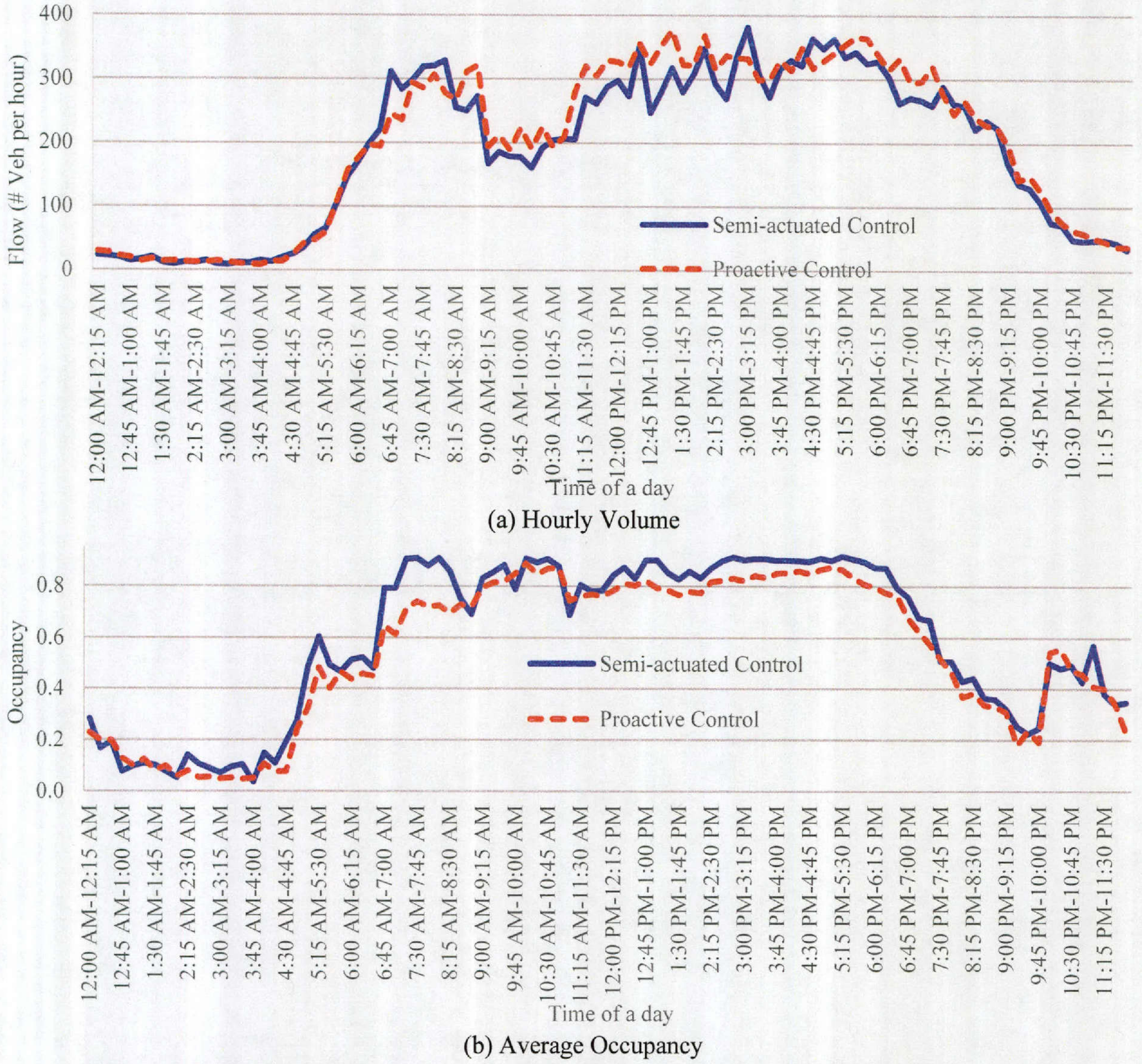
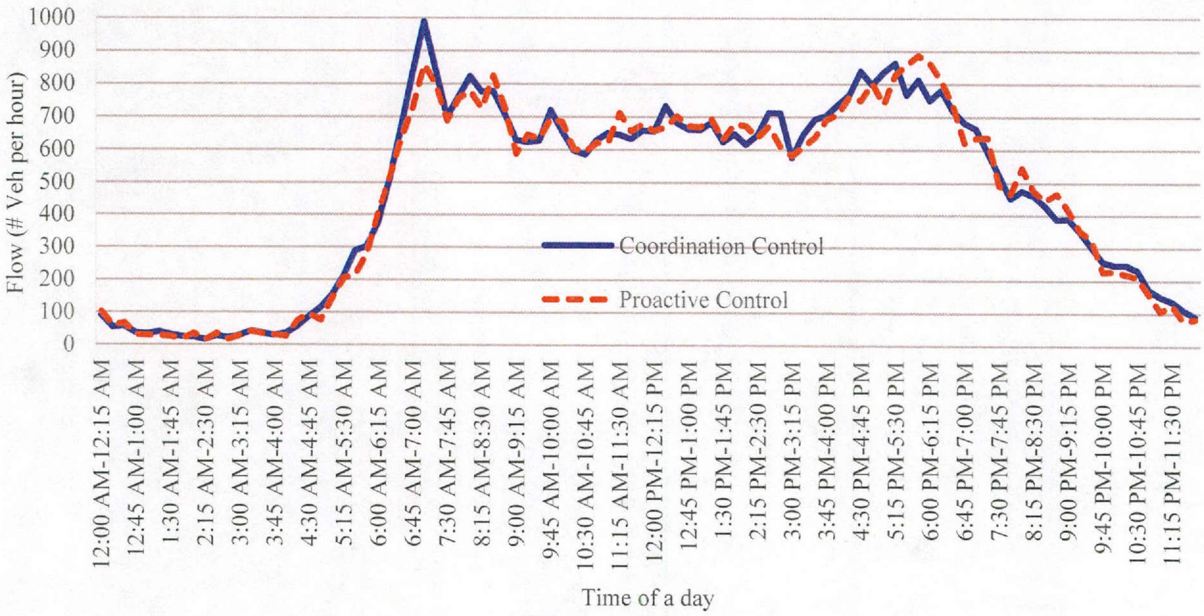
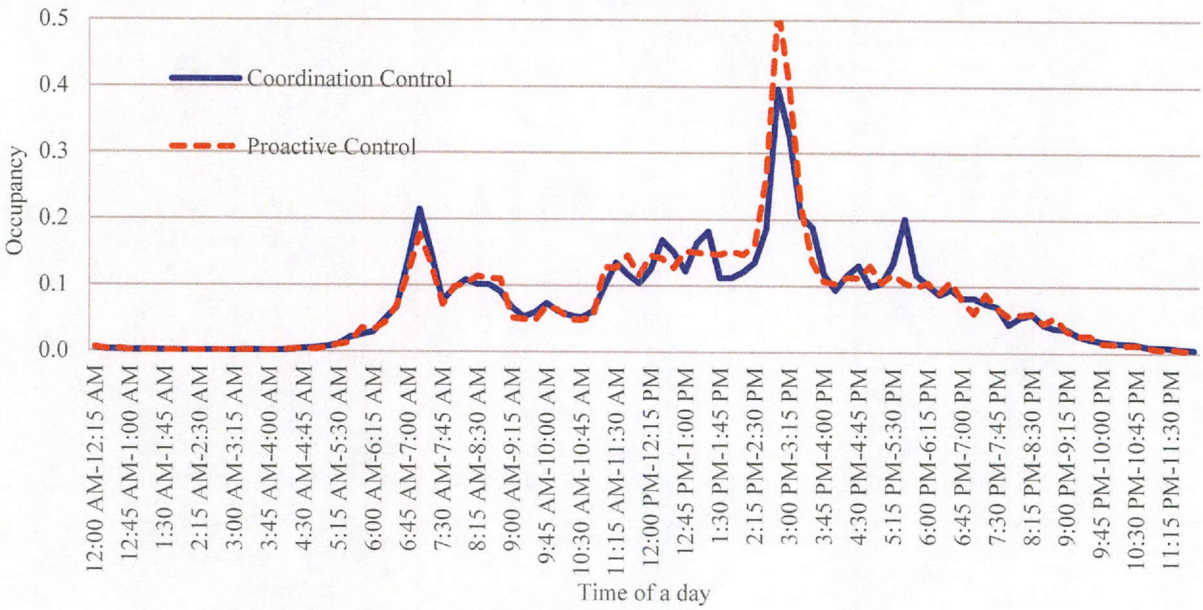


Figure 4.6.4 *Multi-day average westbound left-turn traffic toward Gosling Rd. on weekday*



(a) Hourly Volume



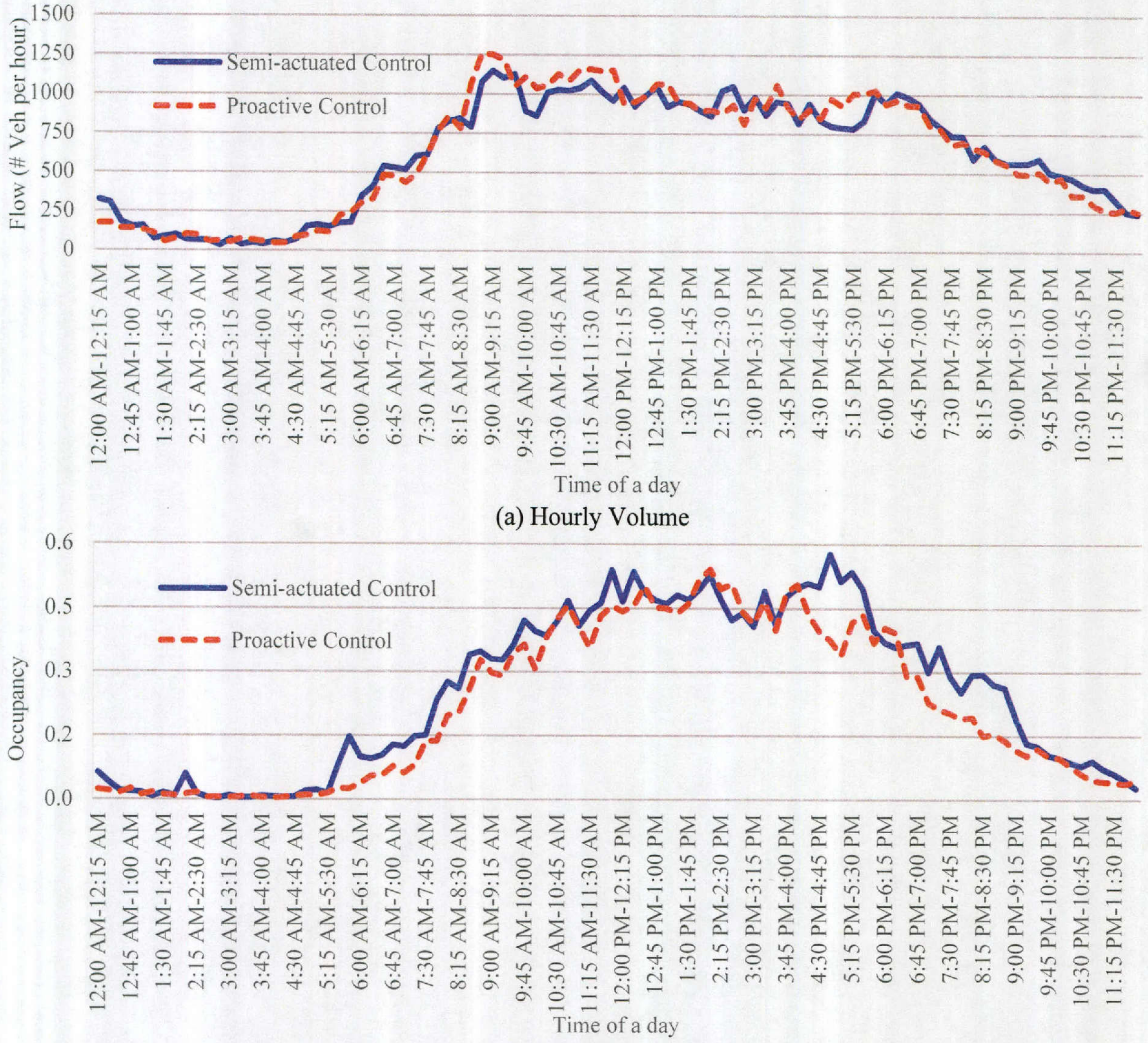
(b) Average Occupancy

Figure 4.6.5 *Multi-day average eastbound traffic of the intersection at Achievement Dr. on weekday*

4.6.3 Weekend Traffic Flows on Major Road (SH 242)

Data comparisons for the traffic on SH 242 at four three intersections (Green Bridge Dr., Alden Wood and Gosling Rd.) on weekend can be found in Figures 4.6.6 through 4.6.8. Note that the occupancy data under the proactive signal control system, reported on Figure 4.6.8, apparently are in error. The data analysis for westbound left-turn traffic of the intersection at Gosling Rd. is reported in Figure 4.6.9.

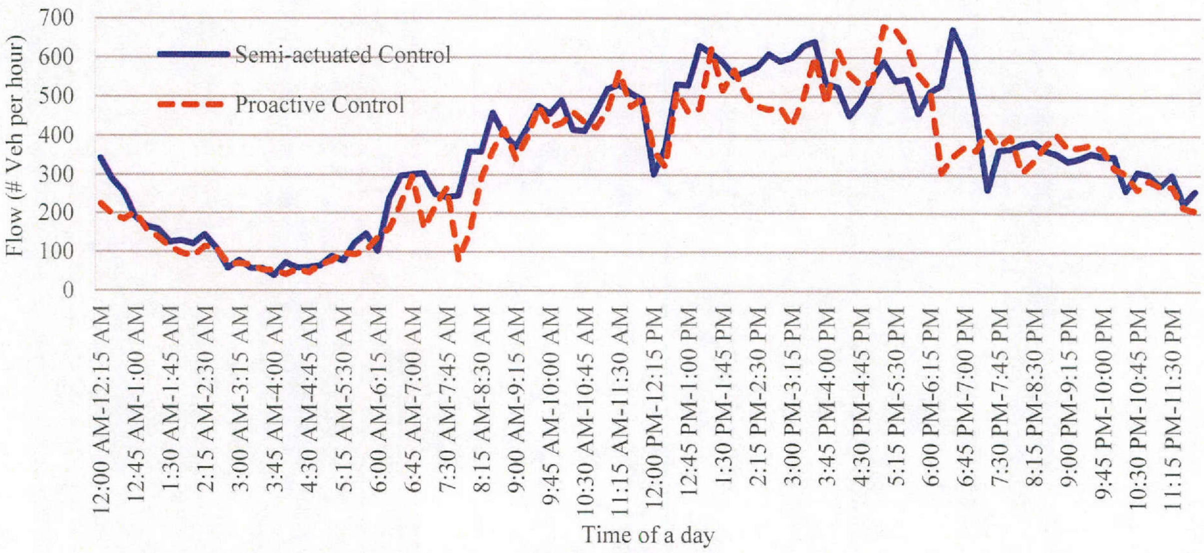
We can see that in weekend, the results are good at the intersections of Green Bridge Dr. and Gosling Rd.: the traffic flow increased while the occupancy level dropped. The flow at Alden Wood, however, remained almost at the same level, but the occupancy level also dropped. Therefore, it seems the proposed proactive signal control is effective at these three intersections.



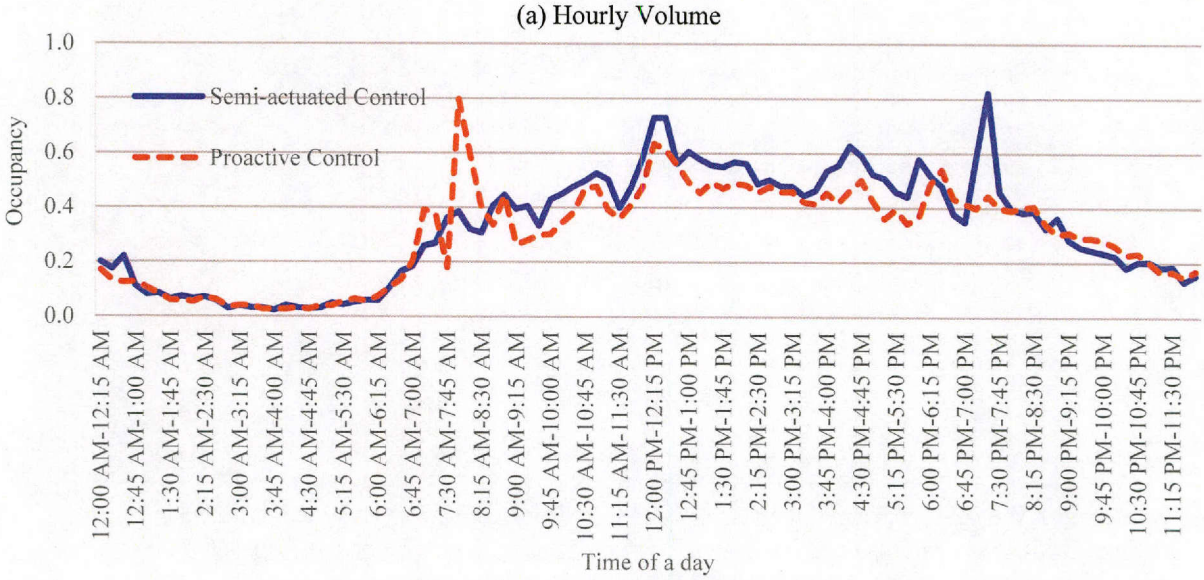
(a) Hourly Volume

(b) Average Occupancy

Figure 4.6.6 *Multi-day average eastbound traffic of the intersection at Green Bridge Dr. on weekend*

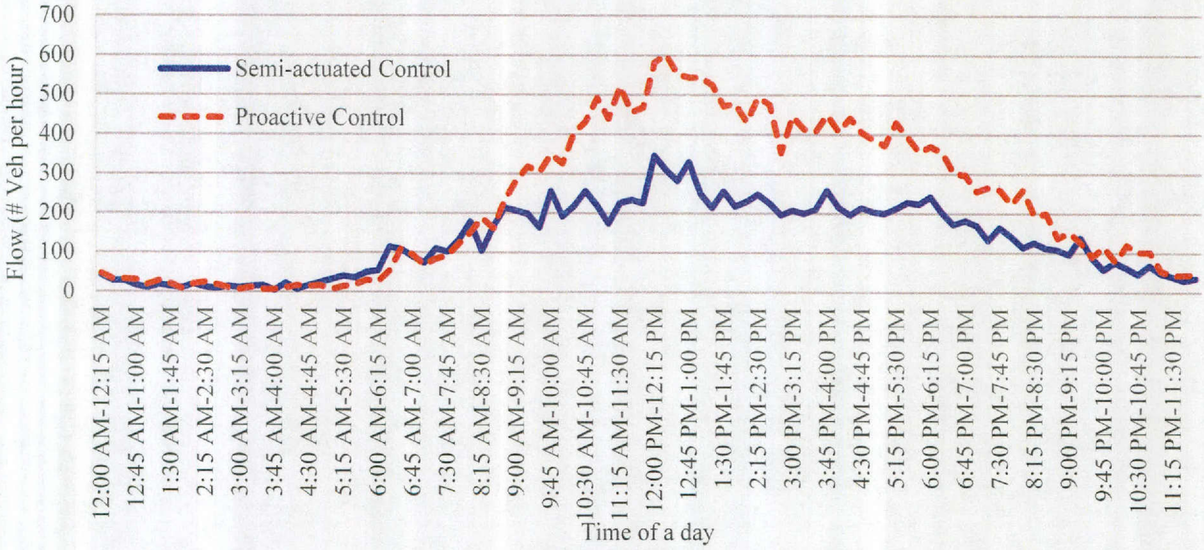


(a) Hourly Volume

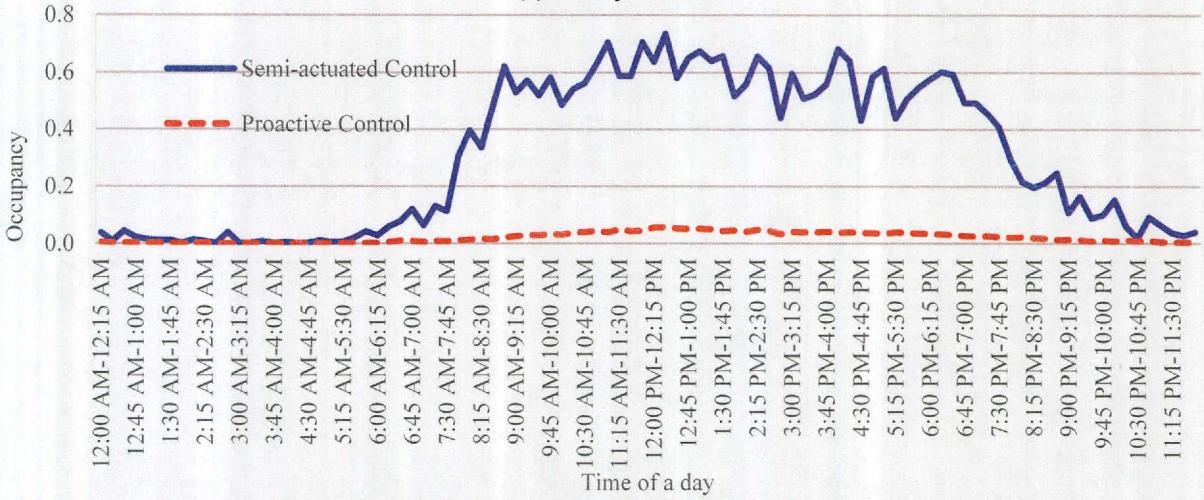


(b)Average Occupancy

***Figure 4.6.7** Multi-day average eastbound traffic of the intersection at Alden Woods on weekend*

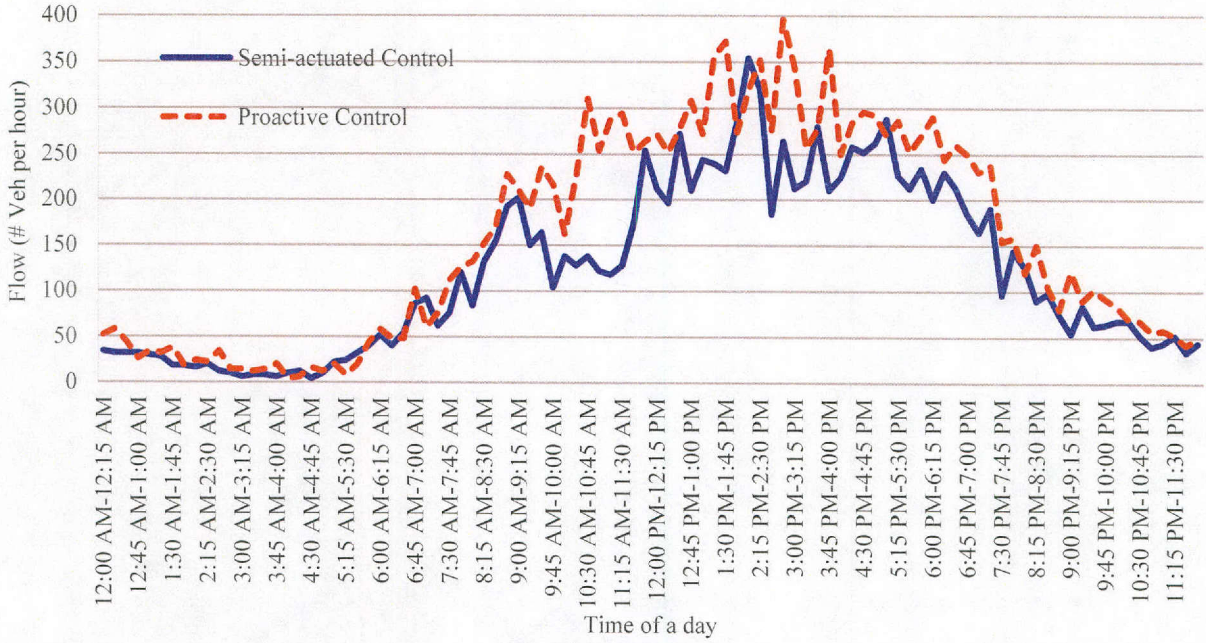


(a) Hourly Volume

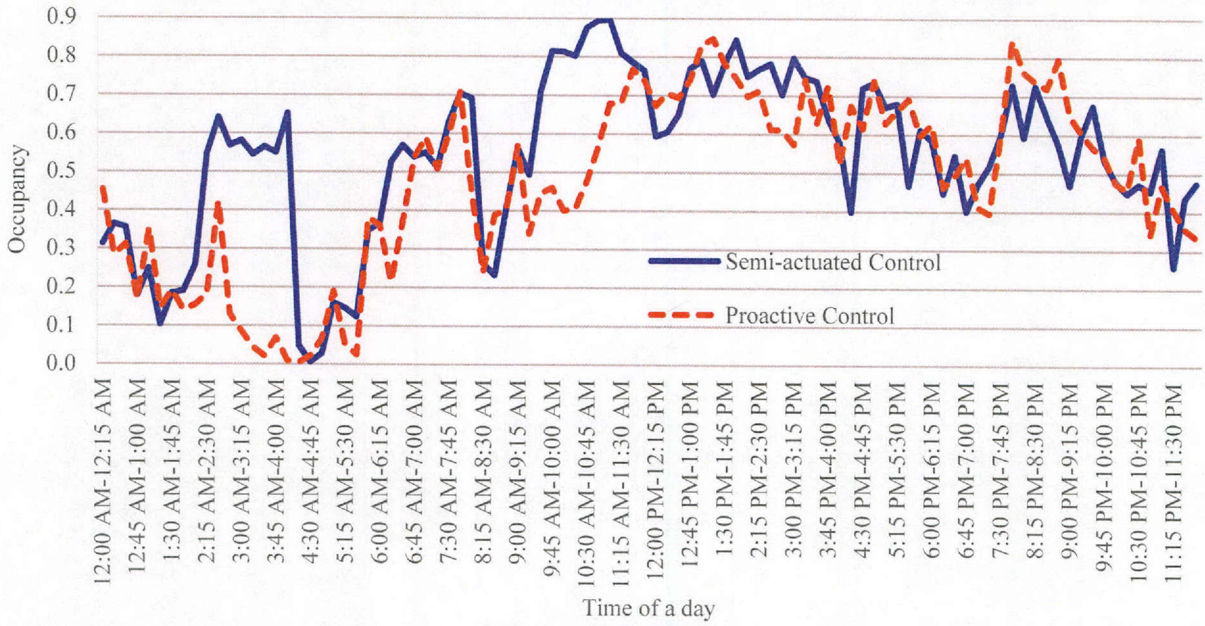


(b) Average Occupancy (the data under the proactive signal control system are error)

Figure 4.6.8 *Multi-day average eastbound traffic of the intersection at Gosling Rd. on weekend*



(a) Hourly Volume

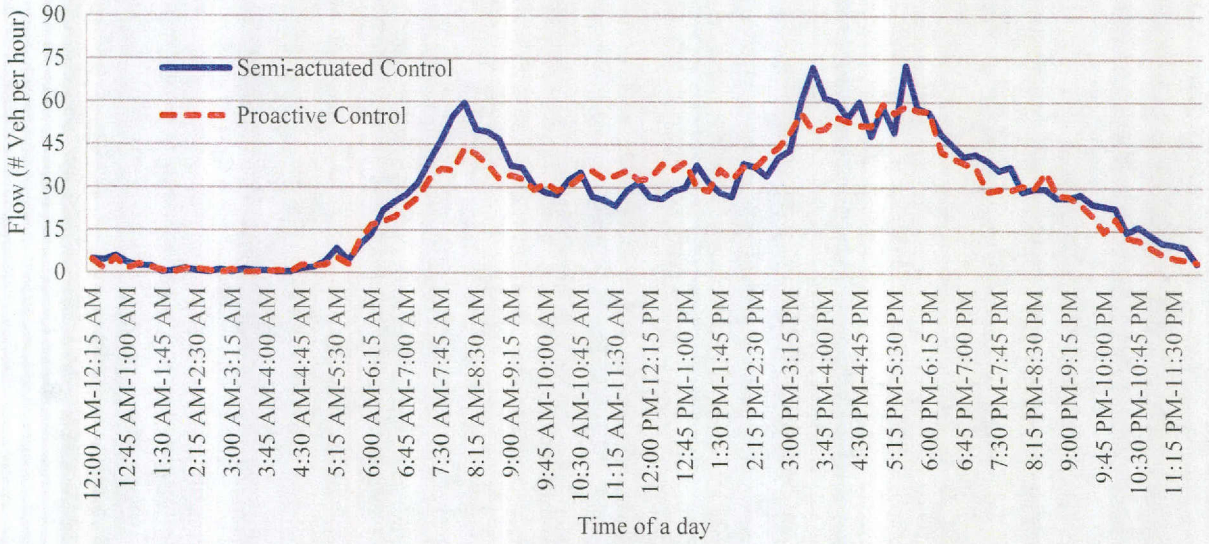


(b) Average Occupancy

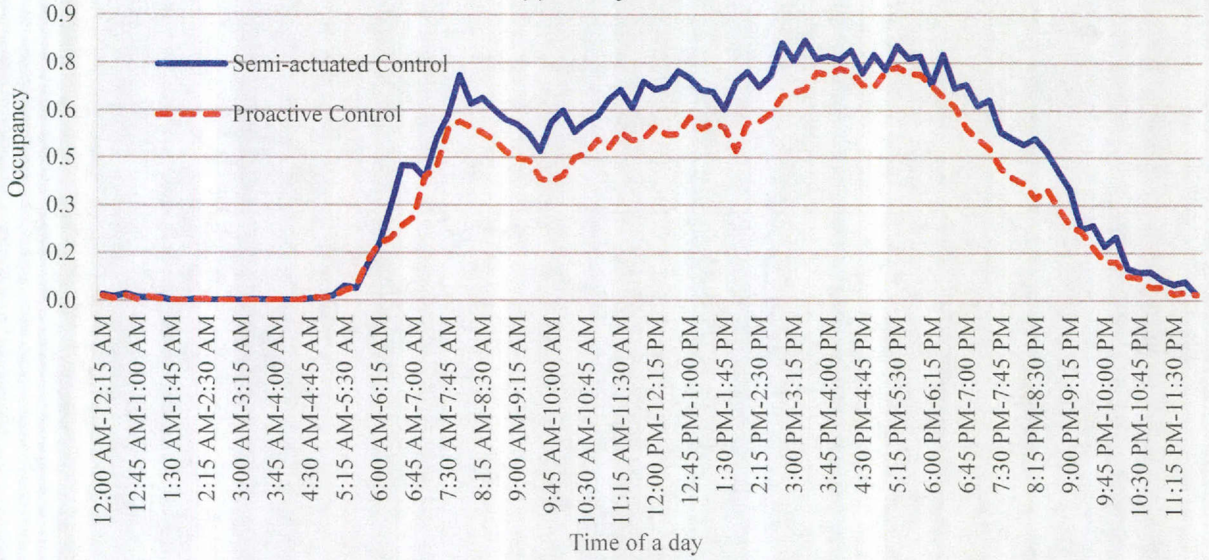
Figure 4.6.9 *Multi-day average westbound left-turn traffic toward Gosling Rd. on weekend*

4.6.4 Traffic Flows on Minor Roads

The traffic data on minor roads were also analyzed. The results are reported in Figures 4.6.10 through 4.6.16. It is also seen that the results are good on the minor roads at the intersections of Green Bridge Dr., Alden Wood, and Gosling Rd. Note that except the left-turn phases at Gosling Rd., all other minor roads at these three intersections see a much smaller traffic flow than those at the major road. Therefore, their impacts are not significant. But from these figures, we can still see the traffic flow increased and the occupancy decreased in most of the cases.

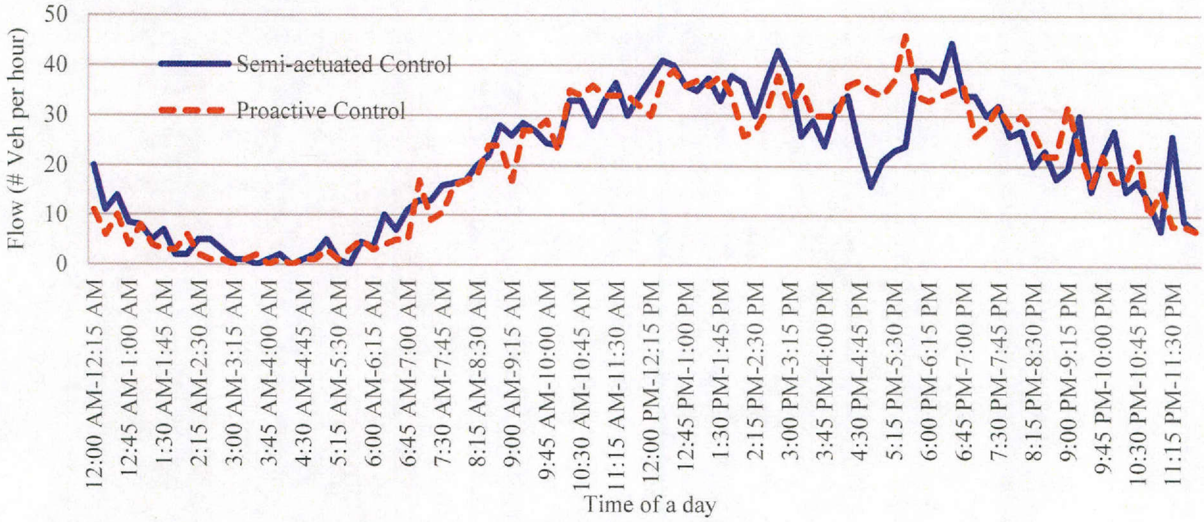


(a) Hourly Volume

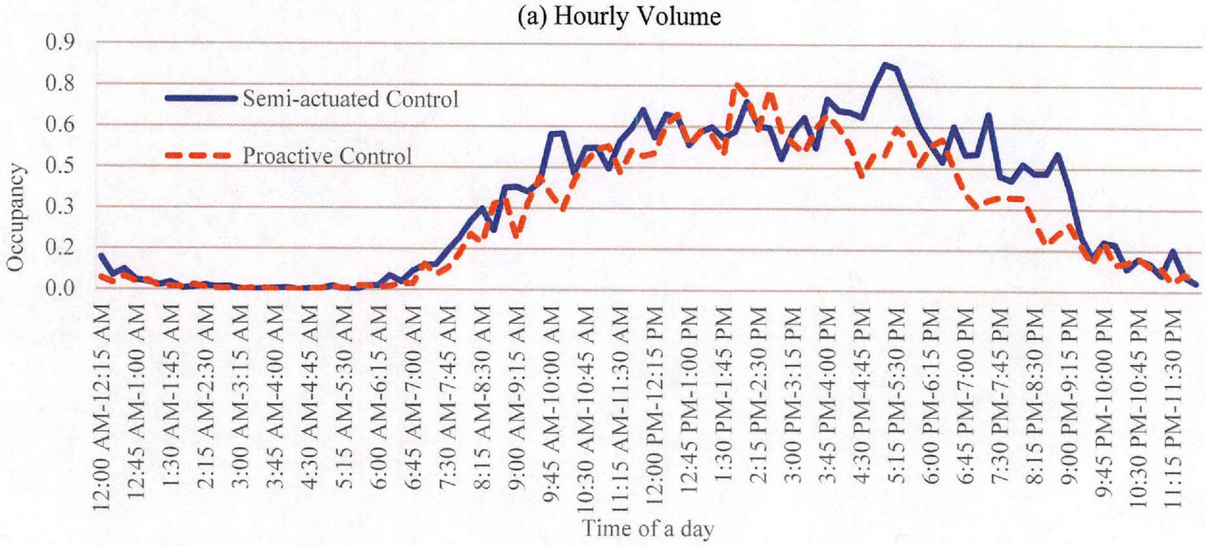


(b) Average Occupancy

Figure 4.6.10 *Multi-day average northbound traffic from Green Bridge Dr. on weekday*

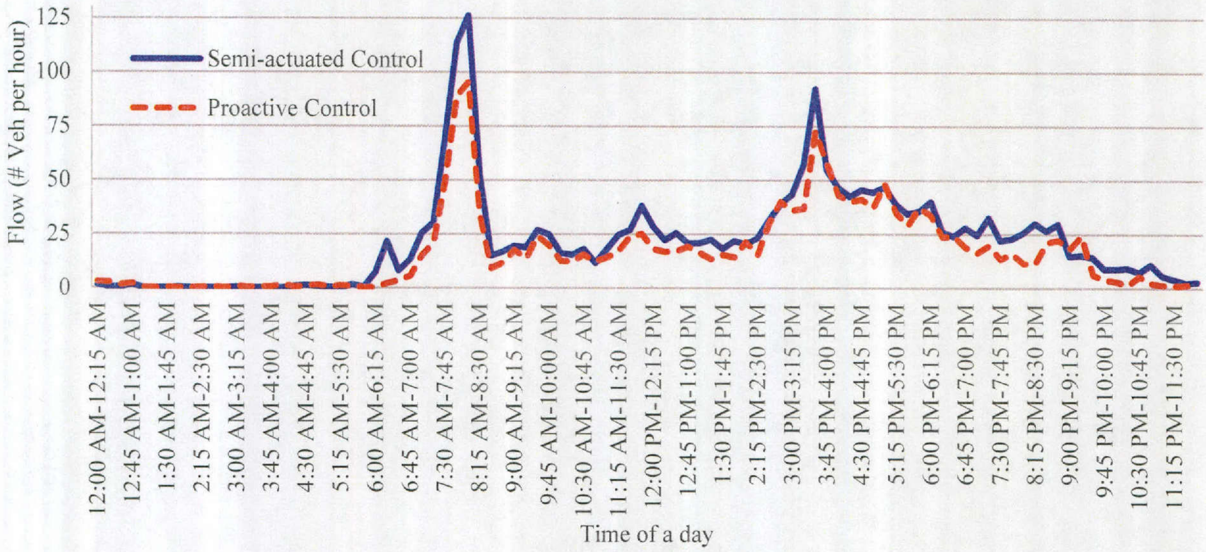


(a) Hourly Volume

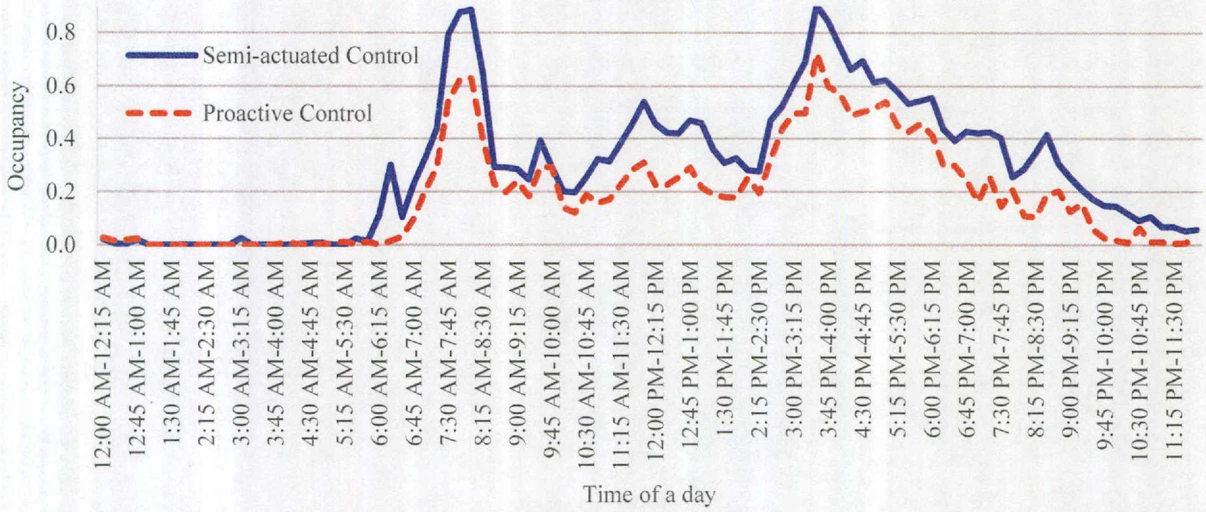


(b) Average Occupancy

Figure 4.6.11 *Multi-day average northbound traffic from Green Bridge Dr. on weekend*

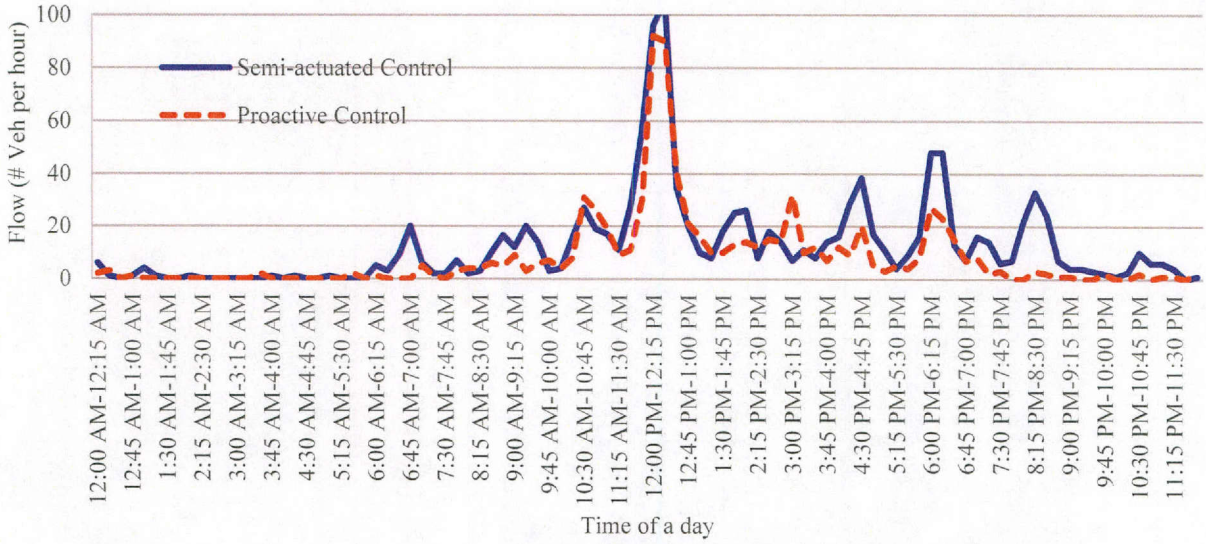


(a) Hourly Volume

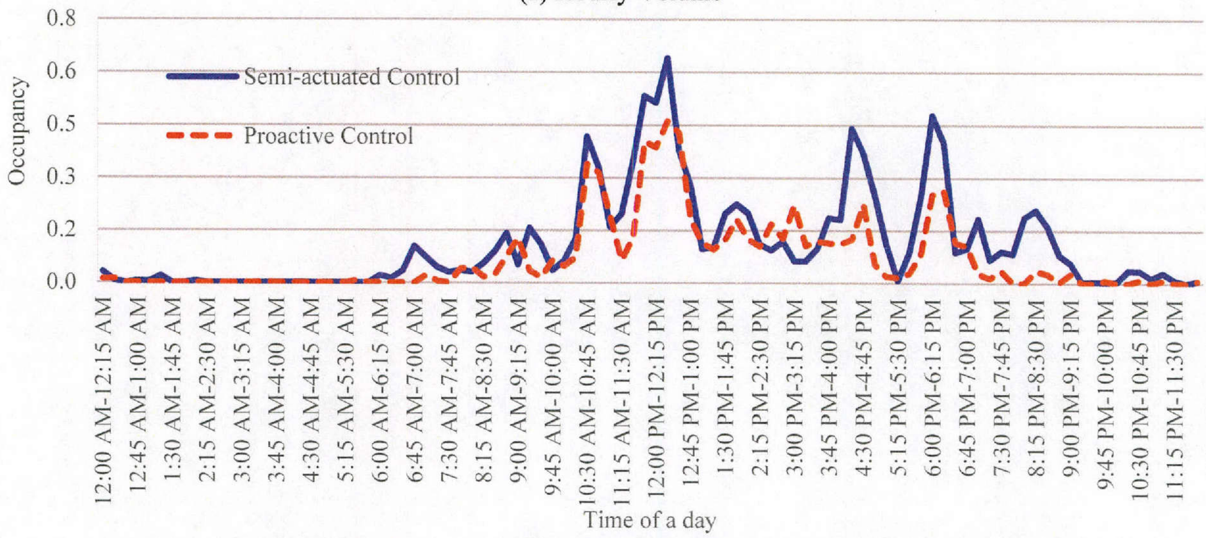


(b) Average Occupancy

***Figure 4.6.12** Multi-day average northbound traffic from Alden Woods on weekday*

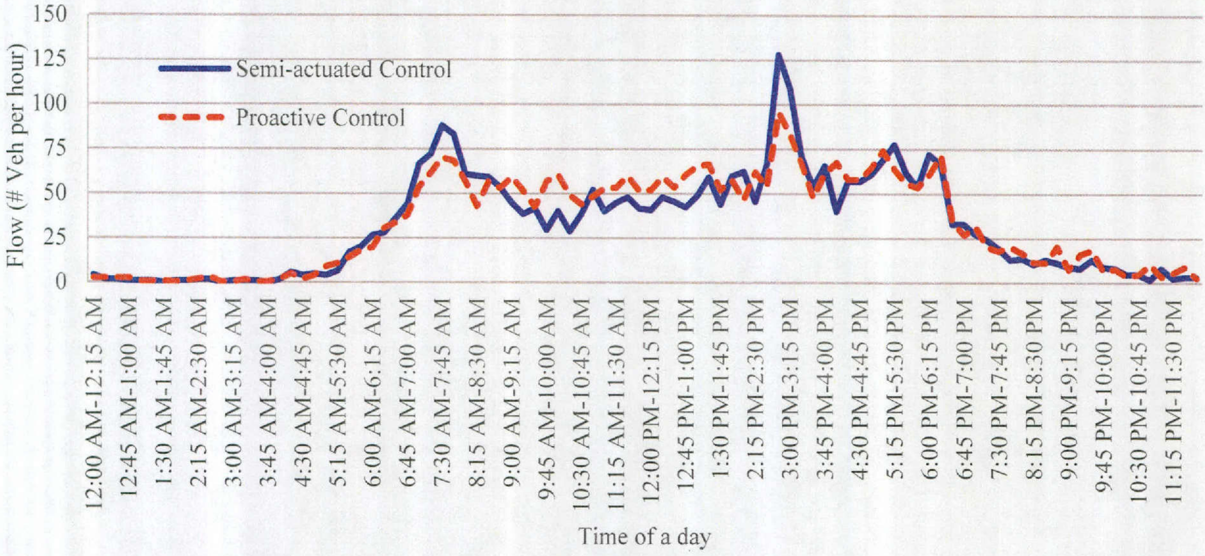


(a) Hourly Volume

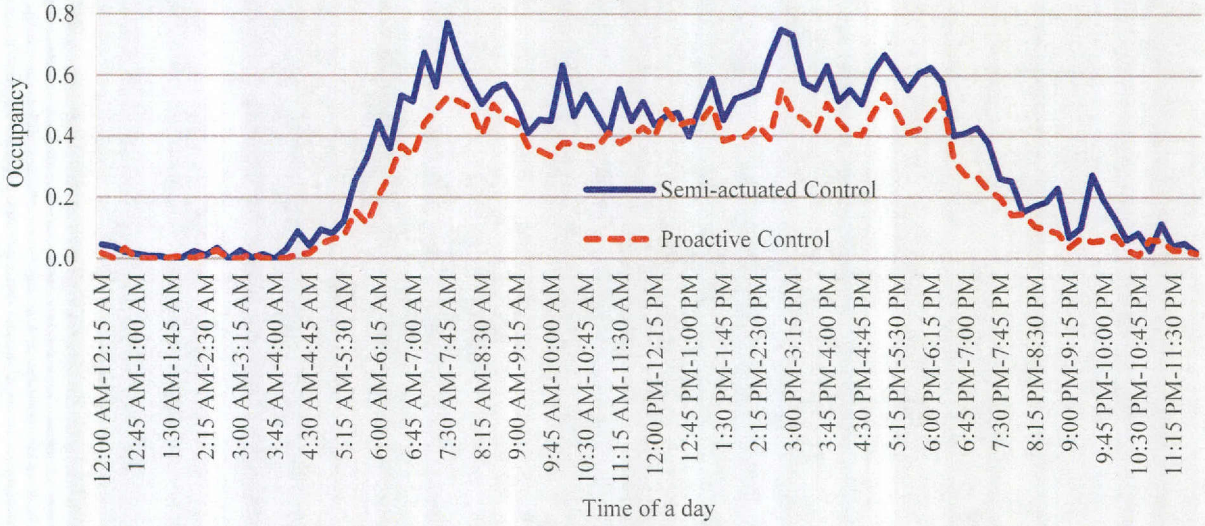


(b) Average Occupancy

Figure 4.6.13 *Multi-day average northbound traffic from Alden Woods on weekend*

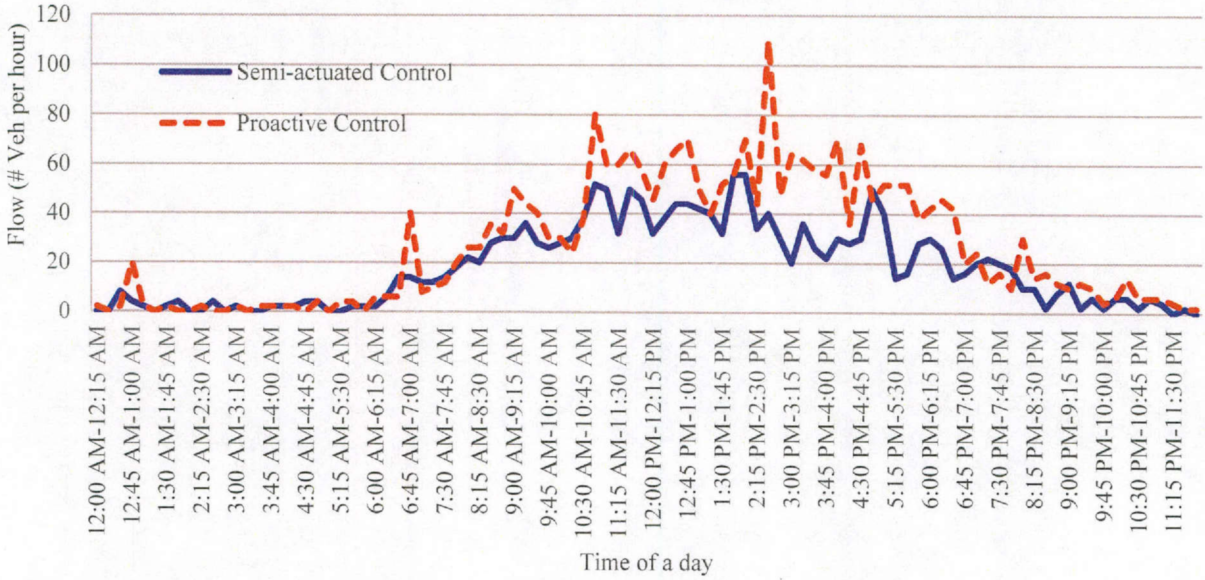


(a) Hourly Volume

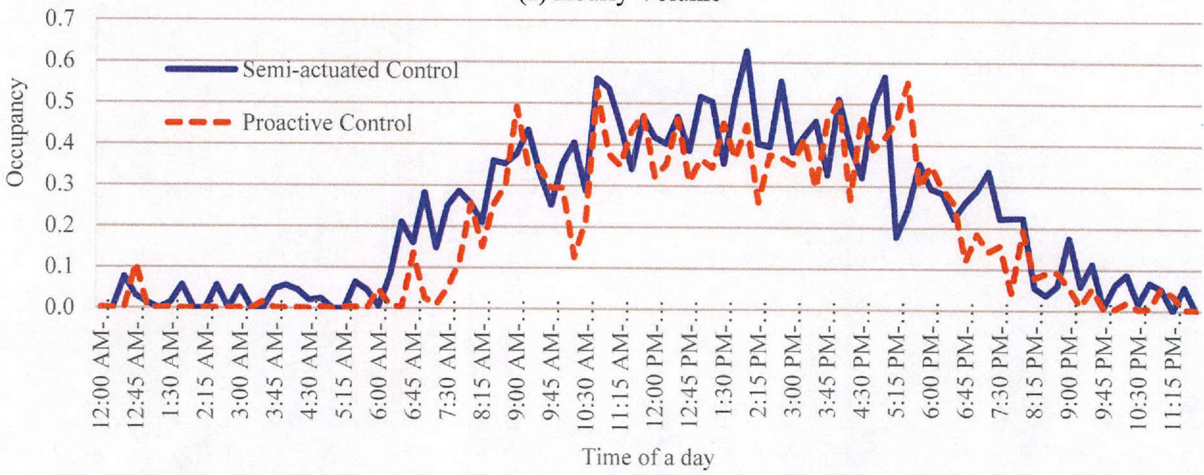


(b) Average Occupancy

Figure 4.6.14 *Multi-day average southbound traffic from Gosling Rd. on weekday*

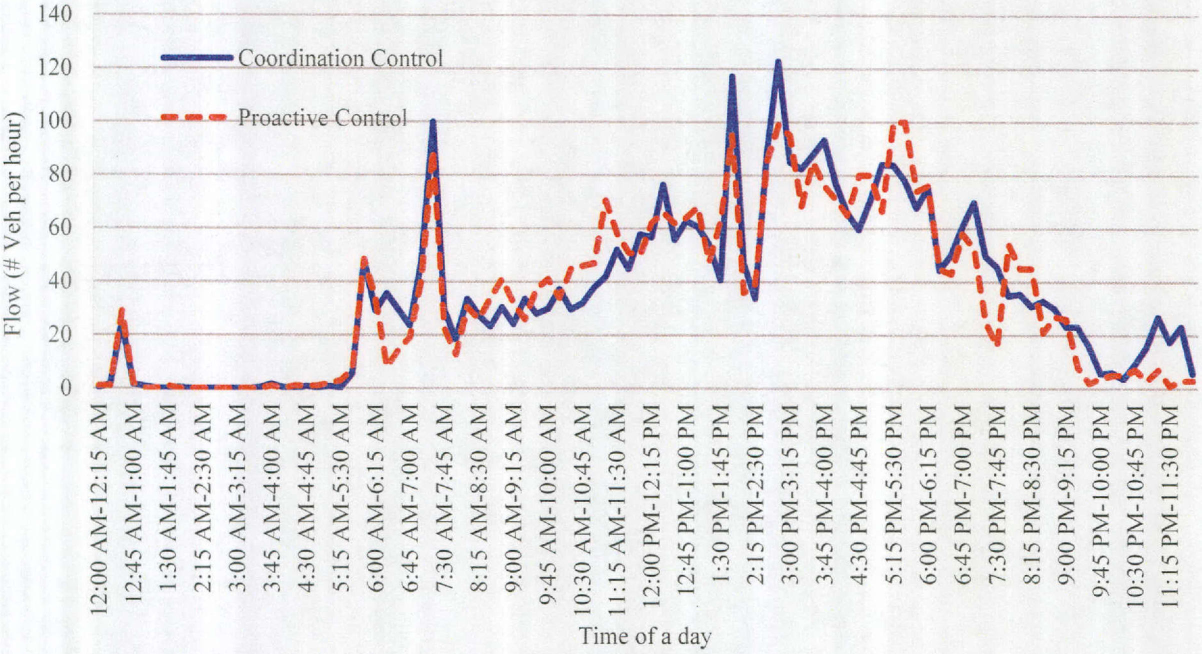


(a) Hourly Volume

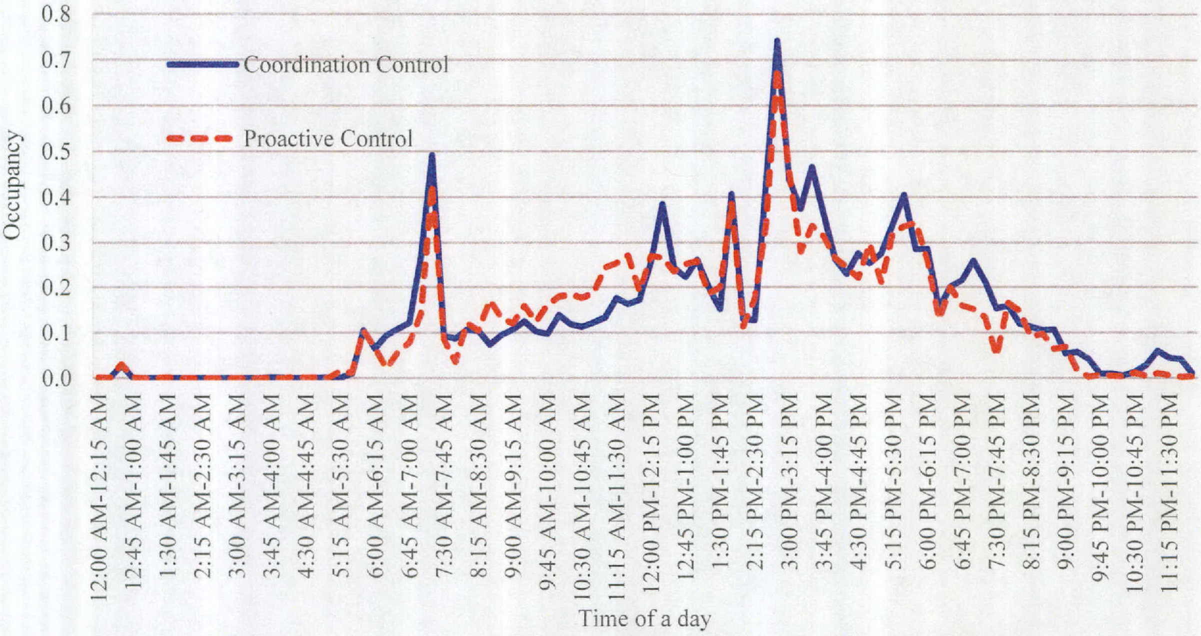


(b) Average Occupancy

Figure 4.6.15 *Multi-day average southbound traffic from Gosling Rd. on weekend*



(a) Hourly Volume



(b) Average Occupancy

Figure 4.6.16 *Multi-day average southbound traffic from Achievement Dr. on weekday*

Of the intersection at Achievement Dr., from Figure 4.6.16, we can see that at some time in the middle of day, the traffic volume increased, however, the occupancies during this time also increased. As mentioned above, the coordination mode is on all the time with just the proactive logic imbedded in. Therefore, as discussed before, the coordination mode dominates the flow control, the effect of the proactive logic is not so apparent.

4.7. SUMMARY

The research team from the Performing Agency developed the proactive signal control system for the testbed of SH 242 based on the data collected by loop detectors. With the great helps offered by the Receiving Agency, the research team conducted field tests on the testbed in October and November 2017. Revisions were made to fit the feature of the specific intersections, because some problems were found in the field tests.

Compared with the first testbed of FM 528 (the implementation was conducted in July 2017), this testbed is much more complicated—both traffic flow and occupancy rates are large, and the coordination is the priority for a series of intersections that are spaced very close. The research team creatively revised the original proactive signal control logic to fit various requirements at this testbed. Especially, a new logic was developed to balance the traffic control system between the original coordinated mode (for the through traffic on the major road) and the new proactive signal control mode (for the left-turn and minor traffic).

Table 4.7.1 summarizes the comparison of the multi-day average flows and occupancies collected under the original and proposed control systems, respectively, at three intersections during different time of day. The results well show the impacts of the proposed proactive signal control on the traffic at this testbed: the flow rates were increased, and the occupancy rates were reduced. It implies more vehicles were able to pass through the intersection in a unit of time; while the congestion levels were relieved at some degree.

Based on the implementation work at this testbed, the following conclusions were made.

1. With the improved algorithm, the data from the detectors show that proactive signal control system worked well for the through traffic on the major road. It helped more vehicles to pass the intersection in a unit time while reducing the occupancies at the intersections.

2. If the demands of left turn and through traffic are both heavy, by considering the balance of waiting time for different phases, the proactive system has also improved the westbound left-turn traffic at Gosling Rd. which suffers one of the heaviest congestions in the entire testbed.

3. Five closely-spaced intersections (Fellowship Dr.–St. Luke's Way) should run on coordination mode all the time. In this case, the original proactive logic was embedded with the existing coordination system. However, since the coordination mode dominates the control logic, the effect of the proactive logic is not so apparent.

4. As reported from Steve Chiu, the Houston District Office of the Receiving Agency has not received any compliant since the improved proactive system was implemented on the testbed of SH 242. It indicates that the improved proactive signal control system has performed well as expected.

Table 4.7.1 Performance comparison based on multi-day average data for three intersections at SH 242

Phase	Day	Time Period		Flow	Occupancy
EB traffic at Green Bridge Dr.	Weekday	6:00:00	9:30:00	14.3%	-64.0%
		11:00:00	15:00:00	79.7%	-61.1%
		15:00:00	19:00:00	69.2%	-60.9%
		6:00:00	22:00:00	55.5%	-62.9%
	Weekend	10:00:00	16:00:00	3.5%	-3.0%
		10:00:00	22:00:00	2.1%	-12.5%
		All Day		1.6%	-15.1%
WB traffic at Green Bridge Dr.	Weekday	6:00:00	9:30:00	-10.9%	-18.0%
		11:00:00	15:00:00	-3.5%	-19.7%
		15:00:00	19:00:00	-1.3%	-10.2%
		6:00:00	22:00:00	-6.3%	-18.7%
	Weekend	10:00:00	16:00:00	-1.3%	-3.6%
		10:00:00	22:00:00	2.0%	-11.1%
		All Day		-4.0%	-12.9%
WB traffic at Alden Wood	Weekday	6:00:00	9:30:00	-1.0%	-18.9%
		11:00:00	15:00:00	4.4%	-5.1%
		15:00:00	19:00:00	-3.7%	-6.0%
		6:00:00	22:00:00	-0.2%	-10.1%
	Weekend	10:00:00	16:00:00	6%	0%
		10:00:00	22:00:00	10%	-3%
		All Day		4%	6%
Left-turn to Alden Wood	Weekday	6:00:00	9:30:00	-12.5%	-30.0%
		11:00:00	15:00:00	12.2%	-13.5%
		15:00:00	19:00:00	1.9%	-18.8%
		6:00:00	22:00:00	15.1%	-22.8%
	Weekend	10:00:00	16:00:00	17%	-15%
		10:00:00	22:00:00	25%	-15%
		All Day		24%	51%
Left-turn to Gosling Rd.	Weekday	6:00:00	9:30:00	-0.2%	-12.8%
		11:00:00	15:00:00	15.4%	-5.9%
		15:00:00	19:00:00	2.8%	-6.9%
		6:00:00	22:00:00	6.8%	-8.1%
	Weekend	10:00:00	16:00:00	46.8%	-10.9%
		10:00:00	22:00:00	37.3%	-1.2%
		All Day		36.4%	-6.8%
EB traffic at Gosling Rd.	Weekday	6:00:00	9:30:00	14.3%	-64.0%
		11:00:00	15:00:00	79.7%	-61.1%
		15:00:00	19:00:00	69.2%	-60.9%
		6:00:00	22:00:00	55.5%	-62.9%

Note: EB and WB traffic refer to the east- and westbound traffic along SH 242 at each intersection, respectively. There are some errors in weekend' data of eastbound traffic at Gosling Rd., so this phase is not reported here.

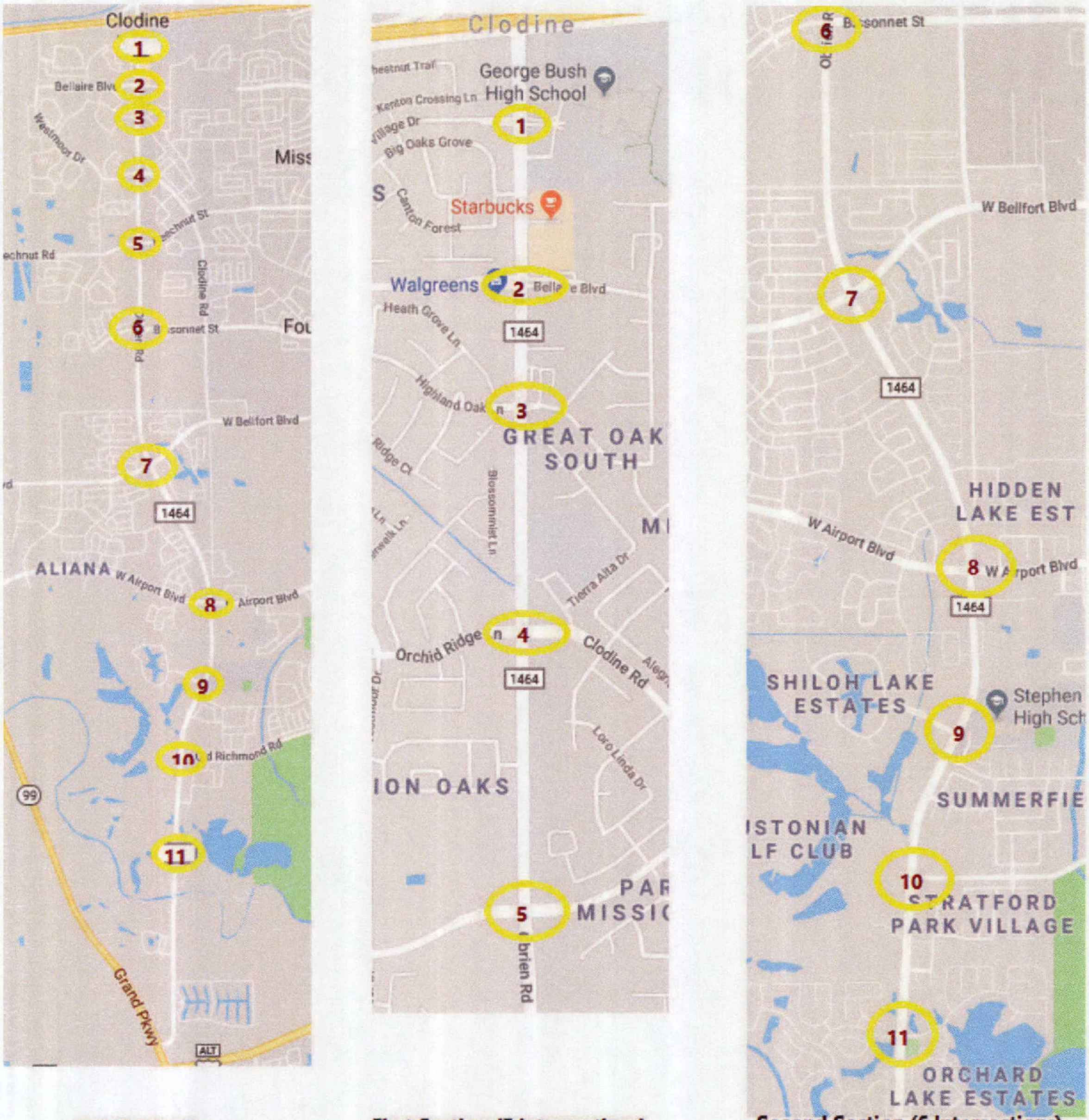
CHAPTER 5

IMPLEMENTATION AT TESTBED OF FM 1464

5.1. INTRODUCTION

The testbed of FM 1464 is located at the southwest suburb of Houston, near Sugarland. The section of FM 1464 used as the testbed is between Westpark Tollway and TX-99 (west-loop).

This testbed is the longest testbed on this project, including 11 intersections: (1) W. Oaks Village Dr. (2) Bellaire Blvd., (3) Highland Oak Ln, (4) Orchid Ridge Ln., (5) Beechnut St., (6) Bissonnet St., (7) W. Bellfort Blvd., (8) W. Airport Blvd., (9) Stephen F Austin High School, (10) Old Richmond Rd., and (11) Orchard Lake Estates Dr. These 11 intersections can be divided into two parts. The first part includes the first five intersections from (1) W. Oaks Village Dr. to (5) Beechnut St., which run in coordination mode (as middle one shown in Figure 5.1.1); and the remaining six intersections from (6) Bissonnet St., to (11) Old Orchard Dr. are independent, and running in actuated mode (regarded as the intersections in part 2 in the following text), as the right one shown in Figure 5.1.1.



Full Testbed **First Section (5 Intersections)** **Second Section (6 Intersections)**

Figure 5.1.1 *Intersections at the testbed of SH FM 1464*

Note that in this testbed, the intersection at Bissonnet St. is a new one. When preparing the task plan before this project started, this one was still in building. Therefore, it was not shown in the original work plan. However, it was included when in the presentation of the task in the kick-off meeting of this project.

Originally, this testbed did not have the loop detectors embedded in each intersection. The data were collected from cameras. In Task 3 (Network Performing Analysis) conducted in Summer 2017, the research team noticed that the data from cameras could not precisely capture the actual number of vehicles passing through the intersection—the flow could be largely underestimated. As the results, the simulations based on these data were not able to represent the real traffic states—the gaps were large. Such large laps make it very difficult to develop the proper signal control algorithm for this testbed, because the test of the proposal algorithm has to be done in the simulation platform first before conducting the field implementation. If the accuracy of the simulations is low, then the bias of the identified optimal parameters for the proactive signal control could be large. Realizing this problem, the Performing Agency urged the Receiving Agency to install the loop detectors on this testbed. Therefore, the implementation on this testbed was heavily delayed.

The newly constructed intersection at Bissonnet St., however, has all loop detectors ready. Therefore, on February 27, 2018, working with Steve Chiu from the Receiving Agency, the research team from the Performing Agency did the first field implementation at this intersection, and they also checked the status of other intersections at this testbed.

Until in March 2018, when all other 10 intersections at this testbed have loop detected imbedded into the pavement, the data were re-collected, and the simulation verifications were completely re-conducted. The second implementation was conducted on April 19, 2018 for the intersections at the first part of intersections, i.e., from (1) W. Oaks Village Dr. to (5) Beechnut St. However, since these intersections are in coordinate mode, some problems were found during the implementation. The research team revised the algorithms based on the suggestions from Steve Chiu, and he revised the algorithm remotely in the first week of May. On May 10, the third implementation was conducted, working on the second part of intersections, from (7) W. Bellfort St. to (11) Old Orchard Dr. Note that the implementation at the first intersection in the second part, (Bissonnet St.) was completed on February 27, 2018.

The reminder of this chapter is organized as follows. Section 5.2 describes the signal timing setting for the proactive signal control system; Section 5.3 describes the latest logic statement settings for the proactive system; Section 5.4 provides simulation testing results; Section 5.5 summarizes and analyzes the data recorded by the detectors on the testbed; and Section 5.6 concludes this chapter.

5.2. SIGNAL TIMING SETTING

5.2.1 Setting of MM (2-1)

As the first step of developing the proactive signal control system for this testbed, the signal timing plans originally used for the 11 intersections at this testbed, provided by the Receiving Agency, were modified for developing the proactive signal control for this testbed. All of these parameters can be edited through the function of MM (2-1) in the ASC/3 Controller.

The research team made similar modifications to the signal parameters of the signal control system at each intersection of this testbed. Mainly, the changes were made to three of the signal parameters: (1) vehicle extension period, (2) minimum green time and (3) maximum green time. For the through traffic on the major road (Phases 2 and 6, respectively), 4.0 seconds were set as the vehicle extension period; while for other approaches (minor roads and left turn movement on major road), a smaller value of 3.0 seconds was set to minimize the overall delay. As to the minimum green time, 4 seconds was set for Phases 2 and 6 (for the through movement on the major road) and 3 seconds for other phases to expedite the skipping or switching a phase. Finally, for the maximum green time, the team only revised for the through movement on major road (Phases 2 and 6, respectively), to make it relatively larger value than the original settings in order to flush the heavy traffic during the peak hours, and specifically, it was increased by 10 seconds than the original setting for Phases 2 and 6, respectively, at all of the intersections.

As mentioned at the beginning, the first part of the testbed: (1) W. Oak Village Dr., (2) Bellaire Blvd., (3) Highland Oak Ln., (4) Clodine Reddick and (5) Beechnut St., originally run on the coordination mode, because they are closely spaced together, as seen in Figure 5.1.1. The second part: (6) Bissonnet St., (7) W. Bellfort St., (8) W. Airport Blvd., (9) Stephen F Austin High School, (10) Old Richmond Rd., and (11) Orchard Lake Estates Dr., originally run independently in the semi-actuated mode. Therefore, the settings for the intersections at these two parts are different. Because of such difference,

In the following, for brevity, this section just reports the revised signal timing parameters of only four intersections, two running on the coordination mode and another running on the semi-actuated mode. Tables 5.2.1 and 5.2.2 report the revised signal timing for the intersections at W. Oak Village Dr. and Beechnut St. (in the coordination mode) respectively; and Tables 5.2.3 and 5.2.4 report the revised signal timing for Old Richmond Rd. and Orchard Lake Estates Dr. (in the semi-actuated mode), respectively.

Table 5.2.1 Revised signal timing parameters for the intersection at W. Oak Village Dr.

Phase	1 (old)	**1**	2 (old)	**2**	3 (old)	**3**	4 (old	**4**	5 (old)	**5**	6 (old)	**6**
Minimum green time	5	**3**	15	**4**	7	**3**	7	**3**	5	**3**	15	**4**
Maximum green time	15	15	35	**45**	25	**25**	25	**2**	15	**15**	35	**45**
Vehicle extension time	2.0	**3.0**	1.5	**4.0**	2.0	**3.0**	2.0	**3.**	2.0	**3.0**	1.5	**4.0**

Table 5.2.2 Revised signal timing parameters for the intersection at Beechnut St.

Phase	1 (old)	**1***	2 (old)	**2***	3 (old)	**3***	4 (old)	**4***	5 (old)	**5***	6 (old)	**6***
Minimum green	5	**3**	15	**4**	5	**3**	7	**3**	5	**3**	15	**4**
Maximum green	15	**15**	35	**45**	15	**15**	25	**25**	15	**15**	35	**45**
Vehicle extension	2.0	**3.0**	1.5	**4.0**	2.0	**3.0**	2.0	**3.0**	2.0	**3.0**	1.5	**4.0**

Table 5.2.3 Revised signal timing parameters for the intersection at Old Orchard Dr.

Phase	**1 (old)**	**1***	**2 (old)**	**2***	**4 (old)**	**4***	**5 (old)**	**5***	**6 (old)**	**6***
Minimum green time	5	**3**	15	**4**	7	**3**	5	**3**	15	**4**
Maximum green time	30	**30**	50	**60**	30	**30**	20	**20**	50	**60**
Vehicle extension time	1.5	**3.0**	1.0	**4.0**	3.0	**3.0**	1.5	**3.0**	1.0	**4.0**

Table 5.2.4 Revised signal timing parameters for the intersection at Old Orchard Dr.

Phase	1 (old)	**1***	2 (old)	**2***	3 (old)	**3***	4 (old)	**4***	5 (old)	**5***	6 (old)	**6***
Minimum green time	5	**3**	15	**4**	7	**3**	7	**3**	5	**3**	15	**4**
Maximum green time	20	**20**	50	**60**	25	**25**	25	**25**	20	**20**	50	**60**
Vehicle extension time	1.5	**3.0**	2.0	**4.0**	2.0	**3.0**	2.0	**3.0**	1.5	**3.0**	2.0	**4.0**

Note: "old" in the phase refers to the existing signal timing parameter values used by TxDOT and "*" in the phase refers to the modification made by the Performing Agency to those signal timing parameters.

5.2.1 Setting of MM-6

In the ASC/3 Controller, MM-6 mainly deals with detectors associated with different phases. From the main menu, the type and function of detectors can be programmed, such as the record of vehicle volume, percentage occupancy, etc.

MM 6-1 is used to assign different detectors to its respective phases. MM 6-2 assists to set different function of the detectors. In this study on the testbed of FM 1464, the research team chose the NTCIP mode of the detectors, and enabled the Volume and Occupancy logs. To set the updating interval, the NTCIP log interval was set as 5.0 seconds for all of the independent intersections that runs in the semi-actuated mode. On the other hand, for those running in the coordination mode, the NTCIP log interval was set as 60 seconds via MM 6-4.

5.3. DEVELOPMENT OF LOGICAL STATEMENTS

5.3.1 General Logical Statements

Basically, the research team from the Performing Agency developed two algorithms for this testbed. The first algorithm is applicable to all of the intersections at the testbed of FM 1464; and the second was used at the first five coordinated intersections (which runs in the coordination mode originally). They are further discussed below. In the ASC/3 Controller, using the Logic Statement by MM 1-8-2, the algorithms described below were written through the ASC/3 controller. MM 1-8-1 can be used to activate or de-active any logic statement.

Basic Logic/Algorithm

The basic logic/algorithm of the proposed proactive control system is similar to the ones described in Chapters 3 and 4 (see Figures 3.3.1 and 4.3.1, respectively). This basic algorithm was implemented of the intersection at Bissonnet St. on February 27, 2018. Then on April 19, 2018 and May 10, 2018, the implementation was focused on the first and second parts of intersections, respectively (see Section 5.1). Figure 5.3.1 illustrates the flowchart of this basic proactive signal control logic/algorithm.

New Logic Statements for the First Five Coordinated Intersections

Like some intersections at the testbed of SH 242, the first five intersections at the testbed of FM 1464 run in the coordination mode with semi-actuated system, i.e., the major phases are fixed timed while the left-turn phases and minor phases are actuated. Therefore, as did to the testbed of SH242, for these intersections, the research team from the Performing Agency added new logic statements along with the basic logic statement. The purpose of this new logic statement is to *provide an automatic switch between coordination mode and actuated control mode, based on the traffic flow in major through movement in Phase 2 and Phase 6*. However, please note that the parameters and the phases used in the logic for switching are different from those used in the testbed of SH 242, making the logic of switching completely different from that used in SH242.

In this case, the research team employed the parameter of *detector volume* for Phases 2 and 6 (through movement on FM 1464), respectively, to control the switch between the coordination and actuated modes. The logic is shown in Figure 5.3.2, which are used in the first five coordinated intersections (from W. Oak Village Dr. to Beechnut St.) at this testbed. As illustrated in Figure 5.3.2, it is seen that when the flow of the through traffic is high on the major road (FM 1464) in either direction (north- or southbound), i.e., during peak hours, the control system is kept in the coordination mode to enable vehicles to pass through these closely spaced intersections. When the flow is low, then the system moves back to the actuated control mode. Note that at three intersections (Greenbridge St., Alden Wood and Gosing Rd.) at SH 242, however, the focus is the left turn traffic and the traffic on the minor roads: when the traffic flow in these directions are high, the system switches to the actuated control mode to release the vehicles. It is because the left turn and minor road at these three intersections (especially of the intersection at Gosling is heavy).

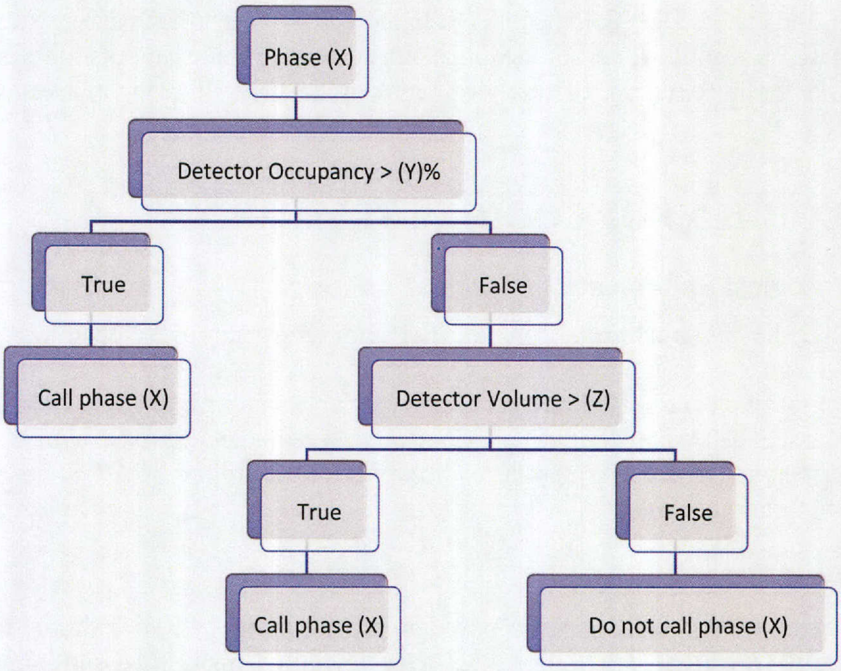


Figure 5.3.1 *Flow chart for the basic proactive signal control logic applied to all intersections*

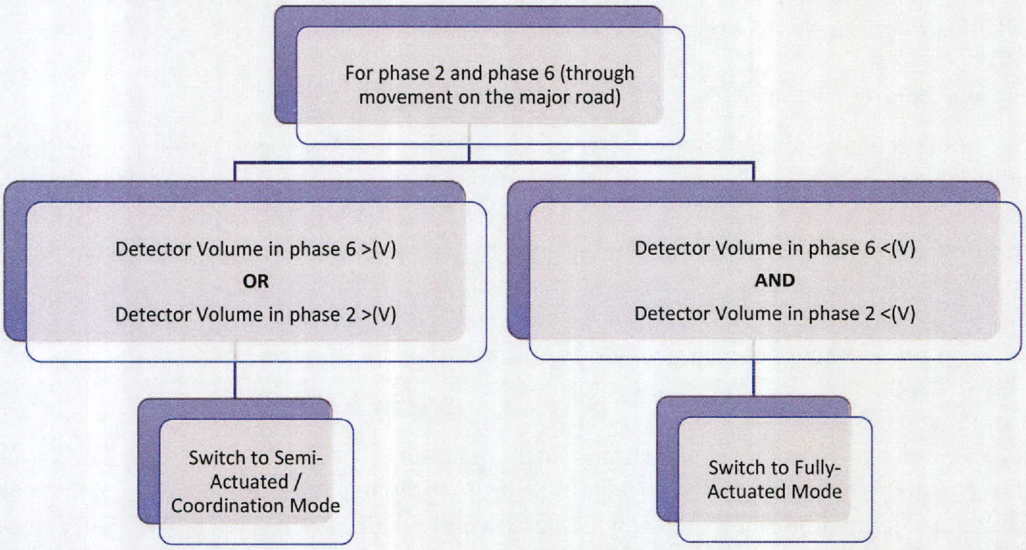


Figure 5.3.2 *Flow chart for the proactive signal control applied to the five coordinated intersections*

Note that here the "actuated control mode" is different from the original actuated mode implemented at some intersections. Originally, the actuation is simply based on the arrivals of individual vehicles. In this implementation, however, if upstream detectors are available at an intersection, the "actuated mode" is more like a "proactive mode", i.e., a short-term prediction is used to control the signal accordingly for the arrival of a platoon of vehicles.

5.3.2 Logic Statements in ASC/3 Econolite Controller

For brevity, this section only reports the values of these parameters at the intersections at W. Bellfort St. and W. Airport Blvd., respectively, because similar values of detector volume and occupancy were used at other 9 intersections. Tables 5.3.1 and 5.3.2 reports the value of the parameters set in VISSIM for the basic logic (as shown in Figure 5.3.1) at these two intersections, respectively, and Figures 5.3.3 and 5.3.4 show the application of the basic logic statements in the ASC/3 Econolite emulator in VISSIM.

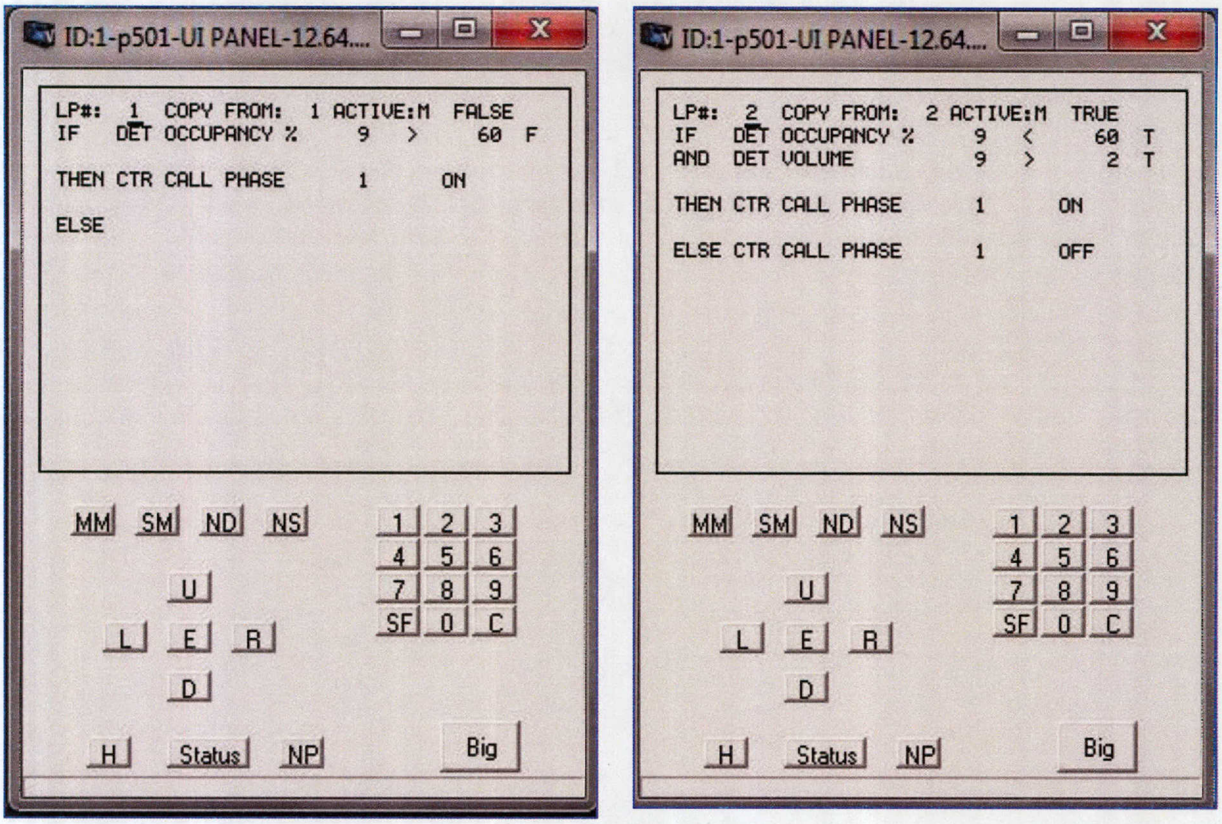
On the other hand, Figure 5.3.5 is an example of the application of logic statements in VISSIM on how to automatically switch the ASC/3 controller's operation mode between the coordination and actuated modes. As described in last section, basically, we want the controller to run in the coordination mode when the through traffic flow (Phases 2 and 6) is high on the major road (FM 1464) in either direction; while if they are low in both directions, the controller will switch to the actuated control mode to reduce the unnecessary delay. To achieve it, this algorithm uses the parameter 'Detector volume' to identify the flow condition on FM 1464. If the detector volume in either Phase 2 or 6 (through traffic on the major road) is greater than 4, then the controller's operation switches to the coordination mode from the actuated control mode; on the other hand, it switches back to the actuated control mode when the detector volumes in Phases 2 and 6, respectively, are both less than 4. Therefore, it implies that in non-peak hours, the controller runs in the actuated control mode, runs in the coordination mode during peak hours. Table 5.3.3 gives the values of Detector Volume used in the logic statements (for switching between the coordination and actuated control mode) for each intersection.

Table 5.3.1 Selected occupancy and volume margins for the intersection of W. Bellfort St.

Phase Number	Detector	Occupancy (%)	Volume
1	1 (SD)	60	2
3	3 (SD)	60	2
4	4 (SD)	60	2
5	5 (SD)	60	2
7	7 (SD)	60	2
8	8 (SD)	60	2

Table 5.3.2 Selected occupancy and volume margins for the intersection of W. Airport Blvd.

Phase Number	Detector	Occupancy (%)	Volume
1	5 (SD)	60	2
3	7 (SD)	60	2
	8 (SD)	60	2
	9 (SD)	60	2
5	6 (SD)	60	2
4	10 (SD)	60	-
	11 (SD)	60	-
	12 (SD)	60	-
	13 (VD)	60	2
	14 (VD)	60	2
	15 (VD)	60	2



(a) (b)

Figure 5.3.3 *Logic statements for Phase 1 (Stop-bar Detector (SD)) of the intersection at W. Oak Village*

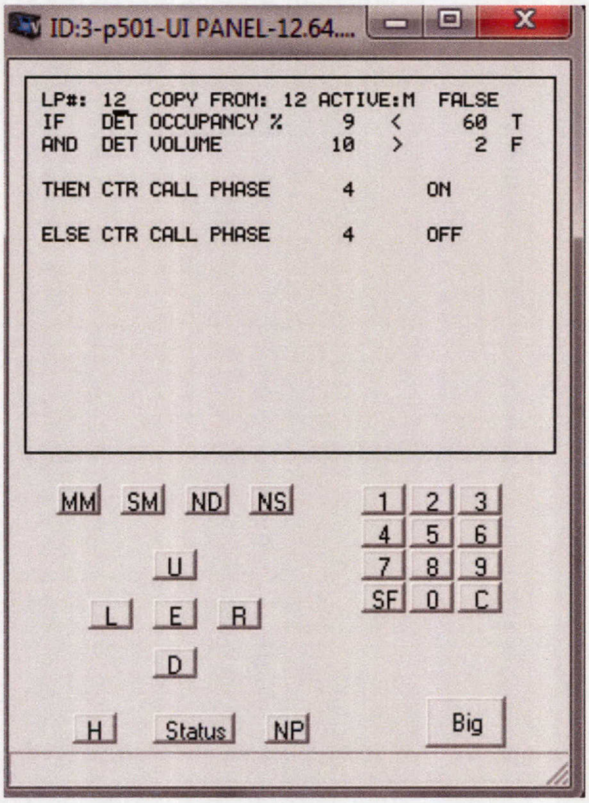
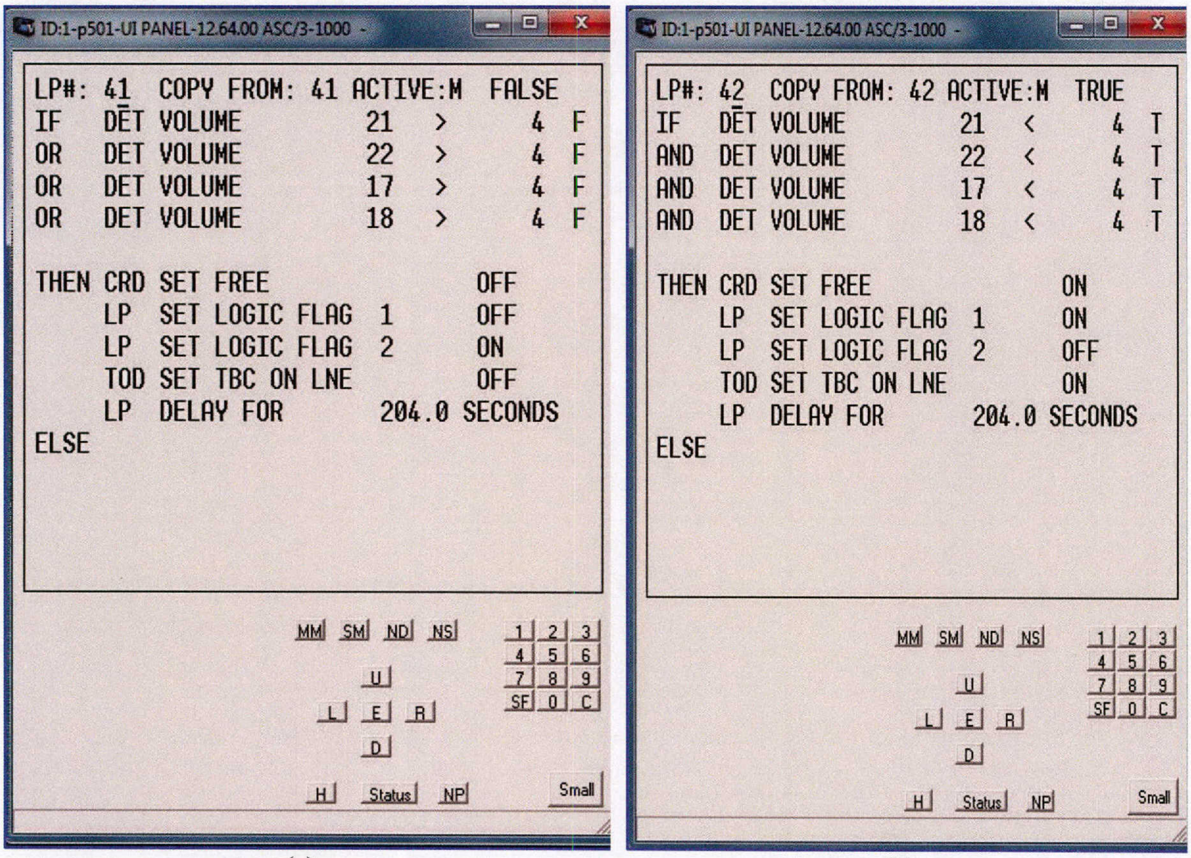


Figure 5.3.4 *Logic Statements for Phase 4 (Volume Detector (VD)) of the intersection at Highland Oak*



(a) (b)

Figure 5.3.5 *Logic Statements to switch between the coordination and actuated control modes*

Table 5.3.3 Selected values of Detector Volume in logic statement

Intersection	Phase	Detector Volume to switch to Coordination Mode	Detector Volume to switch to Actuated Mode
1.W. Oak Village Dr.			
2.Bellaire Blvd.	2	> 4 OR	< 4 AND
3. Highland Oak Ln.	6	> 4 OR	< 4 AND
4. Orchid Ridge Ln.			
5. Beechnut			

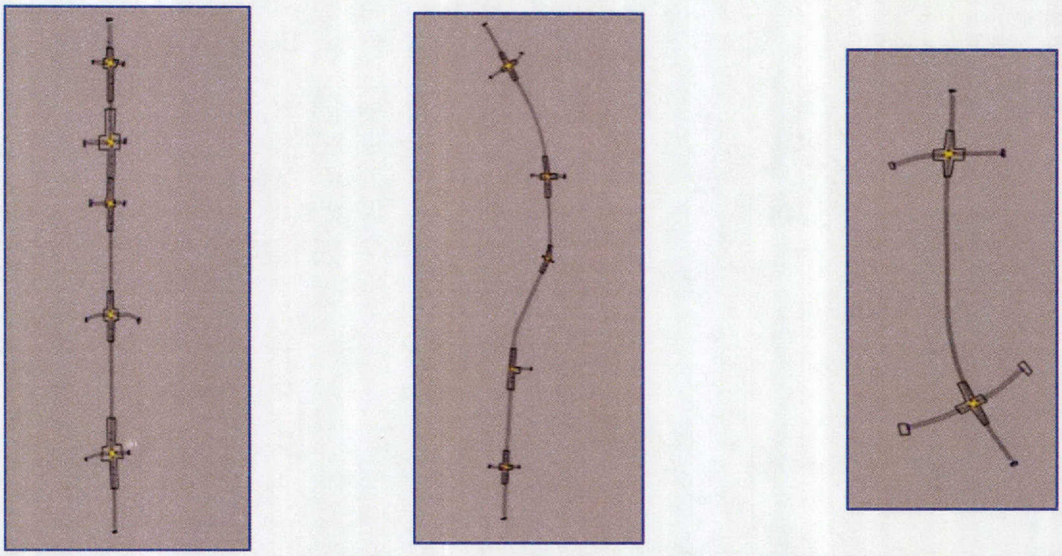
Note: the conditions are used in the logic statements for switching between the coordination and actuated modes

5.4. TEST VIA SIMULATIONS

Before the field experiment, the research team employed the simulation package VISSIM to simulate the traffic flow on the testbed with the proposed proactive signal control (as described in Section 5.3) for the purpose to evaluate the effectiveness of this proposed proactive signal control system.

As mentioned above, the implementation to this testbed was conducted three times, partially because the loop detectors became available in different time: the first one was for the intersection of Bissonnet St., because it was first ready; the second time was for the first five intersections that run in the coordination mode (i.e., part 1); and the third time was for the remaining five (in part 2). For convenience, the whole testbed was divided into three sections: (a) first five coordinated intersections (see Figure 5.1.1, i.e., the intersections in part 1, and see the model in Figure 5.4.1(a)); (2) last five independent intersections starting from W. Belford Blvd. (see Figure 5.1.1 as those in part 2, and see the model in Figure 5.4.1 (b)); and (3) two intersections at Bissonnet St. and W. Belfort Blvd., respectively (see the model Figure 5.4.1

(c)), as the research team received the loop detector data for these two intersections first. However, the first field implementation (on 2/27/2018) did not apply to the intersection of W. Belfort Blvd. To check its compatibility with others, this intersection was simulated again together with other four intersections south of it (see Figure 5.1.1).



(a) For 5 coordinated intersections

(b) For 5 independent intersections (starting from W. Belford Blvd.)

(c) For the intersection of Bissonnet St. and W. Belford Blvd.

Figure 5.4.1 *Three testbed models built in VISSIM for 11 intersections at FM 1464*

5.4.1 Simulation Verification

Note that in Task 3, for each testbed (FM 528, SH 242 and FM 1464), the simulated traffic flows under the existing signal control were expected to compare with the observed ones to ensure that VISSIM can simulate the traffic flow accurately for this specific testbed. However, unlike the testbeds of FM 528 and SH 242, the loop detectors at the testbed of FM 1464 were not fully ready until April 2018, the research team had to re-conduct the simulation verification for this testbed in this task.

The data used for the simulation verification cover from 2/16/2018 8:35:00 am to 3/9/2018 7:23:00 am. The data reported by loop detectors are the volume and occupancy per minute. The research team aggregated the volume for each 15 minutes, starting from 00:01:00 am of each day. That is, for a particular day, the first 15-minute section is from 00:01:00 am to 00:15:00 am, and the second is from 00:15:01 am to 00:30:00 am. If no enough data for one 15-minute section, no data exists for this section. Finally, the multi-day average of volume for each 15-minute section was calculated respectively (as the hourly volume, i.e., 15-minute volume multiplied by 4) for weekday and weekend, respectively. These data were firstly used to estimate the origin-destination matrix of each testbed (such demand data serve as the input parameters for simulations in VISSIM), and then they were used as the base for comparing the simulated flows at this testbed. For brevity, this section just reports the simulated and observed flows at two intersections: Orchid Ridge Ln. from part 1 intersections, and Bissonnet St. from part 2 intersections.

The simulated and observed traffic flows for Phases 2 and 6 (through movements) at these two intersections are reported in Figures 5.4.2 through 5.4.5, and 5.4.6 through 5.4.9, for weekday and weekend, respectively. It is seen that in most cases, the simulated traffic flows are quite close to the observed ones, except those reported in Figure 5.4.8, where the simulated flow is smaller than the observed one, but the patters are quite similar. Such gap may be due to the underestimated demand for this direction. Also, as shown in Figures 5.4.2–5.4.5, we can see there are some sudden jumps from the observed flow. Note that they are the outliers because some data were missing for some particular time sections. The simulations of other intersections also exhibit good results: close to the observed ones, indicating that VISSIM simulations are good to capture the traffic flow pattern at this testbed.

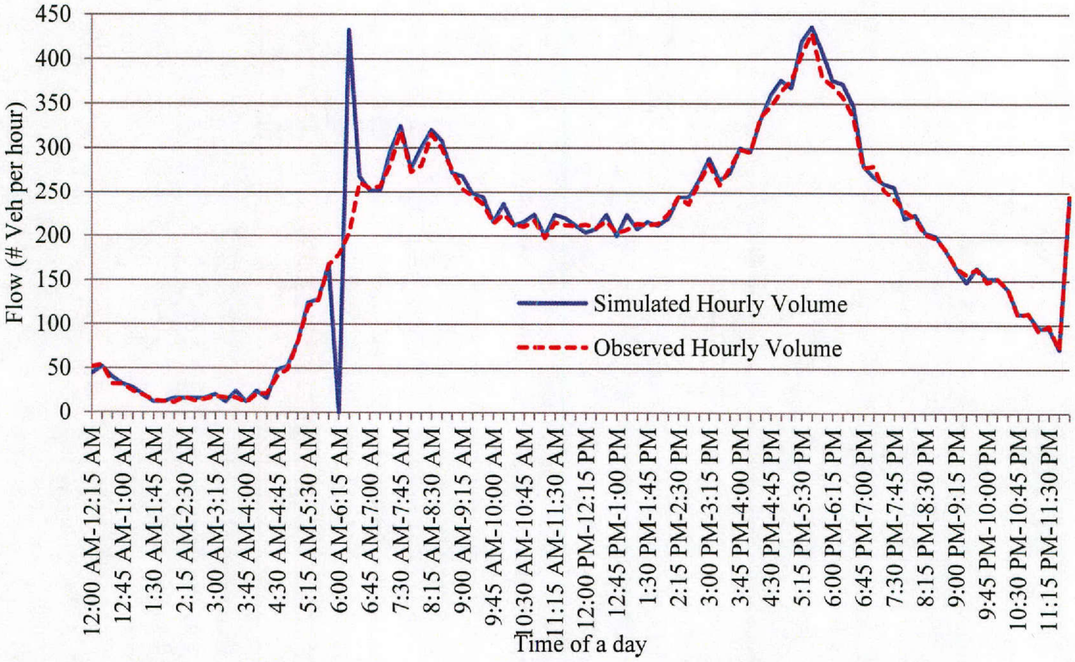


Figure 5.4.2 *Weekday's traffic volume on the major road, FM 1464 (Phase 2) of the intersection at Orchid Ridge Ln.*

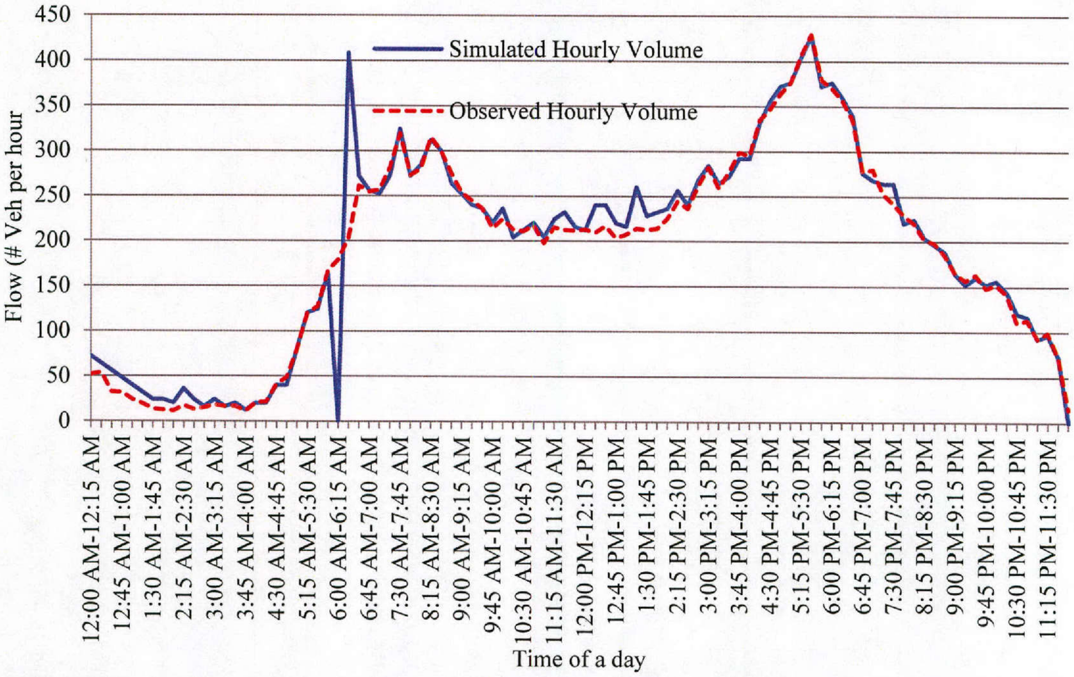


Figure 5.4.3 *Weekend's traffic volume on the major road, FM 1464 (Phase 2) of the intersection at Orchid Ridge Ln.*

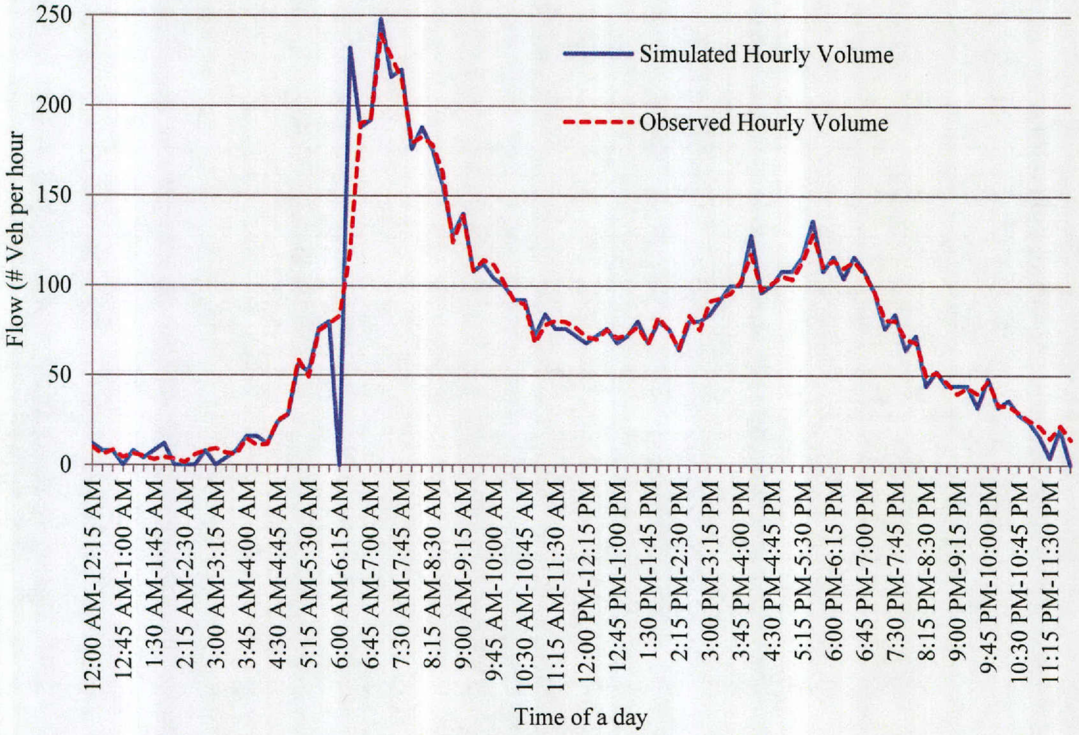


Figure 5.4.4 *Weekday's traffic volume on the major road, FM 1464 (Phase 6) of the intersection at Orchid Ridge Ln.*

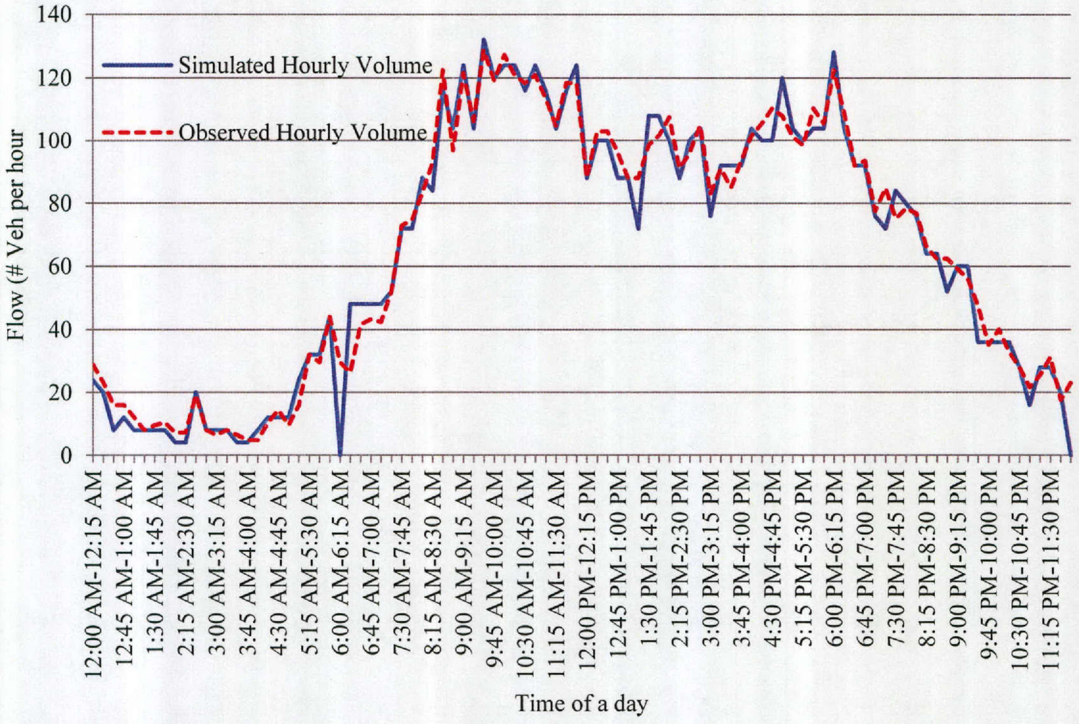


Figure 5.4.5 *Weekend's traffic volume on the major road, FM 1464 (Phase 6) of the intersection at Orchid Ridge Ln.*

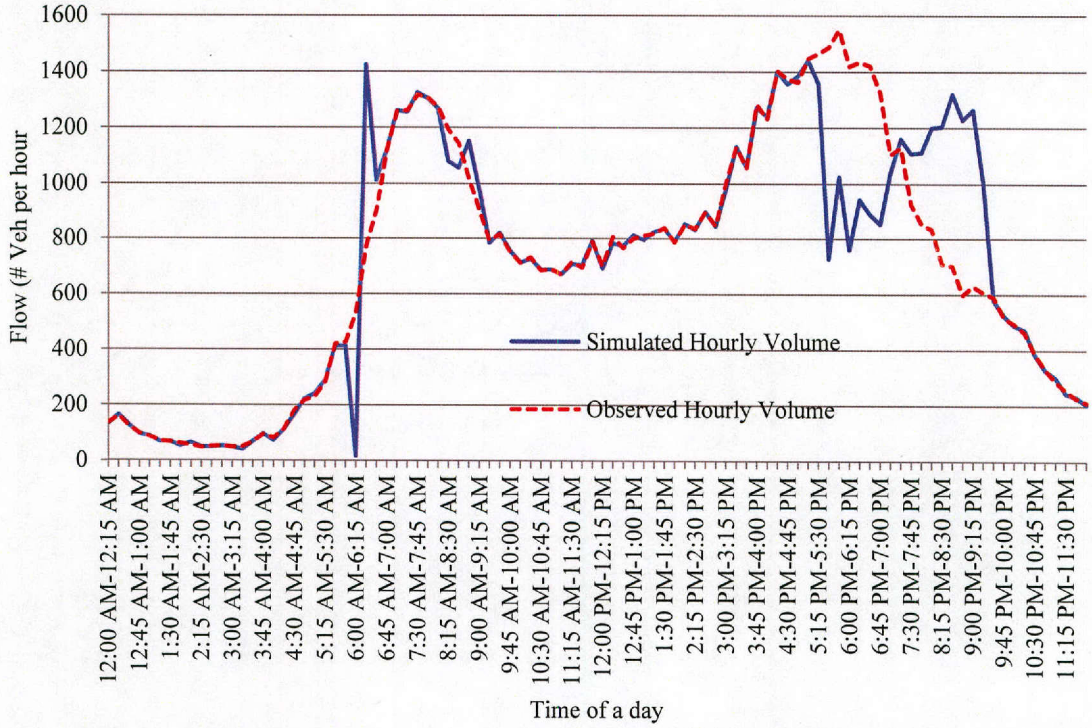


Figure 5.4.6 *Weekday's traffic volume on the major road, FM 1464 (Phase 2) of the intersection at Bissonnet St.*

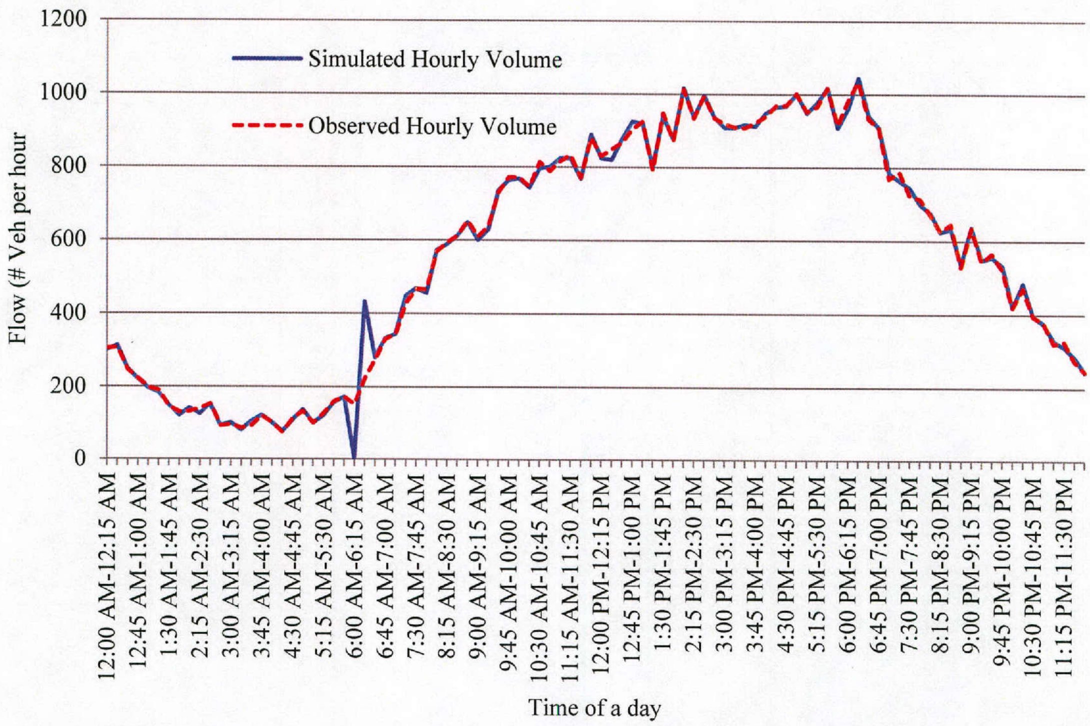


Figure 5.4.7 *Weekend's traffic volume on the major road, FM 1464 (Phase 2) of the intersection at Bissonnet St.*

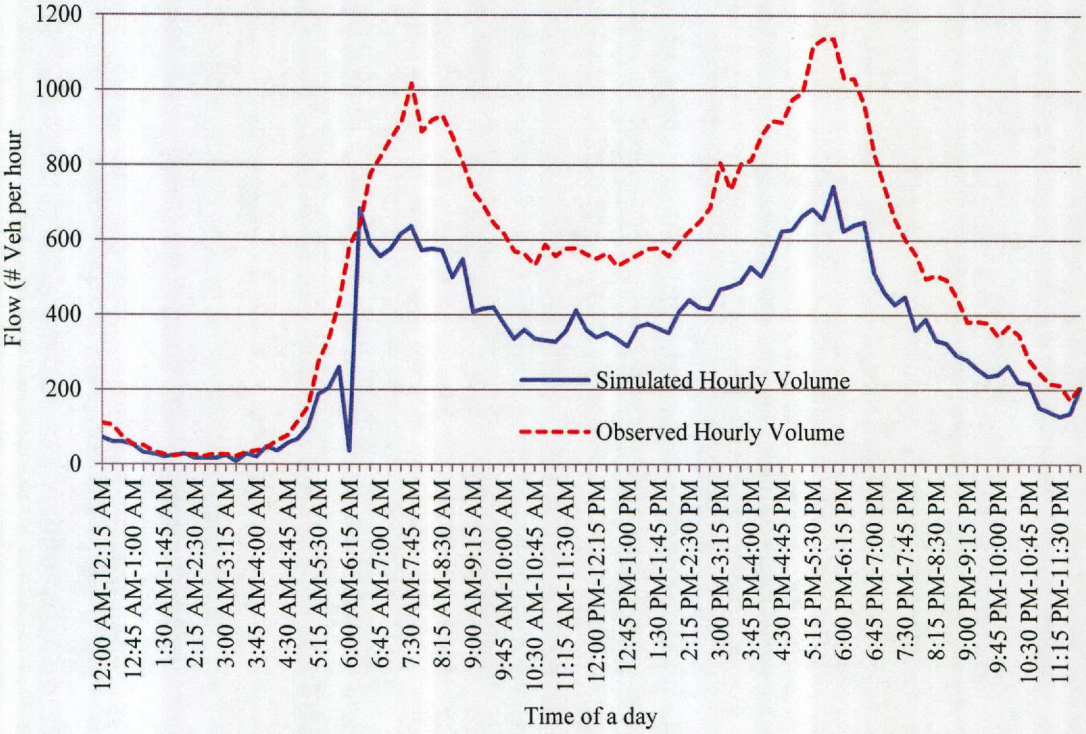


Figure 5.4.8 *Weekday's traffic volume on the major road, FM 1464 (Phase 6) of the intersection at Bissonnet St.*

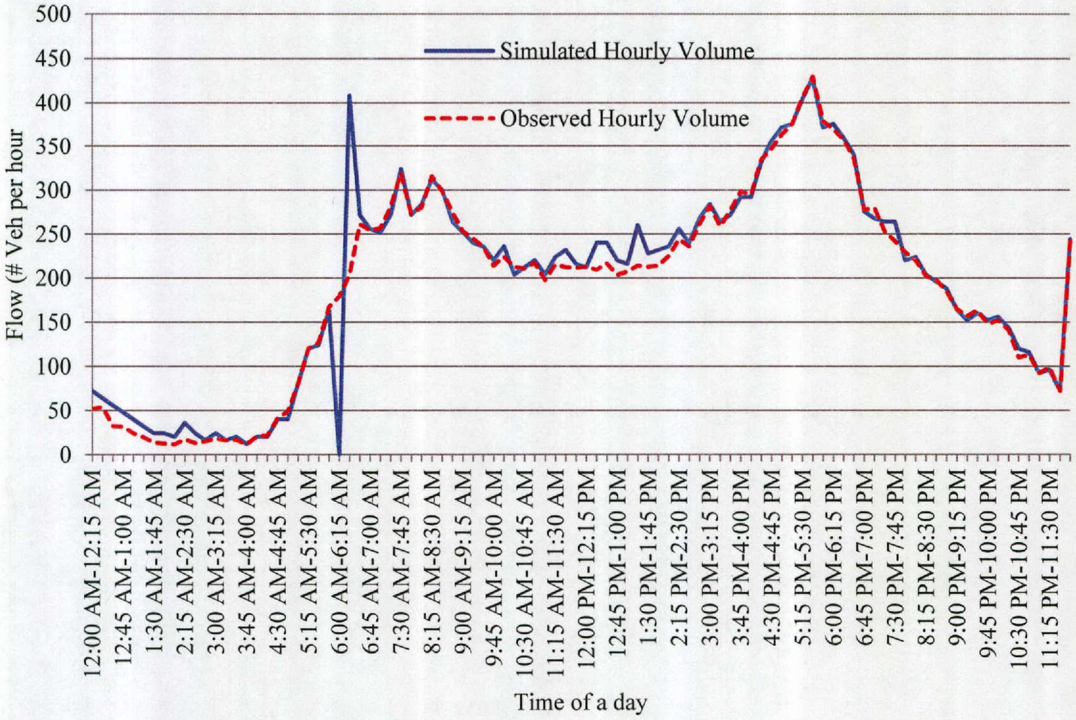


Figure 5.4.9 *Weekend's traffic volume on the major road, FM 1464 (Phase 6) of the intersection at Bissonnet St.*

5.4.2 Simulation Results

Still using the estimated origin-destination demands used for simulation verification, the research team simulate the traffic under the proposed proactive signal control algorithms (see Section 5.3). Note that when doing the simulation verification, the simulations were under the original signal control plans, provided by the Receiving Agency. Note that in addition to the multi-day average hourly volumes, the multi-day average occupancy was also employed. That is: based on the original occupancy data reported at each minute for a number of days, an average for each 15-minute section of each day were calculated (starting from 00:01:00 am of each day), and then multi-day's average for each 15-minute section is calculated for weekday and weekend, respectively.

Based on the multi-day average hourly volume and occupancy pattern in peak hours collected from detector data, the research team sets different values of the critical volumes and occupancies, and the extension time, to define the proposed logic for each detector through the simulator of the ASC/3 controller imbedded in VISSIM. The settings with the highest savings in vehicle delay were recommended in the field experiment.

As mentioned in Section 5.4.1, the whole testbed of FM 1464 was modeled in three separate testbeds. The first testbed model consists of five coordinated intersections; the second one consists of five independent intersections starting from W. Belford St.; and the third one covers two intersections at Bissonnet St. and W. Bellfort Blvd. The simulation covers a whole day (24 hours) of weekday or weekend for three models, since the traffic pattern in weekday and weekend is quite different. Using the simulation results under the original semi-actuated signal control as the base, the performance of the proposed proactive signal control systems for three testbed models are reported in Tables 5.4.1 through 5.4.3, respectively.

These tables indicate that the proposed proactive signal control systems perform well, especially at the first testbed for five coordinated intersections: it helps increase the average speed and reducing the average delay at each testbed mode. For example, for the first testbed, the average speeds were increased by 31% and 56% but the average delays were reduced by 41% and 58%, for a weekday and weekend, respectively. On the other hand, however, on the second testbed, the performance is not as good as that in the first testbed: the average speeds were increased by only 2.91% and 2.24%, for a weekday and weekend, respectively, but the average delays were still reduced by -14.45% and -11.39%, for a weekday and weekend, respectively. Such reductions can still be considered as a significant improvement. Finally, for the first testbed covering two intersections, the performance of the proactive signal control systems also looks good in a weekday—the average speed was increased by 23% and the average delay was decreased by 23%, though the performance in a weekend is not that good.

Table 5.4.1 Performance of the proactive signal control systems on the first testbed model (5 coordinated intersections) for a whole day of weekday or weekend

Trip Measurement (per trip)	Weekday			Weekend		
	Before	After	% Change	Before	After	% Change
Average Speed (mph)	30.89	40.42	31%	25.43	39.77	56%
Average Delay Stop (sec)	69.87	31.41	-55%	91.64	29.59	-68%
Average Delay (sec)	98.78	58.05	-41%	117.2	48.92	-58%
Distance Total (m)	84508.33	84952.53	1%	48199.69	48655.31	1%
Travel Time Total (sec)	9848442.5	7566557.79	-23%	6823369.79	4403875.2	-35%
Delay Total (sec)	5631893.2	3328672.08	-41%	4225853.56	1781246.55	-58%
Stops Total (sec)	143890	111965	-22%	89478	60687	-32%

Table 5.4.2 Performance of the proactive signal control systems on the second testbed model (5 independent intersections) for a whole day of weekday or weekend

	Weekdays			Weekends		
Trip Measurement (per trip)	Before	After	% Change	Before	After	% Change
Average Speed(mph)	57.43	59.15	2.91%	57.68	59	2.24%
Average Delay Stop(sec)	13.19	9.76	-35.14%	11	8.35	-31.74%
Average Delay (sec)	26.77	23.39	-14.45%	23.47	21.07	-11.39%
Distance Total(m)	63741.77	63459.48	-0.44%	40801.63	40802.14	0.00%
Travel Time Total(sec)	3995776	3862032.6	-3.46%	2546684.7	2489490.8	-2.30%
Delay Total(sec)	948850.27	829343.17	-14.41%	557253.7	500181.95	-11.41%
Stops Total(sec)	38498	38075	-1.11%	26059	26133	0.28%

Table 5.4.3 Performance of the proactive signal control systems on the third testbed model (2 intersections) for a whole day of weekday or weekend

	Weekdays			Weekends		
Trip Measurement (per trip)	Before	After	% Change	Before	After	% Change
Average Speed(mph)	12.88	15.90	23%	52.28	53.46	2%
Average Delay Stop(sec)	237.50	178.74	-25%	20.03	17.80	-11%
	363.34	280.15	-23%	33.86	31.47	-7%
Distance Total(m)	53379.32	53391.59	0%	47400	47401.17	0%
Travel Time Total(sec)	14924169	12091620	-19%	3263935	3192168	-2%
Delay Total(sec)	12403918	9570521	-23%	1021012	949162	-7%
Stops Total(sec)	524041	411181	-22%	33453	32645	-2%

5.5. ANALYSIS OF FIELD DATA FROM DETECTORS

The results reported in Tables 5.4.1 through 5.4.3 were from simulations. The real field test results may be different. Therefore, the raw data collected from the loop detectors before and after the implementations were used to evaluate the performance of the implemented proactive signal control system at each intersection.

5.5.1 Coverage of Time

The implementation was conducted at different times at intersections in the testbed of FM 1464. The volume and occupancy data were recorded before and after the each implementation for the analysis.

- The first implementation was applied to the intersection of Bissonnet St. on February 27, 2018. This intersection was a newly opened one, and it has loop detectors ready when it was open. For this reason, implementation was made to this intersection earlier than the other intersections at this testbed.
- The second implementation was applied at all the first five coordinated intersections (the first section in Figure 5.1.1) on April 19, 2018. Later on the research team decided to modify the algorithm as per the suggestion from Mr. Steve Chiu.
- The third and final implementation was completed for all the remaining intersections at the

testbed of FM 1464 on May 10, 2018. Note that the new revised logics were implemented at first five coordinated intersections.

Table 5.5.1 reports the periods when the data were collected, for the performance evaluation for different intersections at the testbed of FM1464.

Table 5.5.1 Time interval of detector data collection based on controller's operation mode

Intersection	Period when the data were collected	
	Under original semi-actuated control system	**Under the proposed proactive control system**
W. Oak to Beechnut (first 5 intersections)	7/16/2018 to 7/31/2018	7/2/2018 to 7/14/2018
Bissonnet	2/16/2018 to 2/27/2018	2/27/2018 to 3/16/2018
West Bellfort to Orchard Lake (last 5 intersections)	4/27/2018 to 5/4/2018	5/11/2018 to 5/18/2018

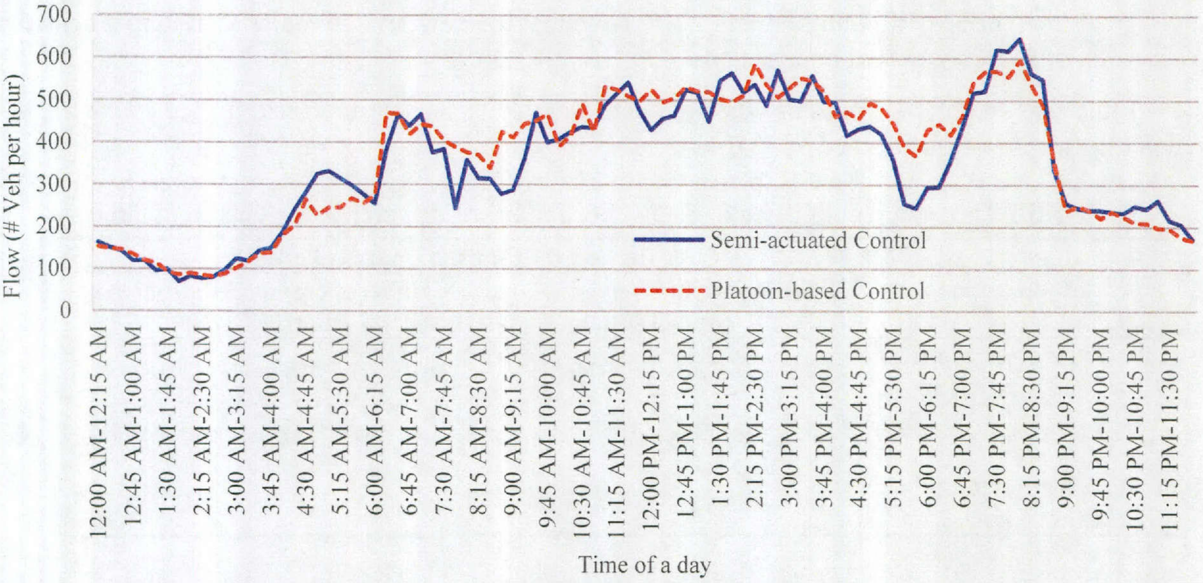
Using exactly the same method described in Section 5.4.2, the raw data reported from loop detectors were summarized into multi-days'15-minute aggregated volumes and 15-minue averaged occupancies for weekday and a weekend, respectively, under either control system (original semi-actuated system or proposed proactive signal control system). For example, if the data covers a week's data, then there are five weekdays and two weekend days. Note that 15-minute volumes were all scaled to hourly volumes by a multiplier of 4. In the following, *please note that both "flow" and "occupancy" are actually the average of multi-days' values.*

For brevity, Section 5.5.2 only shows the comparison of the flows and occupancies under some major phases of four selected intersections: W. Oak Village, Bissonnet St., West Bellfort Blvd., and Orchard Lake Dr. The summary of comparison of the weekdays' performance before and after the implementation of all 11 intersections are given in Table 5.5.1.

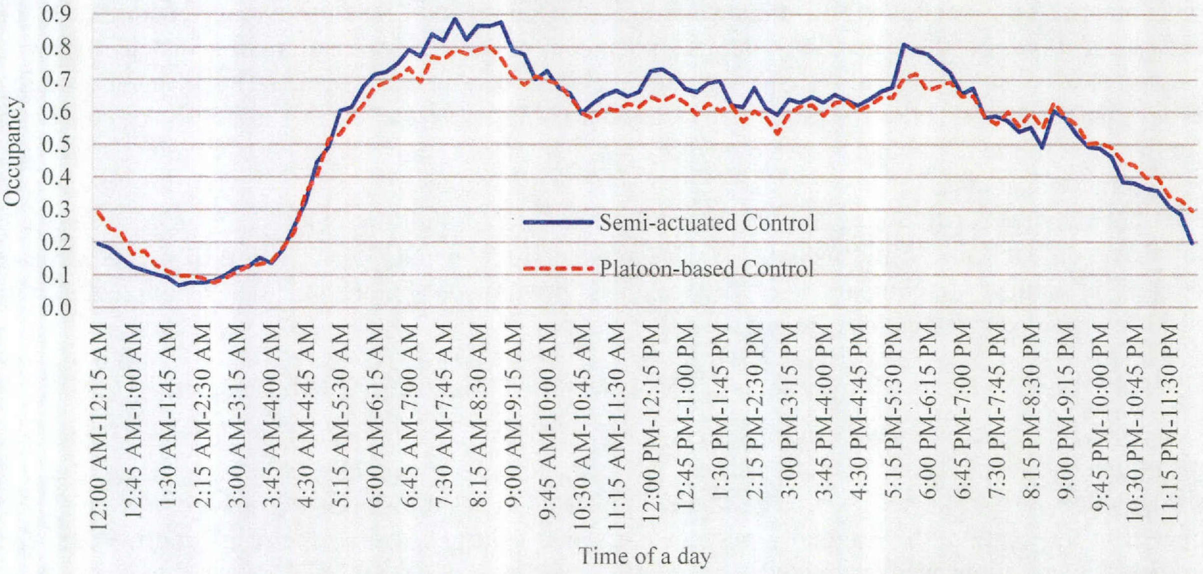
5.5.2 Weekday Traffic on Major Road (FM 1464) for Coordinated Intersections

With the proactive signal control system, the intersection is supposed to facilitate more vehicles to pass through the intersection. Therefore, if the performance of the proposed system is good, after the implementation, it is expected to see the higher vehicle volumes and lower occupancy in a unit time, compared to those under the original semi-actuated signal control. At first, the results on the first five coordinated intersections were examined.

For brevity, we focus on the weekday's traffic conditions on the major road (FM 1464): northbound and southbound through traffic, and the left-turn traffic from the major road. Note that at this testbed, the major traffic movements are north- and southbound traffic along FM1464. For example, as shown in Figure 5.5.1, of the intersection at W. Oak Village, the hourly flow of the northbound traffic along the major road increases but the occupancy decreases. Therefore, it is seen that under this phase, the implemented proactive signal control system can help more vehicles pass the intersection while reduce the congestion at this intersection. Consequently, it implies that the proposed proactive signal control system can help relieve the traffic congestions the traffic under this phase at this intersection.



(a) Hourly Volume



(b) Average Occupancy (in percentage)

Figure 5.5.1 *Multi-day average northbound traffic of the intersection at W. Oak Village on weekday*

For all five coordinated intersections in Part 1, the performance of through traffic and left-turn traffic from the major road (FM 1464) before and after the implementations were reported in Table 5.5.1. It is seen that in most cases (especially during morning and evening peak hours), the implemented proactive signal control system can help relieve the congestion at these intersections: the hourly flows increase but the occupancy does not increase or even reduce. Note that from Table 5.5.1, it is also seen that the intersections of Beechnut, Bellaire, Clodine Reddick, High Land Oak, the performance of the proposed proactive signal control system during the midday period is not as good as that during morning and evening peak hours. During the midday hours, the traffic flow is not high. Note that the parameters of the proactive signal control system were determined through simulating peak-hour traffic (using collected peak-hour data). Therefore, the performance during the midday hours may not so good.

Table 5.5.2 Performance comparison based on multi-day average data for five intersections at FM 1464

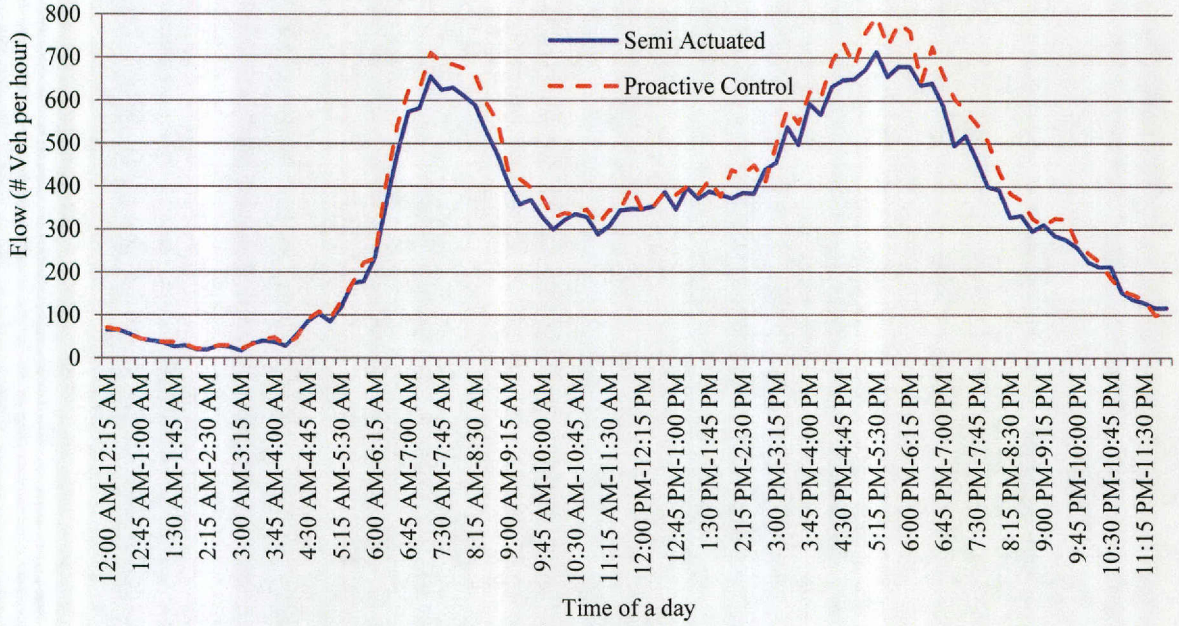
Phase	Day	Time Period		Flow	Occupancy
SB Traffic near Beechnut St.	Weekday	6:00:00	9:30:00	5.3%	-6.6%
		11:00:00	15:00:00	-1.1%	2.7%
		15:00:00	19:00:00	14.4%	-2.2%
		6:00:00	22:00:00	6.5%	-1.0%
LT from Beechnut	Weekend	10:00:00	16:00:00	2.3%	-4.9%
		10:00:00	22:00:00	4.9%	-5.1%
NB Traffic near Bellaire Blvd.	Weekday	6:00:00	9:30:00	9.0%	-5.5%
		11:00:00	15:00:00	-4.5%	0.1%
		15:00:00	19:00:00	3.5%	-1.4%
		6:00:00	22:00:00	1.0%	-1.6%
LT from Bellaire	Weekend	10:00:00	16:00:00	9.5%	3.8%
		10:00:00	22:00:00	6.0%	-0.1%
SB Traffic near Clodine Reddick	Weekday	6:00:00	9:30:00	-5.5%	-13.7%
		11:00:00	15:00:00	-3.2%	-1.2%
		15:00:00	19:00:00	4.4%	-6.0%
		6:00:00	22:00:00	-1.1%	-6.5%
LT from Clodine Reddick	Weekend	10:00:00	16:00:00	10%	9%
		10:00:00	22:00:00	5%	4%
NB near High Land Oak	Weekday	6:00:00	9:30:00	3.4%	-5.6%
		11:00:00	15:00:00	-3.1%	-1.0%
		15:00:00	19:00:00	-1.7%	-1.1%
		6:00:00	22:00:00	-2.0%	-1.3%
LT from High Land Oak	Weekend	10:00:00	16:00:00	2%	-1%
		10:00:00	22:00:00	-1%	-4%
SB near W. Oak Village	Weekday	6:00:00	9:30:00	8.0%	-9.7%
		11:00:00	15:00:00	5.9%	-4.5%
		15:00:00	19:00:00	14.0%	-2.3%
		6:00:00	22:00:00	7.3%	-4.1%
LT from W. Oak Village	Weekend	10:00:00	16:00:00	8.1%	4.9%
		10:00:00	22:00:00	11.3%	4.3%
NB near W. Oak Village	Weekday	6:00:00	9:30:00	18.1%	-8.2%
		11:00:00	15:00:00	3.4%	-8.2%
		15:00:00	19:00:00	19.1%	-5.5%
		6:00:00	22:00:00	9.2%	-4.8%

Note: SB and NB traffic refer to the north- and southbound traffic along FM 1464 at each intersection, respectively. LT refers to left turn from the major road (FM 1464) to minor road at each intersection (north to west or south to east).

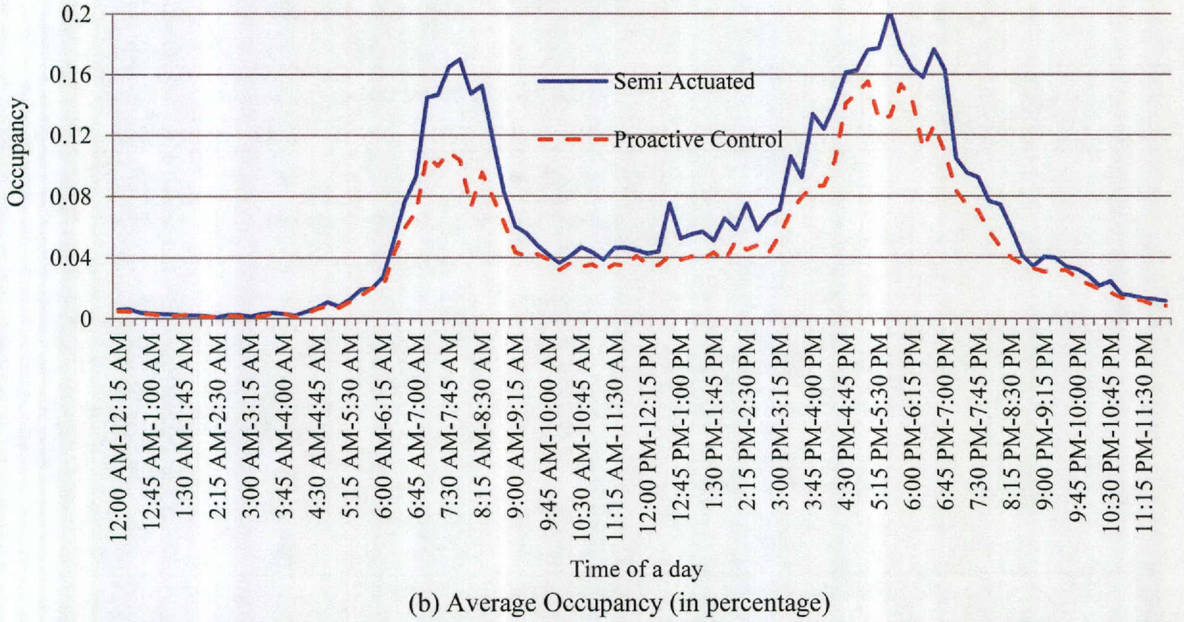
5.5.3 Weekday Traffic on Major Road (FM 1464) for Non-coordinated Intersections

The same analysis was also conducted for the intersections in Part 2, where these intersections were not coordinated. Still, for brevity, this report looks at the northbound and/or southbound traffic on the major road (FM1464), as reported in Figures 5.5.2 through 5.5.8.

As reported in Figure 5.5.2, it is seen that the implemented proactive signal control can help increase the traffic flow while reduce the occupancy for the southbound traffic of the intersection at Bissonnet St.



(a) Hourly Volume



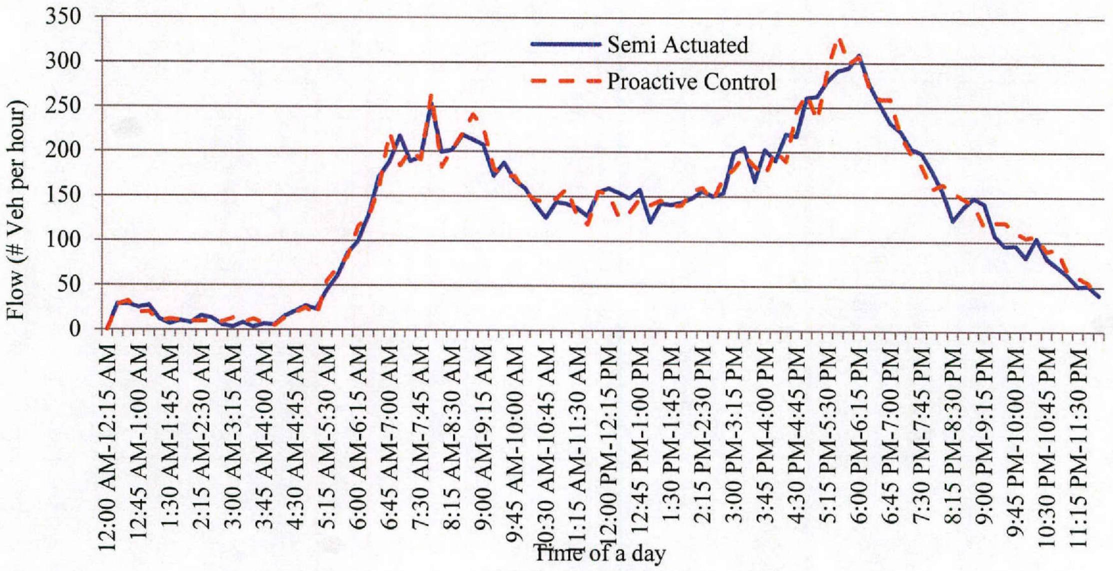
(b) Average Occupancy (in percentage)

Figure 5.5.2 *Multi-day average southbound traffic of the intersection at Bissonnet St. on weekday*

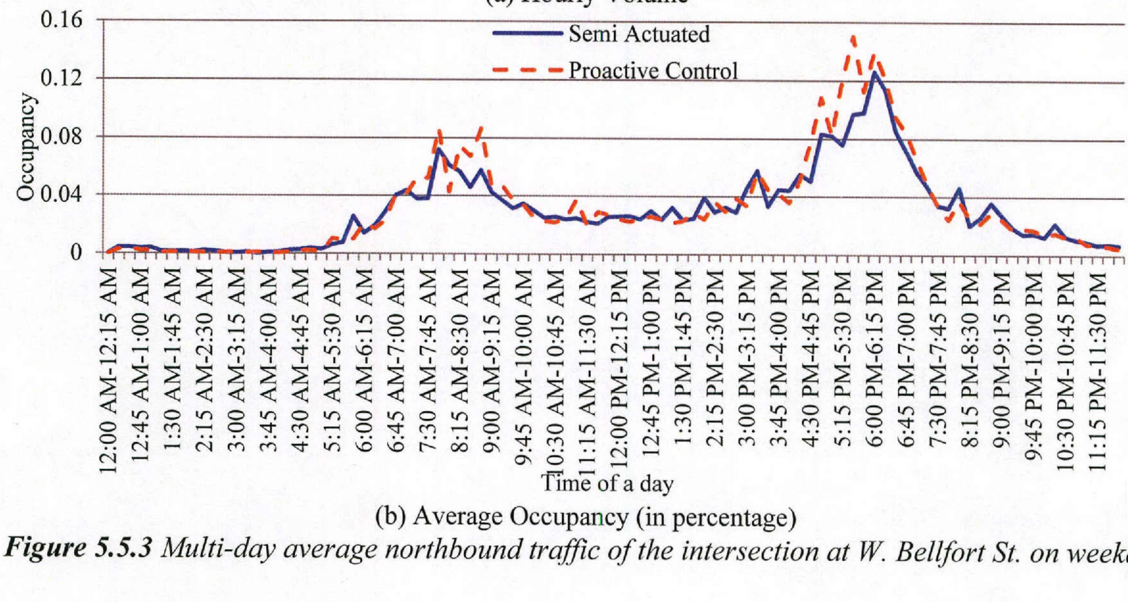
Further, the comparisons of flow and occupancy of north- and southbound traffic of the intersection at W. Bellfort Rd., Orchard Lake and Austin High School are shown in Figures 5.5.3 through 5.5.8. However, it is seen that the effectiveness of the implemented proactive signal control system at these intersections is not so apparent as those in coordinated intersections.

For many situations, the traffic patterns are close under two control system. The reason is that the traffic volumes through these intersections (north- or southbound) are much smaller compared with those through the coordinated intersections. In most cases, the peak hourly flow rate is less than 500 at these intersections; while the peak hourly volumes at those coordinated intersections (plus the intersection of Bissonnet St) is more than 700.

More importantly, it can be seen that from Figures 5.5.3–5.5.8, even the peak values of occupancy is no more than 20% (except a few extreme cases of the intersection at W. Bellfort St.). This pattern is similar to the traffic pattern on the testbed of FM 528 (occupancy rate is no higher than 0.2), as reported in Chapter 3. Such low occupancy rates imply that the vehicles running through these intersections are not largely impacted by others, and the platoon of vehicles is not frequently seen at these intersections. Note that the proposed proactive signal control is particularly designed for helping a platoon of vehicles smoothly pass through the intersection. Therefore, for this reason, the impacts of the proposed proactive signal control system on the traffic through these intersections are insignificant.

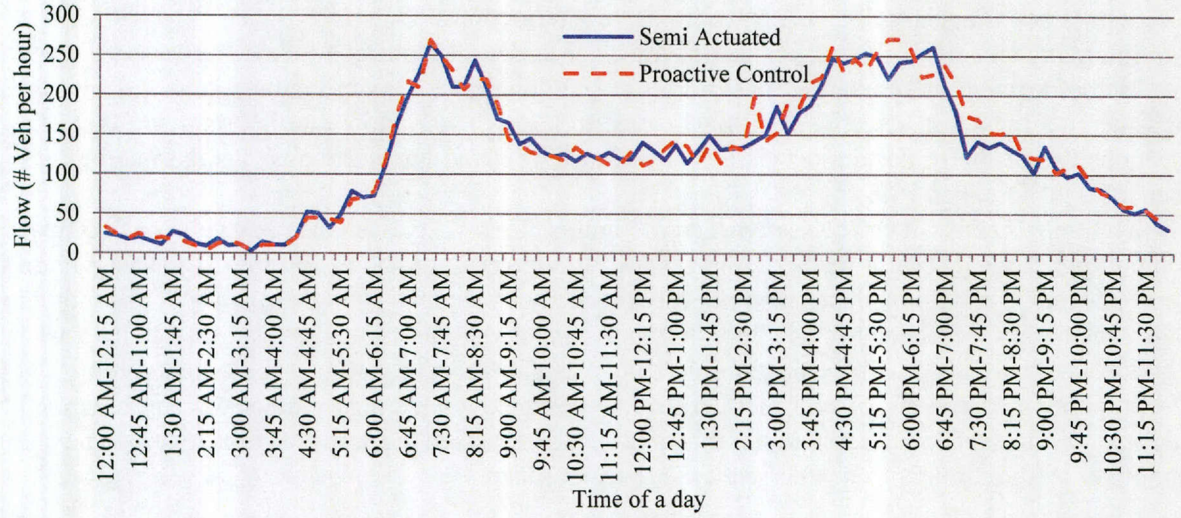


(a) Hourly Volume

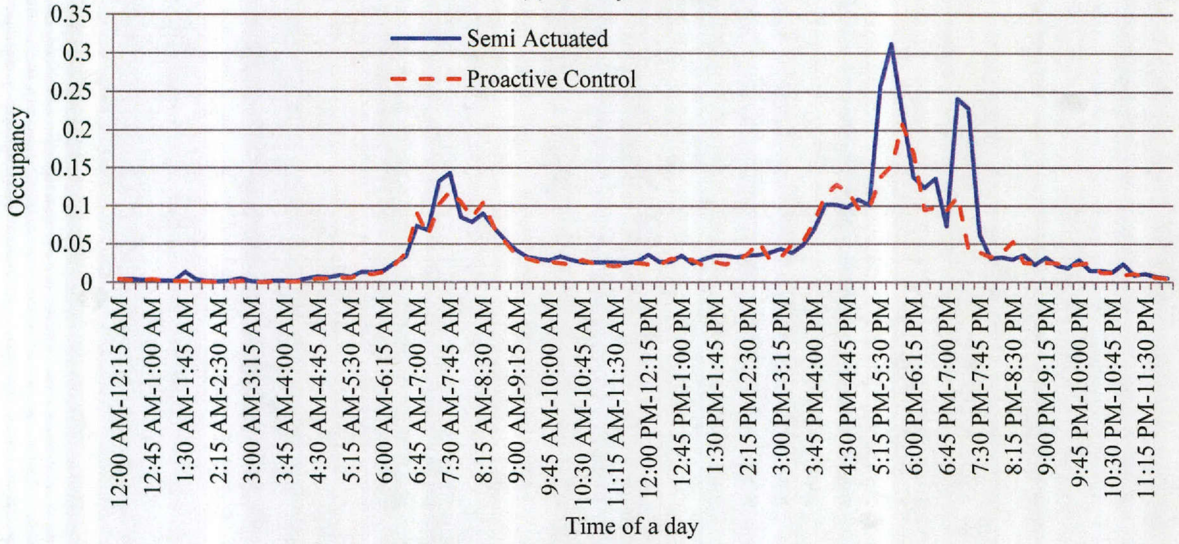


(b) Average Occupancy (in percentage)

Figure 5.5.3 *Multi-day average northbound traffic of the intersection at W. Bellfort St. on weekday*

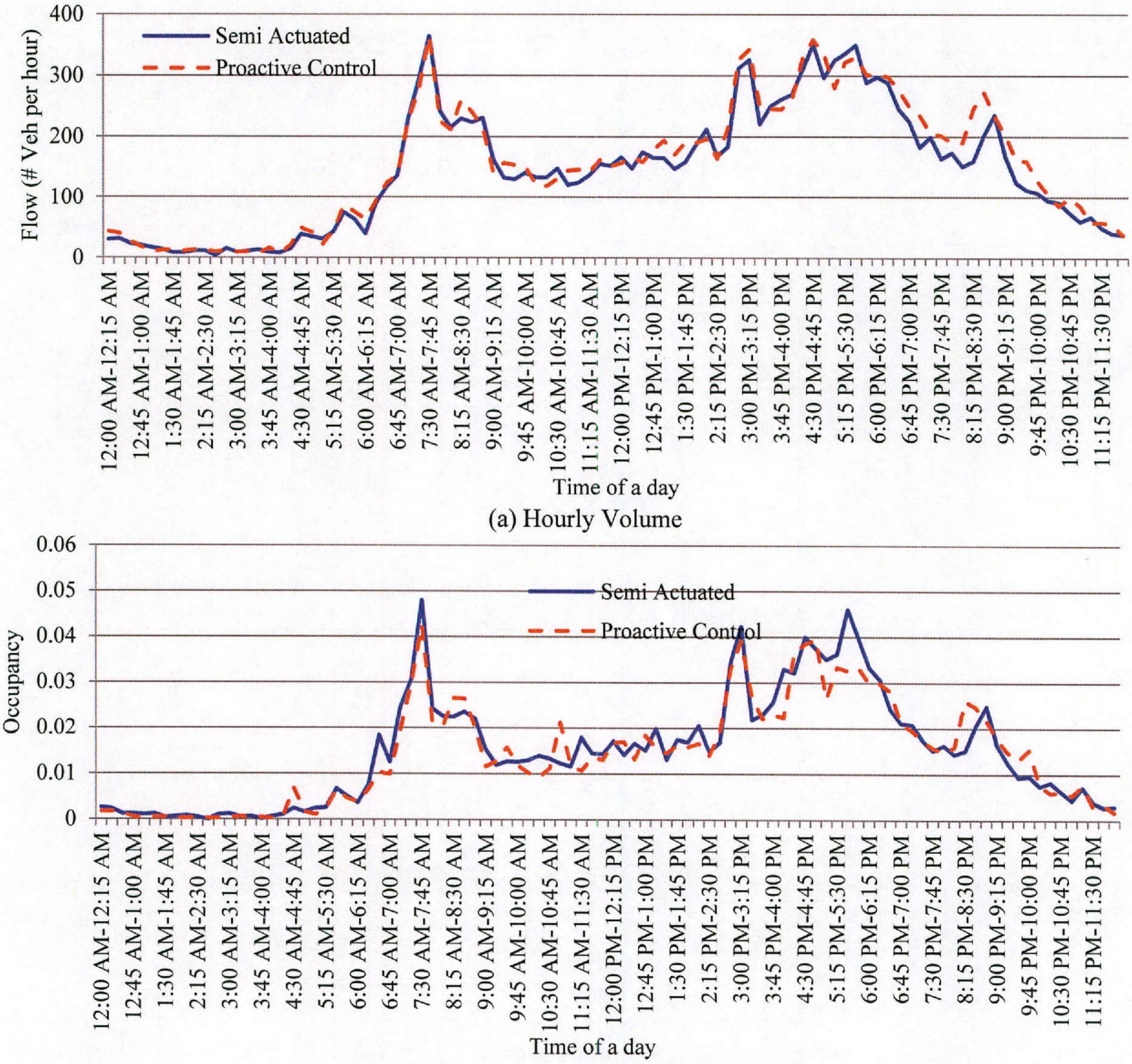


(a) Hourly Volume



(b) Average Occupancy (in percentage)

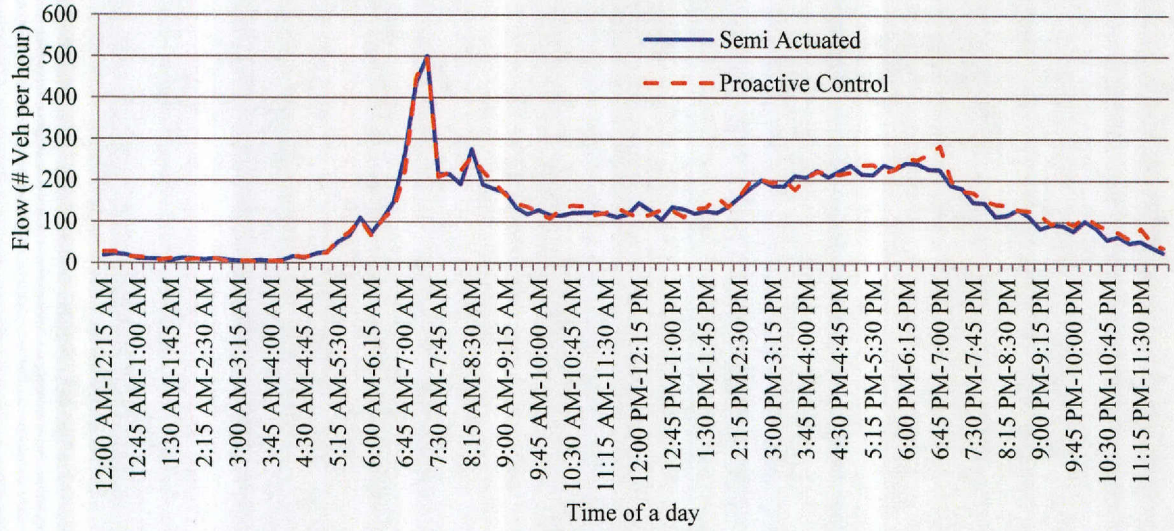
Figure 5.5.4 *Multi-day average southbound traffic of the intersection at W. Bellfort St. on weekday*



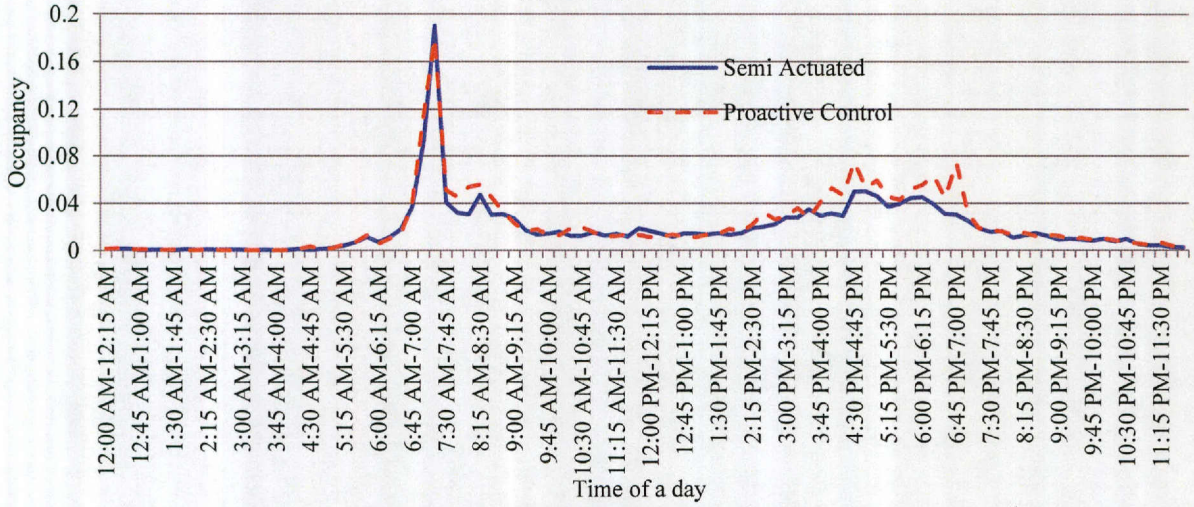
(a) Hourly Volume

(b) Average Occupancy (in percentage)

Figure 5.5.5 *Multi-day average northbound traffic of the intersection at Orchard Lake on weekday*

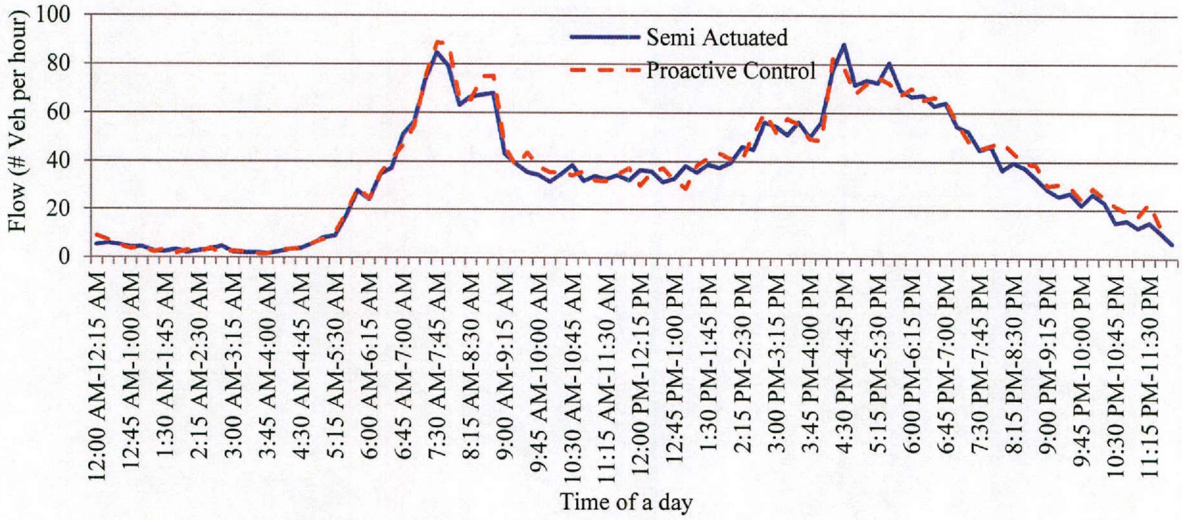


(a) Hourly Volume

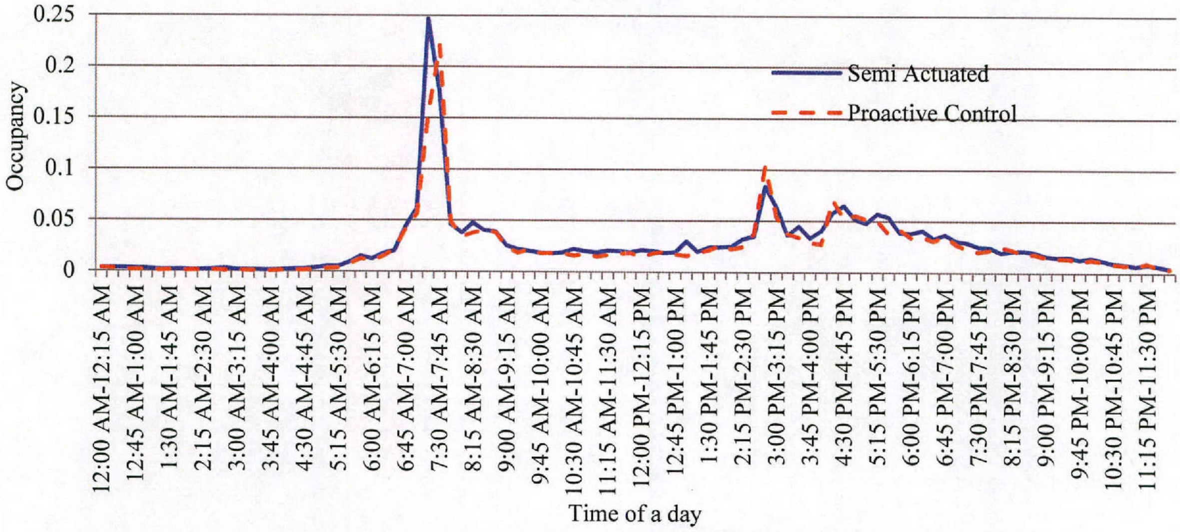


(b) Average Occupancy (in percentage)

Figure 5.5.6 *Multi-day average southbound traffic of the intersection at Orchard Lake on weekday*

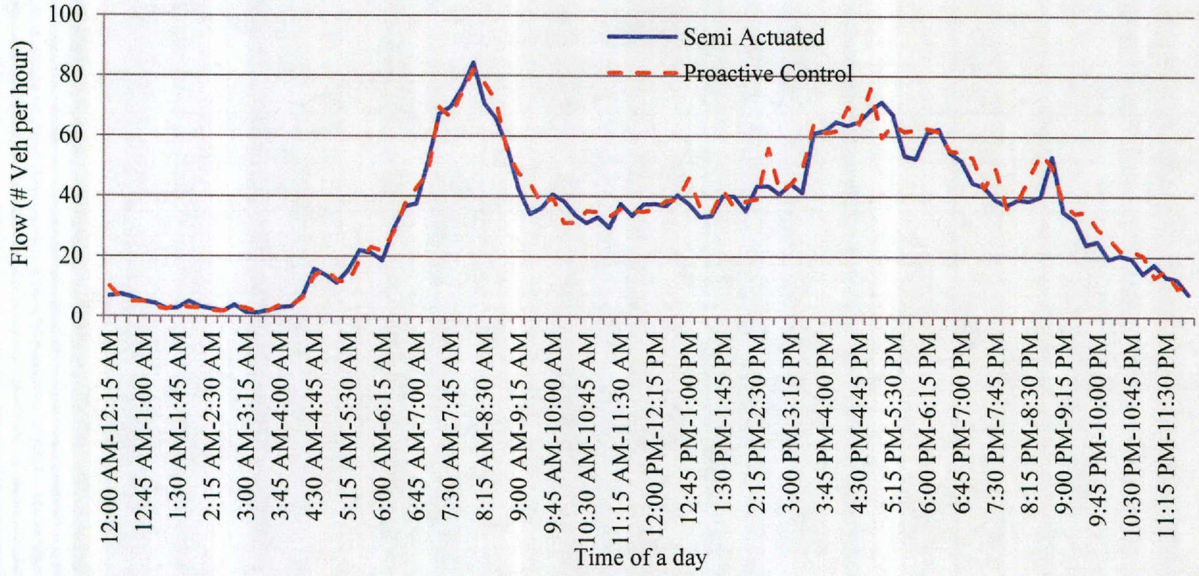


(a) Hourly Volume

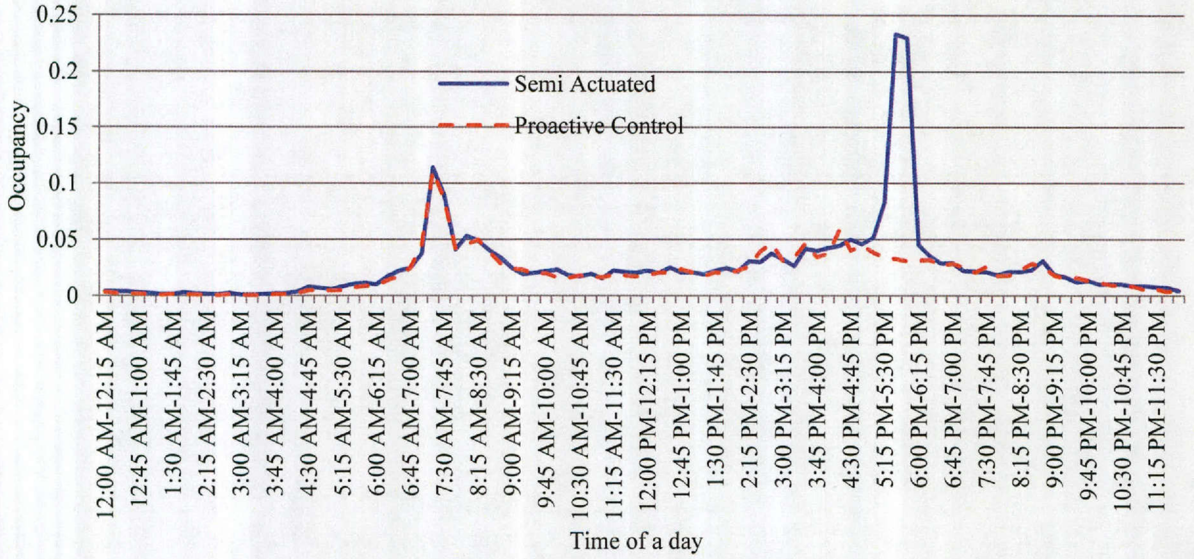


(b) Average Occupancy (in percentage)

Figure 5.5.7 *Multi-day average northbound traffic of the intersection at Austin High School on weekday*



(a) Hourly Volume



(b) Average Occupancy (in percentage)

Figure 5.5.8 *Multi-day average southbound traffic of the intersection at Austin High School on weekday*

5.5.4 Traffic Flows under Other Situations

The above analysis covers the weekday's traffic flow on major road, with a focus on the through traffic along the major road. In addition, the research team also analyzed the traffic data for the major road on weekend, as well as the traffic data on minor roads on weekday and weekend, respectively. For brevity, the figures are not shown here. Readers are referred to TM 4.5 Phase-II Field Experiment Analysis Report (Testbed of FM 1464) for the details.

During weekend, there is not much change in the flow and occupancy in the intersections of Bissonnet St. and West Bellfort Blvd. However, for the eastbound traffic at the intersections of Orchard Lake DR., large occupancy drop can be seen, indicating the improvement in the flow and releasing vehicles through this intersection. For the intersection of Austin High School, the traffic flow seems to be increased with decrease in occupancy level. On the other hand, for minor roads, only significant improvement is seen of the intersection at Orchard Lake Dr. The traffic flow is very small for most of the minor roads. Therefore, for most of the cases, the flow and occupancy levels seem to be unaffected by the proactive control system due to the low traffic volumes.

Generally speaking, the traffic volumes in weekend at this testbed, and the traffic from the minor roads are smaller than those on major road during weekday peak hours. Therefore, the improvement of the implemented proactive signal control system in these situations is not so significant.

5.6. SUMMARY

The research team from the Performing Agency developed the proactive signal control system for the testbed of FM 1464 based on the data collected by loop detectors. With the great help offered by the Receiving Agency, the research team conducted field tests on the testbed in February 27, April 19, and May 10, 2018. The observations were also conducted on these days. During the field implementation, revisions were made to fit the feature of the specific intersections, because some problems were found in the field tests.

Compared with the first testbed of FM 528 (the implementation was conducted in July 2017) and the second testbed of SH 242 (the implementation was conducted in October–November 2017), this testbed is longest, and it has the largest number of intersections (11). The traffic flow at this testbed is not as heavy as that on the testbed of SH 242. But the through traffic for the first five coordinated intersections is still high. The whole testbed is divided into two sections: the first five intersections were closely coordinated due to the close spacing distance, and the other six are independent. The intersections in the second part, south of the intersection of Bissonnet see quite small traffic flow, and very low occupancy rate (less than 20% except a few extreme cases of the intersection at W. Bellfort St.)

For those coordinated intersections, however, different from the testbed of SH 242, on this testbed, the focus is the traffic flow on the major road, because unlike the tested of SH 242, the traffic to and from the minor road is relatively small.

Table 5.5.1 gives a summary of comparing the multi-weekday average flows and occupancies, collected under two control systems: original semi-actuated and proposed proactive control plans, for some phases of five correlated intersections (i.e., the intersections in Part 1, see Figure 5.1.1) for different time of day. These five correlated intersections are the focus on this testbed due to its complexity, as well as its high traffic volumes. It is found that for most intersections (except the southbound traffic near Clodine Reddick), the proposed algorithm can help increase the flow on the major road in most time of day (though not necessarily all the time of a day), and at some intersection, the occupancy is even reduced, such as the southbound traffic at Beechnut St.

On the other hand, it was found that the proposed proactive control system is not so effective for some intersections in the second part (except the intersection at Bissonnet St.). The reason is that the traffic flow through these intersections is quite small, and especially the occupancy level is also very small (less than 20%), implying that the vehicles are less impacted by others. The potential space for improvement is not large. Such phenomenon is also found at some intersections at FM 528 where the occupancy level is low.

CHAPTER 6

IMPLEMENTATION AT FRONTAGE ROAD OF IH-10

6.1. INTRODUCTION

The last set of eight intersections for implementation are completely different from the intersections in other three testbeds: FM 528, SH 242 and FM 1464. As mentioned in Section 1.1 of Chapter 1, the set of these diamond intersections were largely replaced by new ones due to the connection problems. Only two intersections—at Westgreen Blvd. and Greenhouse Rd.—were kept in the final implementation.

Finally, the eight intersections are all along IH-10 west of Houston (Energy Corridor and Katy): Westgreen Blvd., Greenhouse Rd., Katy Fort Bend, Mason Rd., Mason Access Rd., Fry Rd., Fry Access Rd., and Barker Cypress Rd. Their locations are shown in Figure 6.1.1. Among them, the intersections at Mason Rd., Mason Access Rd., Fry Rd. and Fry Access Rd. were determined at the end of November 2018, and the intersection at Barker Cypress Rd. was determined in June 2018. The details of the change were reported in Table 1.1.2 (in Chapter 1).

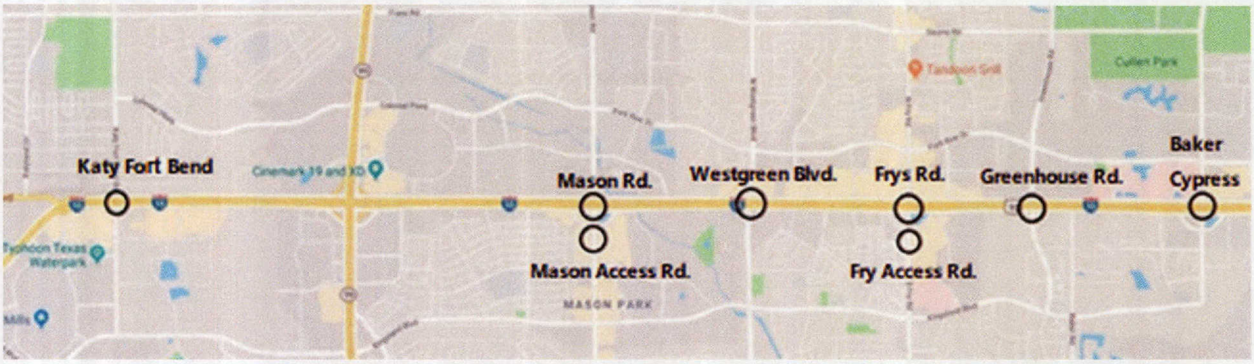


Figure 6.1.1 *Eight intersections at the testbed of IH-10*

Also, among these eight intersections, six are diamond intersections and two are four-way intersections (Mason Access Rd. and Fry Access Rd.). These two are located about 500 ft south of the diamond intersections at Mason Rd. and Fry Rd., respectively, as shown in Figure 6.1.1. Also, three intersections at Mason, Fry and Barker Cypress have only video detectors.

Due to this change, the implementation to these set intersections was separate. For the last five, due to the change, the research team went to the testbed twice for the purpose of implementation. The first time is for the testbed inspection and planning of implementation. Table 6.1.1 gives the implementation details.

Table 6.1.1 Implementation time to eight intersections

Corridors	Implementation Time
Westgreen Blvd. and Greenhouse Rd.	Nov 2, 2018
Katy Fort Bend	Nov 14, 2018
Mason Rd., Mason Access Rd., Fry Rd., Fry Access Rd., Barker Cypress Rd.	Dec 11, 2018 Dec 17, 2018

Originally, according to the latest revised work plan, the implementation report on these eight intersections (TM 4.6a) should be submitted by the end of November 2018. However, due to the change of intersections, the report (TM 4.6a), which was submitted on December 1, 2018, only covers the first three intersections: Westgreen Blvd., Greenhouse Rd., and Katy Ford Bend. Also, because the implementation at the intersection at Katy Fort Bend was on November 14, 2018, and the data collected at this intersection (for the purpose of evaluating the performance of the system) was not available until the end of November, the report (TM 4.6a) provided to the Receiving Agency at that time only covers the data analysis for two intersections: Westgreen Blvd. and Greenhouse Rd.

However, this chapter covers all intersections. The reminder of this chapter is organized as follows.

Section 6.2 describes the properties of these eight intersections including the phase settings and signal plan settings. Section 6.3 introduces the logic statement settings for this proposed system; Section 6.4 summarizes the findings from the field observation; Section 6.5 summarizes and analyzes the data recorded by the detectors on the testbed; and Section 6.6 summarizes this chapter.

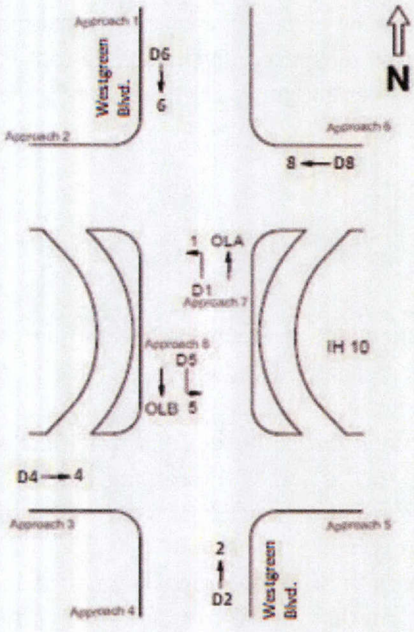
6.2. PROPERTIES OF INTERSECTIONS

6.2.1 Phase Settings

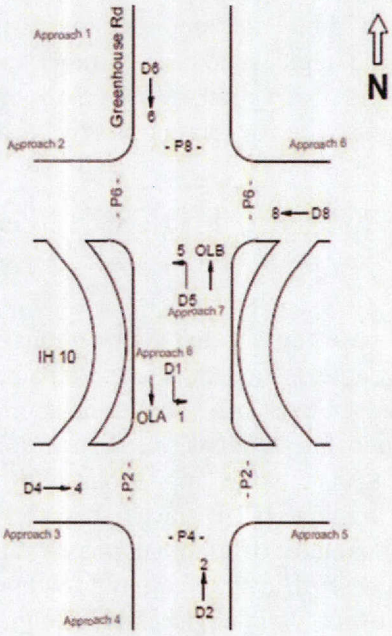
A diamond intersection is more complicated than a normal four-way intersection. The formal can be regarded as a combination of two intersections: the arterial road intersects the frontage road twice when passing through underneath a freeway. Figure 6.2.1 shows the approaches and phases associated with each intersection. Especially, for diamond intersections, because it combines two intersections together, an important feature about phase settings is the overlapping phase for the traffic across the frontage roads on both sides of a freeway. As shown in Figure 6.2.1, at each diamond intersection, there are two overlapping phases: OLA and OLB. Phases 2 and 6 have overlapping phases A and B, respectively.

For diamond intersections, the traffic on the arterial road (across the frontage road) is regarded as the major road, though its traffic flow may not be necessarily higher than that on the frontage road of IH-10. This is because the traffic on these arterial roads suffers more serious congestions.

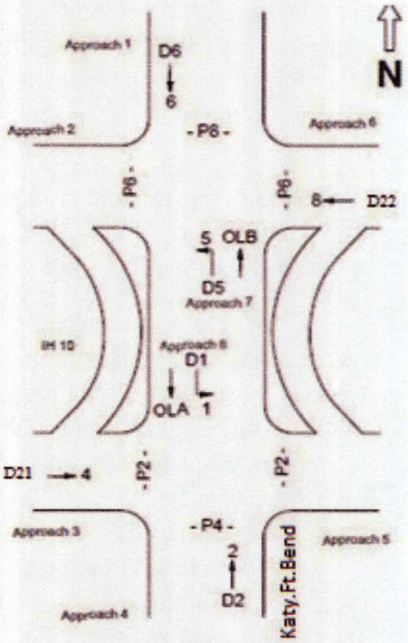
Also, for three diamond intersections at Westgreen Blvd., Greenhouse Rd. and Katy Fort Bend, which have loop detectors. There are two sets of loop detectors on the approach of arterials roads: one set is located about 250 ft upstream of the stop bar (when the left-turn bay starts), and another is located near the stop bar (around 20 ft away from the stop bar). However, at the frontage road, there is only one set of loop detectors around 100 ft away from the stop bar. For the other three diamond intersections at Mason Rd., Fry Rd. and Barker Cypress Rd., as mentioned in Section 6.1, only video detectors are available. While for two four-way intersections, both loop detectors and video detectors are available.



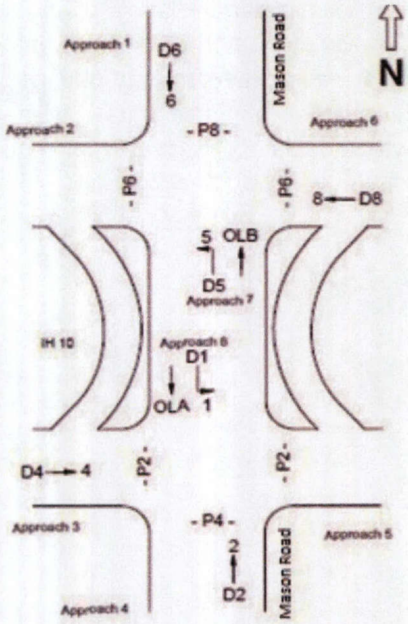
(a) Westgreen Blvd.



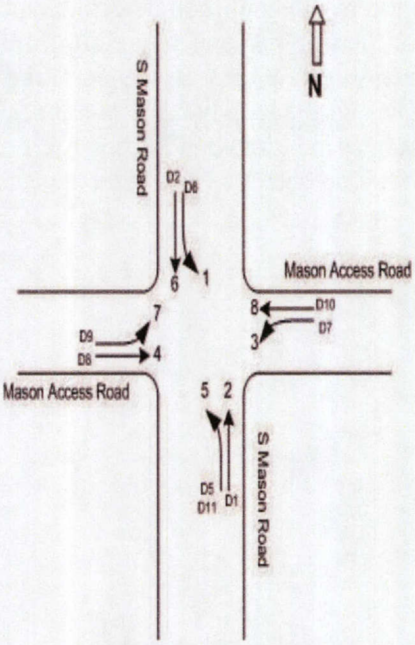
(b) Greenhouse Rd.



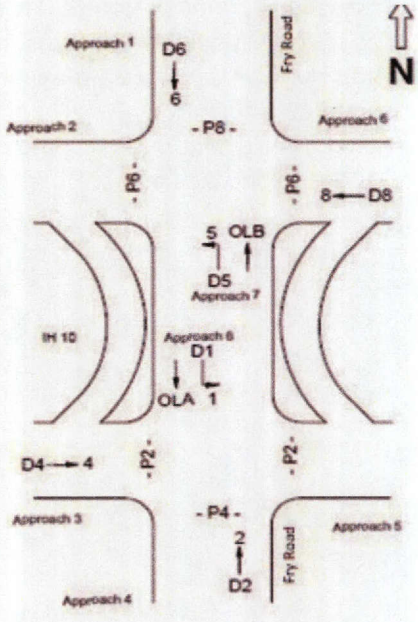
(c) Katy Fort Bend



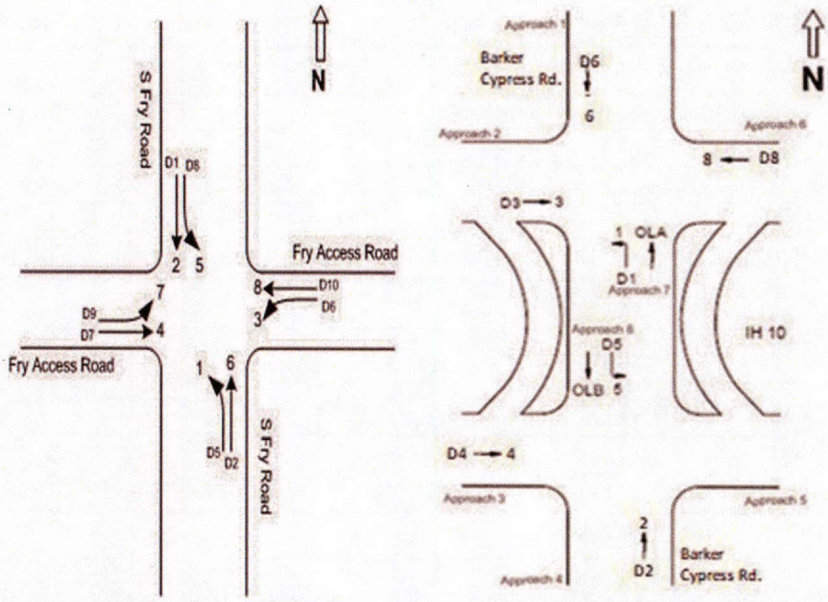
(d) Mason Rd.



(e) Mason Access Rd.



(f) Fry Rd.



(g) Fry Access Rd. (h) Barker Cypress Rd.

***Figure 6.2.1** Specification of approaches and phases associated with eight intersections.*

Note: "D" in the figure refers to the loop detectors. Just for the purpose of demonstration, not all detectors are shown. OLB is the overlapping phase. The number associated with the end of an arrow refers to the phase number.

6.2.2 Signal Timing setting (MM 2-1)

Steve Chiu from the Houston District Office of the Receiving Agency provided the original signal timing plans of these eight intersections to the research team of the Performing Agency for the purpose of analyzing and conducting the simulation.

Since a diamond intersection usually has more phases than those ordinary four-way intersection, especially the former has some overlapping phases, for brevity, in this section, only the parameters of major phases are reported. For example, as shown in Figure 6.2.1, Phases 2 and 6 refer to the north- and southbound through traffic along the arterial road, and Phases 4 and 8 refer to the west- and eastbound through traffic along the frontage road.

Tables 6.2.1 through 6.2.8 show the major parameters (for several major phases) in the ASC/3 Controller for six diamond intersections and two four-way intersections, respectively.

Table 6.2.1 Signal timing parameters for the intersection at Westgreen Blvd.

Phase	**2**	**4**	**6**	**8**
Minimum green time	7	5	7	5
Maximum green time	35	55	55	55
Vehicle extension time	2.0	2.2	2.0	2.0

Table 6.2.2 Signal timing parameters for the intersection at Greenhouse Rd.

Phase	**2**	**4**	**6**	**8**
Minimum green time	7	5	7	5
Maximum green time	35	35	35	50
Vehicle extension time	2.0	2.0	2.0	2.0

Table 6.2.3 Signal timing parameters for the intersection at Katy Fort Bend

Phase	2	4	6	8
Minimum green time	5	8	5	8
Maximum green time	50	50	50	50
Vehicle extension time	2.0	1.5	2.0	1.5

Table 6.2.4 Signal timing parameters for the intersection at Mason Rd.

Phase	2	4	6	8
Minimum green time	10	7	10	10
Maximum green time	45	50	50	80
Vehicle extension time	1.7	1.7	1.7	1.7

Table 6.2.5 Signal timing parameters for the intersection at Fry Rd.

Phase	2	4	6	8
Minimum green time	15	5	15	5
Maximum green time	75	35	75	35
Vehicle extension time	2.0	2.0	2.0	2.0

Table 6.2.6 Signal timing parameters for the intersection at Barker Cypress Rd.

Phase	2	4	6	8
Minimum green time	13	7	12	10
Maximum green time	45	60	50	80
Vehicle extension time	1.7	1.3	1.7	1.3

Table 6.2.7 Signal timing parameters for the intersection of the intersection at Mason Access Rd.

Phase	1	2	3	4	5	6	7	8
Minimum green time	7	10	7	7	7	10	7	7
Maximum green time	35	80	35	45	40	80	35	45
Vehicle extension time	2.0	2.5	1.8	1.8	1.7	2.5	1.8	1.8

Table 6.2.8 Signal timing parameters for the intersection of the intersection at Fry Access Rd.

Phase	1	2	3	4	5	6	7	8
Minimum green time	7	10	7	7	7	10	7	7
Maximum green time	35	80	35	45	40	80	35	45
Vehicle extension time	2.0	4.0	2.0	2.0	2.0	4.0	2.0	2.0

Note: Phases 2 and 6 refer to the south and northbound traffic on arterial roads, and Phases 4 and 8 are west and eastbound traffic on frontage roads.

6.3. DEVELOPMENT OF LOGICAL STATEMENTS

6.3.1 General Logical Statements

Originally, these diamond intersections were operated under a fixed time model without any detectors. The loop detectors were installed in the summer of 2018 at three diamond intersections at Westgreen Blvd., Greenhouse Rd., and Katy Fort Bend.

As mentioned in Section 6.2.1, at the intersections of Westgreen Blvd., Greenhouse Rd., and Katy Fort Bend on the arterial road (the one across the freeway underneath the bridge), there are two sets of sensors: one set is located near the stop bar (called stop-bar detector); and another is about 250 feet away from the stop-bar (upstream), and these detectors are also called vehicle detector. On the other hand, as to the frontage road, only stop-bar detectors (but not exactly located at the stop-bar, but about 100 ft away) are installed. For this reason, the proposed signal control focuses on the traffic on the arterial roads (the data from two types of detectors are both used).

Also, as noted before, no loop detectors are available at the diamond intersections of Barker Cypress Rd., Mason Rd. and Fry Rd. They only have video detectors (i.e., cameras) in each of the four phases along arterial and frontage roads.

Finally, at the four-way intersections at Mason Access Road and Fry Access Rd., respectively, both loop detectors and video detectors are used together.

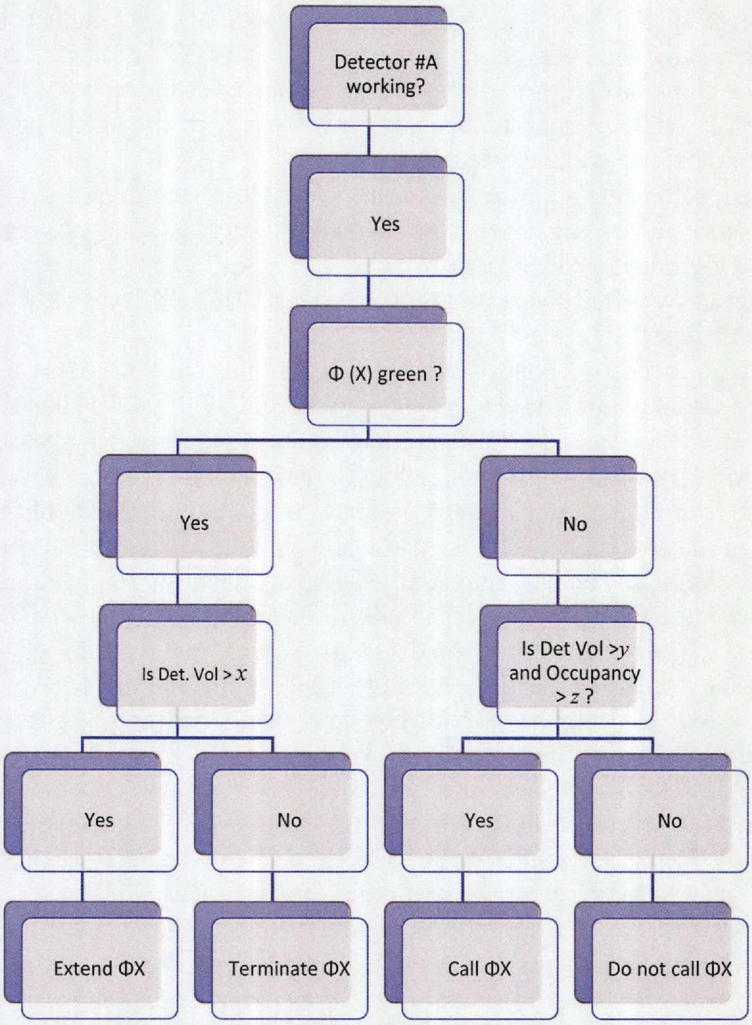
The basic idea of proactive control system is to use detectors to report the traffic volume and occupancy, so that the signal control system can adjust accordingly. It is like such a case: detectors find a platoon of vehicles are moving toward the intersection, so the system gives a green light to this platoon to help this vehicle platoon pass the intersection smoothly (without needing to slow down). By helping a platoon of vehicles more smoothly pass through an intersection, the efficiency of the intersection will be increased largely as the headways of vehicles within a vehicle platoon are smaller than other not in a platoon, implying more vehicles can pass through this intersection.

Since there are three types of detectors in use, three logic statements were developed because the feature of video detectors is different from loop detectors. The general logic of proactive signal control is already described in Chapters 3-5 on the testbeds of FM 528, SH 242 and FM 1464, respectively. However, since a diamond intersection basically has two intersections for arterial roads, the research team revised the original design. Figure 6.3.1 (a-c) illustrate the flowchart of the proactive signal control logic used in diamond intersections (with loop detectors and video detectors, respectively) and four-way intersections, respectively.

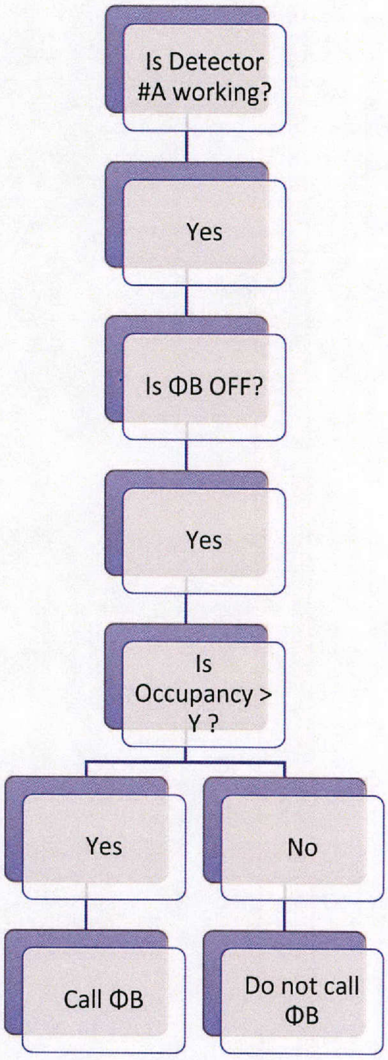
Note that video detectors cannot accurately capture the volume—actually, usually they will underestimate the volume. For this reason, the data of occupancy is used in the logic design: as shown in Figure 6.3.1 (b), this logic is different from the logic implemented to the intersections with loop detectors, as shown in Figure 6.3.1(a), which employs the values of both volumes and occupancies reported from detectors. On the other hand, for four-way intersections, both volume and occupancy were used in the logic because at these intersections, loop detectors and video detectors are used together.

As shown in Figure 6.3.1 (a), the logic for the intersections at Westgreen Blvd., Greenhouse Rd. and Katy Fort Bend (where loop detectors were used), respectively, the phase is either extended or called, if the traffic volume is large when the phase is on, or when the phase is off and the occupancy is found high. On the other hand, as shown in Figure 6.3.1 (b) for the intersections at Mason Road, Fry Road and Barker Cypress Rd. (where only video detectors were used), respectively, *only occupancy was used* to determine if a call for a certain phase is needed or not. Finally, as shown in Figure 6.3.1 (c) for the four-way intersections at Mason Access Rd. and Fry Access Rd., respectively, both volume and occupancies are used to determine whether to call phase or not.

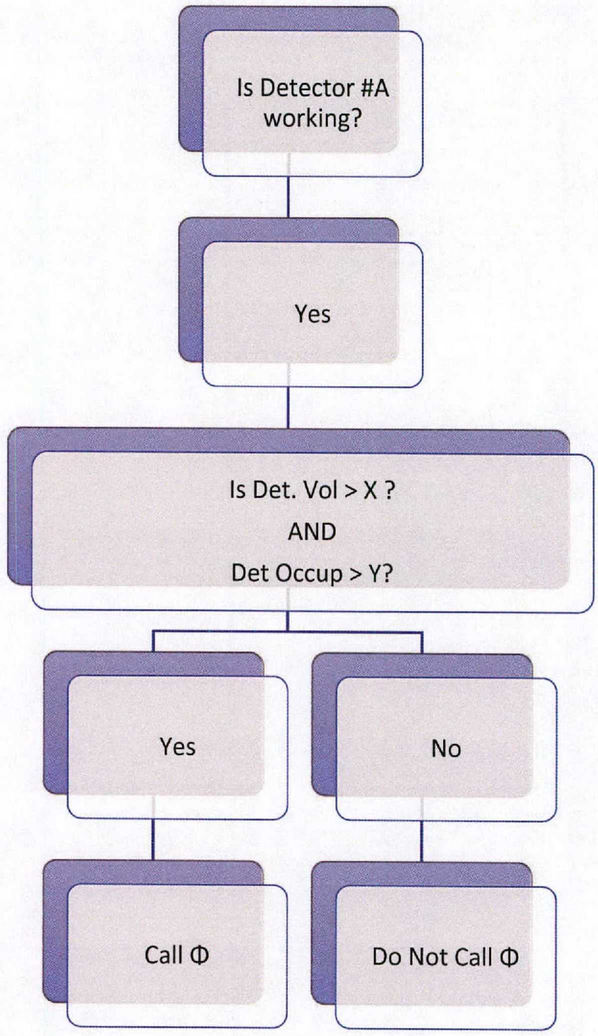
In these three types of control model, it is needed to determine two parameters: x and y, which are the control thresholds. In the previous implementations on FM 528, SH 242 and FM 1464, these thresholds were determined by running the simulations in VISSIM. For the eight intersections in this chapter, however, due to the delay of the data collection from loop detectors (no data were available until the middle of November 2018 and the data for two intersections were not available until January 2019), the parameters are based on field test observations (the simulations are still in preparation) and the experience the research team had in the previous implementations. These values are reported in Tables 6.3.1 through 6.3.6.



(a) For diamond intersections at Westgreen Blvd, Greenhouse Rd. and Katy Fort Bend (with loop detectors)



(b) For diamond intersections at Mason Rd., Fry Rd. and Barker Cypress Rd. (with video detectors only)



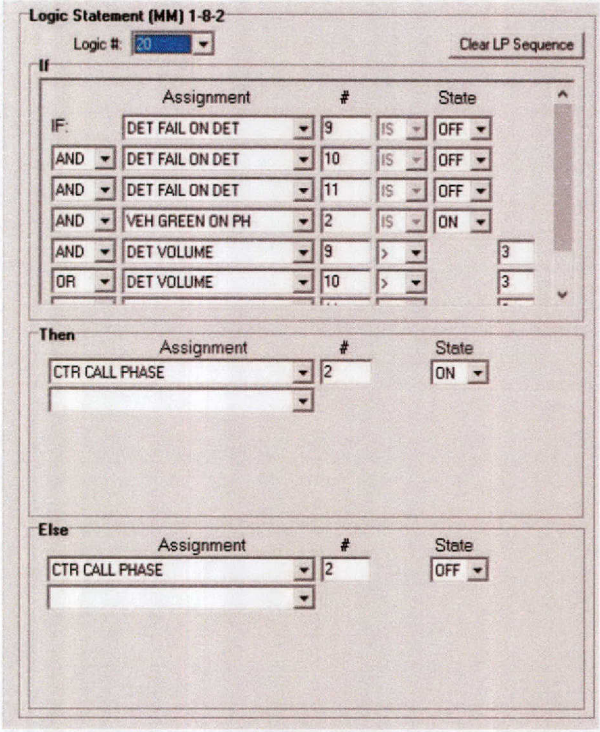
(c) For four-way intersections at Mason Access Rd. and Fry Access Rd. (with both loop detectors and video detectors)

Figure 6.3.1 *Three logic statements of proactive signal control systems implemented at eight intersections*

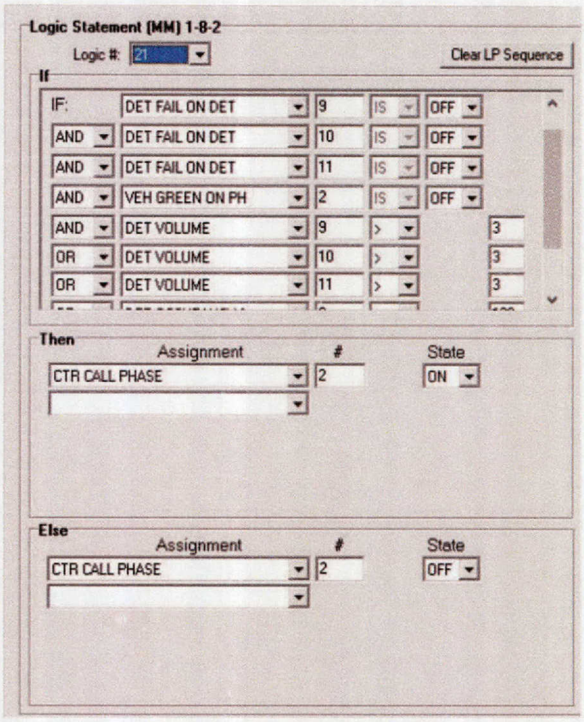
6.3.2 Logic Statements in ASC/3 Econolite Controller

The logics described in Figures 6.3.1 were imported into the ASC/3 Econolite Controller at the three sets of intersections, respectively, for the purpose of implementation.

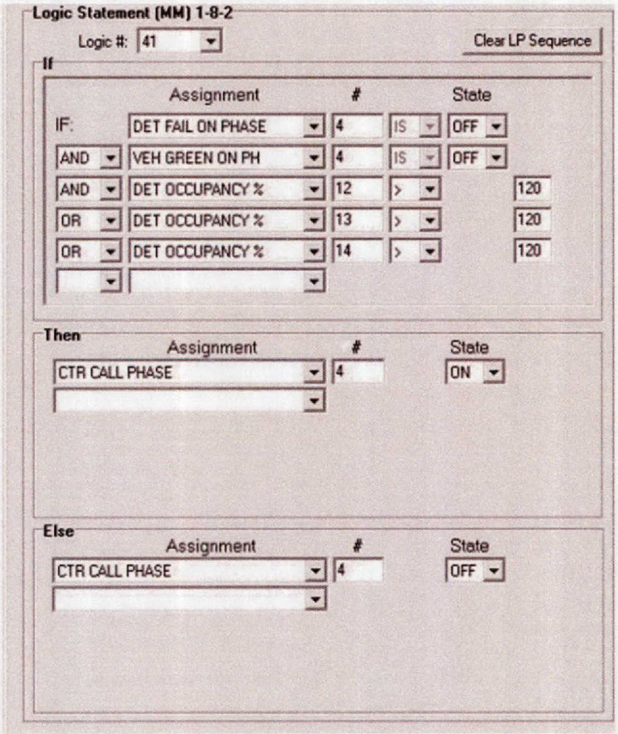
Figures 6.3.2 through 6.3.4 give the examples of the logic statement setting in the ASC/3 Econolite controller. And as mentioned above, Tables 6.31 through 6.3.6 summarize the values of the parameters used in the logic plan at each intersection, respectively.



(a) Arterial road

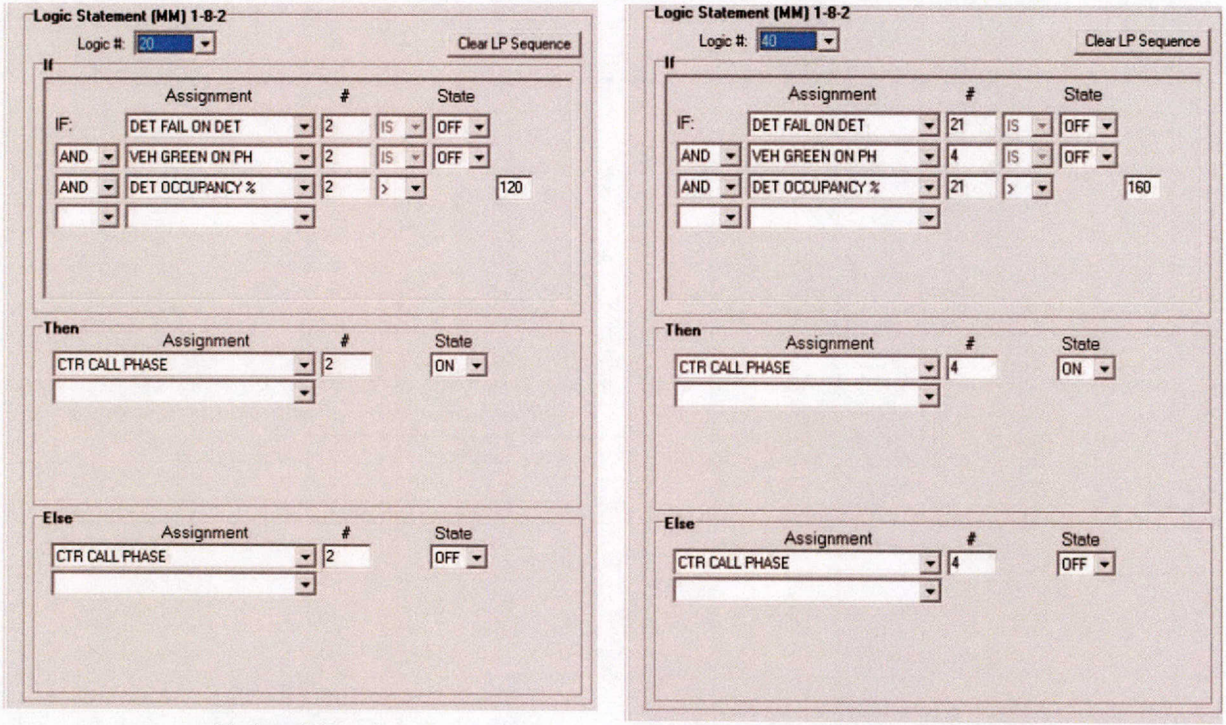


(b) Arterial Road



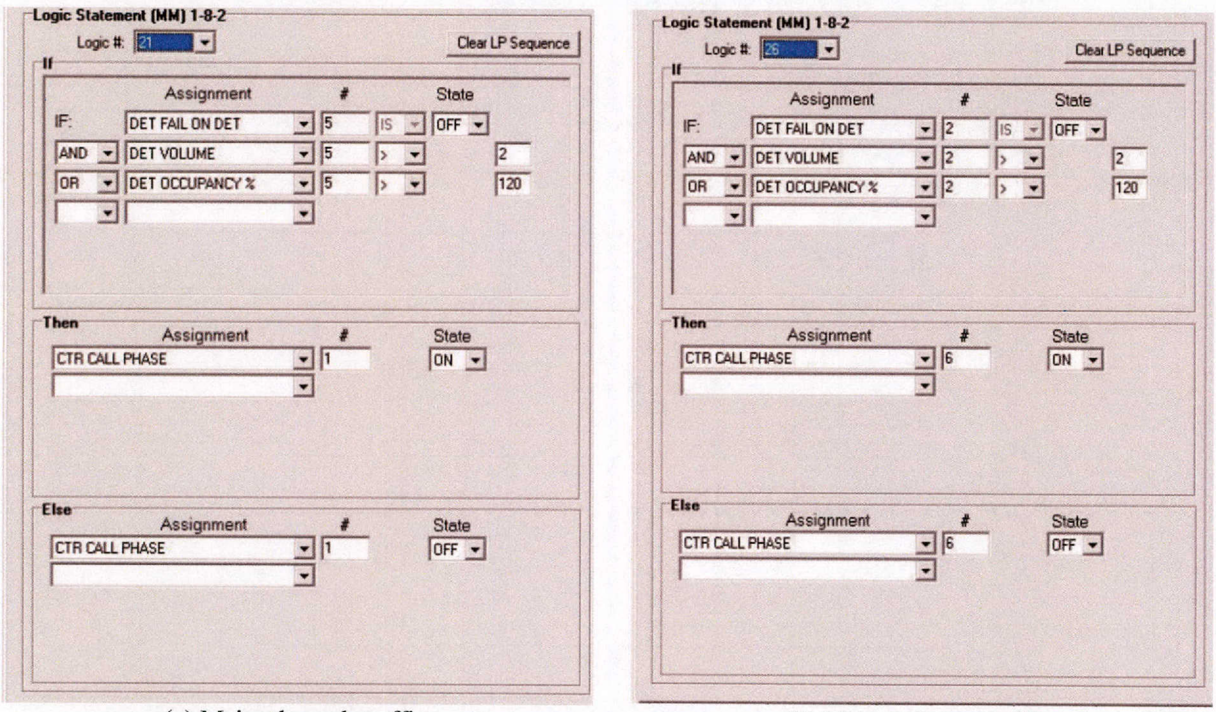
(c) Frontage Road

Figure 6.3.2 *Logic statements for the intersections with loop detectors*



(a) Arterial road (b) Frontage road

Figure 6.3.3 *Logic Statements for the intersections with video detectors*



(a) Major through traffic (b) Major left-turn traffic

Figure 6.3.4 *Logic Statements for the four-way intersection with both video and loop detectors.*

Table 6.3.1 Selected occupancy and volume thresholds used in the logic statement for the intersection at Westgreen Blvd.

Phase Number	Detector	Occupancy (%)	Volume
2	9 (VD)	-	5
	10 (VD)	-	5
4	11 (SD)	60	3
	12 (SD)	60	3
	13 (SD)	60	3
6	14 (VD)	-	5
	15 (VD)	-	5
8	16 (SD)	60	3
	17 (SD)	60	3
	18 (SD)	60	3

Note: SD: stop-bar detector, VD: vehicle detector, Vid.: Video Detector

Table 6.3.2 Selected occupancy and volume thresholds used in the logic statement for the intersection at Greenhouse Rd.

Phase Number	Detector	Occupancy (%)	Volume
2	9 (VD)	60	3
	10 (VD)	-	5
4	11 (SD)	60	3
	12 (SD)	60	3
	13 (SD)	60	3
6	14 (VD)	60	3
	15 (VD)	-	5
	16 (VD)	-	5
8	17 (SD)	60	3
	18 (SD)	60	3

Table 6.3.3 Selected occupancy and volume thresholds used in the logic statement for the intersection at Katy Fort Bend

Phase Number	Detector	Occupancy (%)	Volume
2	9 (VD)	60	3
	10 (VD)	60	3
	11 (VD)	60	3
4	12 (SD)	60	3
	13 (SD)	60	3
	14 (SD)	60	3
6	15 (VD)	60	3
	16 (VD)	60	3
	17 (VD)	60	3
8	18 (SD)	60	3
	19 (SD)	60	3
	20 (SD)	60	3

Table 6.3.4 Selected occupancy and volume thresholds used in the logic statement for the intersections at Barker Cypress Rd., Mason Rd. and Fry Rd.

Phase Number	Detector	Occupancy (%)	Volume
2	2 (Vid.)	60	-
4	21 (Vid.)	80	-
6	6 (Vid.)	60	-
8	22 (Vid.)	80	-

Table 6.3.5 Selected occupancy and volume thresholds used in the logic statement for the intersection at Mason Access Rd.

Phase Number	Detector	Occupancy (%)	Volume
1	6 (SD)	60	2
2	1 (Vid.)	60	2
3	7 (SD)	80	3
4	8 (SD)	80	3
5	5 (SD)	60	3
	11 (SD)	60	3
6	2 (Vid.)	60	2
7	9 (SD)	80	3
8	4 (SD)	80	3

Table 6.3.6 Selected occupancy and volume thresholds used in the logic statement of the intersection at Fry Access Rd.

Phase Number	Detector	Occupancy (%)	Volume
1	5 (SD)	60	2
2	1 (Vid.)	60	2
3	6 (SD)	80	3
4	7 (SD)	80	3
5	8 (SD)	60	3
6	2 (Vid.)	60	2
7	9 (SD)	80	3
8	10 (SD)	80	3

6.4. ANALYSIS OF FIELD OBSERVATION DATA

As shown in Table 6.1.1, on November 2, 2018 (Friday), the Performing Agency worked with the Houston Office of the Receiving Agency to implement the proactive signal control algorithm to two intersections: Westgreen Blvd. and Greenhouse Rd.; on November 14 (Wednesday), the implementation was conducted to the intersection of Katy Fort Bend; and on December 11 (Tuesday) and 17 (Monday), the implementation was conducted to the remaining five intersections.

During these days, before the implementation, the research team observed the traffic during the morning peak-hours (7:30 am–9:00 am) at the intersections of Westgreen Blvd. and Greenhouse Rd. on November 2 and November 14, respectively; and observed the traffic at the intersections of Katy Ford Bend and Mason Rd., respectively in the morning on December 11 (at this intersection, the implemented proactive system was turn off in the morning of 12/7, and then restored in the morning of 12/14) and December 17, respectively. As to the others four intersections, the observations was made in the afternoon (2:00 pm–4:00 pm) on December 11 (under the original plan) and 17 (when the proactive plan was implemented).

By taking photos and videos at each approach for five cycles, as well as driving through the intersections for five cycles, the research team recorded the queue lengths and delay for the major phases at each approach at two intersections, respectively. It should be noted that both queue length and delay were recorded manually, so the values of queue length are more accurate because they were counted directly from the photos. On the other hand, the human error in recording the delay is large, because it is difficult to find the vehicle who suffered from the longest delay directly from the observations, especially if the congestion is heavy.

Note that on December 11, there were some road works near the intersections of Mason Rd. and Mason Access Rd., causing serious congestion on Mason Rd. Also, the research team found that the traffic flow on Katy Fort Bend (i.e., the arterial road) after 8 am is quite small (no apparent vehicle platoons were observed, and the vehicles arrivals seems to be random) on both days (12/11 and 12/17), so the comparison seems meaningless. As to the intersections of Fry Rd. and Fry Access Rd., the recorded data have errors. For these reasons, this section only reports the data observed at three intersections: Westgreen Blvd., Greenhouse Rd., and Barker Cypress Rd., as shown in Tables 6.4.1 through 6.4.3.

Table 6.4.1 Weekday field observation of the intersection at Westgreen Blvd. (7:30 am–8:15 am)

	Traffic flow	**Average queue length (# vehicles)**			**Average delay (seconds)**		
		Original	**Proactive**	**Difference**	**Original**	**Proactive**	**Difference**
Weekdays	Eastbound (Frontage)	11	7	-36.36%	78	80	2.56%
	Westbound (Frontage)	6	6	0.00%	82	75	-8.54%
	Northbound (Arterial)	17	15	-11.76%	92	85	-7.61%
	Southbound (Arterial)	17	12	-29.41%	90	95	5.56%

Note: the traffic condition under original control was observed on November 2, 2018 and those under the proposed proactive signal control was observed on November 14, 2018.

Table 6.4.2 Weekday field observation of the intersection at Greenhouse Rd. (8:30 am–9:00 am)

	Traffic flow	**Average queue length (# vehicles)**			**Average delay (seconds)**		
		Original	**Proactive**	**Difference**	**Original**	**Proactive**	**Difference**
Weekdays	Eastbound (Frontage)	8	9	12.50%	92	75	-18.48%
	Westbound (Frontage)	10	6	-40.00%	77	92	19.48%
	Northbound (Arterial)	5.5	8	45.45%	76	70	-7.89%
	Southbound (Arterial)	17	15	-11.76%	65	70	7.69%

Note: the traffic condition under original control was observed on November 1, 2018 and those under the proposed proactive signal control was observed on November 14, 2018.

Table 6.4.3 Weekday field observation of the intersection at Barker Cypress (3:00 pm–3:10 pm)

	Traffic flow	**Average queue length (# vehicles)**			**Average delay (seconds)**		
		Original	**Proactive**	**Difference**	**Original**	**Proactive**	**Difference**
Weekdays	Eastbound (Frontage)	2	4	100.00%	74	70	-5.41%
	Westbound (Frontage)	7	5	-28.57%	77	80	3.90%
	Northbound (Arterial)	1	3	200.00%	75	70	-6.67%
	Southbound (Arterial)	8	6	-25.00%	87	67	-22.99%

Note: the traffic condition under original control was observed on December 11, 2018 and those under the proposed proactive signal control was observed on December 17, 2018.

The observations between 7:30 am and 8:15 am on November 2 and 14 of the intersection at Westgreen Blvd. found that the southbound and northbound traffic on the arterial road (Westgreen Blvd.) suffered more serious congestions (many vehicles had to wait for two cycles) than the eastbound and westbound traffic on the frontage road. Note that at this intersection for through traffic, the arterial road has only two lanes but the frontage road on each side of IH-10 has three lanes. However, as reported in Table 6.4.1, the number of vehicles waiting in the queue on the arterial road is much more than those on the frontage road.

As reported in Tables 6.4.1, it seems that the proposed proactive signal control system did help reduce the delay and queue length significantly (12% and 29%, respectively) for the northbound traffic on Westgreen Blvd., and also did help reduce the queue length for the southbound traffic on Westgreen Blvd. But the delay for the southbound traffic increased a little bit (about 6%) on average. Note that as mentioned above, the accuracy of delays is not high, compared with the queue length. Therefore, generally we can say the proposed proactive signal control system helped improve the traffic on the arterial road which suffers the serious congestion in the morning peak hours. On the other hand, the changes of flow patterns on the frontage roads were found not obvious, implying the effectiveness of the proposed proactive signal control system is not apparent. This is reasonable: (1) the logic plan focuses on the traffic flow on the arterial road as there are two sets of loop detectors; and (2) the traffic flow on the frontage road experiences no congestion (especially, the vehicles platoons are not obvious), so the potential room for improvement is limit (similar patterns found at the testbed of FM 528, and five intersections at the testbed of FM 1464).

As to the intersections at Greenhouse Rd., the field observations from 8:30 am to 9:00 am on November 2 and 14, found that the traffic on the arterial road (Greenhouse Rd.) saw more serious congestion than that on the frontage road. However, such congestion on Greenhouse Rd. was not as serious as that observed on Westgreen Rd. It may be due to the observation time: the peak is over after 8:30 am. From Table 4.6.2, it seems that the proposed proactive control plan did help reduce the queue length for the southbound traffic—the reduction is as high as about 12%. However, for the northbound traffic, the queue length increased largely instead. Note that the average numbers of vehicles in the queue in this direction are both small before and after implementation, just 5.5 and 8, implying the northbound traffic during the observation time is not congested at all. Therefore, such numbers of vehicles are more likely to be random, not reflect the effectiveness of the proactive signal control. On the other hand, similarly, the traffic flows on the frontage roads were found small, implying no congestion during the observation time (vehicles seemed to arrive randomly). Note that as the same to the previous intersection, at this intersection., the frontage road has three lanes while the arterial road has only two lanes for through traffic. Since vehicles seemed to arrive randomly at these two directions on the frontage roads, the comparison cannot reflect the effectiveness of the proposed signal control system.

Finally, of the intersection at Barker Cypress Rd., as reported in Table 6.4.3, apparently, it seems that vehicles arrived randomly because the numbers of vehicles in the queue were all small in each direction at this intersection. The comparison cannot reflect the effectiveness of the proposed proactive signal control system.

Note that the onsite observation data may be not accurate, especially the delay data. Also, the onsite observations only reflect one-day's pattern, which may have bias due to various reasons. In the following, the data collected by loop detectors during multiple days will be used to evaluate the performance of the proposed proactive signal control system. These data have much higher accuracy compared with the onsite observation data. Also, since the multi-day average will be used for the comparison, the bias can be reduced.

6.5. ANALYSIS OF FIELD DATA FROM DETECTORS

As did before for the testbeds of FM 528, SH 242, and FM 1464, the data collected by loop detectors during multiple days before and after the implementation were used to evaluate the performance of the proactive signal control systems implemented to the selected eight intersections. Table 6.5.1 reports the time periods when the detector data were collected under the original and proactive control plans, respectively.

Due to the delay of implementation, the time periods for the data collection are short. Especially, the data reporting problems at the intersections at Fry Rd. and Barker Cypress Rd. were not solved until the early January 2019. Due to the short time periods, especially at the intersections at Westgreen Blvd. and Greenhouse Rd., the analysis only focuses on the traffic on weekdays. Loop detectors at these intersections can only report two traffic parameters: volume and occupancy. They are reported every 1 minute. As did before, the research team calculated the 15-minute total volumes and 15-minute average occupancy starting from 00:00:00. That is, if the data are available for the whole day, there are 96 sets of 15-minute flow and occupancy. Then, the multi-day average 15-minute flow and occupancy were calculated for the purpose of comparison.

Table 6.5.1 Time periods when the data were collected under two control plans

Intersection	Under Original Plan	Under Proactive Plan
Westgreen Blvd.	10/29/2018 – 11/01/2018	11/12/2018 – 11/19/2018
Greenhouse Rd.	10/29/2018 – 11/01/2018	11/12/2018 – 11/19/2018
Katy Ford Bend	11/12/2018 – 11/19/2018, 12/07/2018 – 12/14/2018	11/24/2018 – 11/30/2018 12/31/2018 – 01/11/2019
Mason Rd.	12/07/2018 – 12/14/2018	12/31/2018 – 01/11/2019
Mason Access Rd.	12/07/2018 – 12/14/2018	12/31/2018 – 01/11/2019
Fry Access Rd.	01/11/2019 – 01/23/2019	12/31/2018 – 01/11/2019
Barker Cypress Rd.	01/11/2019 – 01/23/2019	12/31/2018 – 01/11/2019

6.5.1 Comparison of Data Collected at Diamond Intersections with Loop Detectors

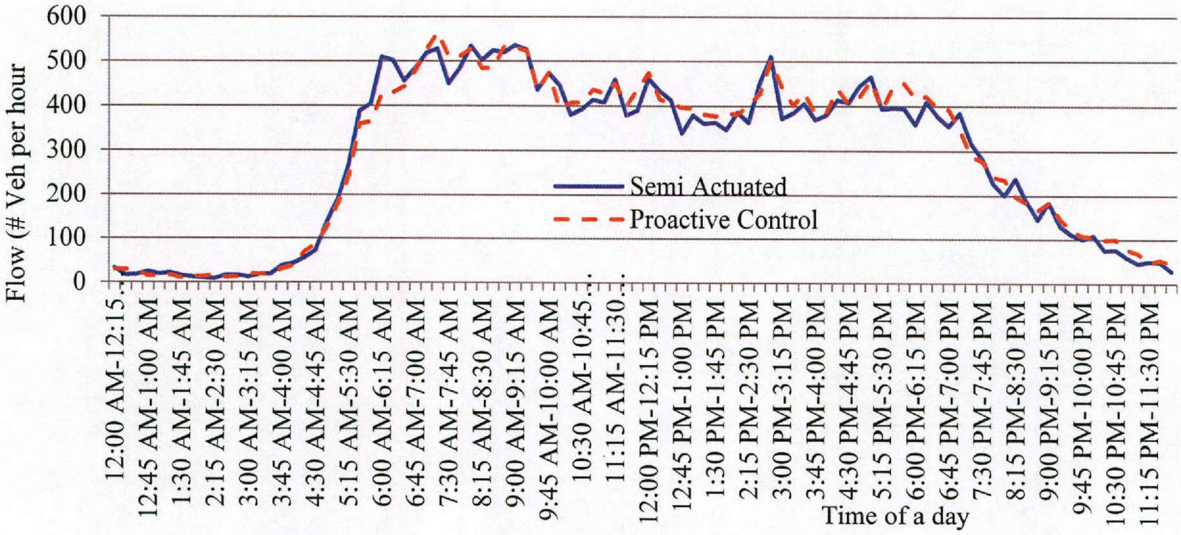
First, the performance of the proactive signal control system implemented to three diamond intersections at Westgreen Blvd., Greenhouse Rd. and Kary Fort Bend were analyzed, respectively, based on the data reported by loop detectors. Note that these three intersections have loop detectors, and the logic shown in Figure 6.3.1(a) were applied, in which both volumes and occupancies were used.

As the data covers short time periods, especially for the intersections at Westgreen Blvd. and Greenhouse Rd., just 5 days' data were reported (see Table 6.5.1); the comparison at these intersections only looks at the data collected in weekdays. On the other hand, as mentioned before (as well as reported in Tables 4.6.1 and 4.6.2), the congestion seems to be more serious than that on the arterial roads. These arterial roads have less numbers of lanes for through traffic, and they have equipped two sets of loop detectors. Therefore, the comparison in this section also focuses on the traffic on these arterial roads, especially the through traffic (northbound or southbound).

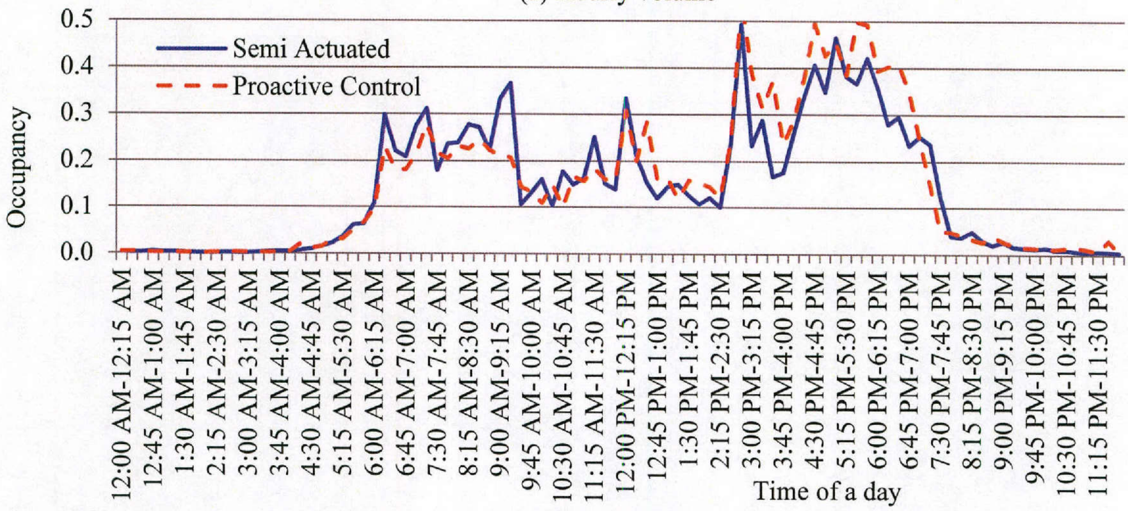
Note in the following figures, *"Semi Actuated"* refers to the original control plan used at these intersections. We use this name just for convenience, no matter the original plan was "actuated" or "pre-timed" mode.

Westgreen Rd. on Weekdays

Figures 6.5.1–6.5.2 report the comparisons of hourly traffic flows and occupancies for the north- and southbound through traffic along Westgreen Blvd., respectively.

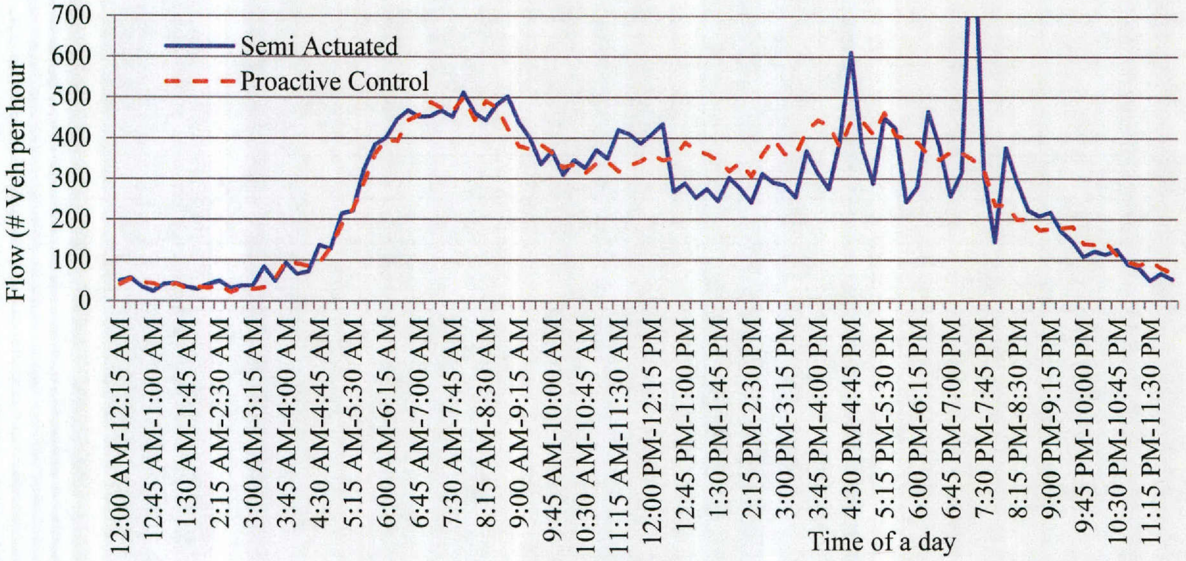


(a) Hourly volume

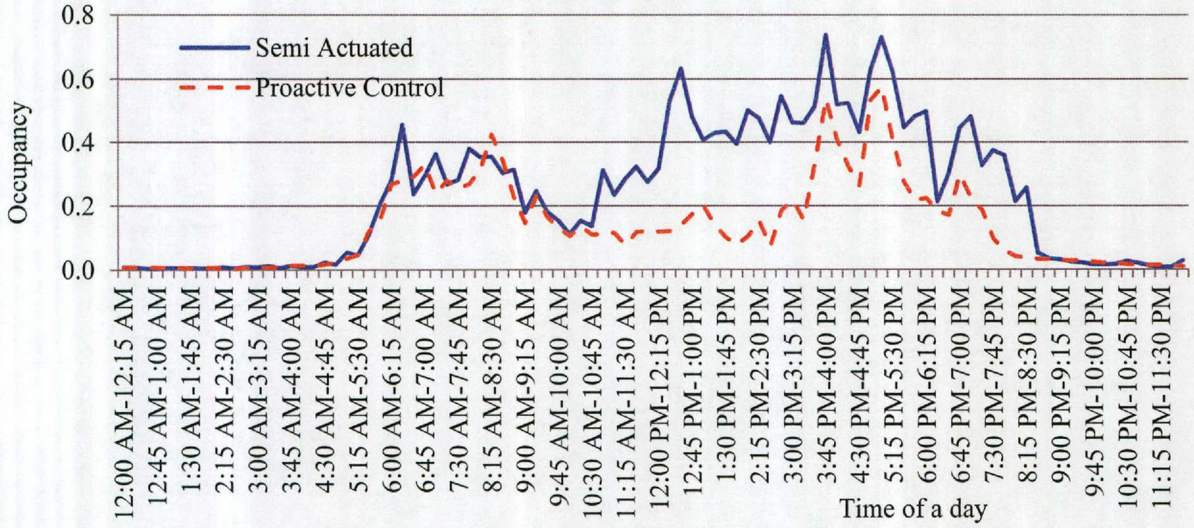


(b) Occupancy

Figure 6.5.1 *Multi-day average northbound traffic of the intersection at Westgreen Blvd. in weekday*



(a) Hourly Volume



(b) Occupancy

Figure 6.5.2 *Multi-day average southbound traffic of the intersection at Westgreen Blvd. in weekday*

It seems that for these two-directions' traffic, after the implementation, the average hourly flow increased; but the average occupancy dropped in most of time, except the evening peak hours for the northbound traffic, which increased a little bit after the implementation. Note during that time, interestingly, the hourly flows were less than those in the morning peak hours, but the approach saw higher occupancies rates before and after the implementation. Note that during the evening peak hours, the flow was not high, just around 400 veh/hr/ln, indicating probably the vehicle platoons were not apparent, so that the impact of the proactive signal control system is not so significant.

Figure 6.5.3 reports the comparison of traffic patterns of the eastbound traffic on the frontage road at this intersection before and after the implementation. It seems that these two patterns are very close. The westbound traffic has the similar pattern, which is not reported here for brevity. The average through traffic on the frontage road seems to be smaller than that on the arterial road, just around 350 veh/hr/ln—such low traffic implies few vehicle platoons. For this reason, as expected, the new logic seems not so effective here.

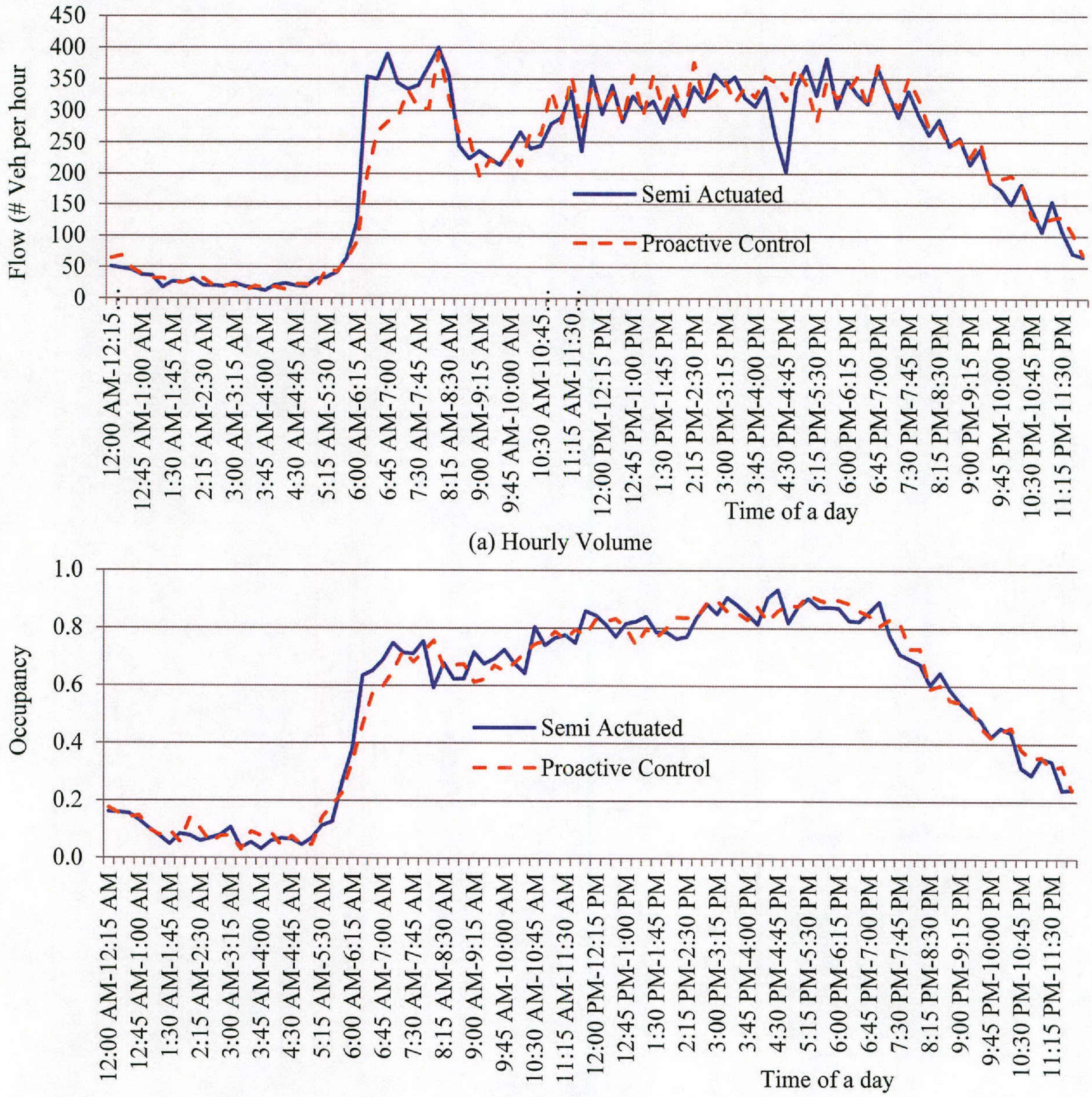
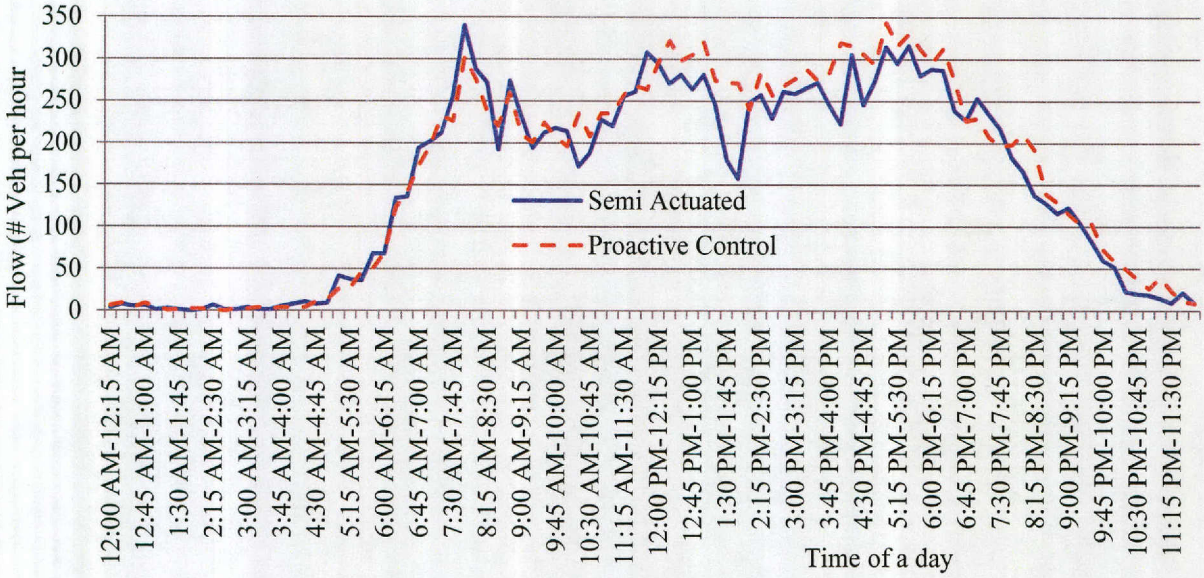


Figure 6.5.3 *Multi-day average eastbound traffic of the intersection at Westgreen Blvd. in weekday*

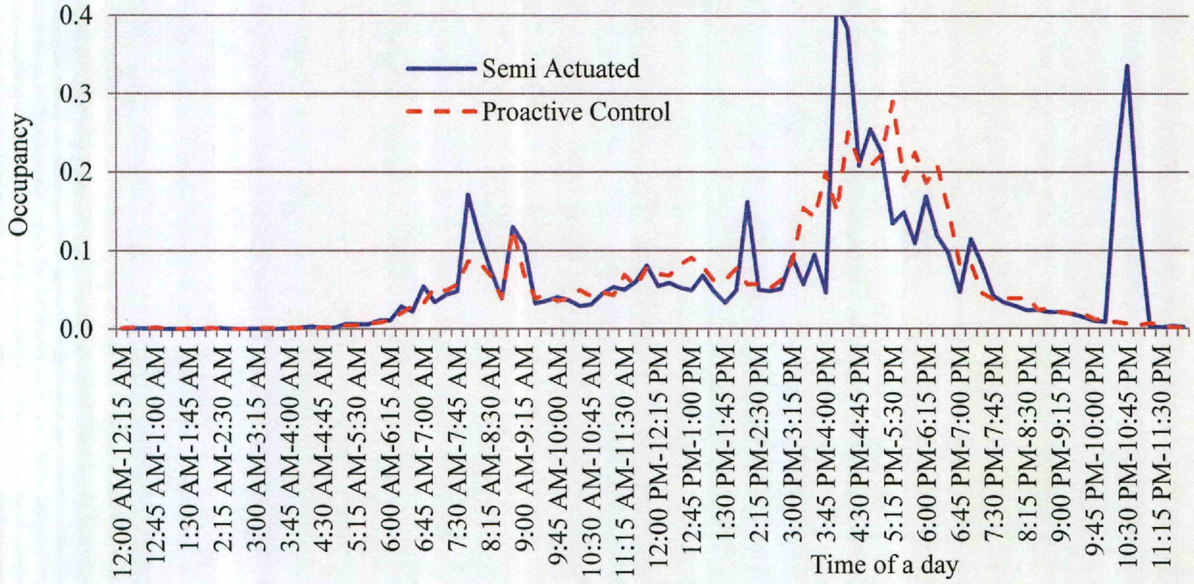
Greenhouse Rd. on Weekdays

Similarly, the north- and southbound traffic patterns on Greenhouse Rd. were analyzed and compared before and after the implementation, as shown in Figures 6.5.4 and 6.5.5, respectively. After the implementation, the average hourly flow increased but the average occupancy rates dropped, especially for the southbound traffic in the morning peak hours, when the flows were as high as 900 veh/hr/ln. At this level, vehicles are formed platoons. Therefore, the impact of the proactive signa control system is clear.

On the other hand, during the evening peak hours, the average flow was just around 350 veh/hr/ln. Such low level indicates that the impact of the proactive signal control system is likely to be insignificant.



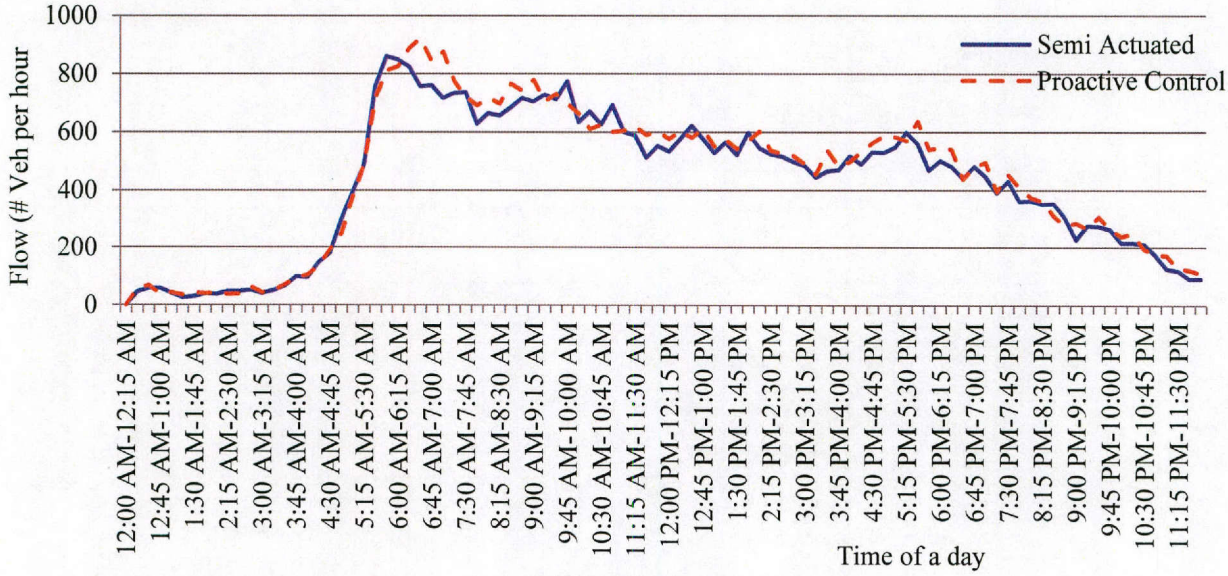
(a) Hourly Volume



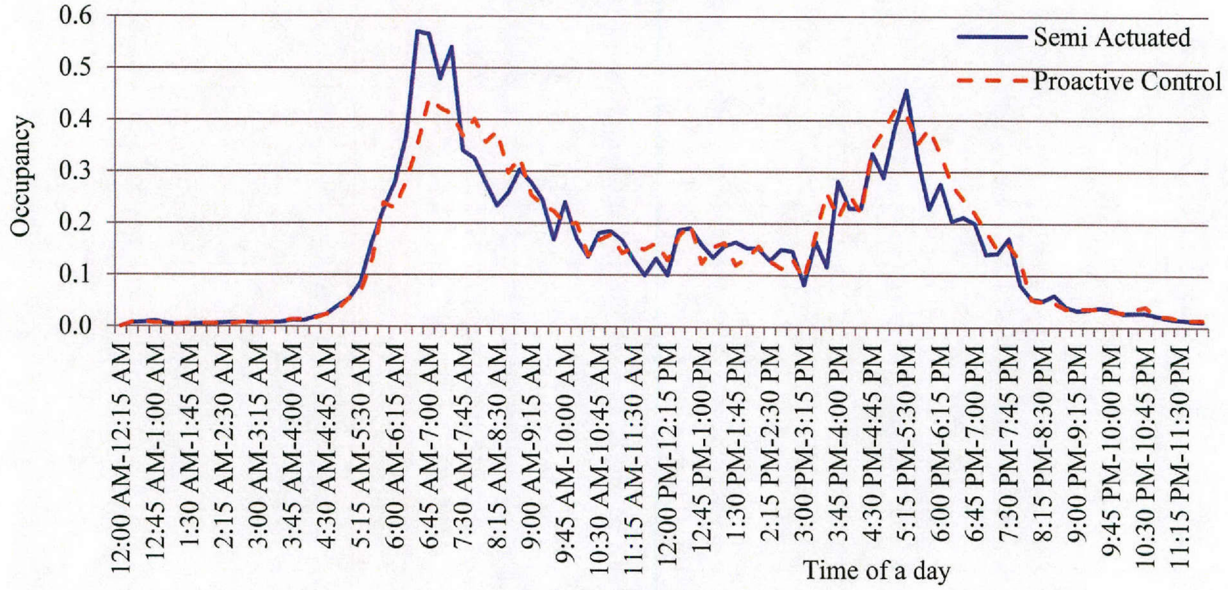
(b) Occupancy

Figure 6.5.4 *Multi-day average northbound traffic of the intersection at Greenhouse Rd. in weekday*

Figure 6.5.6 reports the traffic patterns for the westbound traffic on the frontage road before and after the implementation. Similar to the case of the intersection at Westgreen Blvd., in most cases, the traffic patterns are very close in most cases. Note that it is found that during the evening peak hours, the westbound traffic experienced much higher traffic than the morning hours—around 750 veh/hr/ln on average. During this period, though the flow seems to be very close, the occupancy rates were a little bit smaller after the implementation, indicating that the proposed proactive signal control helped to release more vehicles to pass the intersection. During other time, since the flow was so small (no more than 350 veh/hr/ln), the impact of this proactive signal control system was not so apparent.

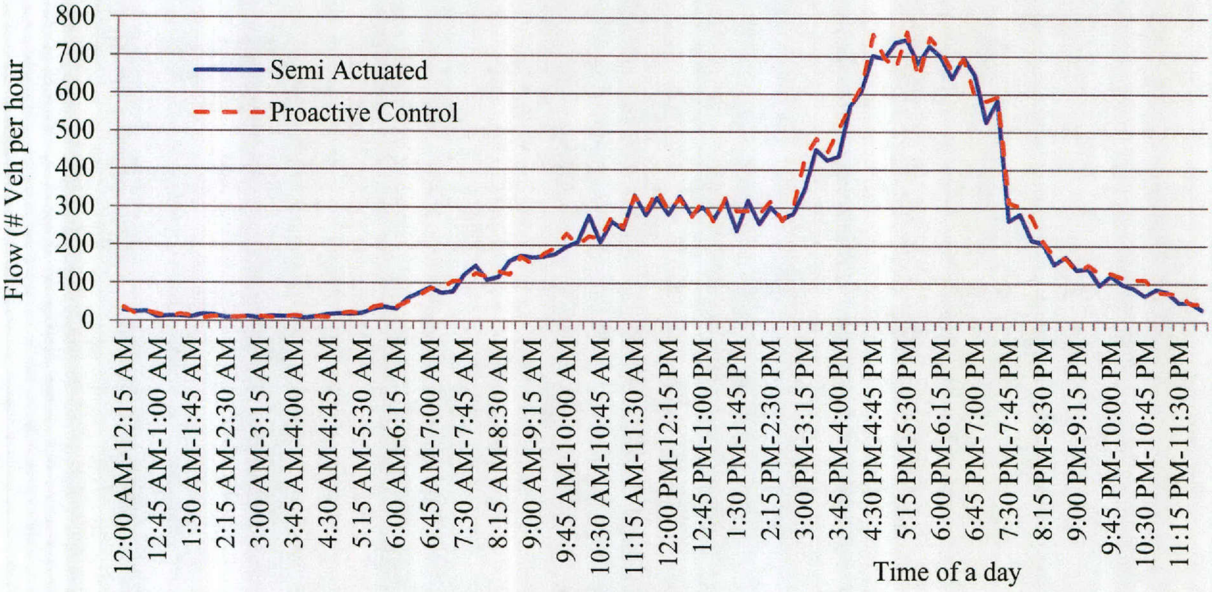


(a) Hourly Volume

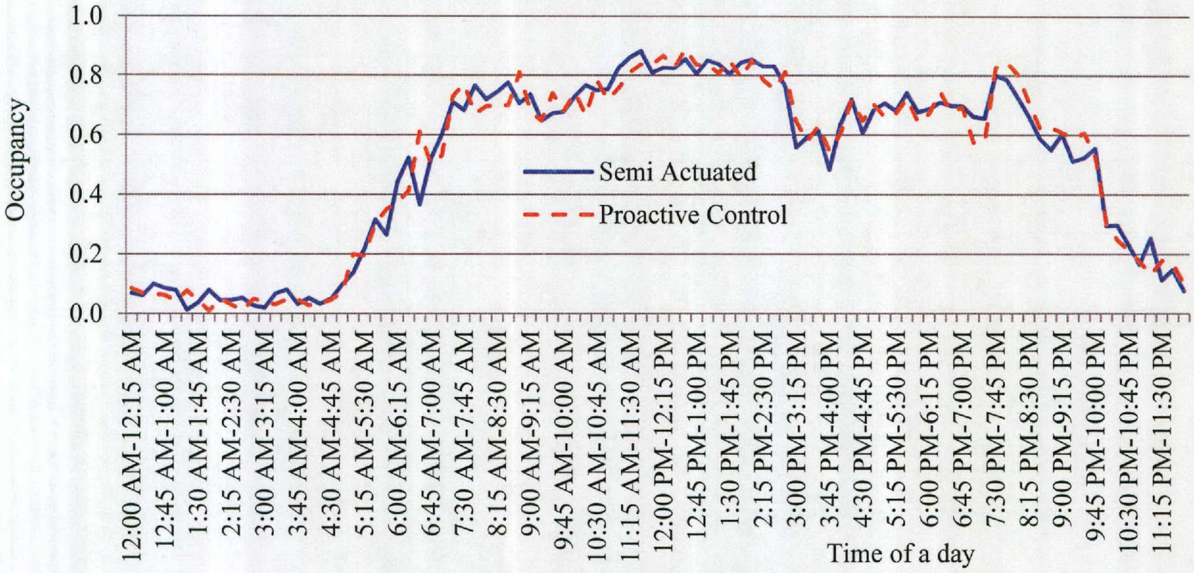


(b) Occupancy

Figure 6.5.5 *Multi-day average southbound traffic of the intersection at Greenhouse Rd. in weekday*



(a) Hourly Volume



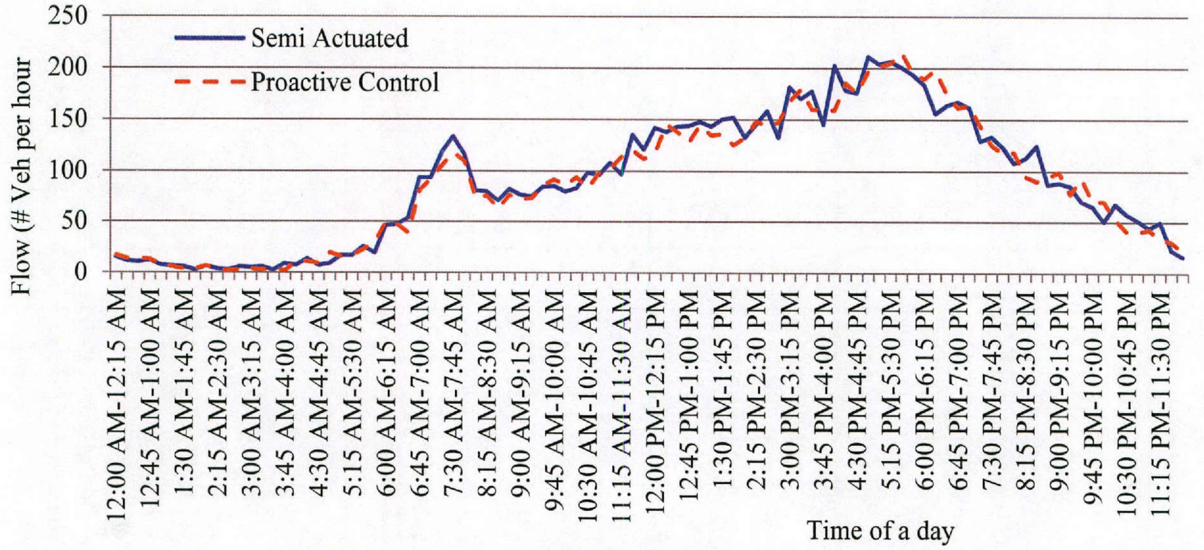
(b) Average Occupancy (in percentage)

Figure 6.5.6 *Multi-day average westbound traffic of the intersection at Greenhouse Rd. in weekday*

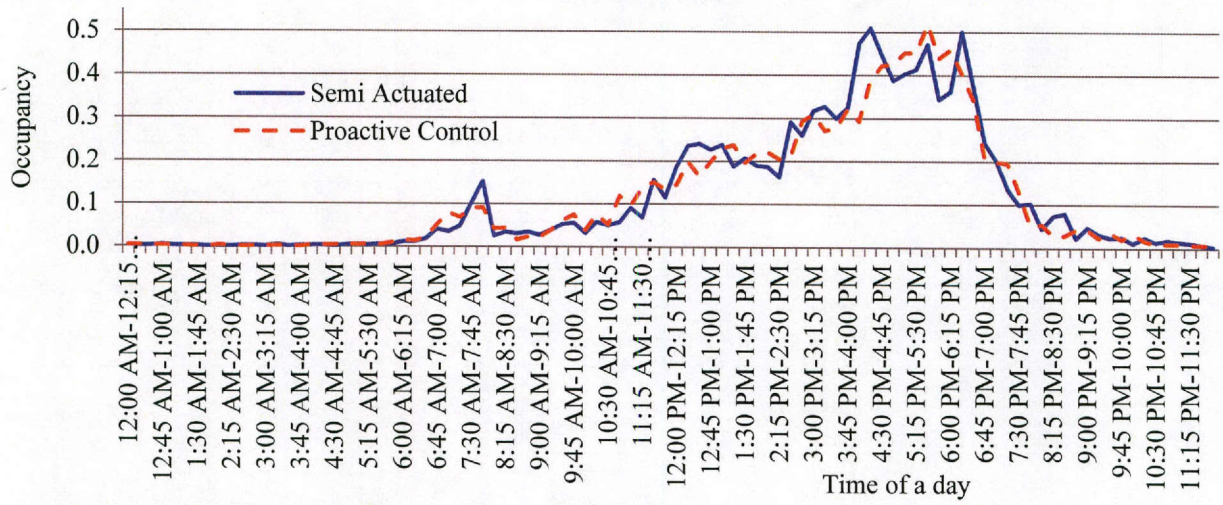
Katy Fort Bend on Weekdays

This intersection is the westmost one of all testbed. The onsite observation during the morning peak hours on December 11 and 17 found that the traffic volumes on the arterial road is quite small. No frequent platoons of vehicles were seen. In most times during the observation, vehicles arrived individually and randomly.

As reported in Figures 6.5.7 through 6.5.8, it is seen that the northbound traffic flows were quite small—only around 200 veh/hr/ln in the peak hours; while the southbound traffic flows were higher but still around 400 veh/hr/ln in peak hours. As mentioned before, given such a low traffic flow, the impact of the proposed proactive signal control system is not significant, because such low flows imply that vehicle platoons are not frequently seen. As mentioned before, therefore, it is expected to see that the impact of the proposed proactive signal control system may be not significant at this intersection.

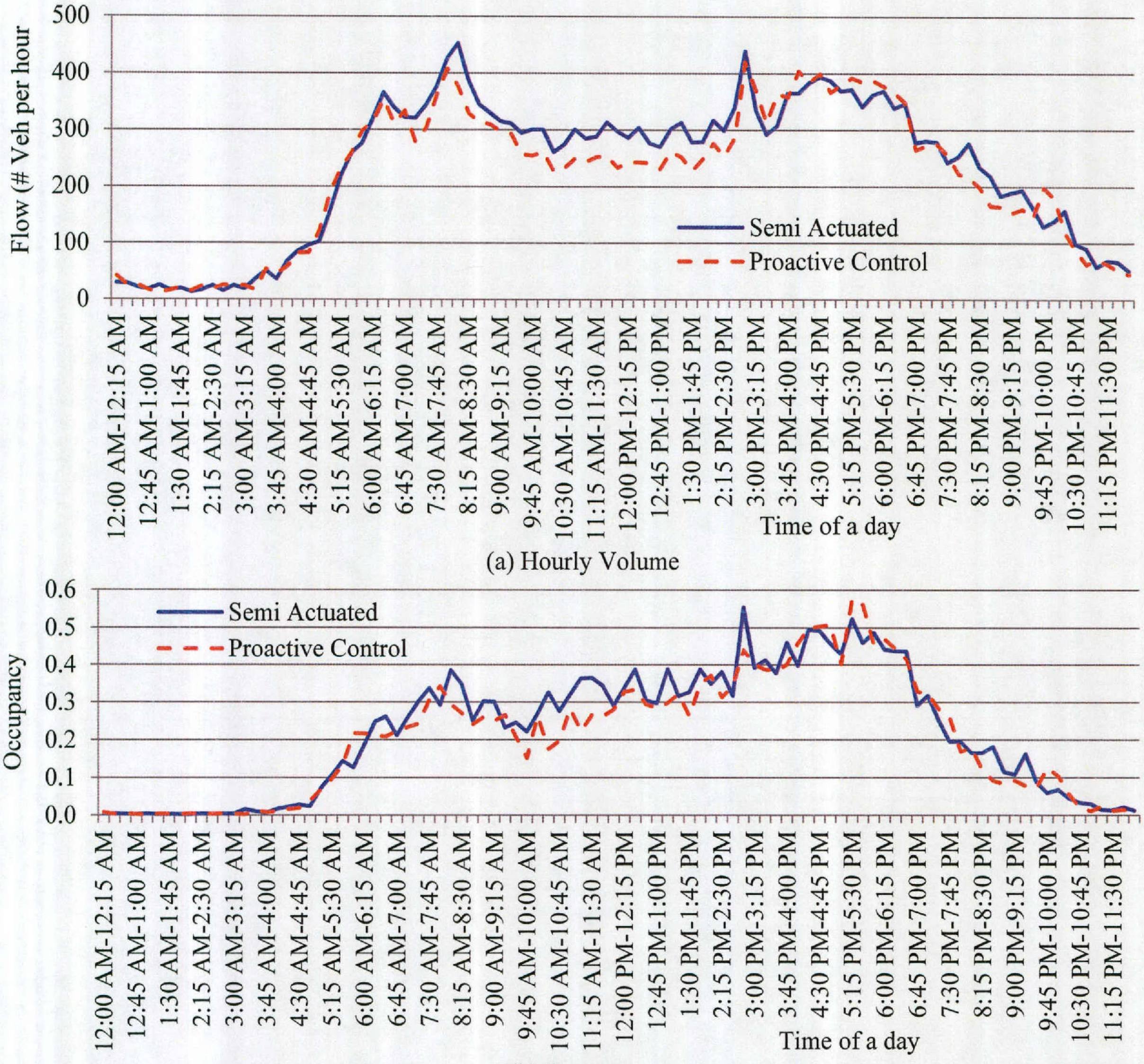


(a) Hourly volume



(b) Occupancy

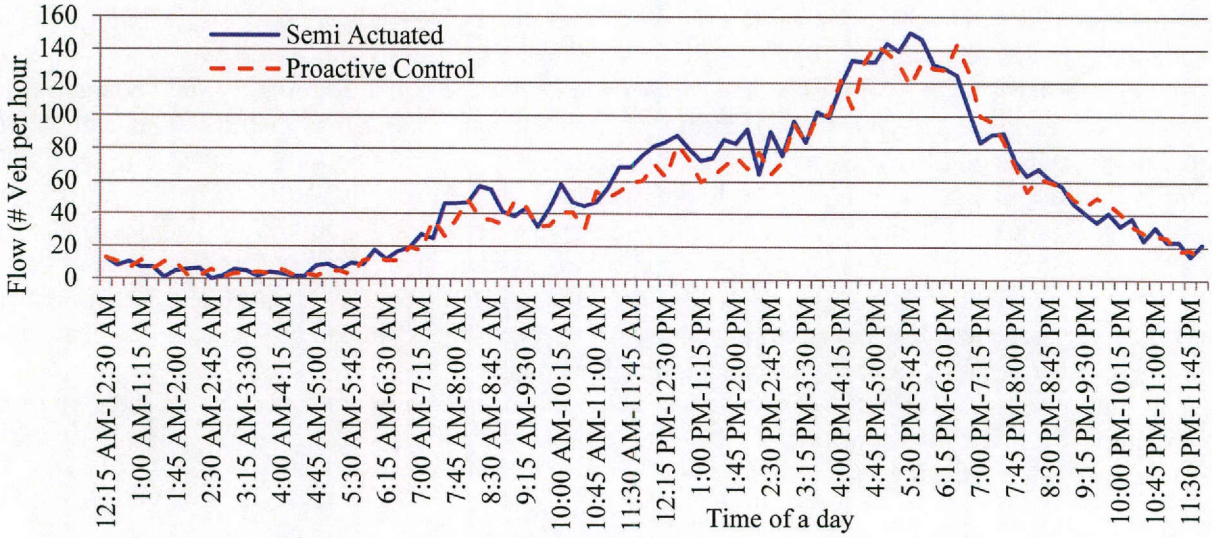
Figure 6.5.7 *Multi-day average northbound traffic of the intersection at Katy Fort Bend in weekday*



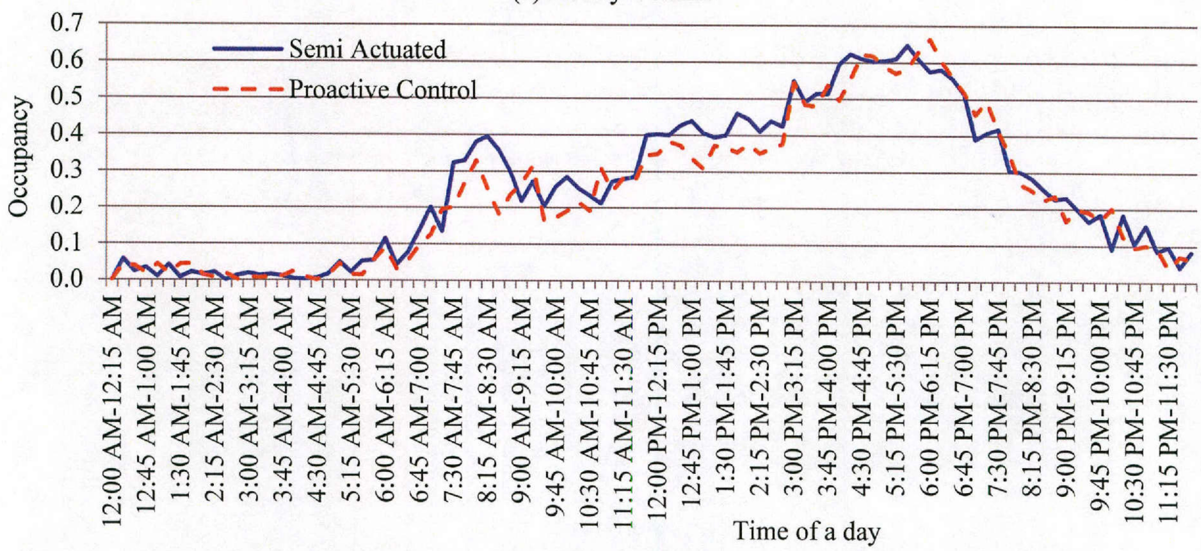
(a) Hourly Volume

(b) Occupancy

Figure 6.5.8 *Multi-day average southbound traffic of the intersection at Katy Fort Bend in weekday*



(a) Hourly Volume



(b) Occupancy

Figure 6.5.9 *Multi-day average westbound traffic of the intersection at Katy Fort Bend in weekday*

As to the westbound and eastbound traffic on the frontage road, some similar patterns to the traffic on the arterial road is seen: the flows were very small: just about 160 veh/hr/ln, even in peak hours. Therefore, we can say the proposed proactive signal control system has insignificant impact on such small traffic flows.

Actually, the research team also analyzed the traffic flow patterns during weekends before and after the implementation. It seems that the traffic flows were also small at this intersection (no more than 200 veh/hr/ln). Therefore, the details of the figures on weekend are not shown here, as the effectiveness of the proposed system is not significant.

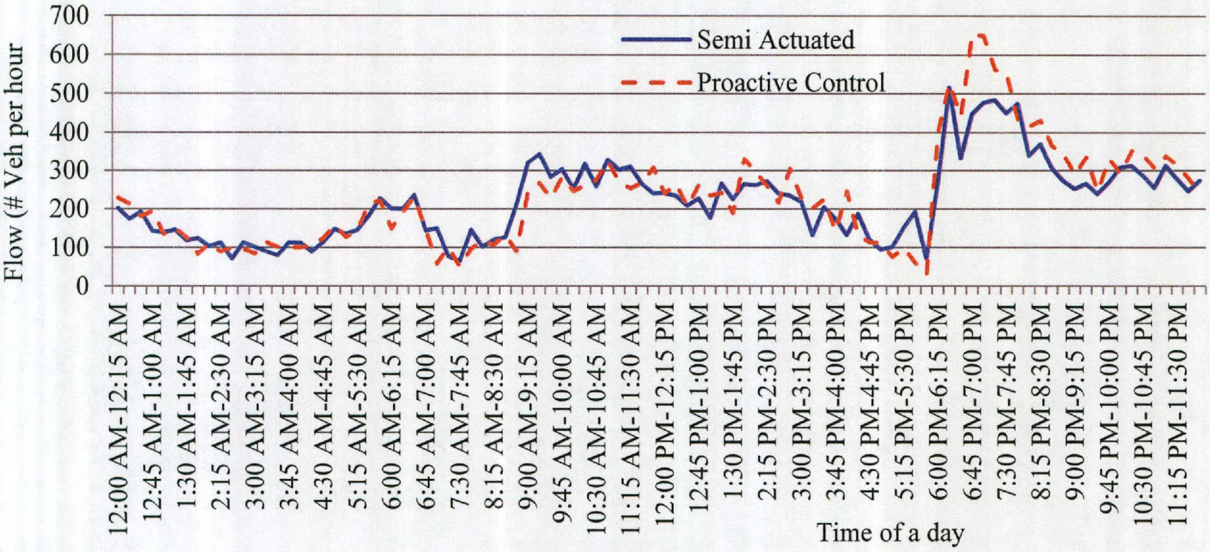
6.5.2 Comparison of Data Collected at Diamond Intersections with Video Detectors

In the second section, the performance of the proactive signal control system implemented to three intersections at Mason Rd., Fry Rd., and Barker Cypress Rd., are analyzed. These three intersections have only video detectors, so the logic applied to them is different from the previous three intersections. The logic here is the one shown in Figure 6.3.1 (b), which only relies on the occupancy, as volumes reported by video detectors are not accurate (largely underestimated by video detectors).

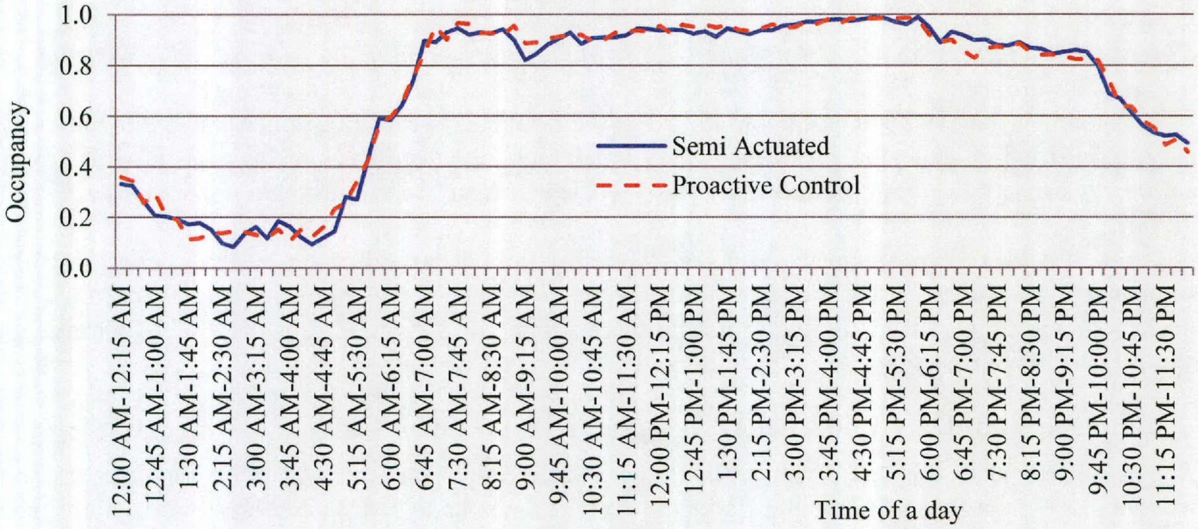
As mentioned in Section 6.5.1, if the flow is not large, it is expected to see that the impact of the proposed proactive signal control is not significant. Therefore, for brevity, only the cases with high flows were reported in this section.

Figures 6.5.10 and 6.5.13 shows the comparison of average traffic patterns for southbound traffic on Mason Rd., and Barker Cypress Rd., respectively, in weekday. It seems that the traffic in this direction at these two intersections saw higher volumes in evening peak hours—the traffic flows were over 500 veh/hr/ln (at Mason) and 300 veh/hr/ln (at Barker Cypress), respectively; while it was no more than 350 veh/hr/ln (at Mason) or 150 veh/hr/ln (at Barker Cypress) in other times. Figures 6.5.10 and 6.5.11 clearly show the significant impacts of the proactive signal control system. While for Fry Rd., Figures 6.5.11 and 6.5.12 show the comparisons of the northbound and westbound traffic patterns, respectively. For these two directions, it is seen that the traffic during the morning peak hours were large, and the implemented proactive signal control significantly helped improve the traffic.

Please note that the traffic flows were largely underestimated by video detectors. Therefore, the aforementioned criteria on flow (around 600 veh/hr/ln) (when the proactive signal control becomes effective) may not be suitable here.

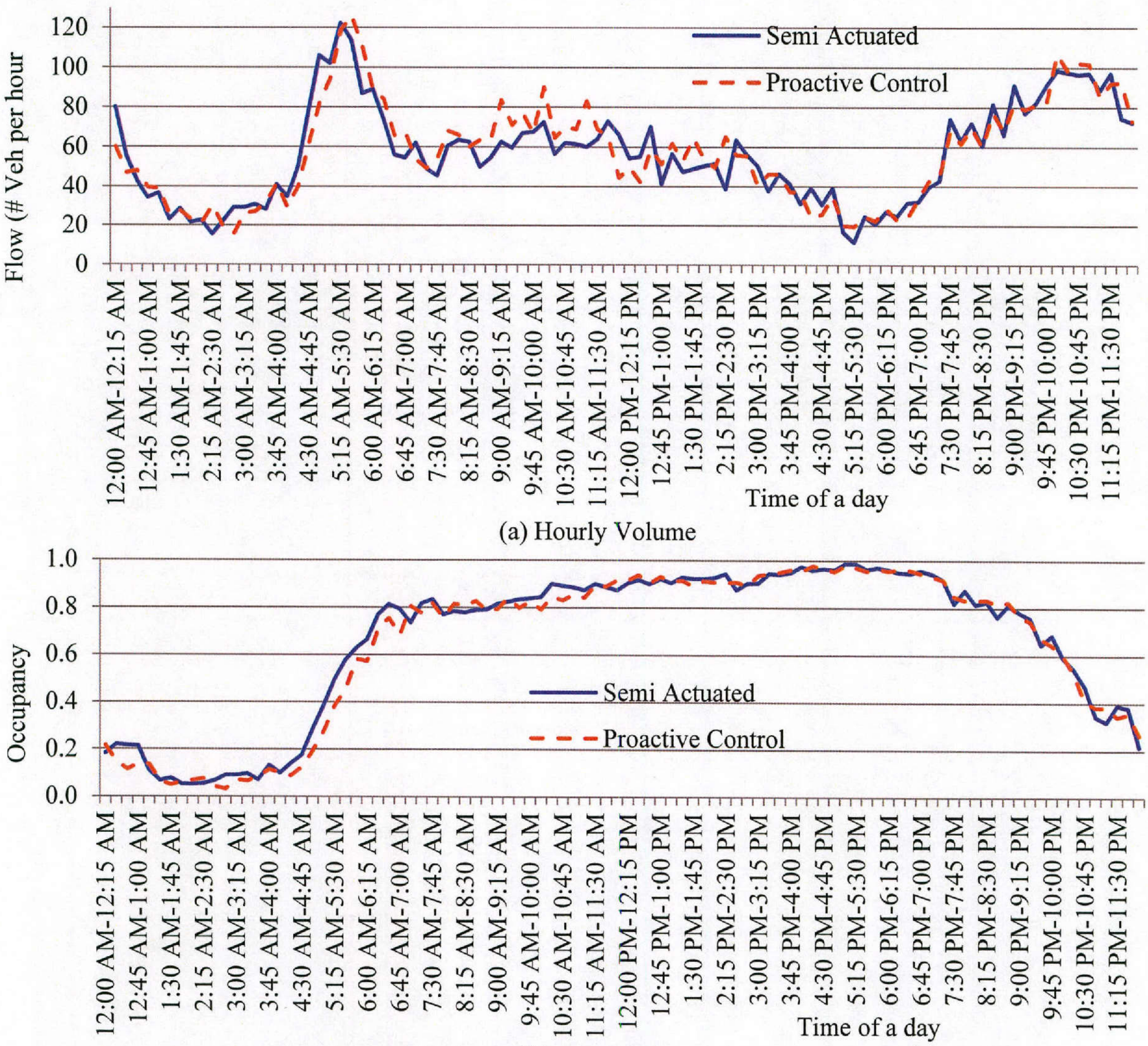


(a) Hourly Volume



(b) Occupancy

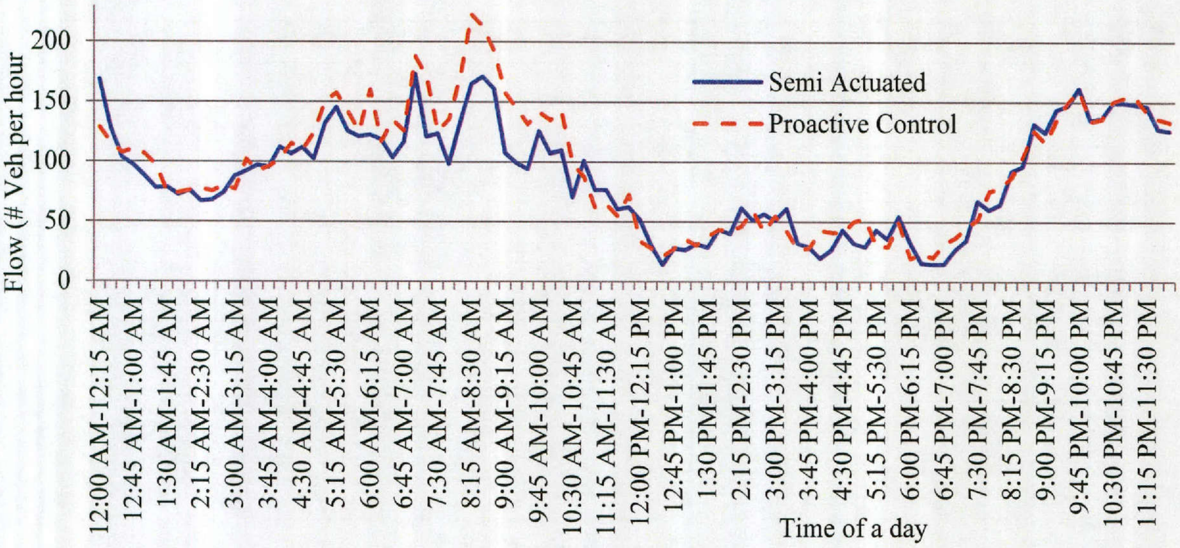
Figure 6.5.10 *Multi-day average southbound traffic of the intersection at Mason Rd. in weekday*



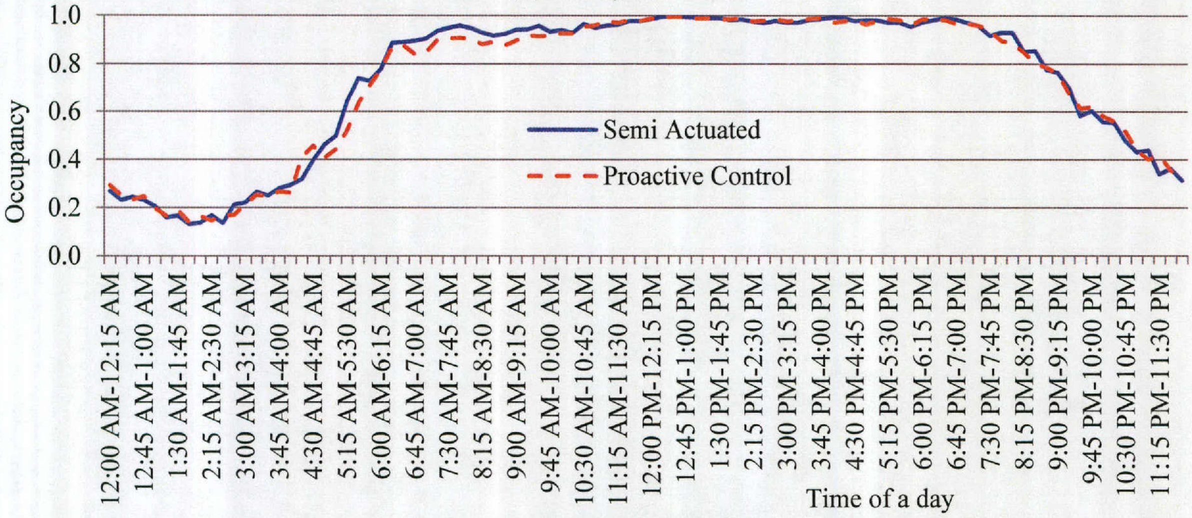
(a) Hourly Volume

(b) Occupancy

Figure 6.5.11 *Multi-day average northbound traffic of the intersection at Fry Rd. in weekday*

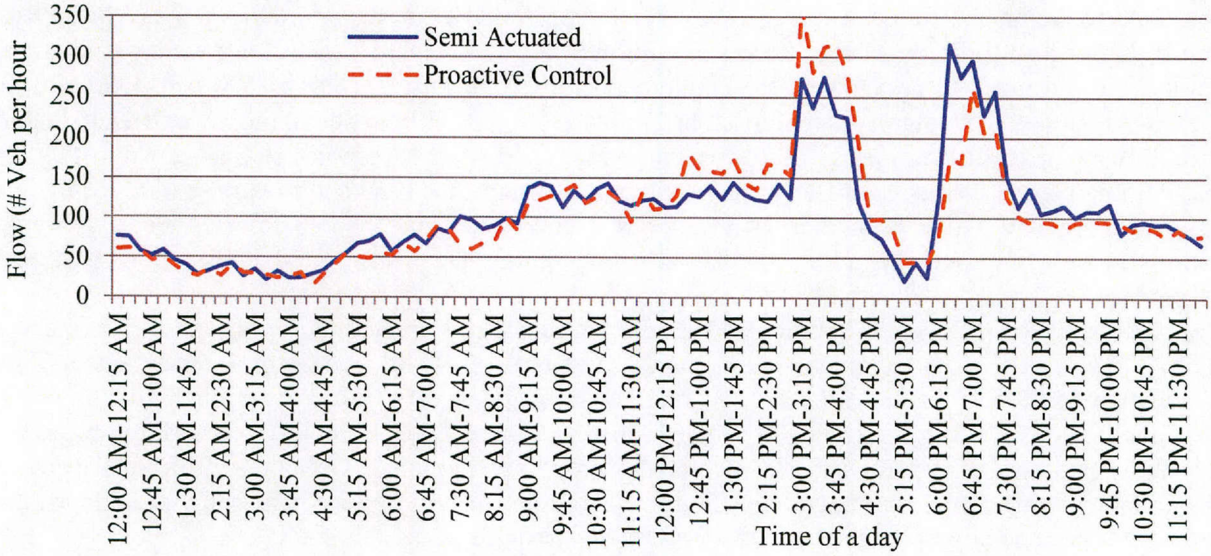


(a) Hourly Volume

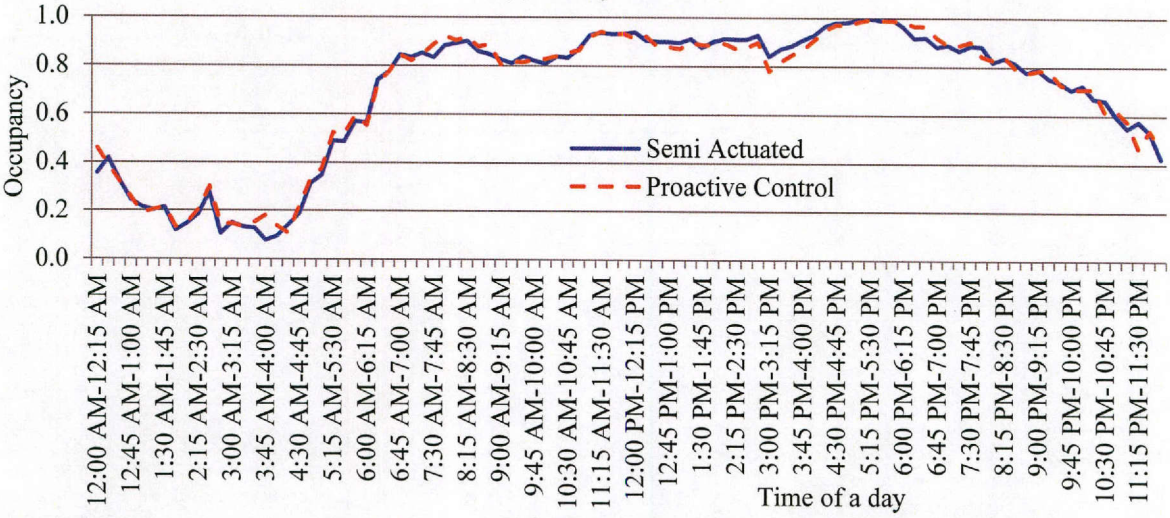


(b) Occupancy

Figure 6.5.12 *Multi-day average westbound traffic of the intersection at Fry Rd. in weekday*



(a) Hourly Volume



(b) Occupancy

Figure 6.5.13 *Multi-day average southbound traffic of the intersection at Barker Cypress Rd. in weekday*

Compared with the performance of the system implemented to three intersections with loop detectors, the impact of the proactive signal control system of these two intersections seems not so significant, even when the volume is large. The reason is that the logic here only depends on occupancy. Actually, the volume is a very important parameter used in logic design, as when the volume data are high, it reflect the passing of the platoons of vehicles. On the other hand, the occupancy, at some sense, more reflects the status when vehicles are waiting for signal. It may not well reflect the platoon of vehicles as volume does.

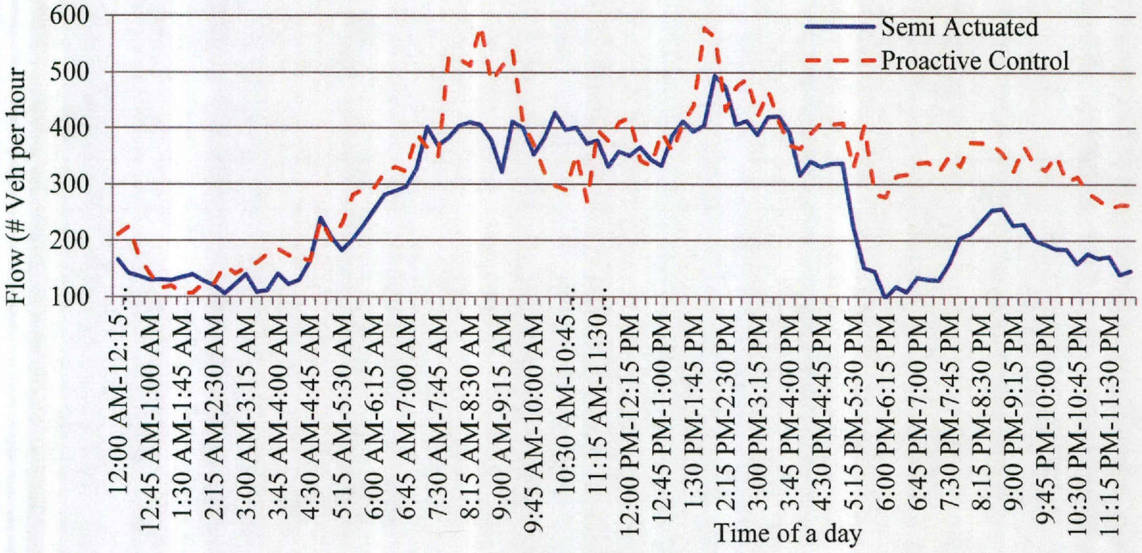
6.5.3 Comparison of Data Collected at Two Four-way Intersections

Finally, we will look at two four-way intersections at Mason Access Rd. and Fry Access Rd. These two intersections are about 500 ft south of the intersections of Mason Rd. and Fry Rd., respectively. These two intersections are four-way intersections, so the logic for these two should be different, as shown in Figure 6.3.1 (c).

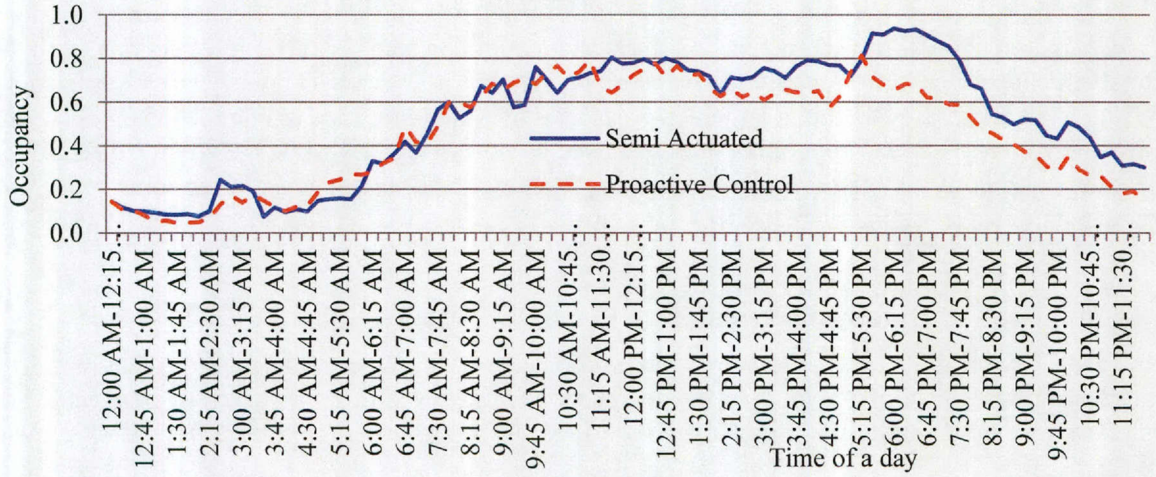
Still for brevity, this section only reports the results with high traffic flows because otherwise the impact of the proactive signal control is expected not significant, as the platoons of vehicles may not be frequent.

As to the intersection at Mason Access Rd., the traffic flows on the arterial road, i.e., Mason Rd., is much larger than those on Mason Access Rd. Further, the southbound traffic flows were found larger than northbound ones. In peak hours, the southbound flows could be as high as 500 veh/hr/ln, but the northbound ones were no higher than 300 veh/hr/ln. Therefore, similarly to Section 6.5.2, only southbound traffic flow comparisons in weekday and weekend were reported here, respectively, as shown in Figures 6.5.14 through 6.5.16. Note that different from other intersections, the weekend flow rates here seem to be as high as those in weekday. From these two figures, it is clearly seen that the implemented proactive signal control did help improve the traffic in that direction during the peak hours: higher flows but less occupancies.

As to the intersection at Fry Access Rd., the traffic patterns are similar to those at Mason Access Rd.: the flows on Fry Rd. were much higher than those on Fry Access Rd. But as this intersection, not like the situation at Mason Access Rd., the southbound traffic rates were just a little bit higher than the northbound: about 350 veh/hr/ln v.s 300 veh/hr/ln in peak hours. Though the difference is not large, the proactive signal control systems implemented to this intersection seems have more significant impact on the southbound traffic than northbound one. Figure 6.5.16 reflects such impact: in morning peak hours, the traffic flows became larger, but the occupancy rates were reduced.

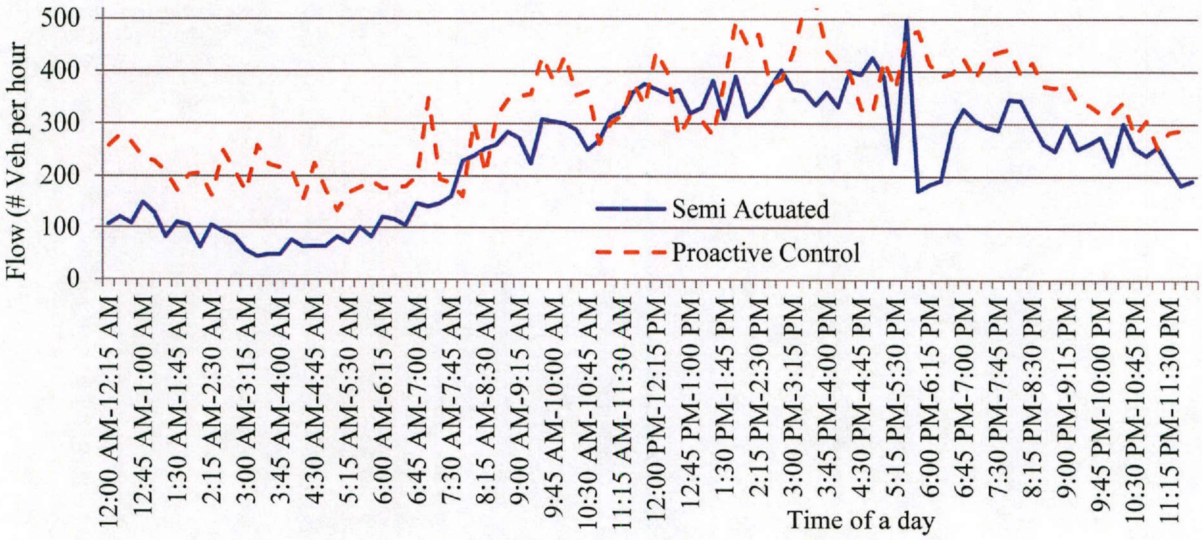


(a) Hourly Volume

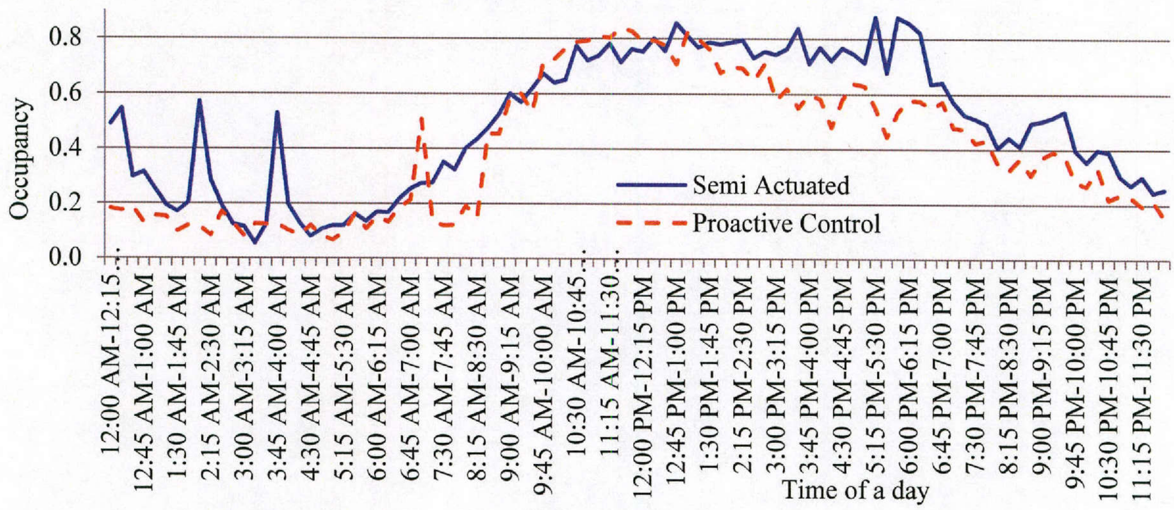


(b) Occupancy

Figure 6.5.14 *Multi-day average southbound traffic of the intersection at Mason Access Rd. in weekday*

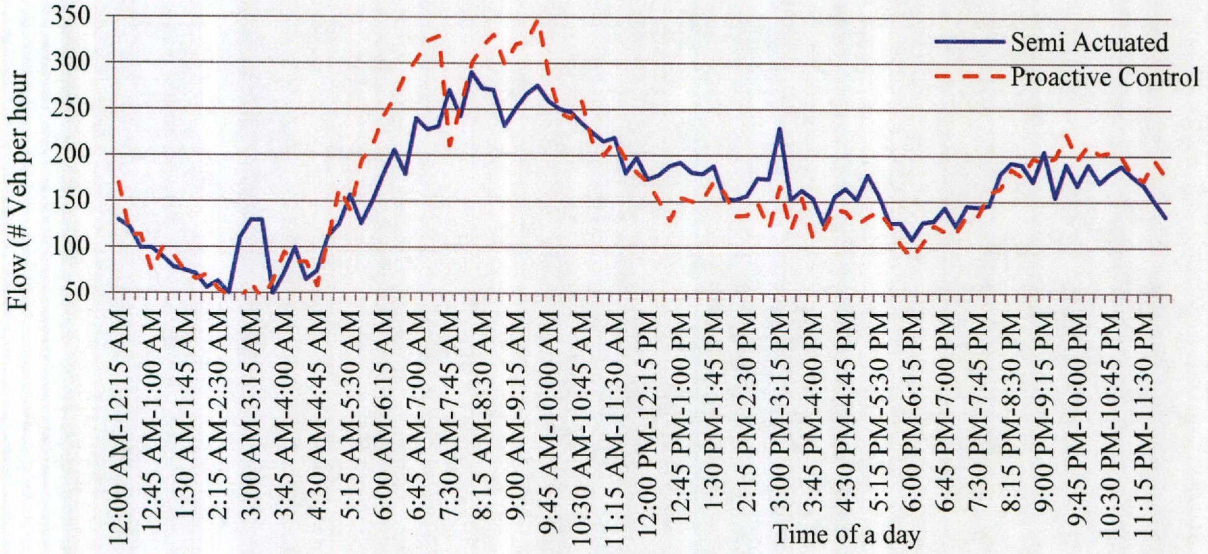


(a) Hourly Volume

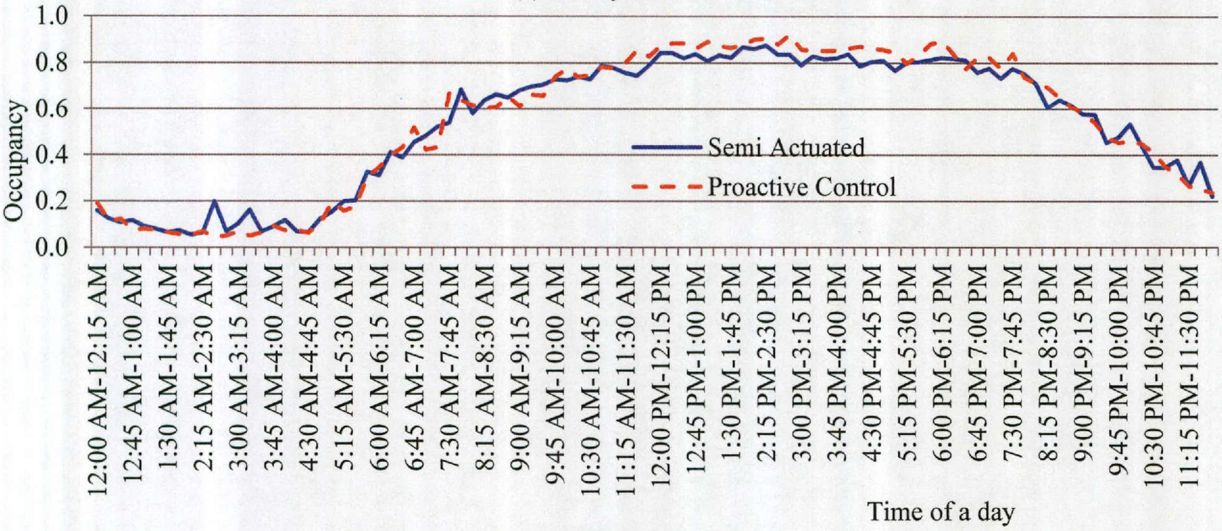


(b) Occupancy

Figure 6.5.15 *Multi-day average southbound traffic of the intersection at Mason Access Rd. in weekend*



(a) Hourly Volume



(b) Occupancy

Figure 6.5.16 *Multi-day average southbound traffic of the intersection at Fry Access Rd. in weekday*

6.6. SUMMARY

This chapter summarizes the field implementations to the last eight intersections. The implementations to these eight intersections were much more complicated than the work to the other intersections. One reason is that six are diamond intersections, which are the combinations of two intersections—overlapping phases exist. Also, due to the connection problems from the detectors, the implementations were seriously delayed—four intersections were replaced almost at the last minute in the scheduled time of implementation.

These eight intersections were divided into three parts: three diamond intersections that have loop detectors, three diamond intersections with video detectors only, and two four-way intersections using both loop detectors and video detectors. Since video detectors cannot accurately report the traffic volumes, three different logics were developed, each of which is for one part of intersections.

For the first part of three intersections, the logic is based on both volume and occupancy reported by loop detectors; for the second part of three intersections, the logic only depends on the occupancy data from video detectors; and the last one depends on both volume and occupancy.

The onsite observations during the day of implementations collected the data of queue length and delays of vehicles. For the approaches that suffered from heavy traffic delays, it was found that the proposed logic did help reduce the queue length. However, it was also found that when the traffic flow is not high, the impact of the proposed proactive signal control system is not significant. This is reasonable: the proactive signal control system was designed for helping the platoon of vehicles more smoothly pass through an intersection. When the flow is low, vehicle platoons are not apparent, so the impact is insignificant.

The data from loop detectors also reflect this feature. For all eight intersections, it was found that the traffic on arterial roads that go through underneath IH-10 suffers more serious congestion than that on the frontage road, except the westbound traffic during the evening peak hours at some intersections.

On the other hand, it was also found that the traffic flow of the intersection at Katy Fort Bend is generally small during all day, and the similar pattern was also found of the intersection at Barker Cypress Rd. (except its southbound through traffic). Therefore, the impact of the proactive signal control system at these two intersections is insignificant (but the impact to the southbound traffic at Barker Cypress Rd. can be still clearly seen in evening peak hours).

Table 6.6.1 summarizes the comparison of traffic flows and occupancies before and after the implementations to some phases at each intersection. For most phases shown below, the benefit of the proposed proactive signal control system is significant when the flow is large (above 500 veh/hr/ln).

It also should be noted that for three intersections using video detectors, the logic is only based on the occupancy. However, the occupancy rate cannot well capture the pattern of platoons of vehicles. The traffic volumes reported by loop detectors are largely underestimated. Therefore, it is seen that the effectiveness of the proactive signal control system at these three intersections are limited. At the intersections at Mason Rd. and Fry Rd., the traffic volumes are large on the arterial roads, but the volume reported by detectors are small. The impact of the implemented proactive signal control seems to be limited to the traffic on these arterial roads.

One issue in the implementations to these eight intersections is the tight time window due to the connection problems at these intersections. For this reason, the parameters used in the logics were not well tested in the simulations, as did to other testbeds. It may impact the performance of the proposed proactive signal control systems applied to these eight intersections.

Table 6.6.1 Summary of comparison before and after the implementation at some intersections

Intersection	Phase	Time Period		Change in Flow	Change in Occupancy
Westgreen Blvd.	SB	06:00:00	09:30:00	-2.7%	-7.6%
		11:00:00	15:00:00	12.7%	-68.4%
		15:00:00	19:00:00	20.1%	-36.8%
		All Day		1.7%	-21.5%
	EB	06:00:00	09:30:00	-12.4%	-4.7%
		11:00:00	15:00:00	3.2%	0.4%
		15:00:00	19:00:00	5.2%	0.8%
		All Day		3.0%	4.6%
Greenhouse Rd.	SB	06:00:00	09:30:00	8.3%	3.2%
		11:00:00	15:00:00	4.2%	3.3%
		15:00:00	19:00:00	6.6%	21.0%
		All Day		5.7%	6.7%
	EB	06:00:00	09:30:00	-16.6%	7.5%
		11:00:00	15:00:00	28.8%	1.1%
		15:00:00	19:00:00	1.3%	-0.9%
		All Day		9.9%	-3.8%
	WB	06:00:00	09:30:00	0.6%	0.4%
		11:00:00	15:00:00	3.1%	-1.1%
		15:00:00	19:00:00	3.2%	1.1%
		All Day		5.5%	5.1%
Mason Rd.	SB	06:00:00	09:30:00	-18.70%	1.91%
		11:00:00	15:00:00	4.44%	1.25%
		15:00:00	19:00:00	10.21%	-0.73%
		All Day		4.74%	0.43%
Fry Rd.	NB	06:00:00	09:30:00	11.31%	-2.14%
		11:00:00	15:00:00	2.95%	-0.20%
		15:00:00	19:00:00	-3.45%	0.12%
		All Day		11.31%	-2.14%
	WB	06:00:00	09:30:00	24.70%	-3.96%
		11:00:00	15:00:00	-5.26%	0.29%
		15:00:00	19:00:00	9.09%	-0.07%
		All Day		8.49%	-1.01%
Barker Cypress Rd.	SB	06:00:00	09:30:00	-13.19%	1.10%
		11:00:00	15:00:00	14.09%	-1.50%
		15:00:00	19:00:00	6.57%	-0.71%
		All Day		-1.46%	0.37%
Mason Access Rd.	SB	06:00:00	09:30:00	22.67%	1.15%
		11:00:00	15:00:00	7.49%	-6.45%
		15:00:00	19:00:00	38.67%	-19.05%

Intersection	Phase	Time Period		Change in Flow	Change in Occupancy
		All Day		24.85%	-12.21%
Fry Access Rd.	SB	06:00:00	09:30:00	22.27%	-1.19%
		11:00:00	15:00:00	-11.60%	5.52%
		15:00:00	19:00:00	-15.84%	5.06%
		All Day	2.34%	2.34%	1.73%

Note: the shaded cell indicates the expected results: higher flow rates but less occupancies.

CHAPTER 7

CONCLUSIONS

7.1. SUMMARY OF FINDINGS

This project (5-6920) is an extension of Project 0-6920, with a focus on the implementation on 30 intersections located at four testbeds at different sites in the Houston Metropolitan Area. As a preliminary study, Project 0-6920 built a proactive signal control system, based on the existing detecting and control technologies employed by the Receiving Agency. This novel control system aims to help vehicles in platoons more smoothly pass through an intersection. Within a platoon, vehicles have smaller headways than those not in a platoon. Therefore, helping vehicles in a platoon smoothly pass through an intersection at or close to the design speed, can largely help increase the capacity of an intersection, and thus relieve the congestion. The proposed proactive signal control system was successfully tested in a single intersection in the end of 2016 (see the Final Report of Project 0-6920).

As to this project (5-6920), however, most selected intersections have much more complicated conditions than the single intersection tested in Project 0-6920, so this project also saw a lot of efforts in new model building and testing. The research team faced two challenges in building the new models:

1. How to deal with the signal coordination between adjacent intersections of a testbed, where the intersections are spaced closely (such as the intersections at FM 528, SH 242, and FM 1464, respectively); and
2. How to develop a proactive control logic for the traffic going through a diamond intersection, which combines two intersections that shares the overlapping phases.

For the first challenge, some intersections on the testbeds of FM 528, SH 242, and FM 1464 originally have coordinated signal control systems, which aims to help traffic move through along the major road smoothly. This pre-timed coordinated signal control focuses on the through traffic on major road, but it could not well handle the left-turn traffic to minor roads or the traffic from minor roads. As mentioned in Section 1.2.2 of Chapter 1, the proactive signal control logic (see Figure 1.2.4) aims to capture the arrivals of vehicle platoons, but since the only available real traffic information near an approach of an intersection is the volume and occupancy (reported by detectors for every 1 minute), this logic only provides an approximate way to capture the arrivals of vehicle platoons. Note that if the platoons of vehicles could be precisely captured by the detectors, then this proactive signal plan must be coordinated at adjacent intersections automatically.

When firstly testing the proactive control logic at the testbed of FM 528, it was found that the coordinated signal control performed better for the through traffic on the major road during peak hours. The proposed proactive logic could not precisely capture the offset of the flow between two closely spaced intersections. Therefore, the proposed proactive signal control system was only applied to the off-peak hours (9:30 am–2:00 pm) on a weekday; on a weekend day, the proposed system replaced the original coordinated system as the flow volumes were found less than weekday peak hours: it works from 7am to 11pm. However, the data collected under the original and new systems, respectively, as well as onsite observations, found that the new system did not outperform the original coordinated system in the midday hours of a weekend day, implying that if the major road's through traffic is large, it is important to have signals coordinated. For this reason, it is critical to develop a logic to switch the control system between the coordinated mode and proactive mode. The switch shall depend on the traffic conditions. Such logics were applied to the testbed of SH 242 and FM 1464. The data collected from the loop detectors show that the implemented control systems work well.

The second challenge came from the diamond intersections along the frontage roads of freeways. A diamond intersection combines two intersections together at each side of a freeway. Two overlapping phases are used to coordinate traffic through two intersections together. Therefore, the traffic situation is quite different from those simple four-way or three-way intersections. Another question came from detectors: for the six diamond intersections selected for implementation, only three have loop detectors and the other three use video detectors, which heavily underestimate the traffic volumes. Note that volume is a critical parameter in the proposed control logic to detect the arrivals of vehicle platoons. Therefore, two separate signal control logics were proposed, respectively: one for those with loop detectors, where both volume and occupancy were used; and another for those with video detectors only, where only occupancy was employed.

The selected intersections on the four testbeds have different levels of traffic flows and occupancies. Therefore, the performance of the implemented signal control system varies at different testbed. It was found that when the flow rate was higher than 500 veh/hr/ln and the occupancy rate was larger than 0.4, the proactive signal control system worked well. If the flow rate is less than 350 veh/hr/ln (not reported by video detectors), then the new system became much less effective. This is because when the flow is low, vehicles are moving in a more independent and random mode, rather than moving in a platoon. On the other hand, if the occupancy is lower than 0.2, even if the flow rate is high, the impact of the new system is also not significant, because such low occupancies rates imply that the flow gets few interruptions, and vehicles move more interpedently.

The following summarizes the findings from the implementations to four testbeds, respectively.

1. For the three intersections on FM 528, the major road (FM 528) see quite high traffic flows (>1000 veh/hr/ln in daytimes) but low occupancy levels (<0.2 in most cases even in peak hours). Such low level of occupancies but high flow rates implies that the traffic on the major road is not often interrupted, the vehicle platoons may not be apparent. The data from loop detectors indicates that after the implementation, the change of the through traffic flows on the major road is small. However, the traffic conditions from minor roads to the major road has been improved—the queue length and delay was greatly reduced, and the hourly traffic flow rates were also increased, implying more vehicles were able to enter the intersection in a unit time.

2. The eight intersections on the testbed of SH 242 are different. The five intersections on the east side close to IH-45 see high volumes (> 800 veh/hr/ln) and they are spaced very closely, so the coordination is very important, and the proactive signal logic was simply imbedded into the original coordinated plan. Since the original coordinated mode was not changed significantly, the data collected before and after the implementation were found to close, implying that the impact of the imbedded proactive signal control logic is not significant to these five intersections.

   However, for the three intersections on the west side, the spacings were not so close, and the flow is not as high (around 500 veh/hr/ln) as those in the east part, but they suffer from high level of occupancies (about 0.5 in most of daytime). Moreover, they have large left-turn traffic from major road; e.g., the left-turn traffic to Gosling Rd. is around 350 veh/hr/ln, and the occupancy rate is high (about 0.8 in daytime).

   Therefore, the new control systems were specifically designed for balancing the left-turn and minor traffic and the through traffic on the major road. The data analysis show that the implemented system works week at these three intersections. For example, in the middle of the weekday (11am–3pm) when the left-turn traffic to Alden Wood and Gosling Rd. is highest, the implemented system helped increase the flow by 12% and 15% and reduce the occupancy by 13.5% and 6%, respectively. On the other hand, the implemented system also improves the traffic going through three intersections on the major road: based on the data collected, the flow was increased up to 55.5% (the best case, 24-hr average) and the corresponding reduction rate of occupancies was reduced up to 63%.

3. The 11 intersections on the testbed of FM 1464 also can be divided into two sections. The five intersections on the north side are spaced closely and see larger through traffic flows (>500 veh/hr/ln in peak hours) and high occupancy rate (>0.6 in peak hours), so the coordination of signals is important. Similarly, the research designed the logic to balance the original coordinated signal control mode and the new proactive control mode. But different from the those at SH 242, the focus here is the through traffic on the major road. The data from loop

detectors indicate that the new system performs well at these five intersections in morning and evening peak hours: in the best cases, the through traffic was increased by 18% and the corresponding occupancy was reduced by 8%.

On the other hand, the other six intersections in the south are spaced far away from each other and they see much lower levels of through-traffic flow (no larger than 300 veh/hr/ln in most cases), except the intersection at Bissonnet St. where the through traffic flow in peak hours is greater than 600 veh/hr/ln, so they were originally controlled by semi-actuated signal control. Also, with the exception of the one at Bissonnet St., these five intersections in the south also see a low level of occupancies (< 0.2 in most cases). Such level of flow and occupancy imply that many vehicles may not move in a platoon. Therefore, the proactive signal control system implemented to those five intersections may not have significant impact on the traffic pattern. Such an expectation is consistent with the analysis based on the data from loop detectors. However, the new system works well at the intersection of Bissonnet St., where the traffic flow and occupancy are both higher.

4. For eight intersections along IH-10, the traffic flow and occupancy also vary. Usually, the arterial roads that cross the frontage road see higher flow occupancy rates than the frontage road. If the flow rates are higher than 500 veh/hr/ln and the occupancy rates are large than 0.4, then it was found that the proactive signal control system performs well, such as the north- and southbound traffic along Westgreen Blvd. and south- and westbound traffic on Greenhouse Rd. In the best case, the flows were increased by 20% while the corresponding occupancy dropped by 37%. Generally speaking, the performance on arterial road (south- and northbound traffic) was found better than that on the frontage road, because the frontage road at each diamond intersection has only stop-bar detectors. If the flow rate is smaller than 350 veh/hr/ln, then the arrivals of vehicles can be regarded randomly, so the impact of the new system is not significant, such as the intersection at Katy Fort Bend.

   As to the intersections using video detectors only, the new proactive signal control system is simply based on the occupancy. The data collected from the Mason Rd. and Fry Rd., shows that when the flow rate was larger than 200 veh/hr/ln (note that video detectors underestimate traffic volume largely), the system performed well. At Fry Rd., it was found that the system performed well even when the flow rate is larger than 150 veh/hr/ln. However, the occupancy rates at these intersections are high—about 0.8 or even higher. Generally speaking, as the accuracy of video detectors is not high, the performance of the new system was found not as good as that applied to the intersections with loop detectors.

5. After the implementations, as reported by Steve Chiu, the Houston District Office has not received any complaint from drivers on these testbeds.

7.2. RECOMMENDATIONS

In theory, if the precision of loop detectors is perfect, the vehicle platoons can be accurately predicted, and the signal coordination between adjacent intersections can be well controlled by the proactive signal control logic—the proactive signal control system at two adjacent intersection would be coordinated automatically.

In the implementations, however, it was found that when the through traffic is large between two closely spaced intersections, the proactive signal control could not work well—it was not able to precisely capture the offset of flow, because this system uses an approximate method to capture the arrivals of vehicle platoons. To overcome this problem, the research team developed a complementary logic to switch the control system between the proactive mode and original coordinated mode, as did in the testbeds of SH 242 and FM 1464.

On the other hand, among 30 intersections, two intersections use video detectors. It was found that the performance of the new system is not stable at these two intersections, because the video detectors underestimate the volumes, which is the key parameter to determine the existence of platoon of vehicles. Therefore, for improving the efficiency of the proactive signal control system, one key issue is how to accurately capture the arrivals of vehicle platoons.

Based on the findings from the project, the research team from the Performing Agency would like to give the following two recommendations for the future work.

1. Currently, both loop detector and video detectors are used in arterial corridors in the Houston Metropolitan Area. The implementation results show that loop detectors are more useful in building an *effective* proactive signal control system for the purpose of reducing traffic delay and congestion, because loop detectors report traffic volumes in much higher accuracy than video detectors do. Also, more loop detectors at an approach of an intersection can help improve the performance of the proactive signal control system.

2. Testing the system on a large scale in the Houston Metropolitan Area shows that this system performs well in most intersections, especially those with high left-turn traffic flows from major roads and/or high traffic flows from minor roads. Therefore, it shows that this system is ready for other cities in Texas.

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