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**California PATH Research Report**
**UCB-ITS-PRR-2009-21**

This work was performed as part of the California PATH Program of the University of California, in cooperation with the State of California Business, Transportation, and Housing Agency, Department of Transportation, and the United States Department of Transportation, Federal Highway Administration.



Final Report for Task Order 6409 Contract # 65A0290

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CALIFORNIA PARTNERS FOR ADVANCED TRANSIT AND HIGHWAYS

# SPRINTER Rail:

# Grade Crossing/Traffic Signal Optimization Study

## Final Report for Task Order 6409


**Guoyuan Wu, Irene Li, Wei-Bin Zhang,**

**Scott Johnston, Meng Li and Kun Zhou**

Prepared by:
California PATH
University of California, Berkeley
In collaboration with

North County Transit
California Department of Transportation
City of Vista
City of Escondido
City of Ocean View
San Diego County

STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

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16. ABSTRACT

The second phase of this project further investigates impacts to local traffic operations at intersections adjacent to signal preemption by SPRINTER commuter trains and comes up with countermeasures that not only minimize such impacts but also take into account the traffic signal coordination. An extended traffic signal optimization model has been developed to minimize overall traffic delays and the weighted width of “green band” along several coordinated traffic signals around the grade crossings. Based on the train’s movement detection at grade crossings and the waiting queue estimation at the end of the preemption operation, the optimized signal timings can quickly clear the queue and still maintain coordination along the corridor of interest. This study also recommends further consideration of countermeasures involving advanced train detections.

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# Abstract

This report investigates impacts to local traffic operations at intersections adjacent to signal preemption by SPRINTER commuter trains and countermeasures that would minimize these impacts. Optimization models have been developed to estimate the waiting queue at the end of the preemption operation and to quickly clear the queue while minimizing overall traffic delay. Based on the optimization models, optimized signal timing plans for a total of 10 intersections were proposed, which are designed to facilitate the clearance of the queue accumulated during the preemption, as well as to assure that no major queue backup develops on the travel direction parallel to the train tracks. The analysis and simulation indicate that the intersection delay can be reduced as much as 24 percent as a result of implementing the optimal timing. This study also recommends further consideration of countermeasures involving advanced train detections.

Keywords: Signal Preemption for Transit Rail, Traffic Signal Optimization, Grade Crossing Signal Coordination

# Executive Summary

Frequent signal preemptions and the resulting traffic and transit operational impacts significantly interrupt coordinated traffic flows. Furthermore, the safety of pedestrians and traffic vehicles at and near rail/highway grade crossings is one of the most significant traffic safety issues because most such accidents cause severe injuries and often result in fatalities. Traffic congestion is already significant along the SPRINTER corridor and forecasted traffic demand indicates that dramatic increases are expected over the next 30 years. With the traffic signal preemption installed for the new SPRINTER train service, traffic congestion and safety problems will further deteriorate if traffic signal control is not optimized.

From the technical perspective, there are two major barriers to making the existing preemption and priority operation more efficient and safe. The first barrier is the short warning time that is a limitation of existing detection technologies. The other barrier is that there are no effective means to detect hazardous crossing conditions; therefore traffic controls at grade crossings typically do not have the capability of controlling the signals and/or warning train drivers of the hazardous conditions to attempt to reduce the likelihood of collisions. Consequently, fatality-causing accidents occur frequently at grade crossings.

PATH, North County Transit District (NCTD), Caltrans District 11, the cities of Oceanside, Vista, San Marcos, and Escondido, and San Diego County have been working on a research project to develop methodologies to mitigate the conflict between the new SPRINTER light rail transit system and highways crossing the rail line.

The Sprinter project has been working on countermeasures that would minimize the impact of SPRINTER operations on local traffic operations. More specifically, it takes a two-step approach as follows: The first step is to perform a basic signal timing

optimization based on the train detection that is currently in place. The second step is to develop a new strategy that can use information from advanced detection and time-to-arrival prediction technologies in order to improve the signal control for better management of the traffic. The work documented in this report represents the first step taken by the research team to address the traffic impact issue.

Since early 2007, the research team has been working with NCTD as well as local jurisdictions to gather traffic control, traffic volume and intersection geometry data. Optimization models have been developed and tested which will minimize overall traffic delay at intersections adjacent to grade crossings after the train preemption operation ends. Based on the time point of train arrival during a traffic signal cycle, the duration of the preemption and intersection-specific information (lane geometry, traffic volume, etc.), models estimate the waiting queue at the end of the preemption operation, and try to clear the queue within as few cycle(s) as possible while minimizing overall traffic delay. The data collected from Caltrans and the cities are fed into the model to develop sets of optimized signal timing plans for the intersections under consideration. Unlike the regular timing plans, the purpose of this newly-generated timing scheme is to facilitate the clearance of the queue accumulated during the preemption, as well as to assure that no major queue backup develops on the travel direction parallel to the train tracks. The model thus balances the competing requirements from all directions and provides a balanced signal timing solution. The analysis of the change in overall intersection delay as a result of implementing the optimal timing offers as much as 24 percent improvement over the initial case.

## Table of Contents

# 1. Background

Urban rail can be an effective solution to mitigate traffic congestion along major urban corridors. However, frequent preemptions at rail/highway grade crossings and the resulting delay can significantly interrupt coordinated traffic flows and threaten the safety of pedestrians and other vehicles.

## 1.1 *Overview of the Sprinter Rail Transit*

The SPRINTER Rail Transit system is located in North San Diego County. It extends nearly 22 miles connecting the four North County cities - Oceanside, Vista, San Marcos, and Escondido, as well as unincorporated areas of San Diego County. It serves 15 stations including a 1.7 mile loop that serves California State University San Marcos (CSUSM). The rail line parallels the heavily-congested SR 78 corridor and was previously a freight only transportation corridor. The SPRINTER started revenue service on March 9$^{th}$, 2008.

The project is built primarily on an existing railroad right-of-way owned by the transit agency - the North San Diego County Transit District (NCTD).

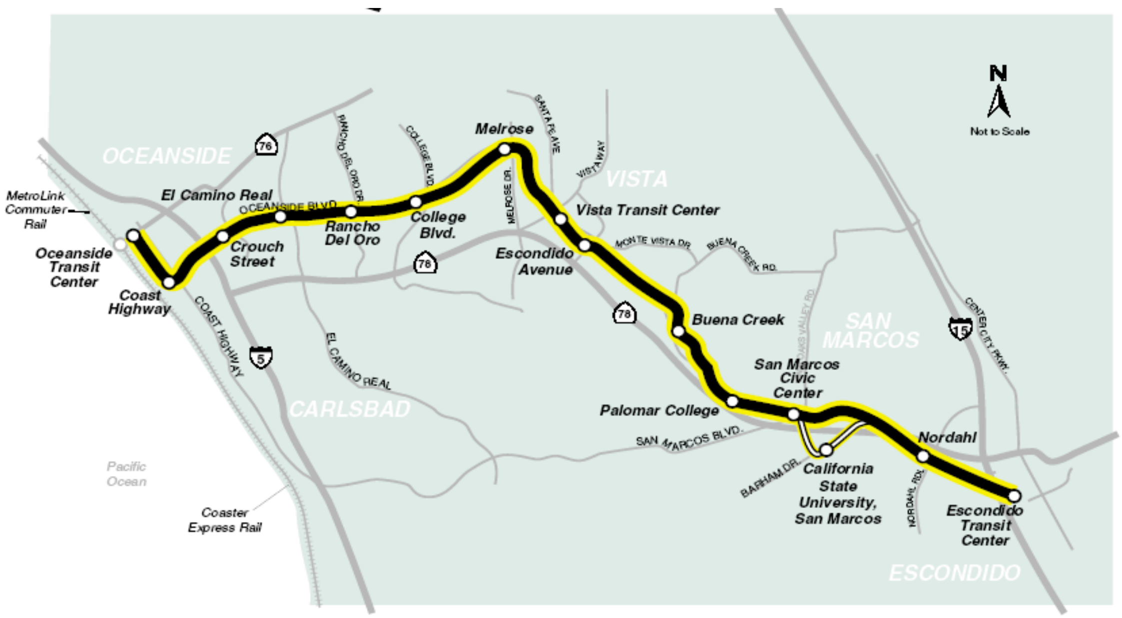


**Figure 1 SPRINTER Project Site**

The existing railroad was previously used by Burlington Northern-Santa Fe for freight transportation.

The freight trains are typically 1,100 feet. long and run only three round trips every week during evening and early morning hours, thus there is very small traffic impact arising from the freight train operation. The current passenger train service shares track operation with the freight train. The passenger and freight operations are temporally and spatially segregated, with the passenger train operating between 5am and 11pm and the freight train operating between 11pm and 5am. The passenger trains are Diesel Multiple Units (DMU), either 85ft. (1 car) or 170ft. (2 cars) long, with approximately 30 minute headways and a maximum operating speed of 55mph. The maximum operating speed of the freight train is 30mph.

The train traffic control system uses traditional blocking systems and a Centralized Train Control (CTC) system. The control center is located at the maintenance facility in Escondido. The track circuits provide train presence detection for interlocking control. Near grade crossings, they also provide predicted time to arrival at the grade crossing.

The corridor served by this project parallels SR78, which currently is already congested for its entire length during morning and afternoon peak periods. The project will serve large intermodal transit centers in both Oceanside and Escondido. The corridor contains a dispersed mix of commercial, industrial, single- and multiple-family residential developments. It has also been estimated that the number of residents living in communities served by the rail line will increase by 74 percent, with employment increasing at nearly the same rate. Thus, current corridor traffic volumes are projected to increase by more than 50 percent by the year 2015, ranging from 150,000 to 200,000 vehicles per day.

In 2004, the consulting firm URS was contracted by the stakeholders along the SPRINTER line to study the possible impacts of train preemption on intersection signal operations and develop strategies to alleviate any negative impacts.

The report focused on 20 intersections along the SPRINTER line. For the base year scenario, *TRAFFIX* traffic analysis software was used to estimate the current LOS at both signalized and unsignalized intersections. The analysis showed LOS of C and D at many intersections. For the future year scenario with train preemption operating, a microscopic simulation software VISSIM was used to evaluate the potential delay and queue length. Based on the simulation result, long term and short term alleviation measures were suggested, ranging from restriping to signal upgrades. However, these measures are only suggested but not further evaluated for their effectiveness. Based on our interviews with the local jurisdictions, the URS study was not performed to the satisfaction of the stakeholders.

## 1.2 Safety and Efficiency Issues at Grade Crossings

The Federal Railroad Administration (FRA) has identified over 260,000 public and private grade crossings in the United States. On average, a pedestrian or a vehicle is hit by a train every two hours in the United States. Among all rail-related fatalities, 90% are connected with grade crossing and trespassing incidents. Additionally, preemption given to trains at grade crossings often generates negative impacts to the safety of pedestrians, cars and the train as well as the efficiency of other traffic.

Section 8A.01 of the Manual on Uniform Traffic Control Devices (MUTCD) defines “the transfer of normal operation of traffic signals to a special control mode” as traffic signal preemption. It is also specified in Section 8D.07 of the MUTCD: “When a highway-rail grade crossing is equipped with a flashing-light signal system and is located within 60 m (200 ft) of an intersection or mid-block location controlled by a traffic control signal, the traffic control signal should be provided with preemption in accordance with Section

4D.13." This provision of the MUTCD is designed to ensure that the preemption sequence reaches the Track Clearance Green interval as soon as possible so that the traffic at the crossing can be cleared prior to a train's arrival. The Track Clearance is treated with higher "relative hazard" over other control events. Consequently, the railroad preemption gives the highest 'priority' from the control aspect at grade crossings and interrupts any other ongoing control events, e.g. emergency vehicle preemption, pedestrian walking and clearance time, minimum and other vehicle green times. As a result, the preemptive treatment at grade crossings may result in crashes at grade crossings.

In addition to the potential hazards at grade crossings, the preemption treatment could jeopardize safety at adjacent signalized intersections. Specifically, the shortening of the pedestrian walking and clearance time can leave a pedestrian walking in the middle of a road while a conflicting track clearance movement receives green. Although not as critical as truncating pedestrian clearance time, the shortening of minimum vehicle green time may also violate drivers' expectancy and lead to unsafe driver behavior. According to a survey of operations engineers and practitioners across the United States[1], the shortening of normal pedestrian clearance and minimum vehicle green time has been ranked one of the most critical safety concerns.

Moreover, the signal control at adjacent intersections could potentially impact the safety at grade crossings. An oversaturated intersection might block traffic within the dangerous zone at a grade crossing.

Furthermore, existing preemptions at grade-crossings could seriously degrade the efficiency of traffic signal control at adjacent intersections. Preemptions often negatively impact the cross street traffic signal progression for the controlled intersections that are

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[1] Roelof J. Engelbrecht, Kevin N. Balke, Srinivasa R. Sunkari, and Steven P. Venglar; *Engineering Solutions to Improving Operations And Safety at Signalized Intersections Near Railroad Grade Crossings with Active Devices;* FHWA/TX–06/0-4265-1, September 2005

operating at or close to capacity. Even worse, standard traffic signal optimization[2,3] approaches do not apply, since the signal operations of these controlled intersections at grade crossings differ from those of urban signalized intersections. The sub-optimality of traffic signal timings at these intersections in the vicinity of highway-railroad grade crossings contributes to non-trivial and unnecessary waiting after a train has passed. The extra long queues might back up to upstream intersections as well. Without awareness of the preemption control, the upstream intersections might waste time on directing traffic toward a fully occupied approach. In the worst scenario, drivers might totally block the upstream intersections for all approaches. A recent study has proposed an improved transition preemption strategy (ITPS)[4] to provide more green time to the phases that will be blocked during preemption, as compared to the normal traffic signal mode[5]. However, the optimality of overall traffic delays cannot be guaranteed. Further research on a better optimization approach is required to improve the performance of nearside intersections adjacent to at-grade crossings.

## 1.3 Literature Review of At-Grade Crossing Operations and Technologies

To better understand the traffic signal operations at intersections near at-grade crossings, the research team collected and reviewed the literature related to regulations and practices

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[2] A. Skabardonis, R. L.Bertini and B. R. Gallagher; *Development and Application of Control Strategies for Signalized Intersections in Coordinated Systems*; In Transportation Research Record 1634, TRB, National Research Council, Washington, D. C., 1998, pp. 110-117

[3] W-H. Lin and C-H. Wang; *An Enhanced 0-1 Mixed-Integer LP Formulation for Traffic Signal Control*; IEEE Trans. On Intelligent Transportation Systems, Vol. 5 (4), December, 2004

[4] Cho, H., and L. R. Rilett; *Improved Transitional Preemption Strategy for Traffic Signals at Intersections Near Highway-Railway Grade Crossings*; Transportation Research Board 83rd Annual Meeting, 2004

[5] Jacobson, M., Venglar, S., and J. Webb; *Advanced Intersection Controller Response to Railroad Preemption – Stage I-IV Report*; Texas Transportation Institute, Texas A&M University, College Station, Tex., May 1999-February 2000

of traffic signal control for such intersections. This section summarizes the findings from the review.

### 1.3.1 Legislations, regulations and guidelines

In order to propose practical enhancements to current control strategies at the twenty-two key intersections along the SPRINTER rail corridor, applicable legislation, regulations and guidelines must be followed. Railroad grade crossing requirements and regulations exist in many government documents. Among them, *Title 49 Transportation from the Code of Federal Regulations*[6] has the authority of defining grade crossing related applications. Secondly, we reviewed *Manual on Uniform Traffic Control Devices (MUTCD)*[7], which is issued by FHWA and has the power of law. This document defines traffic devices used for streets and highways. There are two type grade crossings classified in MUTCD: highway-railroad grade crossing and highway-light-rail-transit (LRT) grade crossing. The SPRINTER passenger rail system will use Diesel Multiple Unit (DMU) light rail vehicle, but the system will share the right-of-way with the existing freight rail system. Therefore, system requirements for both rail and LRT should be followed.

The California Public Utilities Commission (PUC) published and enforces a number of general orders, which described the operation and regulation guidelines for the railroad and LRT vehicles as well as for grade crossings. The other two guidance documents reviewed in this report are issued by FHWA. The first one is *Railroad-Highway Grading Crossing Handbook*[8], which contains information for grade crossing planning, implementation and evaluation. The handbook walks through all the phases in designing

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[6] U.S. Department of Transportation, Revised October, 1 2003, http://www4.law.cornell.edu/uscode/html/uscode49/

[7] FHWA, Manual on Uniform Traffic Control Devices, 2003 Edition, http://mutcd.fhwa.dot.gov/

[8] B.H. Tustin, etc., FHWA, Railroad-Highway Grade Crossing Handbook - 2nd Edition, September 1986, http://www.fhwa.dot.gov/tfhrc/safety/pubs/86215/intro.htm

a grade crossing. The other document is *Guidance on Traffic Control Devices at Highway-Rail Grade Crossing*[9]. This report puts different perspectives together to make a better description of existing requirements and regulations.

### 1.3.2 Traffic Signal Operations near Highway-Rail Grade Crossings

The purpose of this study is to seek improved traffic signal operations near at-grade crossings that are affected by railroad preemption. There are a number of strategies and technologies for railroad preemption, as summarized in Table 1. It is evident that each strategy or technology has its own pros and cons, therefore, different methods or devices should be applied, taking into account the specific situation. For example, in the crossing warning systems, *passive warning devices* are low in cost and quickly implemented but offer low benefit to congestion relief, while active warning systems can provide real-time notification and greater benefit but are much more involved and expensive. Maybe the most illustrative examples are, *adaptive traffic signal phasing* and *variable message sign*s. Both are very useful and valuable in many cases, but they might require a large amount of programming or cost much more than other devices. In some cases, the intersection capacity and reduction of clearance time may offset each other. In general, exploring the selection of various strategies and technologies and the combination of them is not trivial.

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[9] U.S. Department of Transportation, FHWA, GUIDANCE ON TRAFFIC CONTROL DEVICES AT HIGHWAY-RAIL GRADE CROSSINGS, November 2002, http://safety.fhwa.dot.gov/media/twgreport.htm

**Table 1 Summary of Railroad Preemptions**

| | Types | Methods |
|---|---|---|
| Conventional Preemption Strategies | Interaction with Traffic Signal | Single-break, Two-wire Interconnection Circuit |
| | | Double-break, Four-wire Interconnection Circuit |
| | | Double-break, Interconnect Circuit with Supervision and Gate Horizontal Control |
| | Preemption Timing | Minimum Warning Time |
| | | Right-of-Way Transfer Time ( RWTT) |
| | | Queue Clearance Time |
| | | Separation Time |
| Railroad Crossing Protection Technologies | Classic System for Detection | Three-track Circuits |
| | | Motion Detector |
| | | Constant Warning Time (CWT) |
| Crossing Warning Systems | Passive (Independent from train operations) | Warning Signs (Crossbucks) |
| | | Pavement Markings |
| | Active (dependent on or triggered by train) | Crossing Gates |
| | | Flashing Lights |
| Enhancements to Reduce Congestion | Delayed Gate Lowering at Nearside Light-Rail Platforms | |
| | White Transit “T” Indication | |
| | Four Quadrant Gate Systems | |
| | Pre-signal | |

| | |
|---|---|
| | Advanced Preemption |
| Innovative Preemption Strategies & Technologies | Adaptive Traffic Signal Phasing |
| | Traffic Signal Software |
| | Variable Message Sign (VMS) |
| | 2nd Train Warning System |
| | Emergency Vehicle Priority |
| | Radar Obstacle Detection System |
| | Trapped and Stalled Vehicle Detection |
| | Photo Enhancement |
| | Video Detection System |
| | Transit Vehicle Tracking (Mainly Using GPS) |
| | Real-Time Railroad Crossing Status Monitoring |
| | In-Roadway Warning Light at Railroad Crossing |
| | Automated Horn System (AHS) |

Typical railroad preemption procedures at signalized intersections include:

*Prior to the train crossing*: The railroad crossing controller will receive a train approaching signal from detection equipment and then initiate the warning devices and the necessary traffic signal preemption events (including the clearance of tracks).

*During the train crossing*: The warning devices will be activated for at least a minimum amount of time prior to the arrival of the train at the crossing. When the automatic crossing gates are lowered and all movements toward the track have stopped, the traffic signal may implement a limited phasing sequence.

*After the train crossing*: The railroad crossing controller will trigger the automatic gates to rise and the flashing signals and horns to stop. Then, traffic is allowed to move normally.

Recently, the Texas Transportation Institute (TTI) developed the transition preemption strategy (TPS) algorithm[10]. This algorithm was developed to ensure that as the preemption was initiated by approaching trains, the signal would not change to endanger either pedestrians or drivers. In addition to a constant warning time (CWT) detector, the TPS algorithm may require an upstream detector, such as a pulse-coded track circuit, sonic detector, Doppler radar detector, AVI, or some other device, to get the constant advance preemption warning time (APWT). The time between the activation of the two detectors is the TPS operation time.

The data from an APWT detector is fed into a train arrival time prediction algorithm. Because of the variability of the predicted arrival time, the TPS algorithm can be cut abruptly. This may result in safety problems or can apply extra green period in the track clearance phase which may result in excessive intersection delay. Therefore, an improved transition preemption strategy (ITPS)[11] was designed to provide more green time to the phases that will be blocked during preemption, as compared to the normal traffic signal mode and the TPS algorithm.

A signalized intersection along a railway corridor in College Station, Texas was chosen as the test bed for the ITPS algorithm. A Doppler radar detector, located approximately 2.2 km (1.4 mile) upstream, can provide train speed continuously while it is in the detection area. Although no field result from the test bed was discussed in the paper, a simulation network, based on VISSIM plus vehicle actuated programming (VAP), had been set up to duplicate the test bed. Comparing standard preemption and the current

[10] Jacobson, M., Venglar, S., and J. Webb, Advanced Intersection Controller Response to Railroad Preemption – Stage I-IV Report, Texas Transportation Institute, Texas A&M University, College Station, Tex., May 1999-February 2000.

[11] Cho, H., and L. R. Rilett, Improved Transitional Preemption Strategy for Traffic Signals at Intersections Near Highway-Railway Grade Crossings, *Transportation Research Board 83rd Annual Meeting*, 2004.

TPS, the simulation results indicated that the ITPS algorithm with an APWT value of 100, 110, or 120 seconds is the more efficient operation strategy for both safety and efficiency.

### 1.3.3 Existing Grade Crossing Control Technologies

The crossing control system is the logical controller or circuitry which is designed by engineers to control a specific grade crossing location. A control can be implemented by using relay logic or a vital microprocessor based controller or the combination of both. In the case of needing a timing requirement, a Constant Warning Time system (grade crossing predictor) or motion sensor can be used.

The Constant Warning Time system[12] is widely used as a highway grade crossing control system. This type of system is very well designed and has proven to be reliable enough to perform the crossing control. In this section, we will discuss some commonly used equipment. There are still many grade crossing controls using regular fail-safe equipment such as Microlok. This is mainly because of the control issues or cost-benefit concerns.

Track circuits are the most commonly used means for train detection. The requirement and regulation of a track circuit can be found in FRA rules[13]. Table 2 lists the different track circuits.

---

[12] NCHRP Synthesis 271: Traffic Signal Operations Near Highway-Rail Grade Crossings

[13] Title 49, Part 236

**Table 2 List of Different Track Circuits**

| | TYPE of TRACK CIRCUIT | COMMENT |
|---|---|---|
| 1 | D-C coded track | |
| 2 | D-C none-coded | |
| 3 | A-C immunized | Steady energy, used in electrified area |
| 4 | A-C coded track | |
| 5 | A-C none coded track | |
| 6 | AC-DC track | Type-C or rectifier/DC relay |
| 7 | Audio-Frequency-Overlay | AFO-IIC, ATT-20, AFTAC II, PSO-III |
| 8 | Motion Sensor | MODEL 660, 2000, PMD-3 |
| 9 | CWT System | |

The Audio-Frequency Overlay (AFO) track circuit does not require insulated joints. It has an AFO transmitter and an AFO receiver. The transmitter generates a user-selected audio frequency signal sent through the rail. The receiver picks up the frequency signal and energizes the track relay. One of the benefits using AFO is that the track circuit can co-exist with other track circuits as long as they are not in the same frequency. The AFO circuit is widely used in highway crossings, especially with LRT systems.

### 1.3.4 Time-to-Arrival Prediction at Grade Crossings

An efficient signal control strategy requires consistent detection time which comes from the reliable detector and an accurate Time-to-Arrival (TTA) prediction model. The most common system to predict train arrival times, a so-called first generation system, is the one that is integrated with the railroad track circuitry. Second-generation technology uses information available from non-intrusive devices mounted off the railroad right-of-way.

A recent study on third-generation detection systems, which are on-board systems, was conducted by our own team at PATH in the Adaptive Transit Signal Priority (ATSP) project. In the ATSP project, the bus's absolute position and its corresponding coordinated universal time (UTC) are provided by Global Positioning Systems (GPS). The prediction algorithm uses real-time bus location and bus wheel speed information, together with historical AVL data to predict the arrival time of a bus at the next traffic light. The arrival time predictor consists of two models: 1) a historical model that predicts the arrival time based solely on historical data and 2) an adaptive model that adaptively adjusts its filter gain based on the real-time AVL data that are "continuously" made available.

### 1.3.5 System Evaluation and Simulations

Using different control strategies, detection means and prediction models, system evaluation and simulation can be conducted to evaluate their cost-benefit. Stone and Wild (1982) compared level of service (LOS) and total person delay for scenarios with and without signal priority for LRT. Venglar (1995) used field data from Los Angeles, Long Beach, and Portland to calibrate and validate simulation models.

# 2. Problem Identification

With the train preemption interrupting regular traffic operations, delay and queue are expected to increase at intersections adjacent to grade crossings. The research team verified this potential problem through two approaches, including the simulation of the studied intersections and the input of the local jurisdictions. The findings from these two approaches are reported in detail in this Chapter.

## *2.1 Simulation study*

To get further insight into the potential problem, we coded a simulation network in PARAMICS to model the traffic system under the current signal operation. All field data including SPRINTER operations, traffic volumes, road characteristics as well as traffic signal locations at grade crossings, are used to calibrate the microscopic simulation model. The whole SPRINTER railroad line is about 22 miles long along the Highway 78 corridor through Oceanside, Vista, San Marcos and Escondido in San Diego's North County region. More details of simulation setups will be presented in Chapter 5.
We selected the I-5 Southbound Ramp @ Oceanside Blvd., the I-5 Northbound Ramp @ Oceanside Blvd., Enterprise @ Oceanside Blvd., Andreasen Dr. @ Oceanside Blvd., Vista Village Dr. @ Olive, Vista Village Dr. @ Santa Fe Ave., Main @ Santa Fe Ave, Pala Dr. @ Escondido Ave. and Phillips St. @ Escondido Ave. as study sites and compared the system performance of traffic operation with and without preemption impacts. The results are illustrated in the following figures.

**I-5 SB/NB Ramp @ Oceanside Blvd.**

Figure 2.1 shows the comparison results where **65.3 percent** and **69.0 percent** increases in average vehicle delay (sec) during the cycle after the preemption is attributed to the operation of SPRINTER at I-5 Southbound @ Oceanside Blvd and I-5 Northbound @ Oceanside Blvd, respectively.

**Enterprise/Andreasen Dr. @ Mission Rd.**

From Figure 2.1, we can observe that there are **23.3 percent** and **48.7 percent** increases in traffic delay per vehicle (sec) within the cycle right after the preemption, due to the impact of preemption at Enterprise @ Mission Rd. and Andreasen Dr. @ Mission Rd., respectively.

**Vista Village Dr. and Santa Fe Ave**

Figure 2.3 shows the traffic delays along intersections: Vista Village Dr. @ Olive, Vista Village Dr. @ Santa Fe Ave. and Main @ Santa Fe Ave, during the preemption and without preemption under the original signal timings.

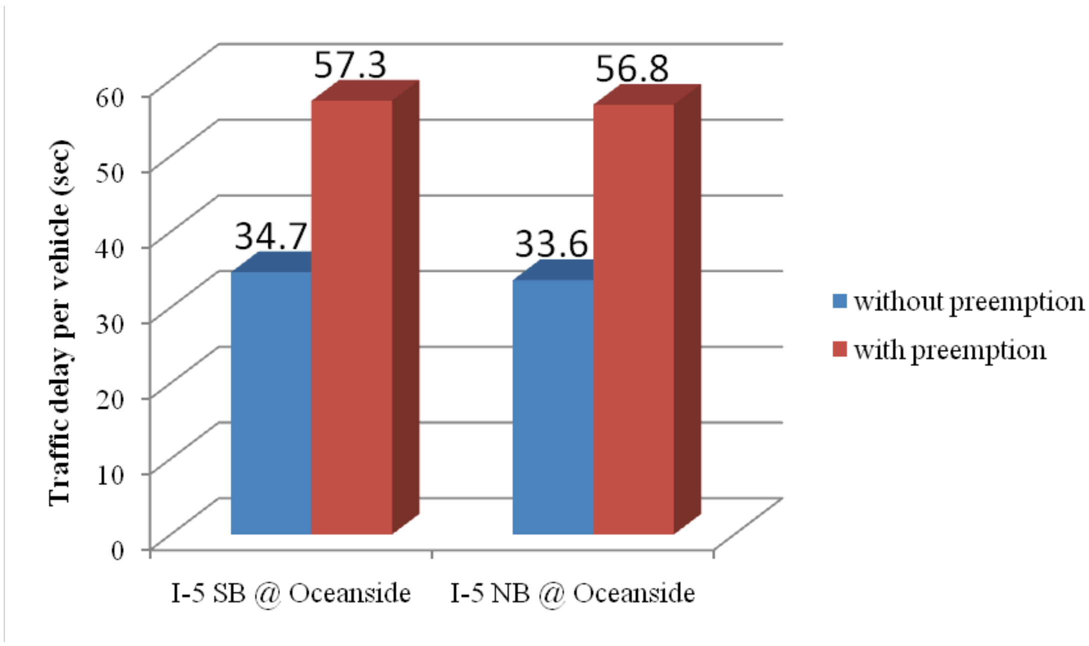



Figure 2.1 Comparison Results from Simulation with and without the Preemption

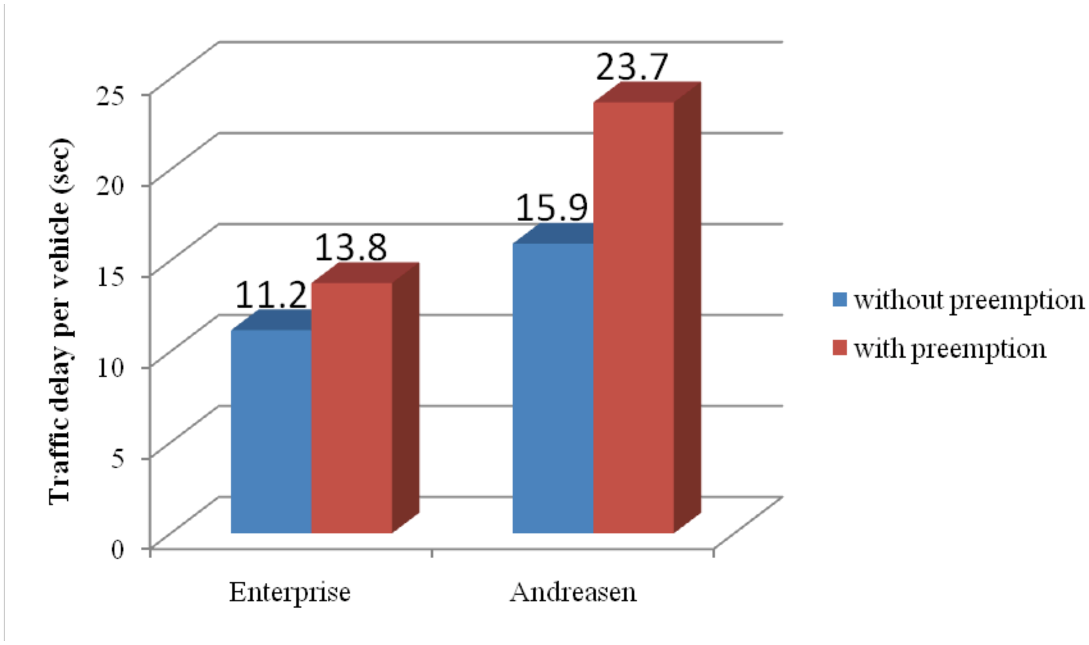



Figure 2.2 Comparison Results from Simulation with and without the Preemption

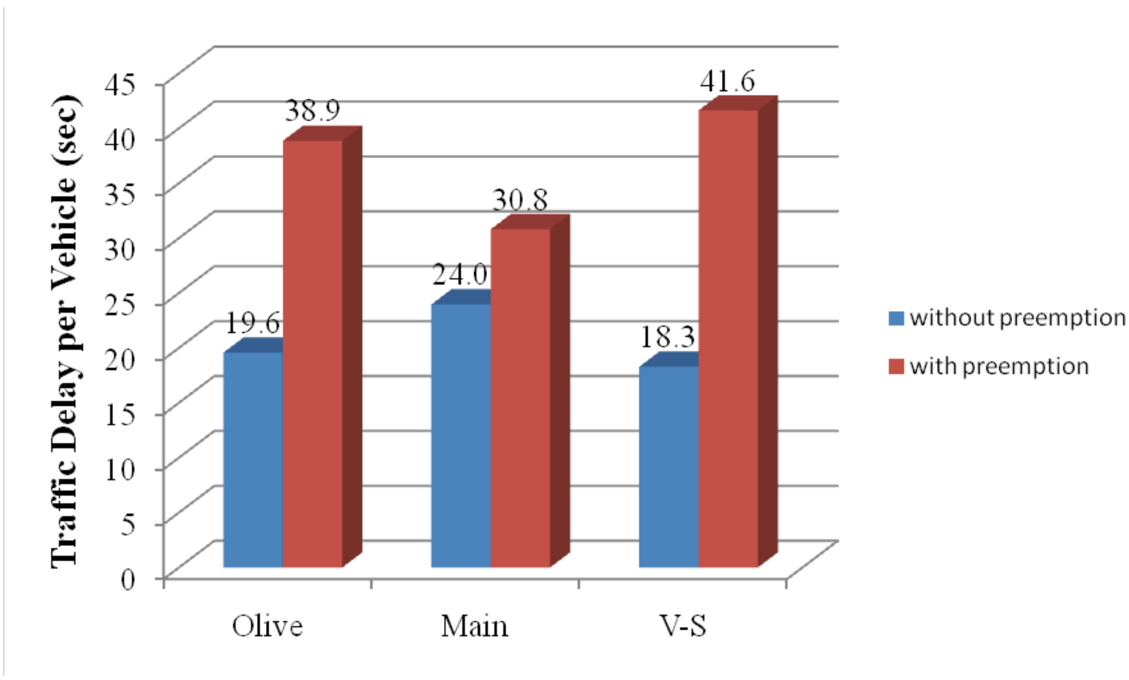



Figure 2.3 Comparison Results from Simulation with and without the Preemption

**Pala Dr./Phillips St. @ Escondido Ave**

In Figure 2.4, the interruption of SPRINTER operation is responsible for the increase of traffic delay by as large as **66.6 percent** and **29.0 percent** at Pala Dr. @ Escondido Ave. and Phillips St. @ Escondido Ave., respectively.

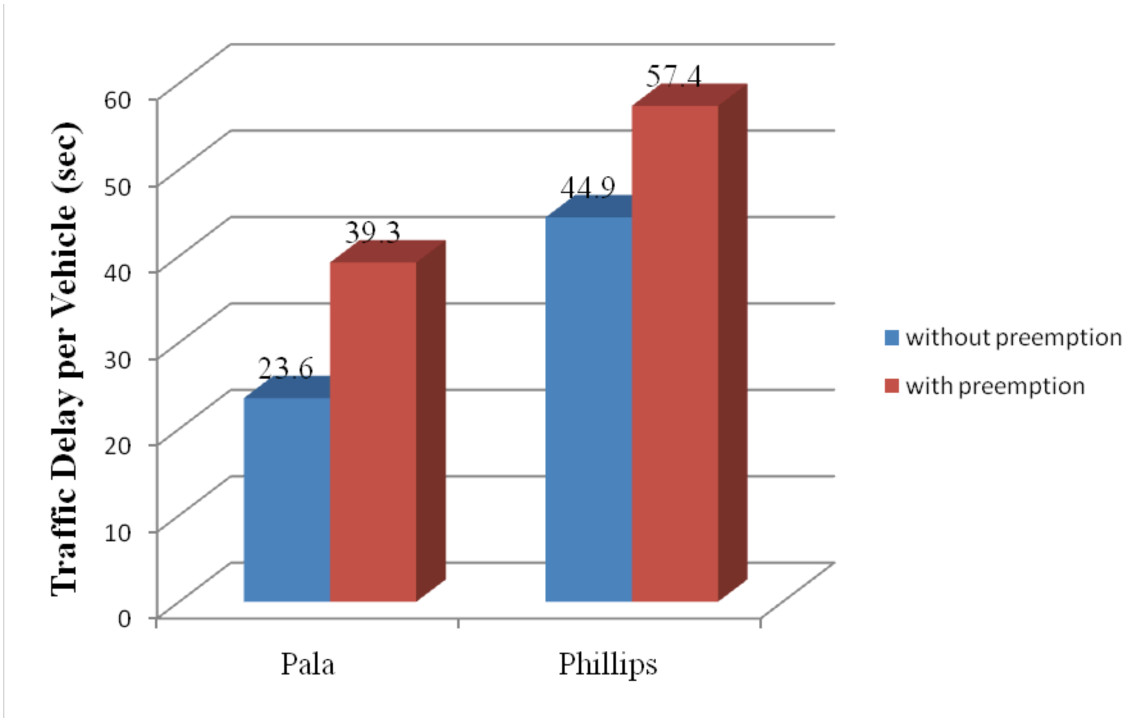



Figure 2.4 Comparison Results from Simulation with and without the Preemption

## *2.2 Inputs from Local Jurisdictions*

In November 2007, the research team scheduled meetings with traffic engineers from the City of Oceanside and City of Escondido to discuss their concerns regarding the impact of preemption operations (refer to Figures 2.3 and 2.4 for the locations of the impacted intersections). Since the track was used only for freight trains during evening hours and did not have significant impact on traffic operations, the concerns expressed were typically based on experience and an expectation of what was going to happen when SPRINTER began service. .

The main concerns were the following and are explained in detail in separate sections:

- Excessive queue and/or delay
- Impact on coordinated signal operations
- Pre-signal (Escondido)
- Pedestrian safety

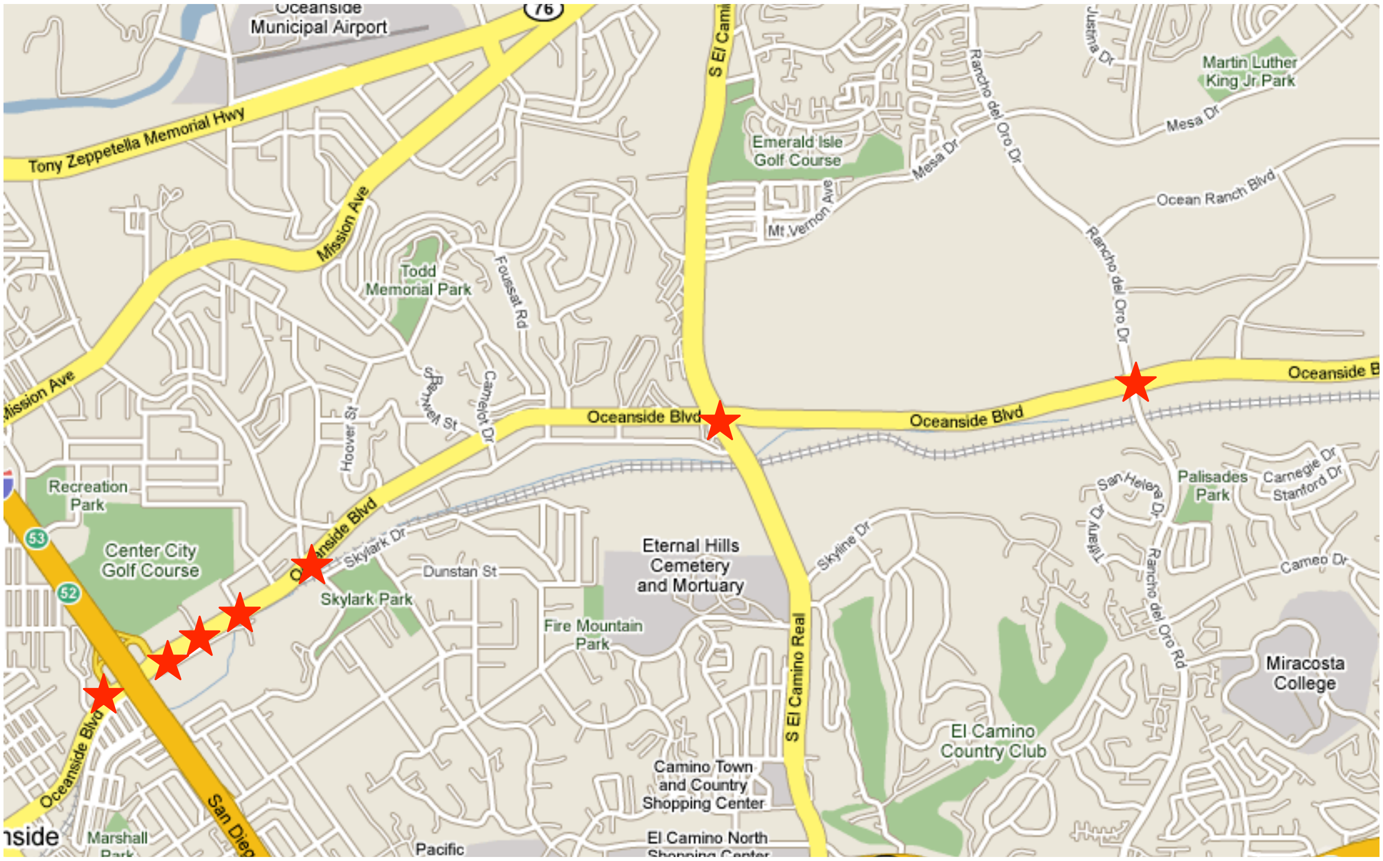



Figure 2.3 Impacted Intersections in the City of Oceanside

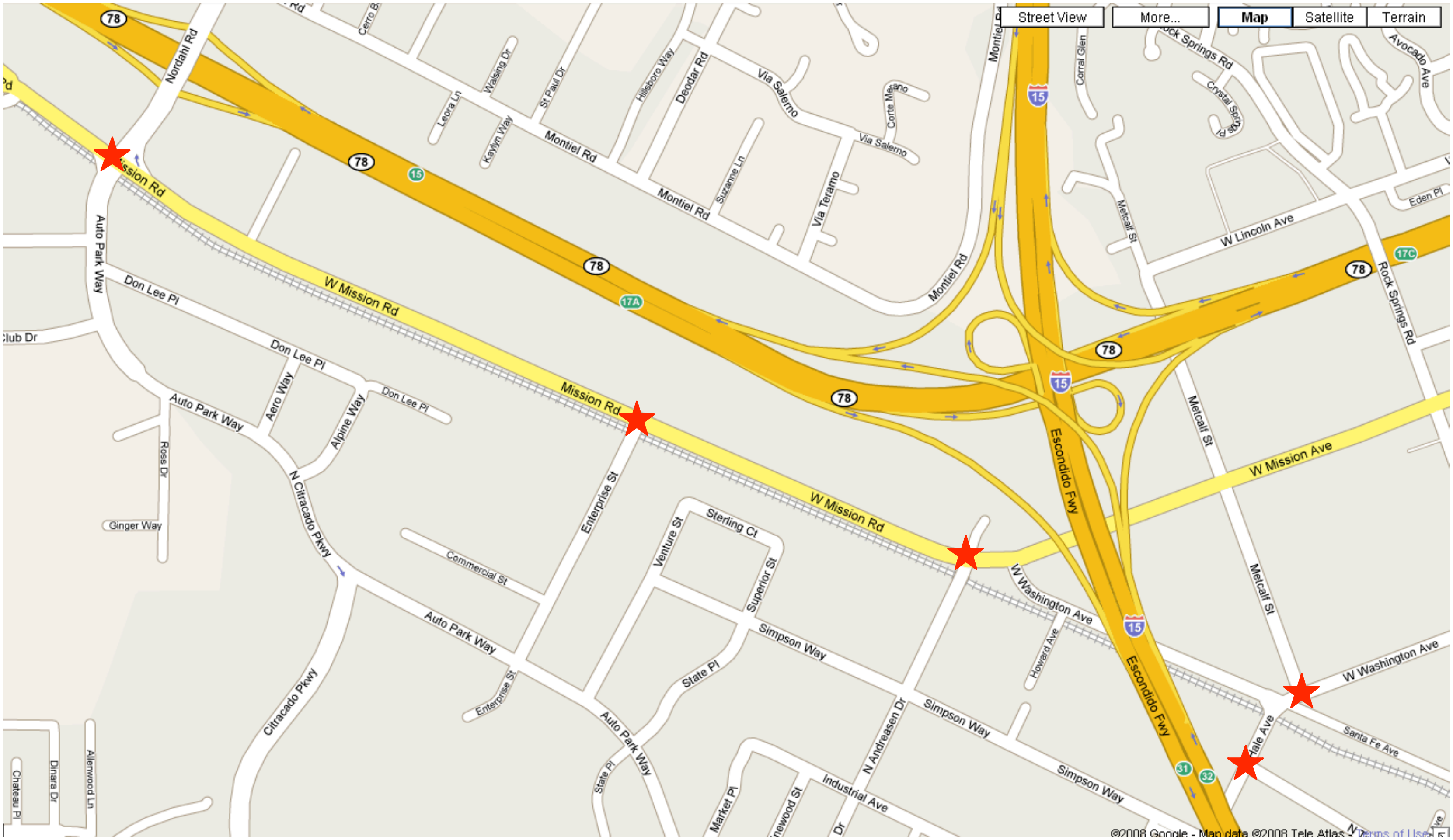



Figure 2.4 Impacted Intersections in the City of Escondido

### 2.2.1 Excessive queue and/or delay

Since the preemption service interrupts regular traffic signal operations at nearby intersections, it is intuitive that the already high-volume and congested intersections would experience excessive queuing and delay during and a couple of cycles after the preemption. This is a concern that was raised by both cities. They were also concerned with the public's reaction to the increased delay, especially during AM and PM rush hours. Related to this concern is the necessary public education regarding railroad safety at the crossings.

### 2.2.2 Impact on coordinated signal operations

Some of the intersections affected by the preemption are in a coordinated signal system (for instance, College Blvd between Oceanside Blvd and Olive Dr). It was a concern how the coordination would be affected and how much time it would take for the signal to return to coordination after preemption.

### 2.2.3 Pre-signals

Pre-signals (required by California Public Utilities Commission) are typically used when, due to horizontal or vertical alignment of the roadway, the signals at the intersection adjacent to a railroad crossing could not be viewed clearly from a distance. Under this situation, a pre-signal is installed ahead of the regular signal to inform and control traffic flowing toward the regular signal. The signal head display on the pre-signal is usually exactly the same as that shown on the related regular signal. The concern with the use of the pre-signal is for the time lost in each cycle, both at the beginning and the end of the phase serving the track crossing approach, since the vehicles will need a few seconds to travel the distance between the pre-signal and the regular signal. Figure 2.5 shows a pre-signal example near the AutoPark Way and Nordahl Road intersection in the City of Escondido. As shown in Figure 2.5a, for northbound vehicles traveling toward the railroad track, the horizontal curve prevents them from having a clear view of the signals at the railroad crossing, thus, a pre-signal is now installed at the location that was previously marked with a railroad crossing sign (Figure 2.5b).


(a) (b)

Figure 2.5 Pre-signal location at Auto Park Way and Nordahl in the City of Escondido

### 2.2.4 Pedestrian safety

Pedestrian safety, especially when the public is still unfamiliar with railroad crossing operations, was a concern expressed by both cities. Public education should be emphasized as well as some pedestrian protection that's built into the traffic signal operations during the preemption period. This is a topic that will be addressed in detail in the 2$^{nd}$ phase of this research project.

## 2.3 GPS data analysis

In March 2008, with assistance from NTCD, PATH installed eight cell phone based GPS trackers (see Figure 6.1) on SPRINTER trains. Data from six GPS trackers are continuously logged and sent back to be stored in the PATH database. The GPS data include both train GPS locations and speeds. This chapter summarizes the analysis and findings from the GPS data.

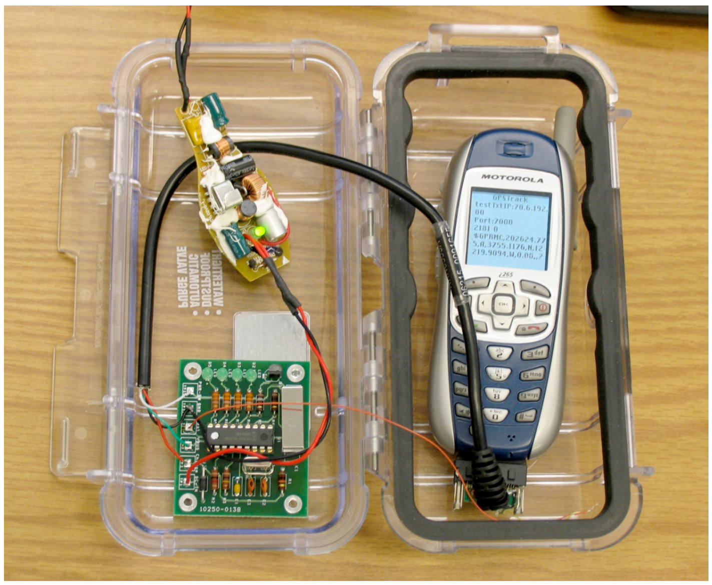
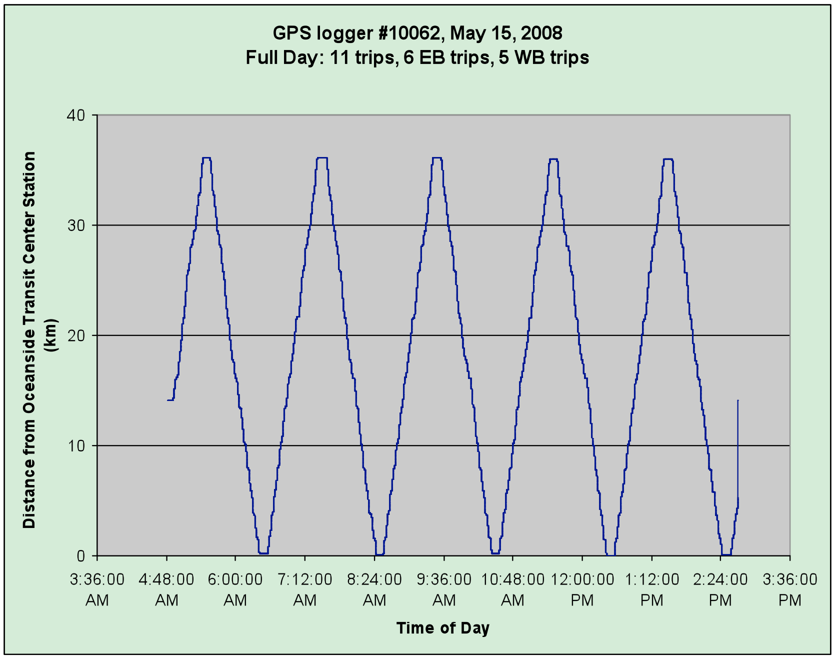


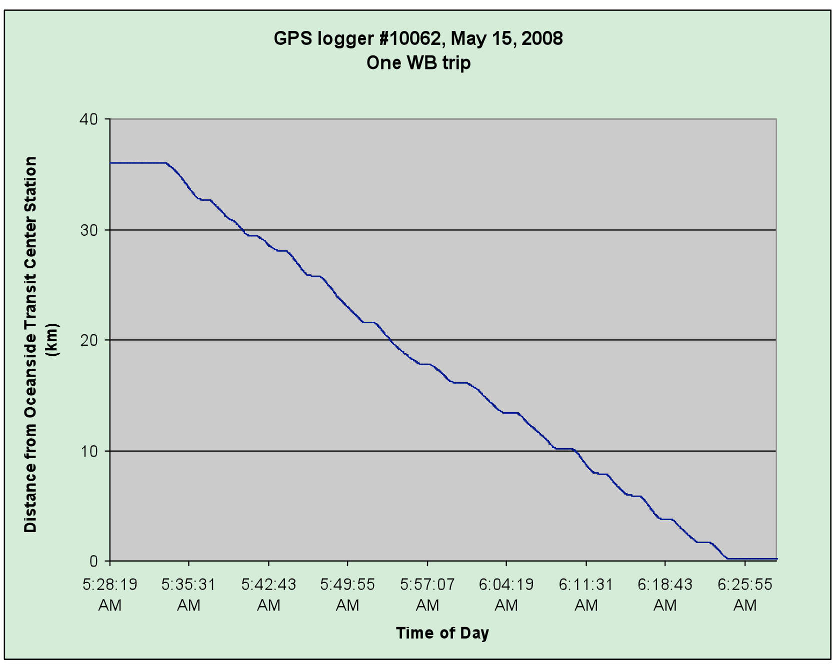

Figure 6.1 Cell phone based GPS tracker

### 2.3.1 Train Trajectory and Speed

The GPS data logged have second-to-second precision and present a detailed picture of the train trajectory. Figure 6.2(a) is a full-day train trajectory plotted based on the GPS location data from 5/16/2008. The trajectories show the number of eastbound and westbound trips made in a day and the duration of each trip. In Figure 6.1(b)), which is a zoomed-in view of one westbound trip, the details of one trip, such as station to station travel time and station dwell times could be easily determined.




a) Train trajectory for one full day




b) One WB trip trajectory

Figure 6.2 Train trajectory from GPS data

The GPS data also include train speed information. Figure 6.3 plots the train speeds for one westbound trip. The changes in train speed reflect the changes in speed limits due to the track's vertical and horizontal alignments.

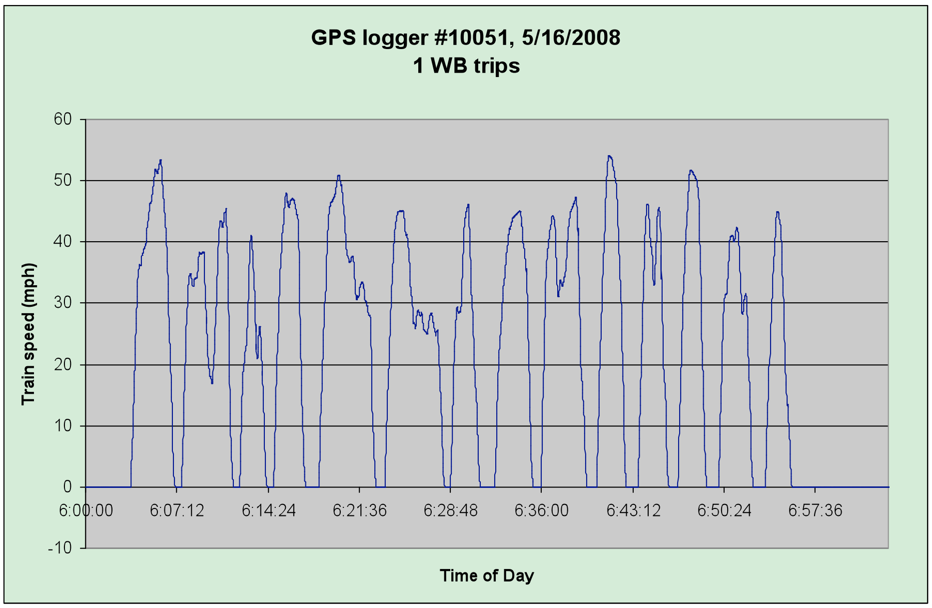



Figure 6.3 Train speed for one WB trip

### 2.3.2 Station Dwell Time

Train speed information is used to calculate the station dwell time. When the train speed was reduced to less than 5 mph, it is assumed that the train is entering a station and preparing to stop. The average station dwell time is shown in Figure 6.4. As seen from the figure, EB trains have no dwell time at the Escondido Ave. station, which is closed for boarding/alighting.

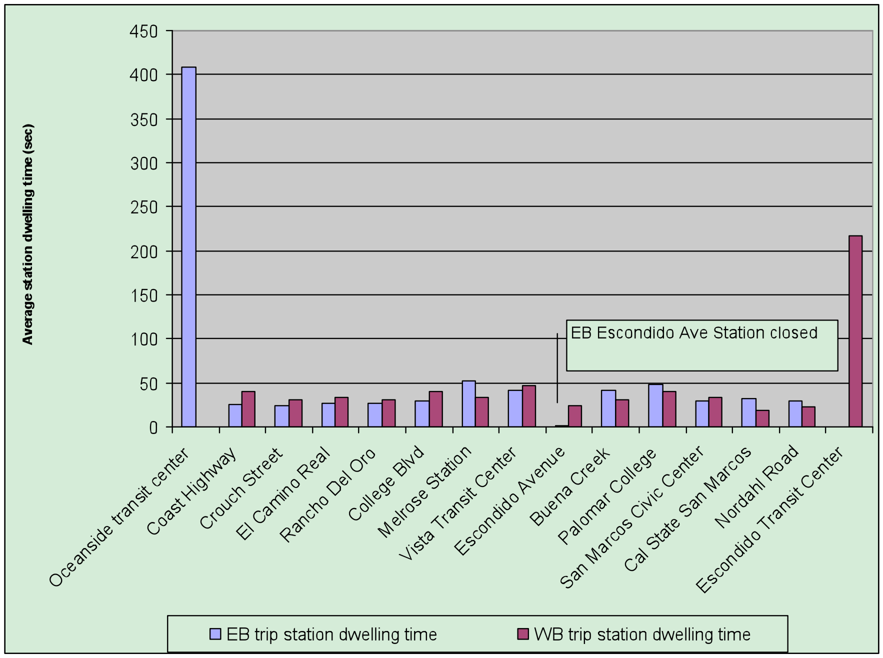
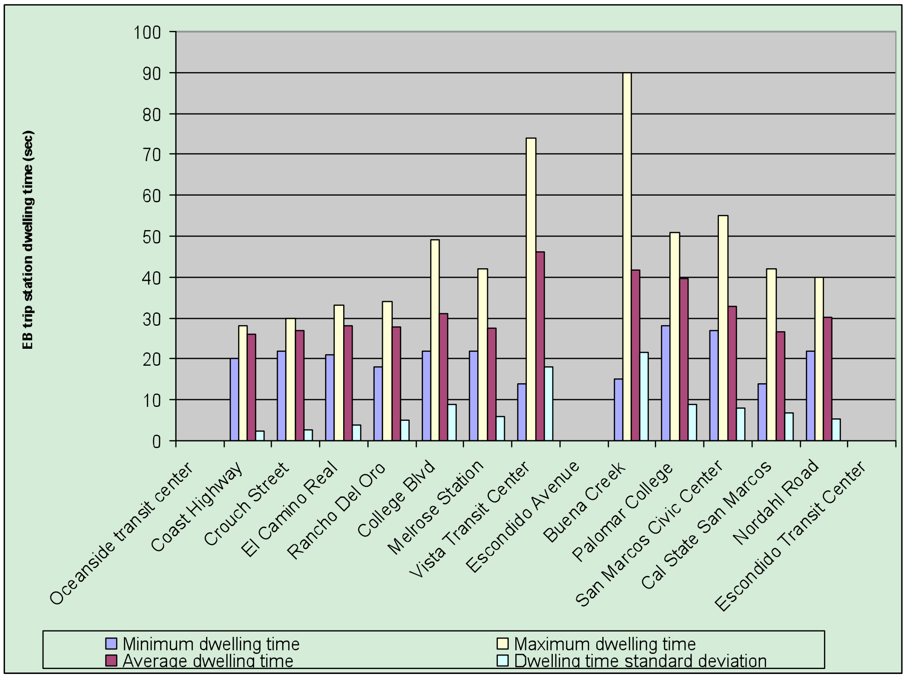
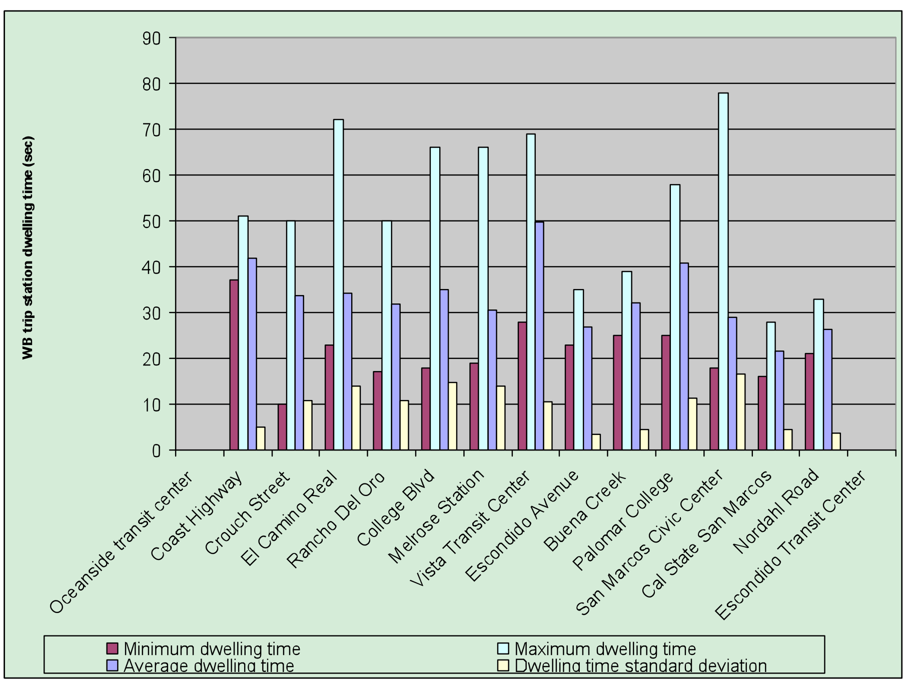



Figure 6.4 Station Dwell Time

More detailed statistics are included in Figure 6.5, which shows the minimum, maximum, average and standard deviation of the station dwell times. The statistics indicate that average station dwell time ranges from 25 to 50 seconds, and that it varies greatly from station to station and also differs by train direction. This observation is important, because currently, for all near-side stations (i.e., the station is located before the grade-crossing), 30 seconds dwell time is assumed for triggering the preemption service. Thus, if the actual dwell time is longer than 30 seconds, then the preemption would start too early, resulting in an unnecessary impact on traffic. If the actual dwell time is shorter than 30 seconds, then the preemption sequence does not have enough time to complete before the train passes the crossing (however, since the observed dwelling time is typically longer than 25 seconds, this is less of a concern than the waste of preemption time).




a) Station dwell time statistics for EB trips




c) Station dwell time statistics for WB trips

Figure 6.5 Station dwell time statistics

### 2.3.3 Future Implementation of GPS

As discussed in Section 6.2, the station dwell times observed from the detailed GPS data could be used to update the preemption parameter and thus help improve the performance of the intersections by delaying an unnecessarily early start of the preemption sequence. In addition, the current optimization algorithm only works with the cycle(s) after preemption, trying to clear the queue(s) that accumulated during the preemption period. With the advanced GPS detection, a train arrival time prediction algorithm could be proposed. With that information, an optimization algorithm could adjust the timing plans before the preemption is initiated, thereby provide better mitigation of the traffic impacts.

# 3. Development of Methodologies for Signal Timing Optimization

As has been identified in the previous chapter, the negative impacts on the motor traffic around the grade crossings due to SPRINTER operations cannot be negligible under the original traffic signal timings. To solve this problem, the approach that we proposed for the current phase of SPRINTER project is to adjust the signal timings around the grade crossings right after the preemption such that the overall intersection traffic delays can be minimized within a certain time window, and/or the queue caused by the preemption along each approach can be dissipated as quickly as possible.

To implement this method and relieve the impacts on normal traffic incurred by the interruption of the SPRINTER train, the queue length along each movement right after the preemption is first estimated. Then, the overall intersection traffic delay can be quantified based on queue length estimation. Then, a delay minimization model is formulated and solved by mathematical programming to obtain a set of optimal traffic signal timings operated within a specified time window right after the preemption at each intersection around the grade crossing. This set of traffic signal timings is aimed to relieve the traffic congestion caused by SPRINTER operations. After the congestion is mitigated, the traffic signal timing can be transitioned back to the original one or a proposed one, which depends on users' needs.

## *3.1 Data Collection*

Data collection is a key step in this project, since these data will not only serve as inputs to both the queue length estimation model and the overall intersection traffic delay minimization model, but also be used in the network construction of the simulation model. At the same time, either the numerical model or the simulation one can only be calibrated by the date we collect.

### 3.1.1 Data Type

According to the requirement, the data can be divided into the following four types:

**SPRINTER Rail/Train Data**

To construct the SPRINTER rail/train in our simulation model, it is necessary for us to obtain the train physical/dynamic parameters, operating schedule, the number and location of stations, as well as the GPS coordinates of the entire SPRINTER trackway.

**Intersection Geometry**

The geometric parameters of intersections around grade crossings, such as number of lanes along each approach, one/two way(s) are indispensable in building up numerical models and the simulation network.

**Traffic Count Data**

Traffic volumes are used in estimating the queue length right after the preemption. This measures the performance of the traffic system under either original signal timings or our proposed signal timing.

**Traffic Signal Timing**

The queue length along each approach immediately after preemption largely depends on the original traffic signal timings at the corresponding intersection near the grade crossing.

### 3.1.2 Data Source

Based on availability, there are three data sources: updated data from each jurisdiction, Google Earth/Map, the SPRINTER website and *Traffic Operations Report* from URS (2004). From documents prepared by all jurisdictions: North County Transit District (NCTD), Caltrans District 11, City of Oceanside, City of Vista, City of San Marcos, City of Escondido, and the incorporated area of San Diego County; traffic counts/ratios and signal timing plans are ready to use. Table 3.1 lists the month and/or year of the most updated data available from each jurisdiction.

Table 3.1. Month/Year of Data Available from Each Jurisdiction

| Jurisdiction | Month/Year of the Most Updated Data | |
|---|---|---|
| | **Traffic Volume** | **Signal Timings** |
| **Caltrans** | 2006 | September, 2006 |
| **City of Oceanside** | August, 2005 | June, 2007 |
| **City of Vista** | July, 2008 | July, 2008 |
| **City of San Marcos** | N/A | April, 2007 |
| **City of Escondido** | May, 2002 | August, 2007 |
| **County of San Diego** | March, 2008 | March, 2008 |

According to the GPS coordinates obtained from Google Earth/Map, we set up the simulation network including SPRINTER rail, grade crossings and intersections of interest with updated road characteristics. The train's physical and dynamic parameters, SPRINTER schedule and the number and location of stations are available from the following link http://www.gonctd.com/sprinter_intro.htm. Traffic counts at the I-5 Southbound Ramp @ Oceanside Blvd. and the I-5 Northbound Ramp @ Oceanside Blvd. are projected from the URS report.

### *3.2 Queue Length Estimation Model (QLEM)*

To quantify the traffic performance at those intersections near the grade crossing right after the preemption, we need to estimate the number of waiting vehicles along each phase. Due to the lack of real time detection on traffic counts, we can only conduct such estimation based on limited historical data. However, if the traffic signal controller is running in "free" mode at the intersection, then the queue length estimation will be very involved. Therefore, we only studied those intersections running fixed timings in the current phase of the SPRINTER project. Under the following assumptions, we proposed two sub-models to refine the queue length estimation by analyzing the data on traffic turning counts.

### 3.2.1 Assumptions

Isolated intersection is considered instead of a coordinated system;

The vehicles arrive uniformly;

The dissipation rate along each phase is a constant related to the road characteristics;

The studied intersections are running fixed timings;

The right-turn traffic can proceed on red;

1. For drivers, the lane choice rule – selecting the lane with shorter queue if not mandatory -- always holds.

Based on the assumptions mentioned above, we can expect to better estimate the number of waiting vehicles or queue length along each lane, thus along each phase, by incorporating the following sub-models.

### 3.2.2 Sub-model I: Queue split on the shared lane

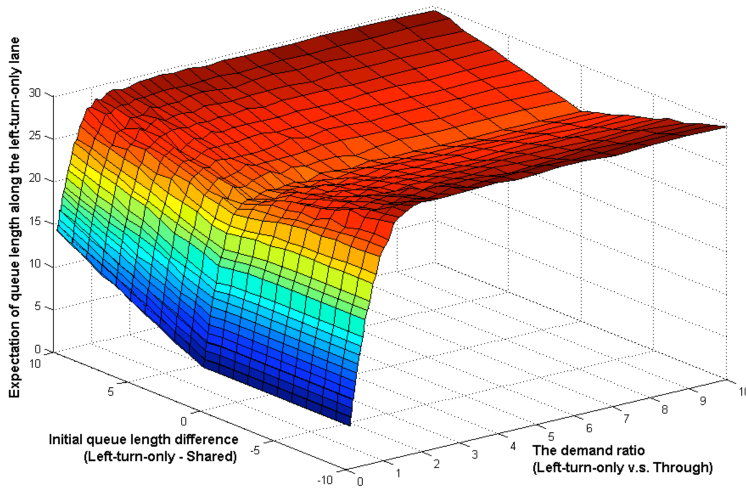
Drivers' behavior will affect the queue development along each lane in the multi-lane case. This results in the variation of the number of waiting vehicles along each phase. Since we assume that the incoming drivers can make decisions on following the queue with the shorter length when waiting for the signal, the relationship between queue split and other factors, such as traffic demand, initial queue and road characteristics, can be further examined by using the simulation model.

We used the Monte Carlo method to simulate such relationships in MATLAB. Without loss of generality, we considered the case that there are two lanes, one of which is a left-turn lane and the other is a shared lane for both left-turn and through traffic. Suppose that both lanes are long enough to accommodate the vehicles arriving within a specified time interval. In the simulation, the repetition is selected as 50 and the total incoming traffic demand is around 50 vehicles. By varying the demand ratio between left-turn and through traffic from 0.1 to 10 with a logistic step of 0.05, we present the simulation results as shown in Figure 3.1 and Figure 3.3, where the difference of initial queue lengths between the through lane and the shared one ranges from -10 to 10 with a step of 1.

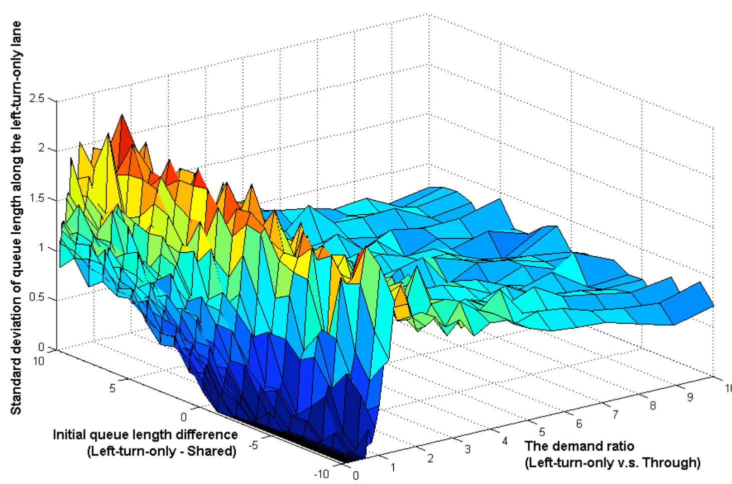
According to these figures, we can summarize our observations below:

If the incoming demand of left-turn traffic is much higher than that of through traffic, then the resulting numbers of waiting vehicles on both lanes are almost the same;

In the reverse case, the numbers of waiting vehicles along the left-turn lane and shared lane approximate the corresponding incoming demands, respectively;

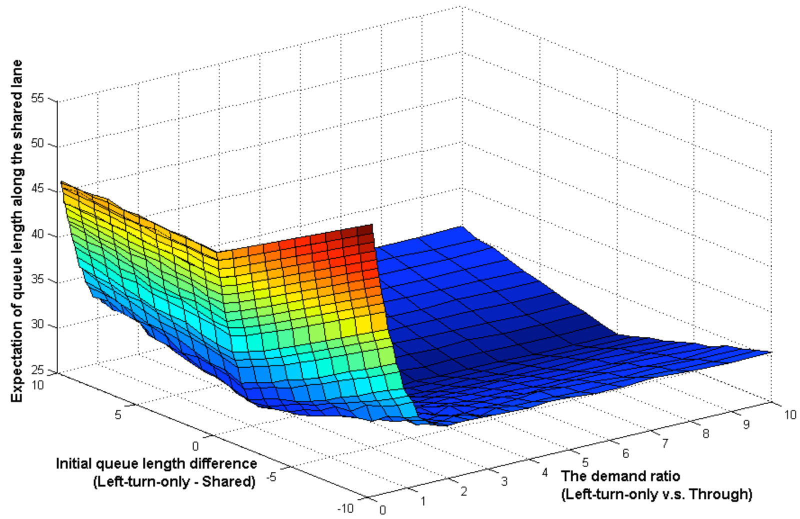



(a)

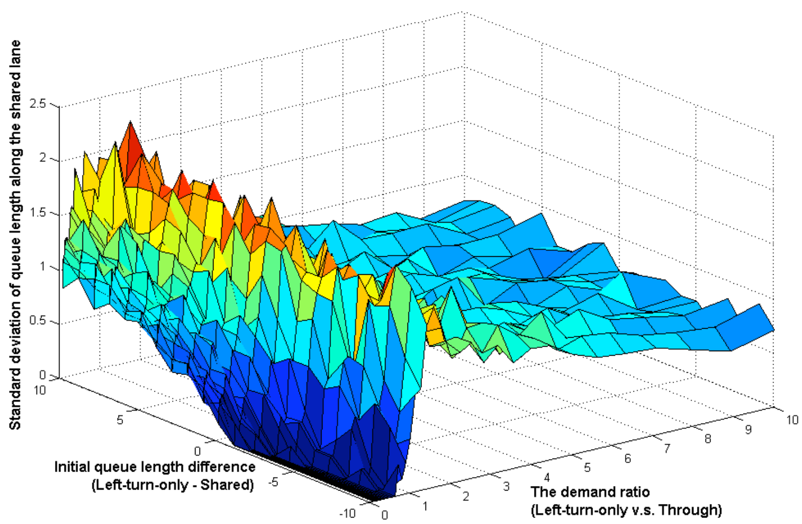



(b)

Figure 3.1 (a) Mean of the number of waiting vehicles along the left-turn lane in simulation. (b) Standard deviation of the number of waiting vehicles along the left-turn lane.

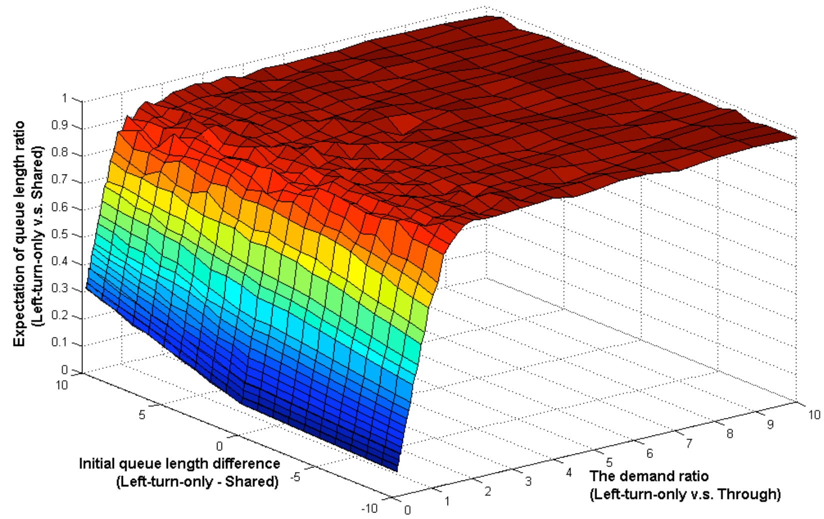



(a)

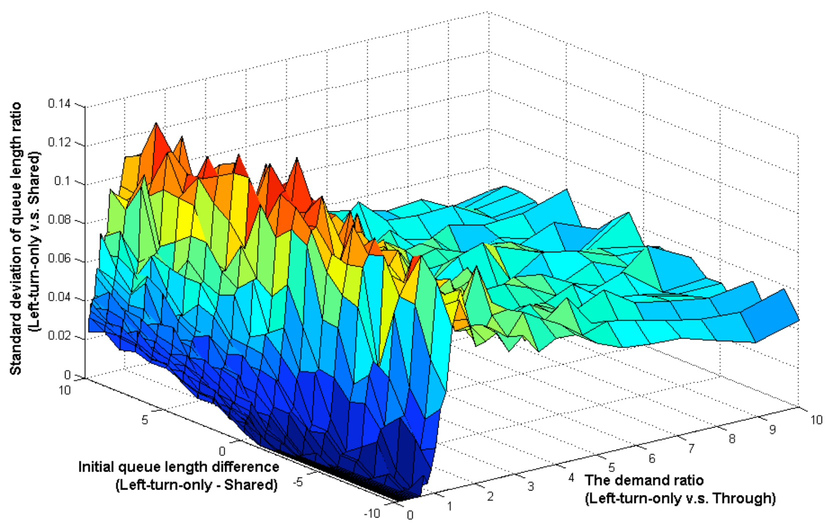



(b)

Figure 3.2 (a) Mean of the number of waiting vehicles along the shared lane in simulation. (b) Standard deviation of the number of waiting vehicles along the shared lane.




(a)




(b)

Figure 3.3 (a) Mean of the queue length ratio (left-turn v. shared). (b) Standard deviation of the queue length ratio (left-turn v. shared).

If the difference in incoming demand is trivial between through and left-turn traffic, then the difference in number of waiting vehicles between these two lanes is also trivial. In addition, the variation of queue length along each lane is noticeably greater than any of those in the two cases mentioned above;

The effect caused by the discrepancy of initial queues is remarkable when the demand of through traffic is much higher than that of left-turn traffic;

### 3.2.3 Sub-model II: Right-turn counts on the shared lane

Based on the assumption that the right-turn traffic can proceed on the right-turn permissive lane even when the phase is red, we need to quantify the number of waiting vehicles along the shared lane for both through and right-turn traffic.

Consider the case where there are $n_1$ through vehicles and $n_2$ vehicles that will make right turns forming a waiting queue along a shared lane. And the random variable, $X$, is defined as the queue index for the first vehicle that will go through the intersection. A probability model is developed to calculate the number of waiting vehicle along this shared lane, by taking into account the impact of right-turn traffic.

*Proposition 1*: The probability that the first through vehicle happens to be the $i$th vehicle along this shared queue with $n_1 + n_2$ vehicles is

$$P(X = i) = \frac{n_1 \cdot n_2! \cdot (n_1 + n_2 - i)!}{(n_2 + 1 - i)! \cdot (n_1 + n_2)!} \qquad 1 \le i \le n_2 + 1$$

*Proposition 2*: Under the assumption that all right-turn vehicles at the very beginning of the queue will make turns in red, the estimated queue length, $L$, of waiting vehicles along this shared lane is

$$E(L) = \sum_{i=1}^{n_2+1} (n_1 + n_2 + 1 - i) P(X = i)$$

The number of waiting vehicles along each phase estimated from the above two sub-models will serve as inputs into the traffic delay minimization model elaborated in the following section.

## *3.3 Delay Minimization*

Our goal is to design green splits for different phases after the preemption, such that the total intersection delay can be minimized over the controlled time period. A deterministic queue model is used for delay calculation. Before we present this model, the following assumptions should be satisfied.

### 3.3.1 Assumptions

Isolated intersection is taken into account;

The arrival rate, $a_i$ for each phase is uniform and constant;

The dissipation rate, $d_i$, is constant and relates to the road characteristics;

In most cases, the controlled time-span is one cycle after the train clears the grade-crossing, but the model can also be extended to the controlled time-span of multiple cycles.

The traffic condition is under-saturated.

Vehicles accelerate and decelerate instantaneously, which implies that all drivers behave identically, i.e. they follow average driving patterns.

In timing optimization, the sequence of phases (lead/lag relationship) keeps untapped during the controlled time-span.

The dual-ring signal controller is used for traffic control at the intersection.

The controlled time-span is a user-defined quantity, and we will present the model extension to the case for the controlled time-span of multiple cycles later in this chapter. Although the deterministic queue model implemented here does not represent normal queuing behavior and may not accurately represent the exact number of queued vehicles at a given instant, it does not bias the delay estimation process over an entire queue formation and dissipation process, and is therefore a valid simplification when only considering delay calculations. In the proposed optimization model, the actual green splits, instead of the effective green splits, are considered at a signalized intersection. However, trivial modification on the constraints of the model can be conducted, such that the effective signal intervals rather than the actual green splits can be taken into account for the additional delays due to driver reaction time and vehicle acceleration/deceleration constraints. In addition, because of the lack of detailed information on traffic, the uniform arrival rate, instead of Poisson or non-Poisson arrival distribution, is adopted to calculate the traffic delay, although the latter model can capture the randomness. In our future research, the sequence of phases will be also incorporated into decision variables for the purpose of signal operation design.

### 3.3.2 Delay quantification

Since the goal is to minimize the overall traffic delay at the signalized intersection near the grade crossing after the preemption, it is important to quantify the traffic delay. As shown in Figure 3, the shadow area represents the delay that vehicles may undergo along a certain phase within a cycle for two cases, respectively: the queue is cleared at the end of green, and the queue remains when the green terminates. Based on fundamental geometric knowledge, we can calculate the shadow area, e.g. in Figure 3.4 (a), as explained in Figure 3.5. Therefore,

$$Area_{shadow} = (Area_1 + Area_2) - (Area_3 + Area_4 + Area_5)$$

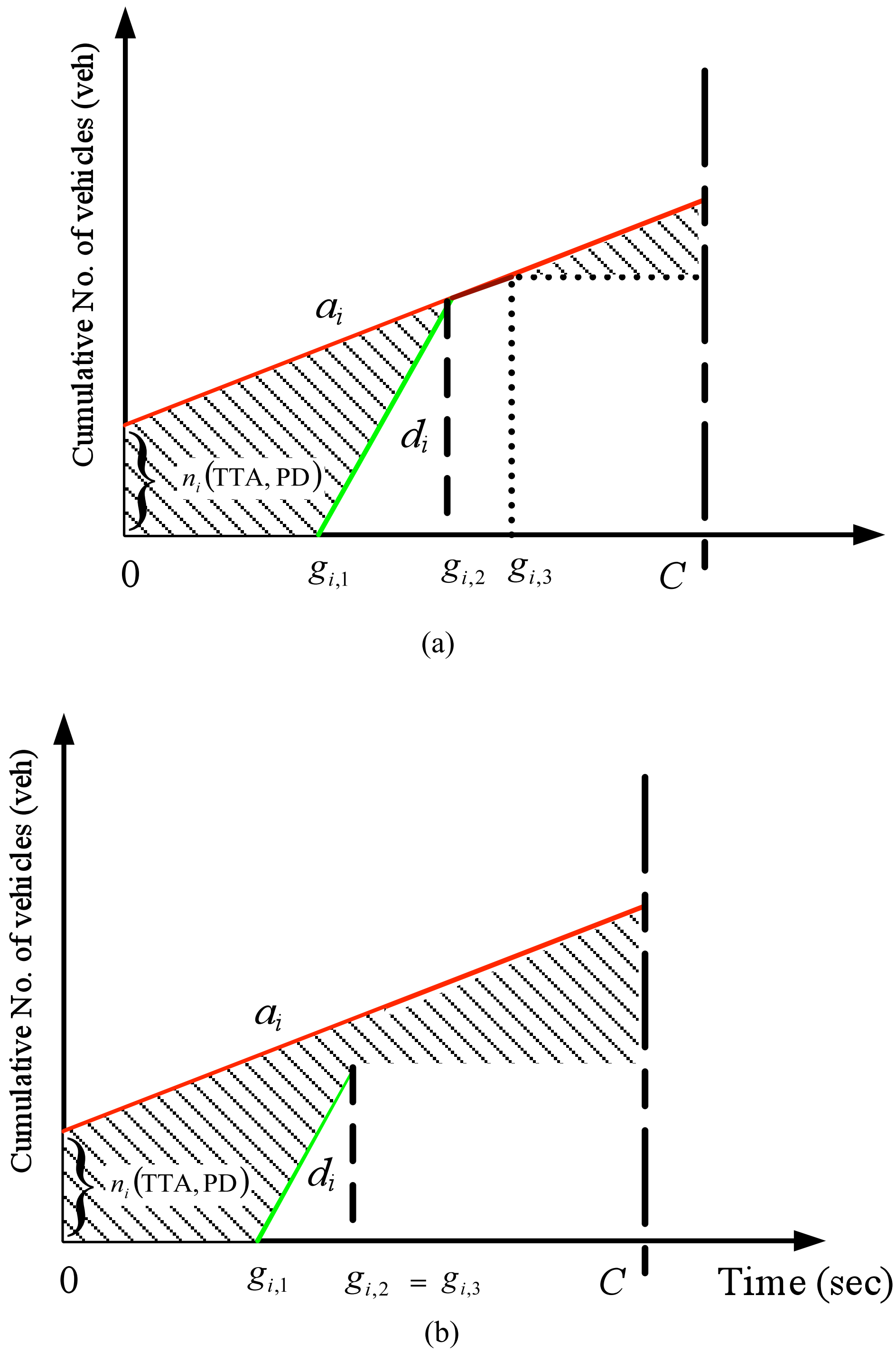
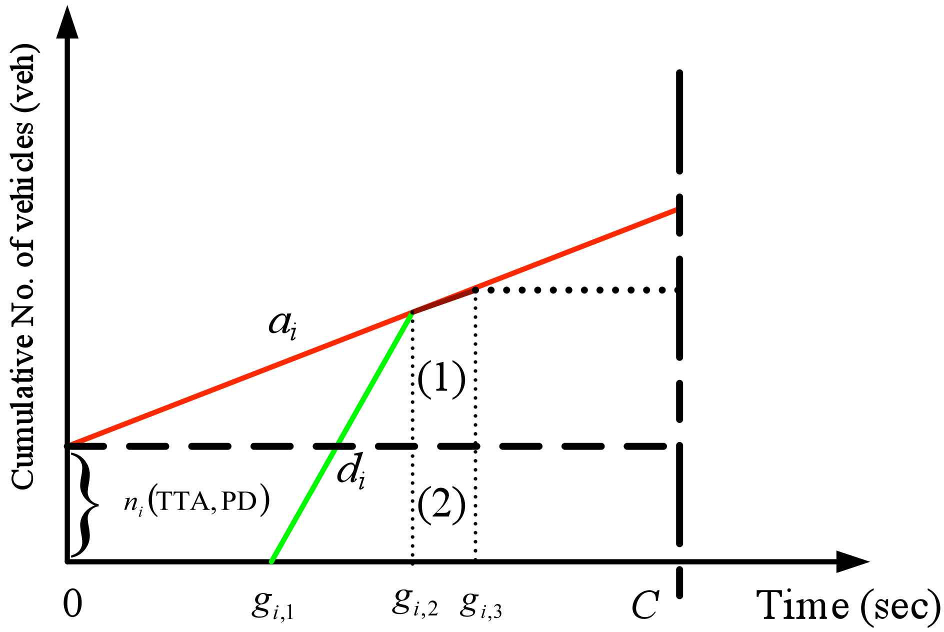
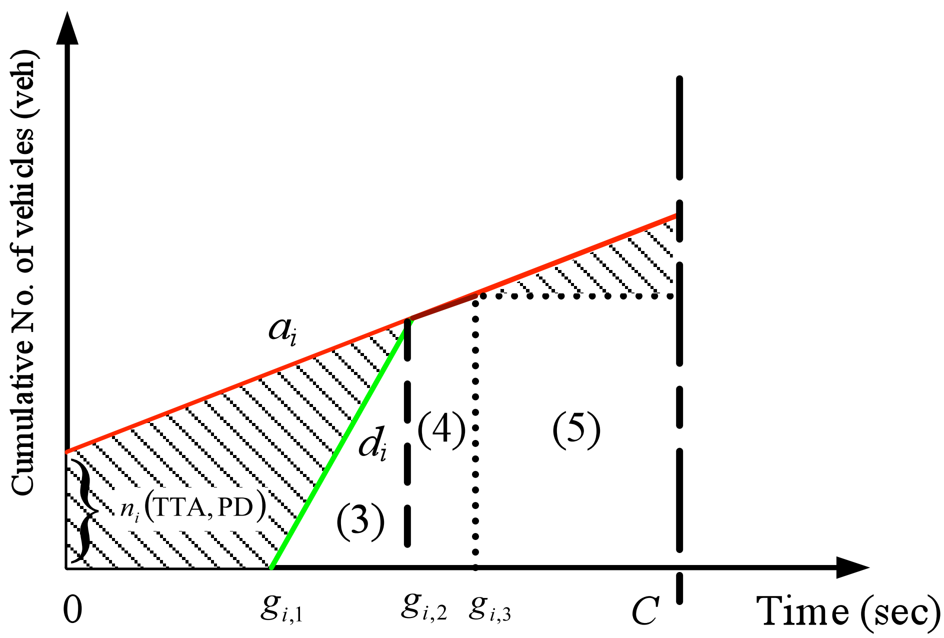



Figure 3.4. Illustration of delay calculation in two cases. (a) The queue is cleared. (b) The queue is not cleared.




(a)




(b)

Figure 3.5 Illustration of the area calculation for Figure 3.4 (a)

### 3.3.3 Constraints

Most of the constraints in the optimization model come from the mechanism of the dual-ring signal controller, such as the sequence of phases, the barrier constraint and the bound on adjustable parameters. The sequence of phases is dependent on the specific site. For example, in the model shown below, there are eight phases and the lag phases are 2, 4, 6, and 8. Modifications on constraints can be easily made for other phase sequences. For the safety issue, the designed length of each green phase should not exceed the maximum green, but must be longer than the minimum green requirement.

### 3.3.4 Mixed Integer Quadratic Programming (MIQP) model

Combining the delay calculation as well as the constraints, we formulate a mixed integer quadratic programming (MIQP) model to minimize the overall traffic delay at the intersection near the grade crossing after the preemption.

$$\min \sum_{j=1,2} \sum_{i \in D_j} \Big\{ n_i(TTA,PD) \cdot C + \tfrac{1}{2} \cdot a_i \cdot C^2 - \tfrac{1}{2} \cdot d_i \cdot (g_{i,2} - g_{i,1})^2 - \tfrac{1}{2} \cdot [2 \cdot d_i \cdot (g_{i,2} - g_{i,1}) + a_i \cdot (g_{i,3} - g_{i,2})] \cdot (g_{i,3} - g_{i,2}) - [d_i \cdot (g_{i,2} - g_{i,1}) + a_i \cdot (g_{i,3} - g_{i,2})] \cdot (C - g_{i,3}) \Big\}$$

subject to

$$n_i(TTA,PD) - d_i \cdot (g_{i,2} - g_{i,1}) \le 0 \qquad i \in D_j \text{ and } j = 1,2 \tag{1}$$

$$d_i \cdot (g_{i,2} - g_{i,1}) - [n_i(TTA,PD) + a_i \cdot g_{i,2}] \le 0 \qquad i \in D_j \text{ and } j = 1,2 \tag{2}$$

$$g_{i,3} - g_{i,1} - G_i^{max} \le 0 \qquad i \in D_j \text{ and } j = 1,2 \tag{3}$$

$$g_{i,1} - g_{i,3} + G_i^{min} \le 0 \qquad i \in D_j \text{ and } j = 1,2 \tag{4}$$

$$g_{p_j^{k+1},1} - g_{p_j^k,3} - y_{p_j^k} - r_{p_j^k} = 0 \qquad j = 1,2, \text{ and } k = 1,2,3 \tag{5}$$

$$g_{p_j^1,1} - y_{p_j^4} - r_{p_j^4} = 0 \qquad j = 1,2 \tag{6}$$

$$g_{p_1^1,1} - g_{p_2^1,1} = 0 \qquad \text{and} \qquad g_{p_1^3,1} - g_{p_2^3,1} = 0 \tag{7}$$

$$g_{p_j^4,3} - C = 0 \qquad j = 1,2 \tag{8}$$

$$0 \le g_{i,1} \le g_{i,2} \le g_{i,3} \le C \qquad i \in D_j \; and \; j = 1,2 \tag{9}$$

Constraint (1) guarantees that vehicles will not wait for more than one cycle, and constraint (2) represents the restriction on the value that $g_{i,2}$ can take. Constraints (3) and (4) relate the safety concerns on minimum and maximum green for each phase. Constraints (5) – (6) are the connectivity (sequence) condition for phases in each ring, where $p_j^k$ means the *k*-th phase in the *j*-th ring. Constraint (7) represents the barrier condition for the dual ring signal controller, which means that phase(s) must terminate their timing and cross the "barrier" together. Constraint (8) ensures the cycle length will not change. The last constraint shows the upper bound and lower bound for each decision variable, where $g_{i,1}'s$ and $g_{i,3}'s$ must be integers.

### 3.3.5 Multiple-Cycle Optimization

When the traffic volumes along the coordinated phases increase and the preemption duration is too long, we might not obtain a feasible solution if we apply the optimization model mentioned in previous sections. This infeasibility is due to the ambition to clear the queue that is backed up during preemption within a single cycle. By modification, we can obtain a more generalized MIQP model, i.e. a multi-cycle optimization model. The queue does not necessarily have to be cleared up within one cycle, but within $m(\geq 1)$ cycles, where $m$is a user-defined value. In addition, it is evident that the optimization model presented above is a special case of this multi-cycle optimization model when $m = 1$.

$$\min \sum_{j=1,2} \sum_{k \in D_j} \Big\{ n_k(TTA,PD) \cdot m \cdot C + \frac{1}{2} \cdot a_k \cdot (m \cdot C)^2$$
$$- \Big\{ \sum_{i \in M} \frac{1}{2} \cdot d_k \cdot \left(g_{i,k,2} - g_{i,k,1}\right)^2 + \frac{1}{2}$$
$$\cdot \left[2 \cdot d_k \cdot \left(g_{i,k,2} - g_{i,k,1}\right) + a_k \cdot \left(g_{i,k,3} - g_{i,k,2}\right)\right] \cdot \left(g_{i,k,3} - g_{i,k,2}\right)$$
$$+ \left[d_k \cdot \left(g_{i,k,2} - g_{i,k,1}\right) + a_k \cdot \left(g_{i,k,3} - g_{i,k,2}\right)\right] \cdot \left(m \cdot C - g_{i,k,3}\right) \Big\} \Big\}$$

subject to

$$n_k(TTA,PD) + a_k \cdot (i-1) \cdot C - d_k \cdot \sum_{l=1}^{i} \left(g_{l,k,2} - g_{l,k,1}\right) \leq 0$$
$$\forall i \in M, j = 1,2, and\ k \in D_j \qquad (1)$$

$$d_k \cdot \sum_{l=1}^{i} \left(g_{l,k,2} - g_{l,k,1}\right) - \left[n_k(TTA,PD) + a_k \cdot g_{i,k,2}\right] \leq 0$$
$$\forall i \in M, j = 1,2, and\ k \in D_j \qquad (2)$$

$$g_{i,k,3} - g_{i,k,1} - G_k^{max} \leq 0 \qquad \forall i \in M, j = 1,2, and\ k \in D_j \qquad (3)$$

$$g_{i,k,1} - g_{i,k,3} + G_k^{min} \leq 0 \qquad \forall i \in M, j = 1,2, and\ k \in D_j \qquad (4)$$

$$g_{i,p_j^{k+1},1} - g_{i,p_j^k,3} - y_{p_j^k} - r_{p_j^k} = 0$$
$$\forall i \in M, j = 1,2, and\ k = 1,2,3 \qquad (5)$$

$$g_{i,p_j^1,1} - g_{i-1,p_j^4,3} - y_{p_j^4} - r_{p_j^4} = 0$$
$$\forall i \in M, and\ j = 1,2 \qquad (6)$$

$$g_{0,p_j^4,3} = 0 \qquad \forall j = 1,2 \qquad (7)$$

$$g_{i,p_1^1,1} - g_{i,p_2^1,1} = 0 \quad \text{and} \quad g_{i,p_1^3,1} - g_{i,p_2^3,1} = 0 \qquad \forall i \in M \qquad (8)$$

$$g_{i,p_j^4,3} - i \cdot C = 0 \qquad \forall i \in M, and\ j = 1, 2 \tag{9}$$

$$(i-1) \cdot C \leq g_{i,k,1} \leq g_{i,k,2} \leq g_{i,k,3} \leq i \cdot C \qquad \forall i \in M, j = 1, 2, and\ k \in D_j \tag{10}$$

## 3.4 Optimization Strategy

In summary, the basic flow diagram of the proposed strategy is shown in Figure 3.6,

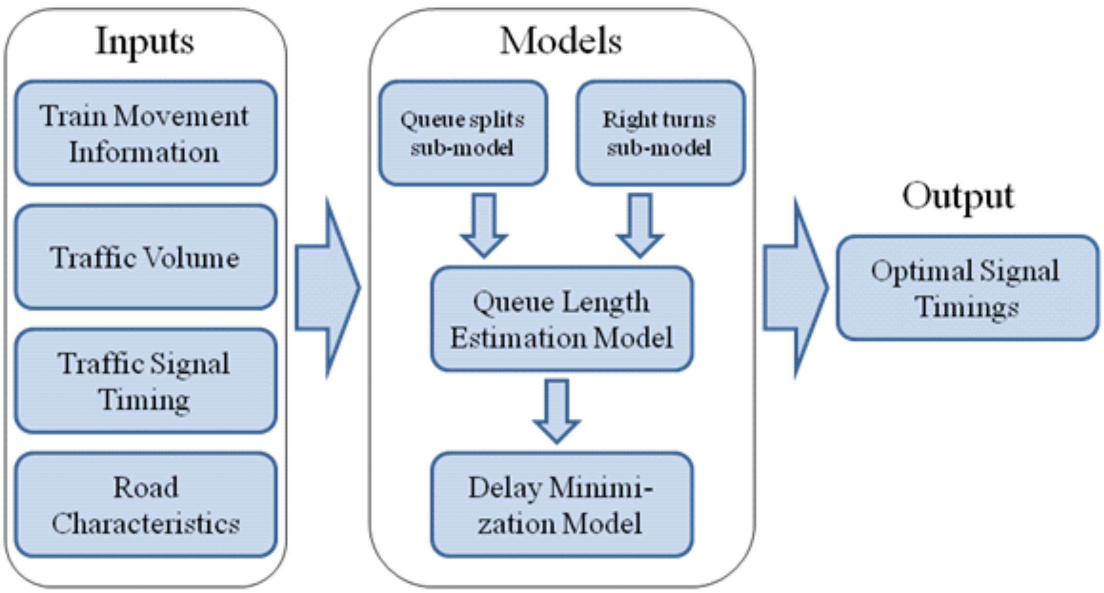



Figure 3.6 The Proposed Optimization Strategy.

## 3.5 Tool

Since the delay minimization model is formulated into a mixed integer quadratic programming (MIQP), we code in LINDO API 5.0 to solve it and obtain the optimal signal timings, i.e. green splits of each phase at the studied sites. Such optimal timings vary with different combination of the train check-in time (with regard to the Local Clock) and the preemption duration.

The following symbols are used in this chapter

$\mathrm{M}$ = The cycle index set, i.e. $\mathrm{M} = \{1, 2, \ldots, \mathrm{m}\}$.

$\mathrm{D}_1$ = The phase set of the first ring in the dual ring signal controller in our case study, $D_1 = \{3,4,1,2\}$.

$D_2$ = The phase set of the first ring in the dual ring signal controller in our case study, $D_2 = \{7,8,5,6\}$.

$C$ = The cycle length (sec).

$TTA$ = The time of the local clock when the train triggers the preemption, or the preemption initiation time (sec).

$PD$ = Preemption duration (sec).

$n_i(\cdot,\cdot)$ = The number of waiting vehicles along the ith phase after the preemption, it is a function of $TTA$ and $PD$ (veh).

$a_i$ = The arrival rate of traffic along the ith phase (veh/sec).

$d_i$ = The departure rate of traffic along the ith phase (veh/sec).

$g_{i,1}$ = The green start along the ith phase on the local clock (sec).

$g_{i,2}$ = The green clear point along the ith movement on the local clock (sec). If the queue is cleared, then $g_{i,2} = [n_i(TTA, PD) + d_i \cdot g_{i,2}]/(d_i - a_i)$, else, $g_{i,2} = g_{i,3}$.

$g_{i,3}$ = The green end along the ith phase on the local clock (sec).

$g_{i,j,1}$ = The green start along the jth phase on the local clock in the ith cycle after the preemption (sec).

$g_{i,j,2}$ = The green clear point along the jth phase on the local clock in the ith cycle after the preemption (sec).

$g_{i,j,3}$ = The green end along the jth phase on the local clock in the ith cycle after the preemption (sec).

$G_i^{max}$ = The maximum green along the ith phase on the local clock (sec).

$G_i^{min}$ = The minimum green along the ith phase on the local clock (sec).

$y_i$ = The yellow duration along the ith phase on the local clock (sec).

$r_i$ = The red clearance along the ith phase on the local clock (sec).

$p_j^k$ = The k-th phase in the j-th ring.

# 4. Evaluation of Effectiveness and Benefits

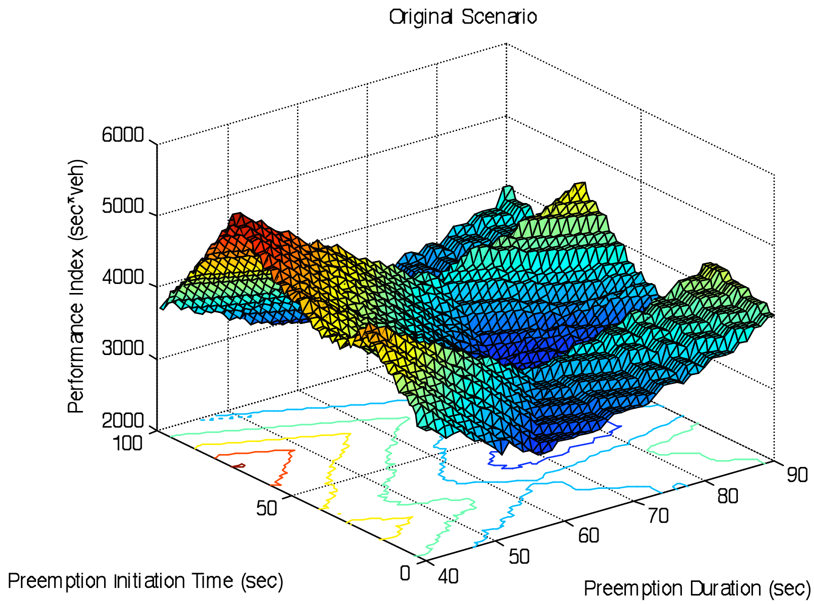
Both numerical analysis and microscopic simulation were conducted to evaluate the effectiveness and benefits of the proposed signal timing optimization strategy.

## *4.1 Numerical Analysis*

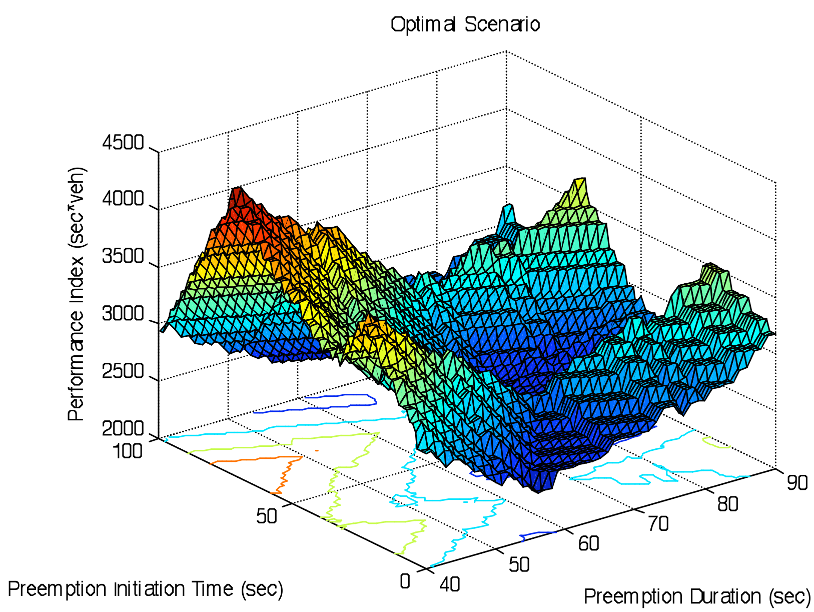
Based on the strategy that we proposed in Chapter 3, we did the numerical analysis for the following nine intersections: I-5 Southbound Ramp @ Oceanside Blvd (Caltrans), I-5 Northbound Ramp @ Oceanside Blvd (Caltrans), College Ave @ Oceanside Blvd (City of Oceanside), Enterprise @ Mission Rd. (City of Escondido), and Andreasen Ave @ Mission Rd. (City of Escondido), Vista Village Dr. @ Olive (City of Vista), Vista Village Dr. @ Santa Fe Ave. (City of Vista), Main @ Santa Fe Ave. (City of Vista), Pala Dr. @ Escondido Ave. (City of Vista) and Phillips St. @ Escondido Ave. (City of Vista)
Due to the preemption logic, the queue length along each phase may vary with different time-to-arrival (TTA) estimates of the SPRINTER train on the Local Clock and different preemption duration (PD) under the assumption of uniformly deterministic traffic arrival and dissipation rates. Therefore, the optimal timings at each intersection may also vary with both the preemption initiation time on the Local Clock and the preemption duration. In the numerical analysis, we compared the performance index, i.e. the overall intersection delay within one cycle right after the preemption, between the scenario under the original signal timings and the one under the proposed signal timings. Then we investigated the numerical analysis results site by site and illustrated them by using a 3-D diagram.

### 4.1.1 I-5 Southbound Ramp @ Oceanside Blvd

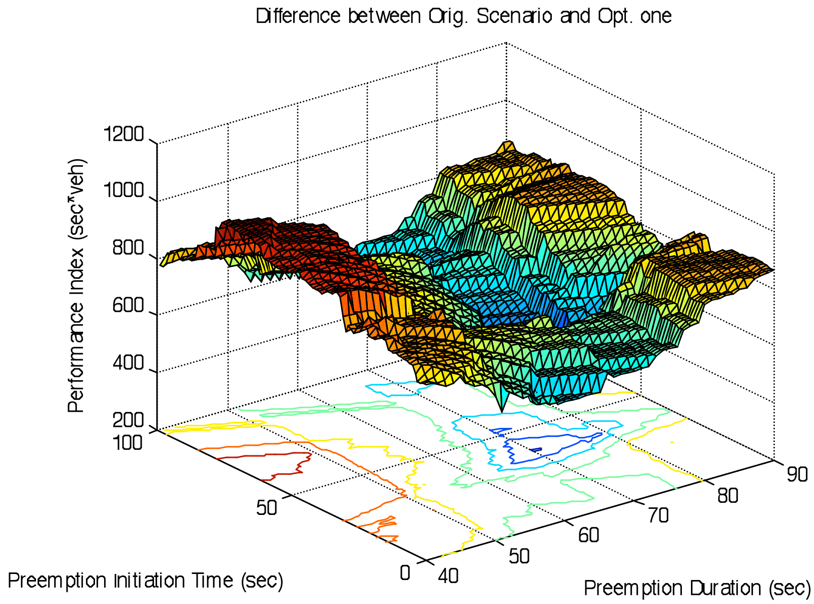
As is shown in Figure 4.1, by implementing the optimal green splits, not only can the queue along each phase can be cleared within one cycle after the preemption, but also the overall intersection delays can be reduced by as much as 24 percent at the I-5 Southbound Ramp @ Oceanside Blvd. Furthermore, we can observe a 20.2 percent reduction in traffic delay per vehicle on average during the cycle after preemption.


Original Scenario
Performance Index (sec*veh)
6000
5000
4000
3000
2000
100
50
0
Preemption Initiation Time (sec)
40
50
60
70
80
90
Preemption Duration (sec)

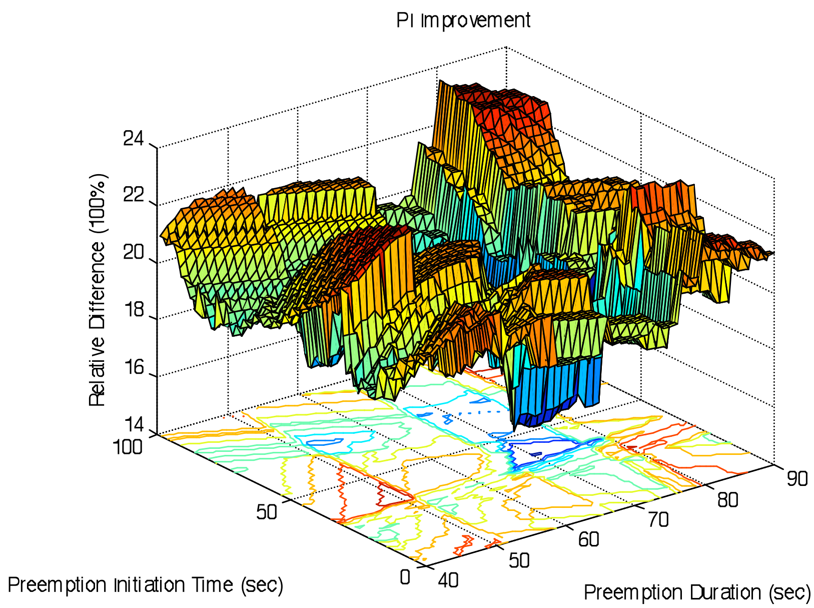
(a)


Optimal Scenario
Performance Index (sec*veh)
4500
4000
3500
3000
2500
2000
100
50
0
Preemption Initiation Time (sec)
40
50
60
70
80
90
Preemption Duration (sec)

(b)




(c)

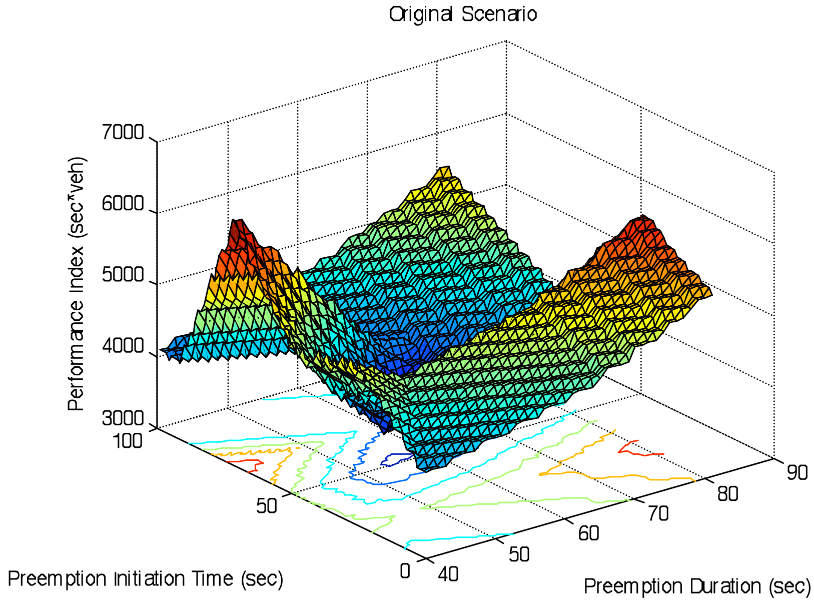



(d)

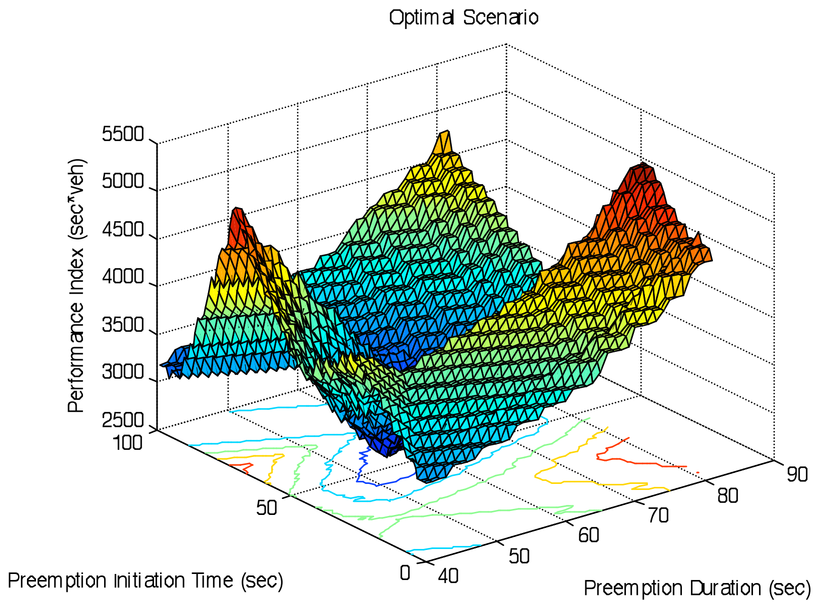
Figure 4.1 Performance index comparison at I-5 SB ramp @ Oceanside Blvd. (a) Performance index using the original signal timings. (b) Performance index using the proposed optimal signal timings. (c) Difference of performance index between the original scenario and the optimal one. (d) Relative performance index improvement.

### 4.1.2 I-5 Northbound Ramp @ Oceanside Blvd

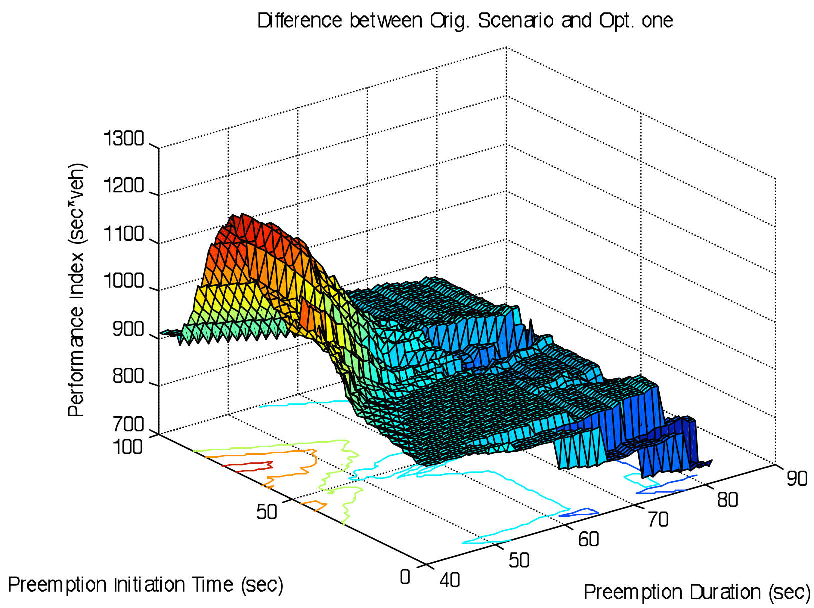
Similarly, from the numerical analysis, as much as a 25 percent improvement in the performance index can be witnessed at I-5 Northbound Ramp @ Oceanside Blvd. if the proposed traffic signal timings are conducted instead of the original ones. Moreover, the results shown in Figure 4.2. indicate that there is, on average, approximately a 19.3 percent improvement in the performance index.



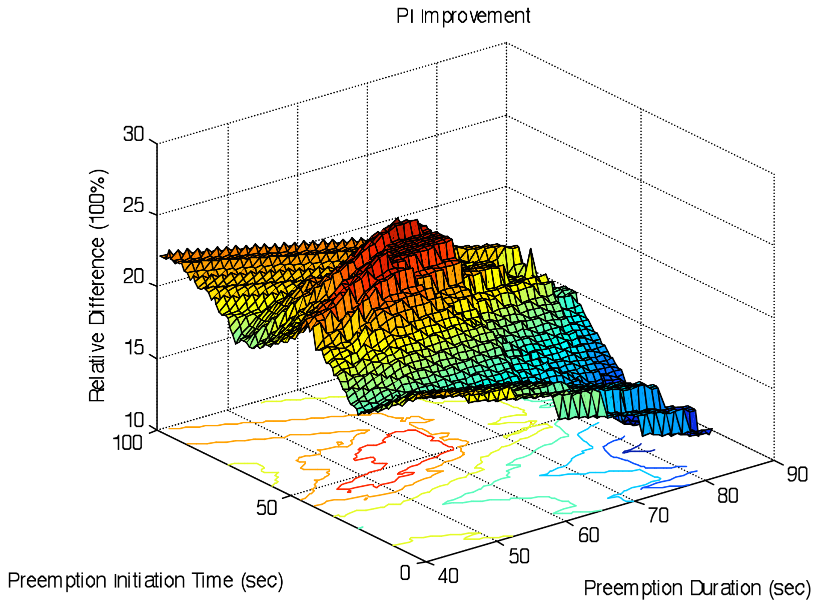

(a)


Optimal Scenario
5500
5000
4500
4000
3500
3000
2500
Performance Index (sec*veh)
100
50
0
Preemption Initiation Time (sec)
40
50
60
70
80
90
Preemption Duration (sec)

(b)


Difference between Orig. Scenario and Opt. one
1300
1200
1100
1000
900
800
700
Performance Index (sec*veh)
100
50
0
Preemption Initiation Time (sec)
40
50
60
70
80
90
Preemption Duration (sec)

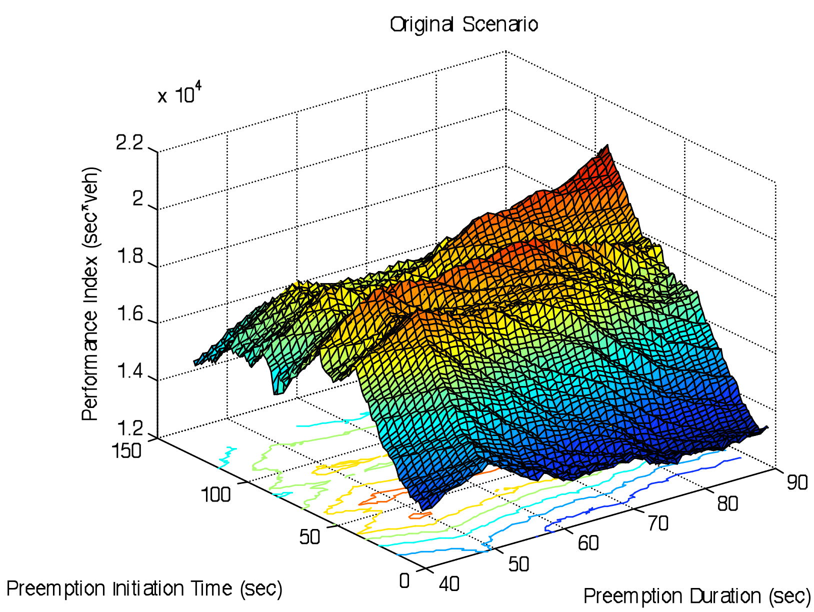
(c)



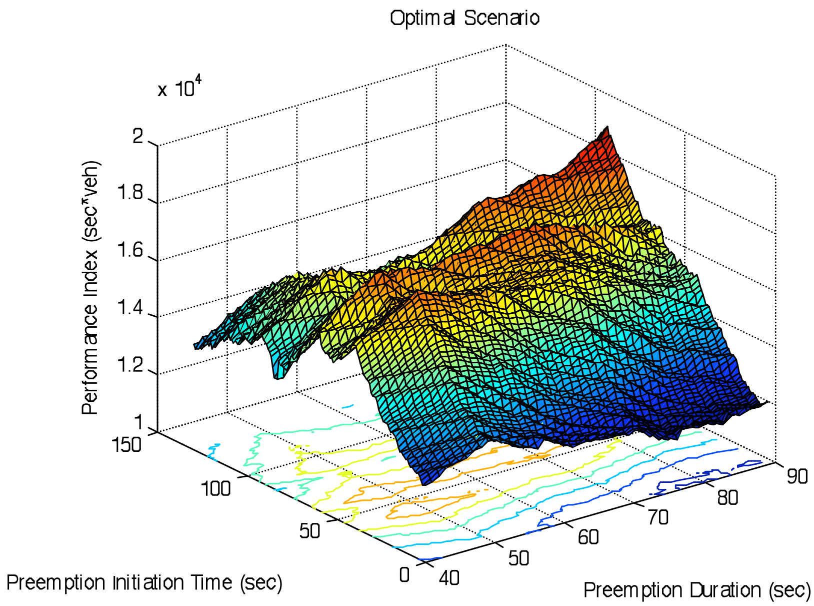

(d)

Figure 4.2 Performance index comparison at I-5 NB ramp @ Oceanside Blvd. (a) Performance index using the original signal timings. (b) Performance index using the proposed optimal signal timings. (c) Difference of performance index between the original scenario and the optimal one. (d) Relative performance index improvement.

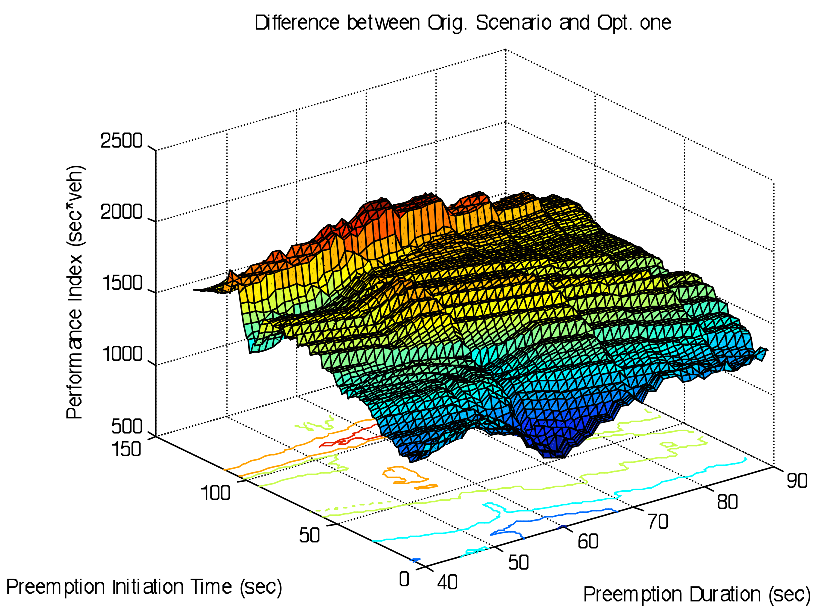
### 4.1.3 College Ave @ Oceanside Blvd.



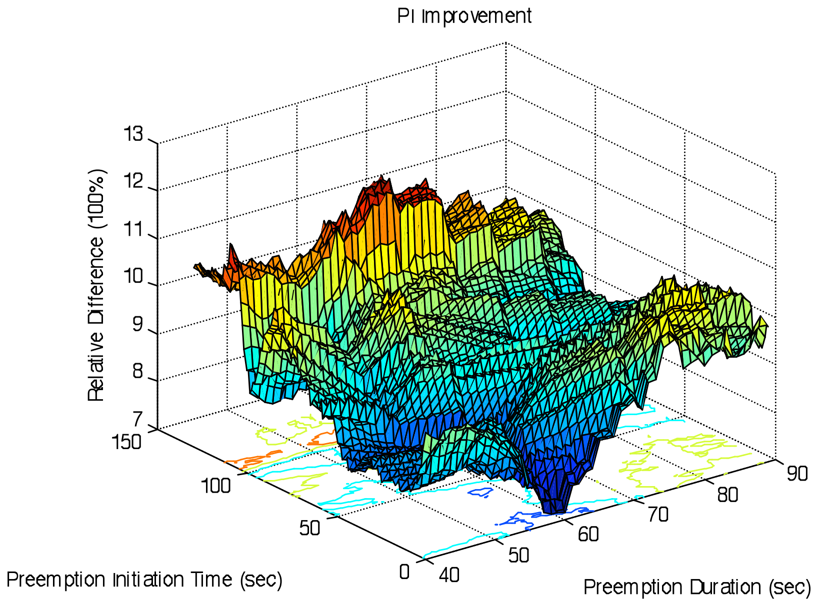
(a)


Optimal Scenario
x 10^4
2
1.8
1.6
1.4
1.2
1
Performance Index (sec*veh)
150
100
50
0
Preemption Initiation Time (sec)
40
50
60
70
80
90
Preemption Duration (sec)

(b)


Difference between Orig. Scenario and Opt. one
2500
2000
1500
1000
500
Performance Index (sec*veh)
150
100
50
0
Preemption Initiation Time (sec)
40
50
60
70
80
90
Preemption Duration (sec)

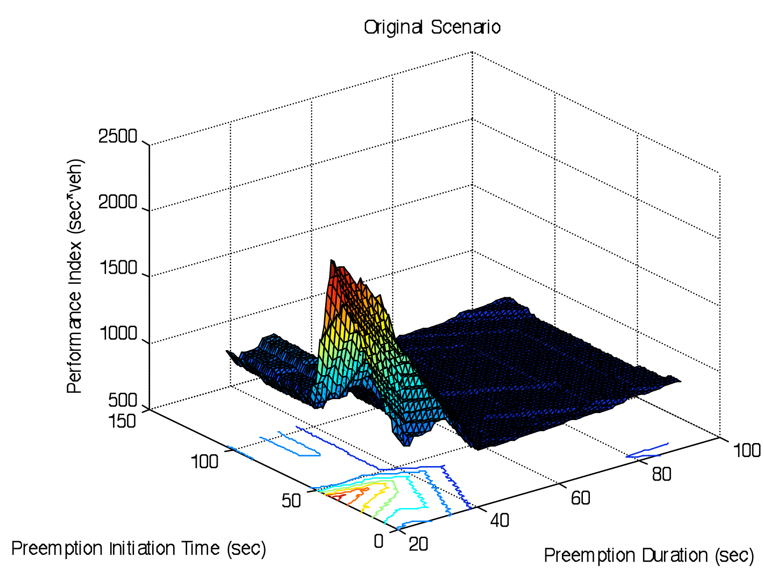
(c)



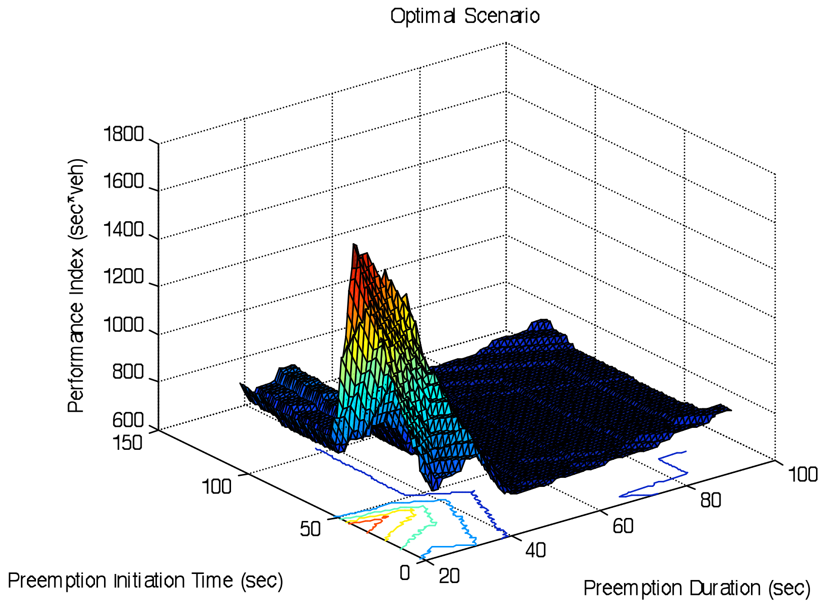

(d)

Figure 4.3 Performance index comparison at College Ave @ Oceanside Blvd. (a) Performance index using the original signal timings. (b) Performance index using the proposed optimal signal timings. (c) Difference of performance index between the original scenario and the optimal one. (d) Relative performance index improvement.

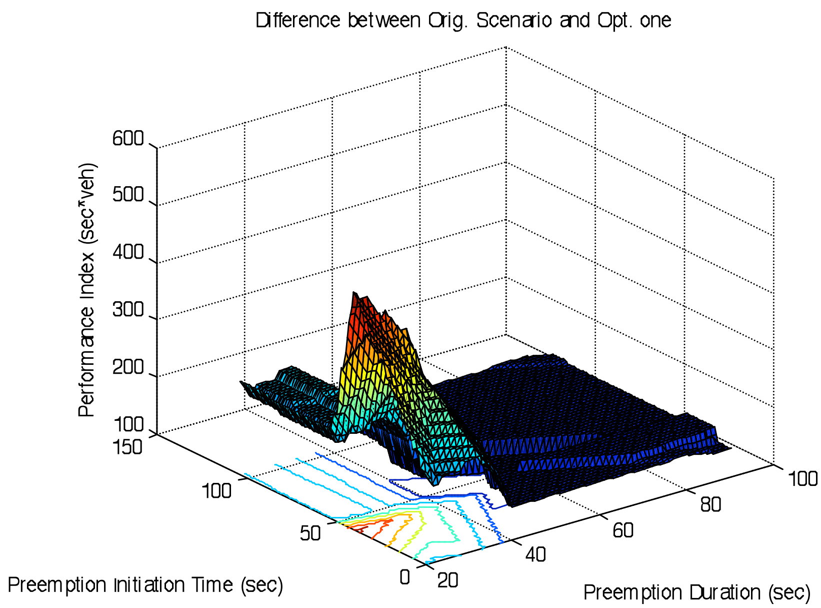
### 4.1.4 Enterprise @ Mission Rd



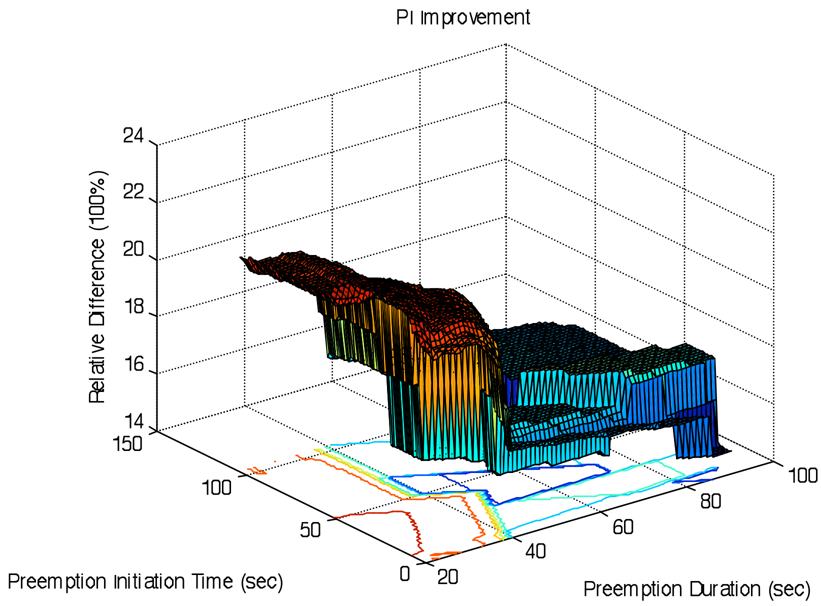

(a)


Optimal Scenario
Performance Index (sec*veh)
1800
1600
1400
1200
1000
800
600
150
100
50
0
Preemption Initiation Time (sec)
20
40
60
80
100
Preemption Duration (sec)

(b)


Difference between Orig. Scenario and Opt. one
Performance Index (sec*veh)
600
500
400
300
200
100
150
100
50
0
Preemption Initiation Time (sec)
20
40
60
80
100
Preemption Duration (sec)

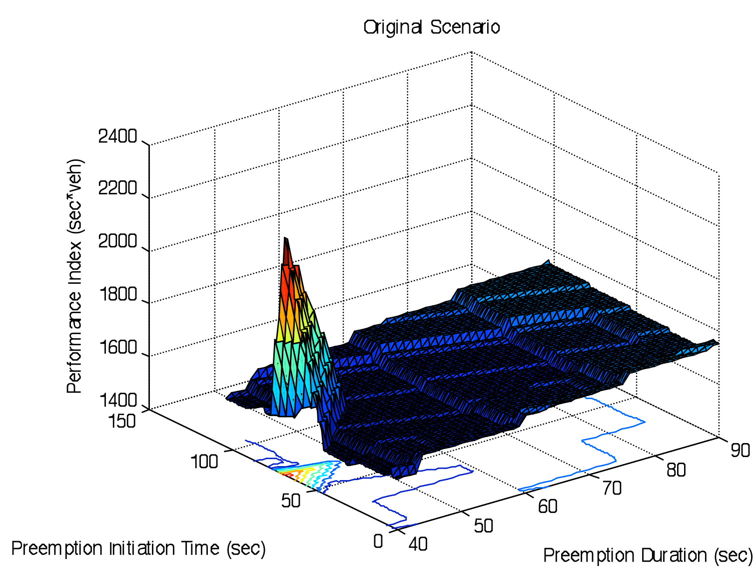
(c)



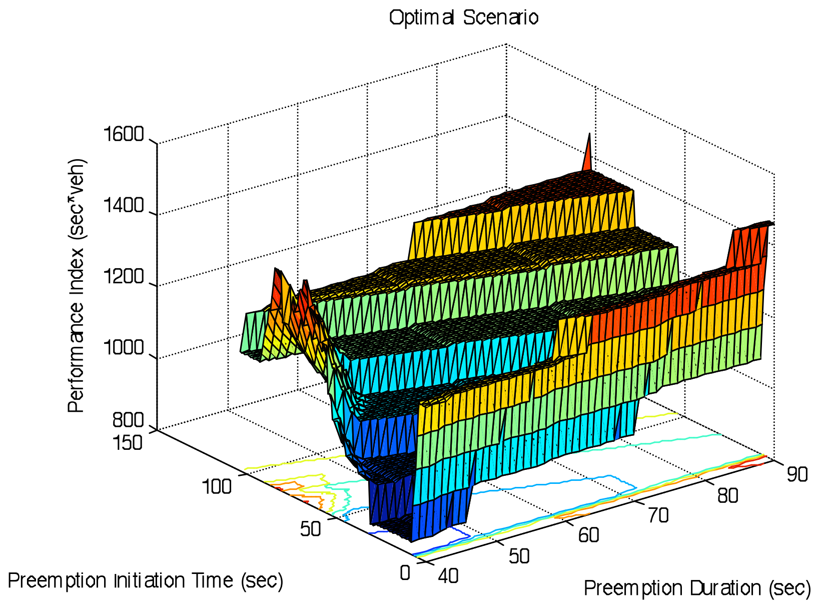

(d)

Figure 4.4 Performance index comparison at Enterprise @ Mission Rd (a) Performance index using the original signal timings. (b) Performance index using the proposed optimal signal timings. (c) Difference of performance index between the original scenario and the optimal one. (d) Relative performance index improvement.

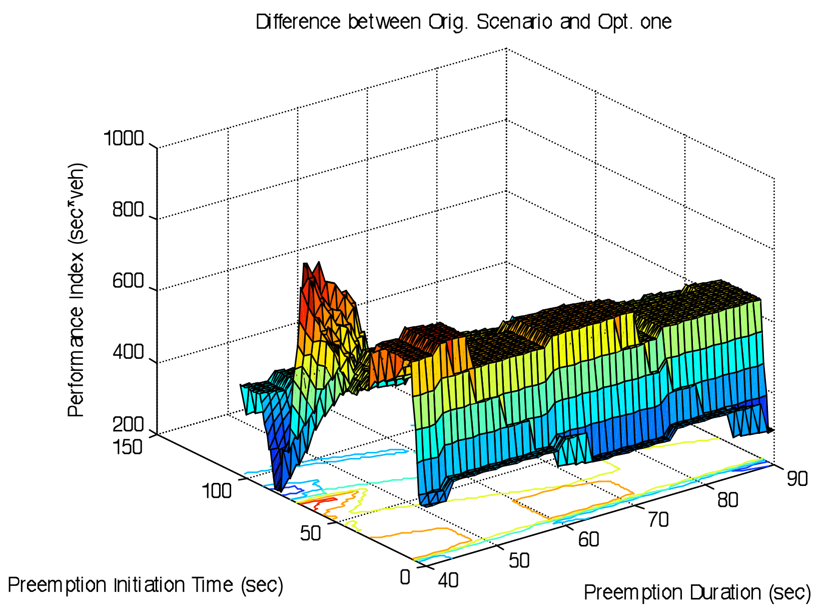
### 4.1.5 Andreasen @ Mission Rd



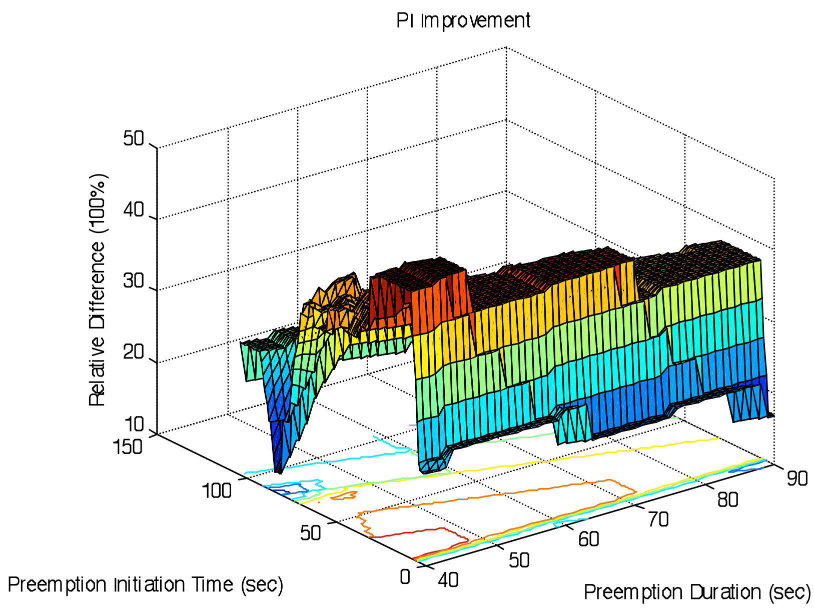

(a)


Optimal Scenario
Performance Index (sec*veh)
1600
1400
1200
1000
800
150
100
50
0
Preemption Initiation Time (sec)
40
50
60
70
80
90
Preemption Duration (sec)

(b)


Difference between Orig. Scenario and Opt. one
Performance Index (sec*veh)
1000
800
600
400
200
150
100
50
0
Preemption Initiation Time (sec)
40
50
60
70
80
90
Preemption Duration (sec)

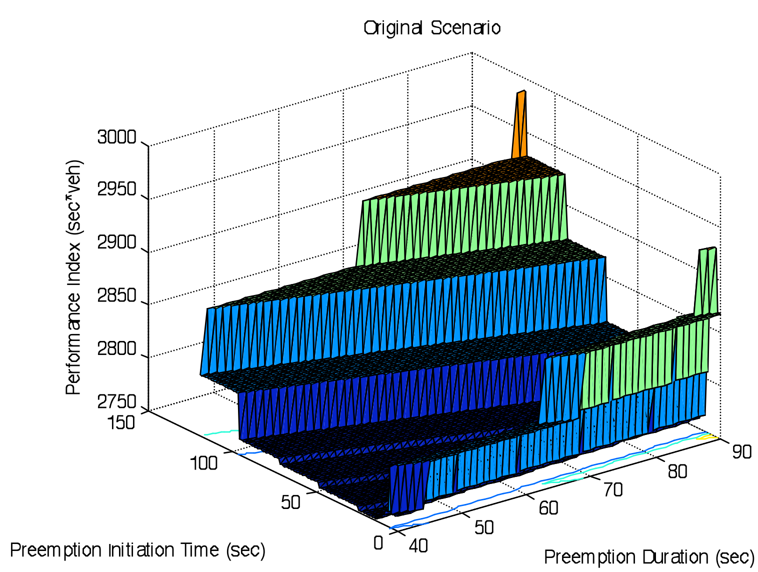
(c)



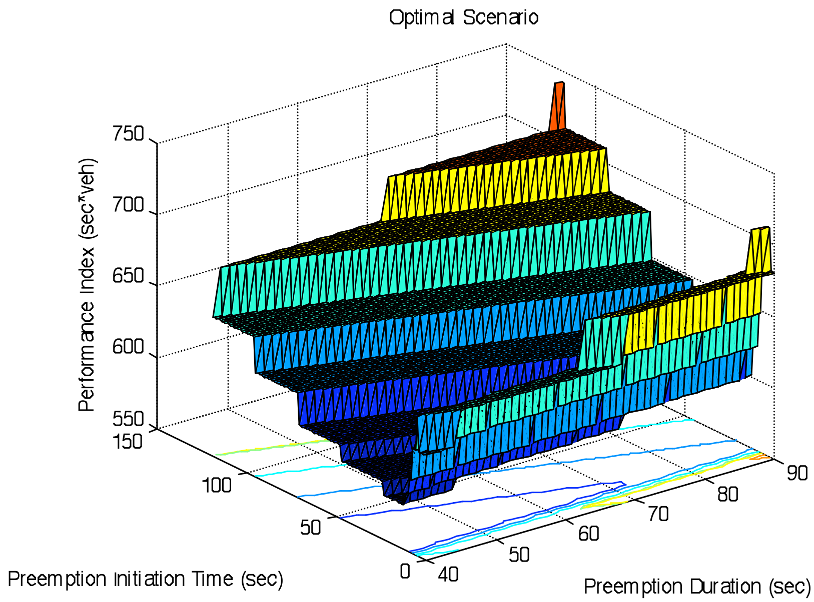

(d)

Figure 4.5 Performance index comparison at Andreasen @ Mission Rd (a) Performance index using the original signal timings. (b) Performance index using the proposed optimal signal timings. (c) Difference of performance index between the original scenario and the optimal one. (d) Relative performance index improvement.

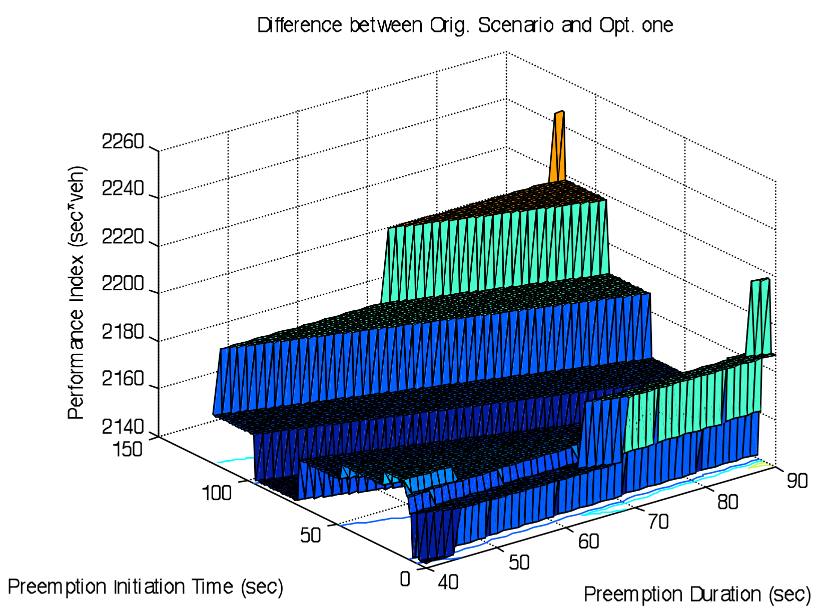
### 4.1.6 Olive @ Vista Village Dr.



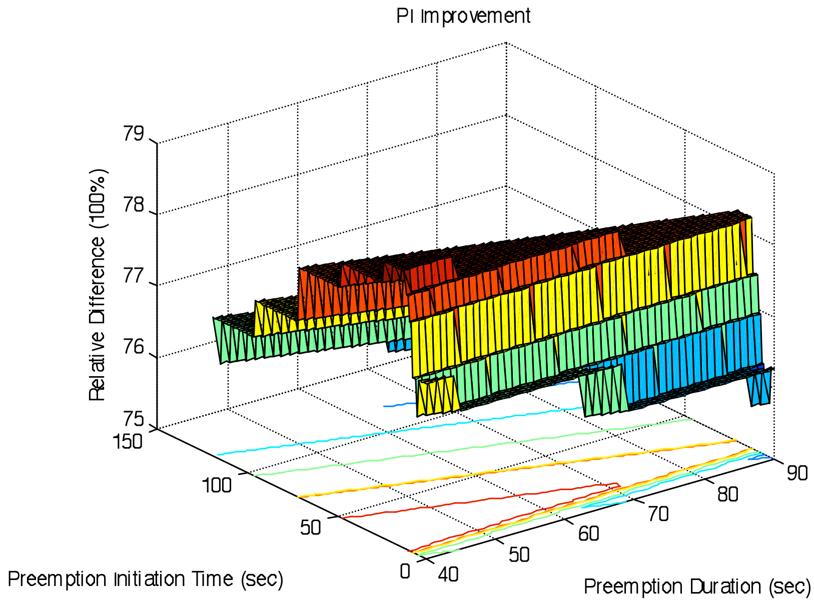

(a)


Optimal Scenario
750
700
650
600
550
150
100
50
0
40
50
60
70
80
90
Performance Index (sec*veh)
Preemption Initiation Time (sec)
Preemption Duration (sec)

(b)


Difference between Orig. Scenario and Opt. one
2260
2240
2220
2200
2180
2160
2140
150
100
50
0
40
50
60
70
80
90
Performance Index (sec*veh)
Preemption Initiation Time (sec)
Preemption Duration (sec)

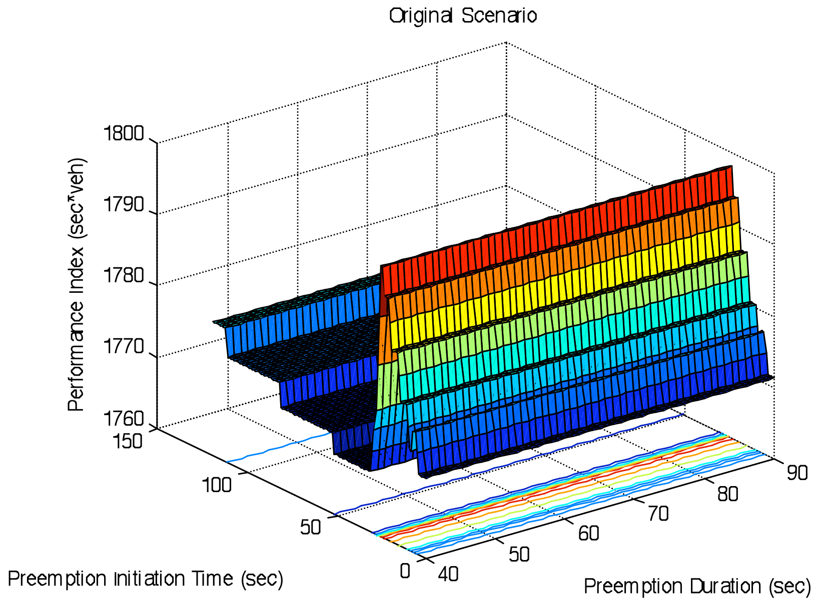
(c)



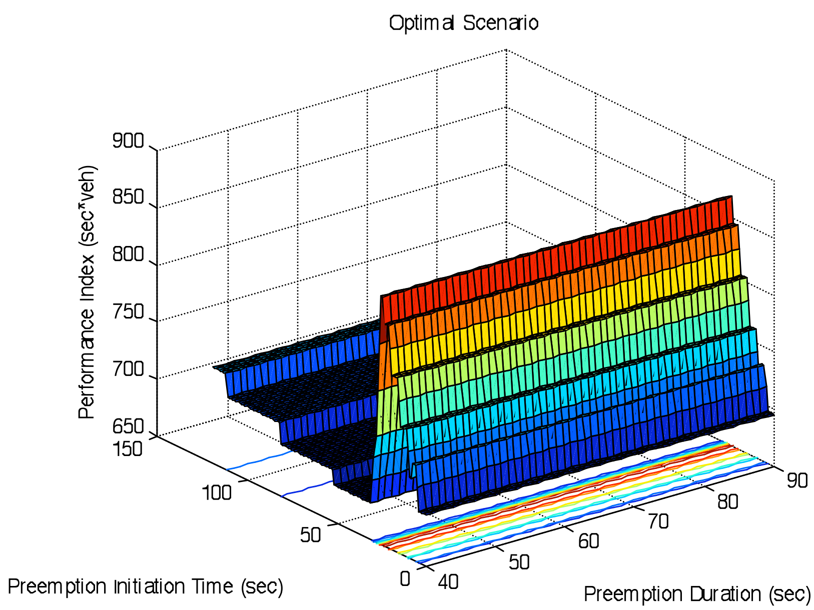

(d)

Figure 4.6 Performance index comparison at Olive @ Vista Village Dr. (a) Performance index using the original signal timings. (b) Performance index using the proposed optimal signal timings. (c) Difference of performance index between the original scenario and the optimal one. (d) Relative performance index improvement

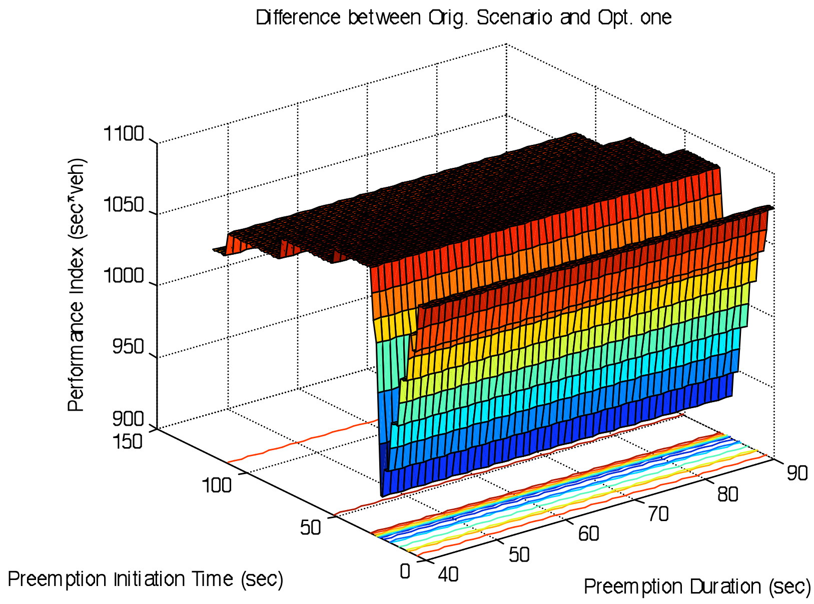
#### 4.1.7 Main St. @ Santa Fe Ave.



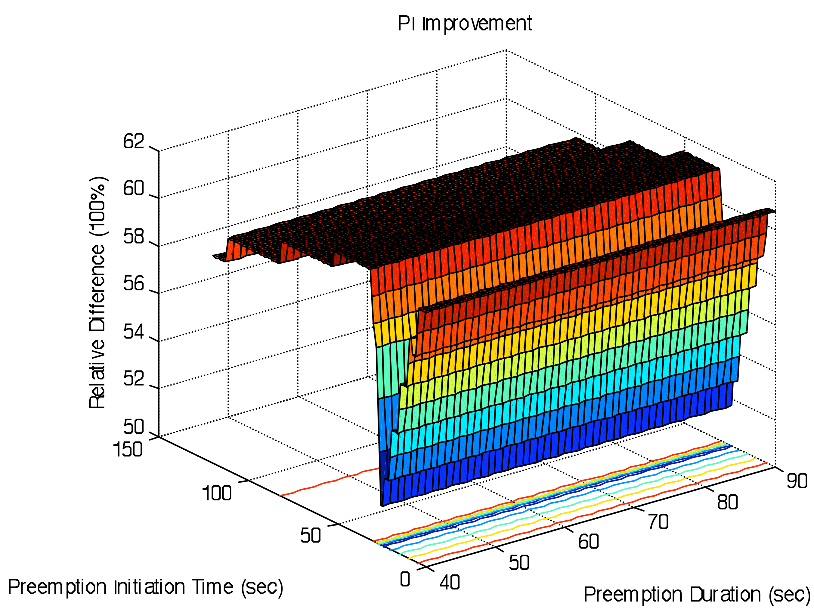

(a)




(b)



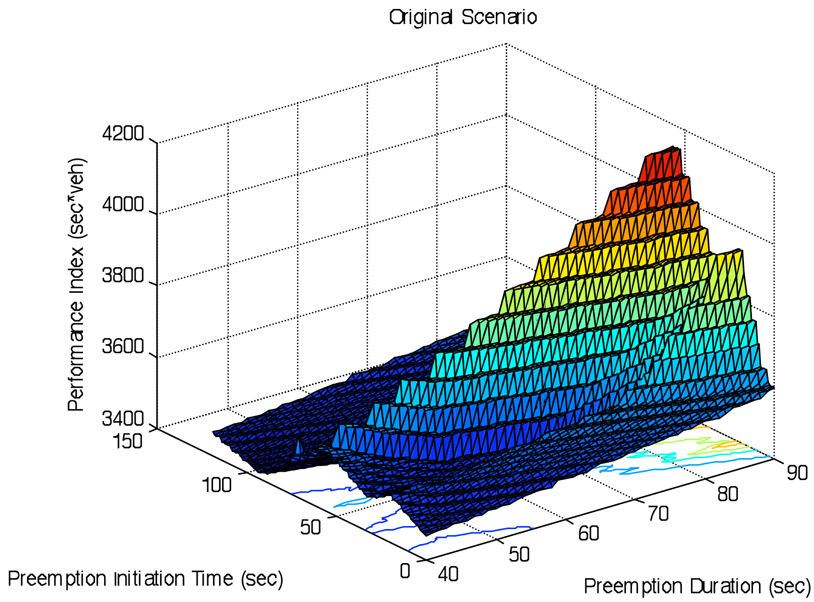

(c)



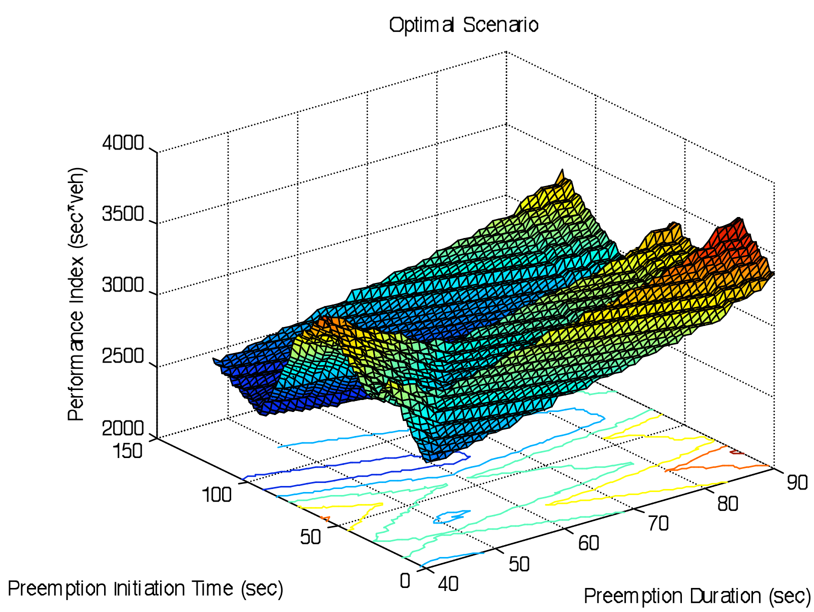

(d)

Figure 4.7 Performance index comparison at Main @ Santa Fe Ave. (a) Performance index using the original signal timings. (b) Performance index using the proposed optimal signal timings. (c) Difference of performance index between the original scenario and the optimal one. (d) Relative performance index improvement

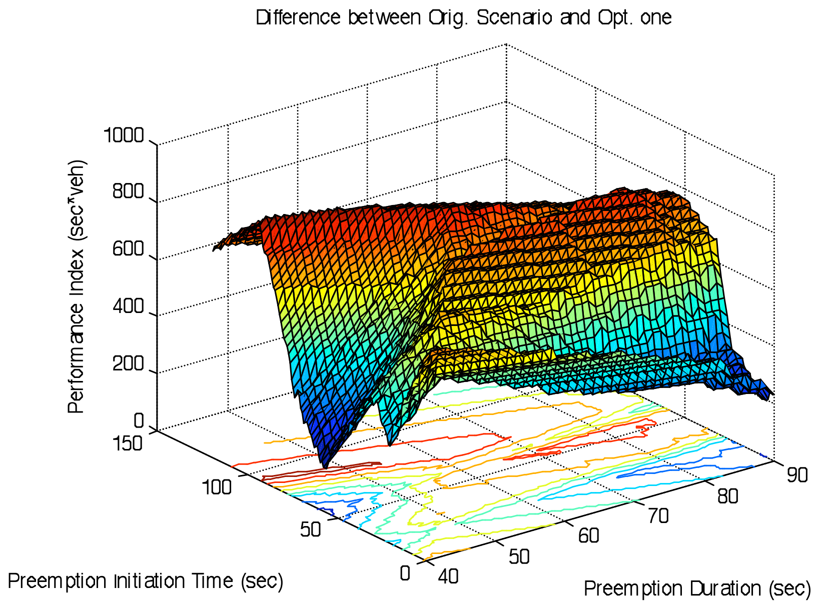
### 4.1.8 Vista Village Dr. @ Santa Fe Ave.



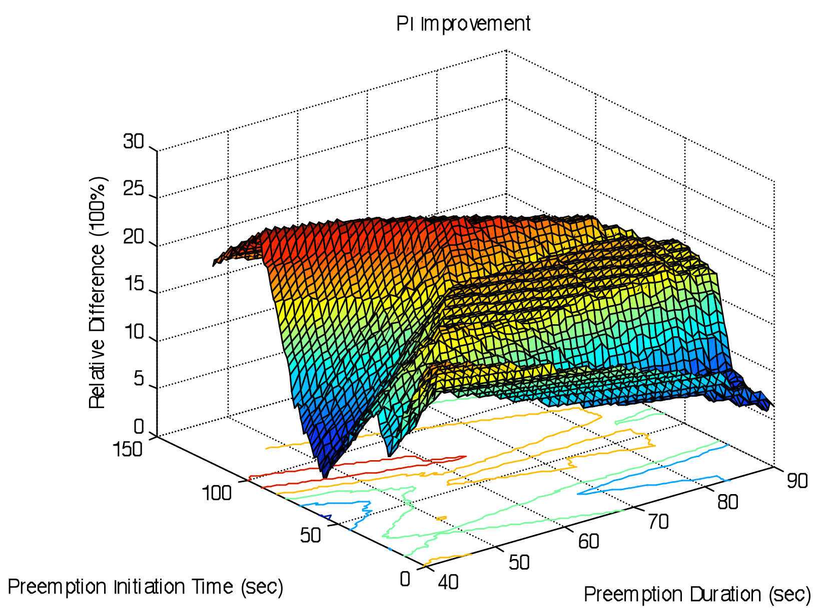

(a)




(b)



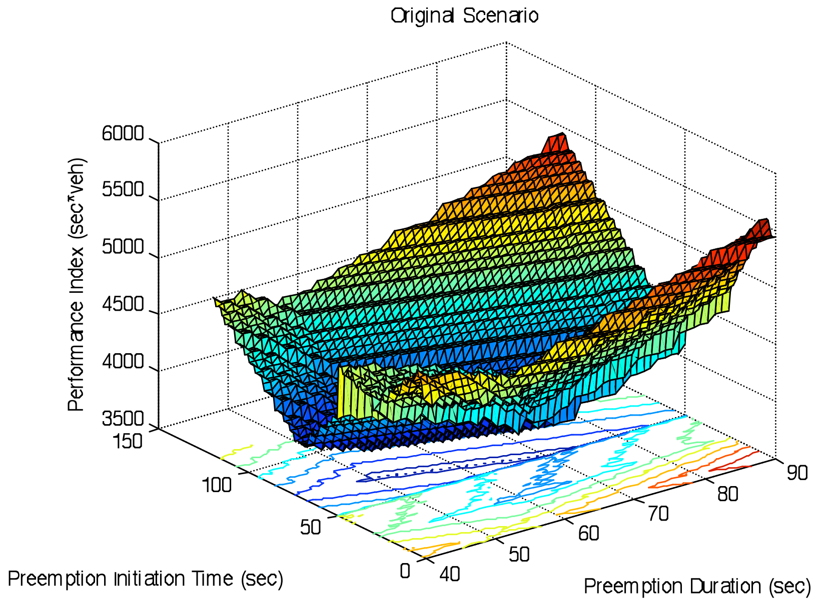

(c)



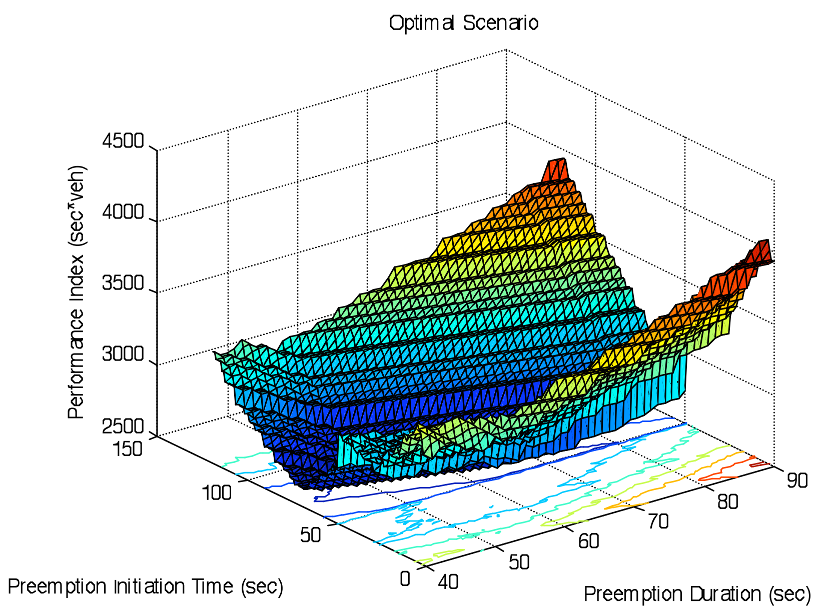

(d)

Figure 4.8 Performance index comparison at Vista Village Dr. @ Santa Fe Ave. (a) Performance index using the original signal timings. (b) Performance index using the proposed optimal signal timings. (c) Difference of performance index between the original scenario and the optimal one. (d) Relative performance index improvement

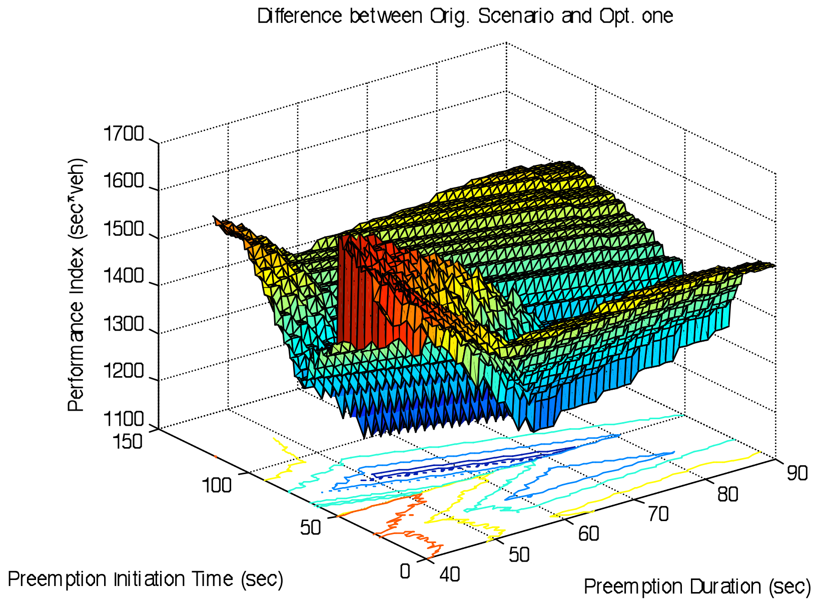
### 4.1.9 Pala @ Escondido Ave.



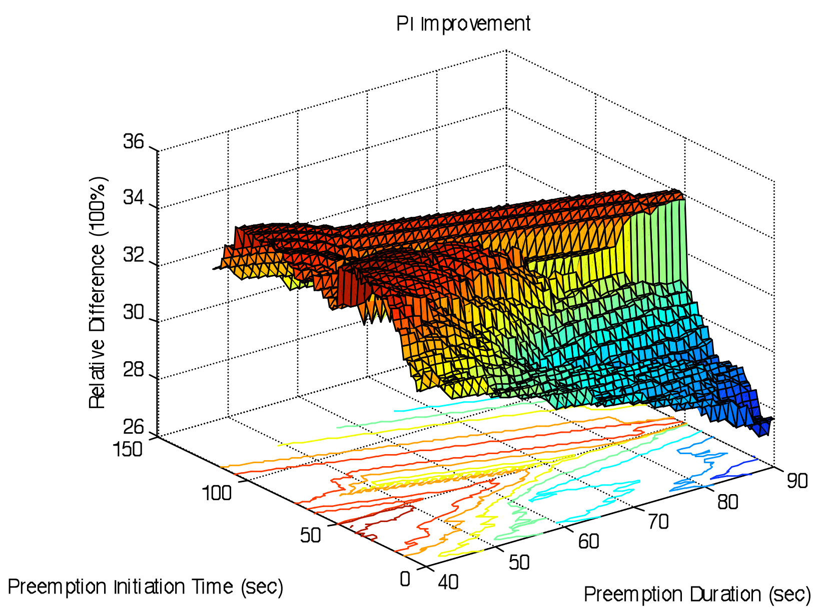

(a)




(b)



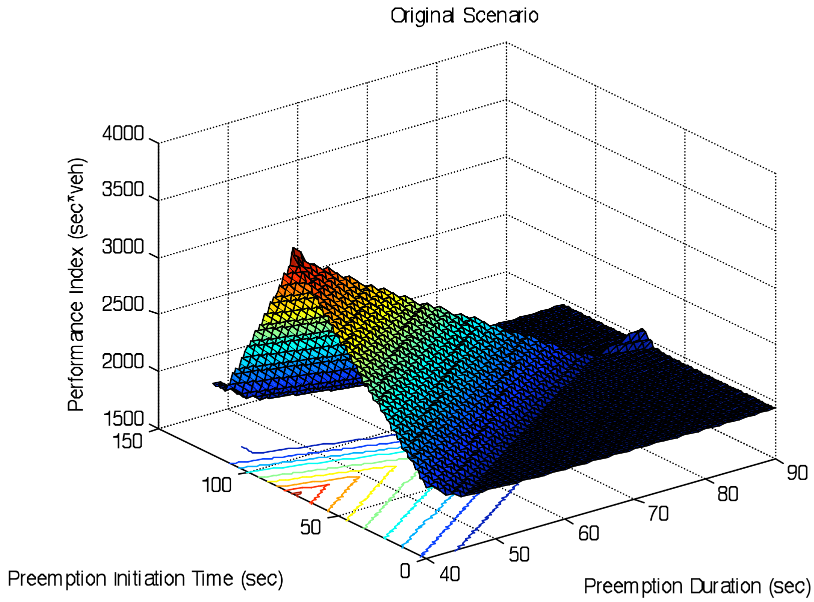

(c)



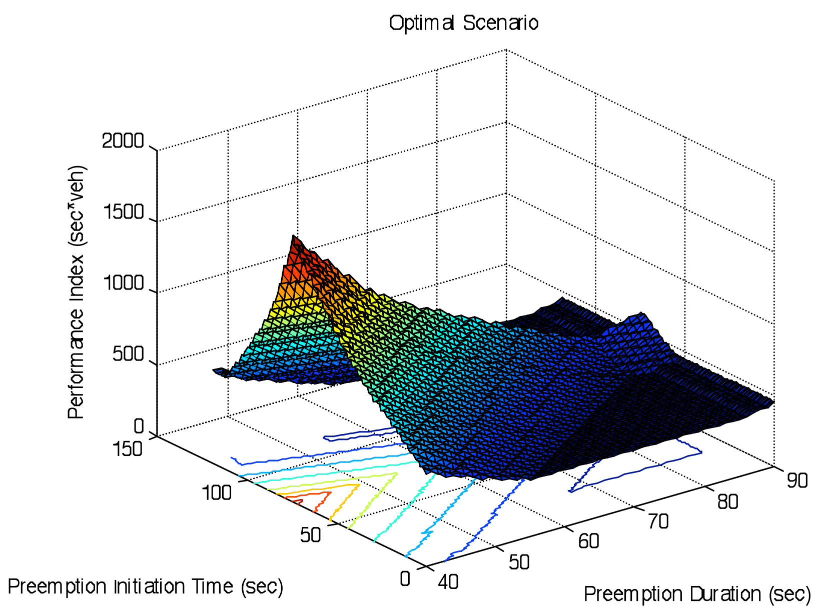

(d)

Figure 4.9 Performance index comparison at Pala @ Escondido Ave. (a) Performance index using the original signal timings. (b) Performance index using the proposed optimal signal timings. (c) Difference of performance index between the original scenario and the optimal one. (d) Relative performance index improvement

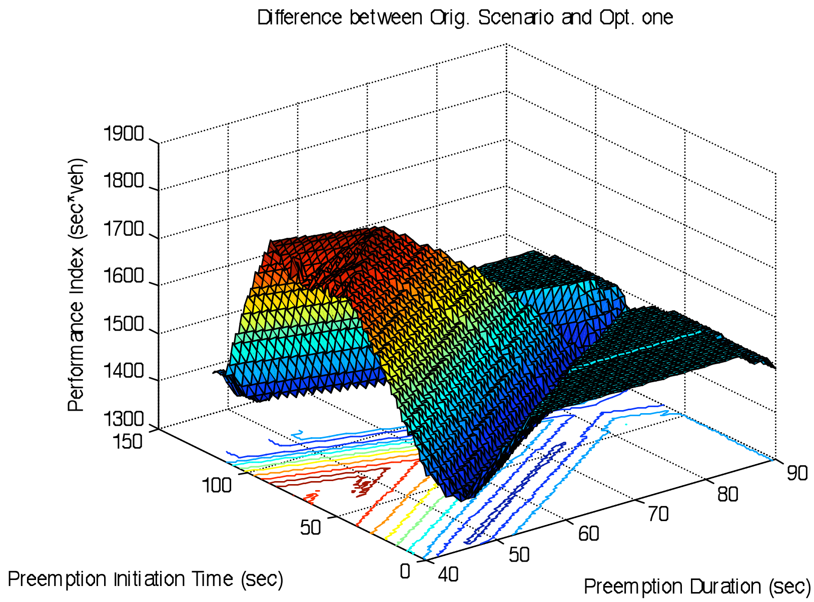
### 4.1.10 Phillips @ Escondido Ave.



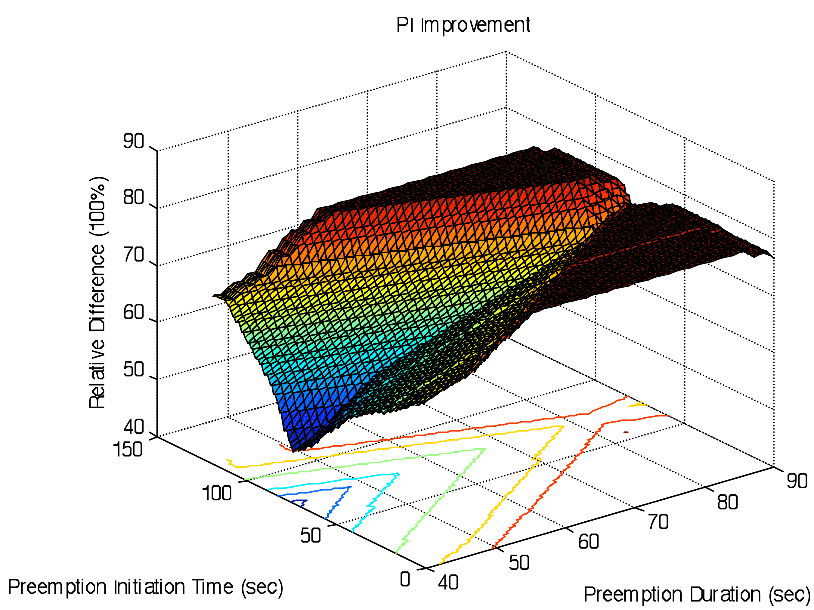

(a)




(b)




(c)




(d)

Figure 4.10 Performance index comparison at Phillips @ Escondido Ave. (a) Performance index using the original signal timings. (b) Performance index using the proposed optimal signal timings. (c) Difference of performance index between the original scenario and the optimal one. (d) Relative performance index improvement

### 4.1.11 Summary of Numerical Analysis Results

To get further insight into the numerical results from the proposed strategy, we not only compared the overall intersection delay (within one cycle right after the preemption) under the original signal timings and under the proposed ones, but also calculated the performance index (for one normal cycle) under the original signal timings without preemption. The results are illustrated in Figure 4.11 through Figure 4.16 for the five study sites mentioned above.

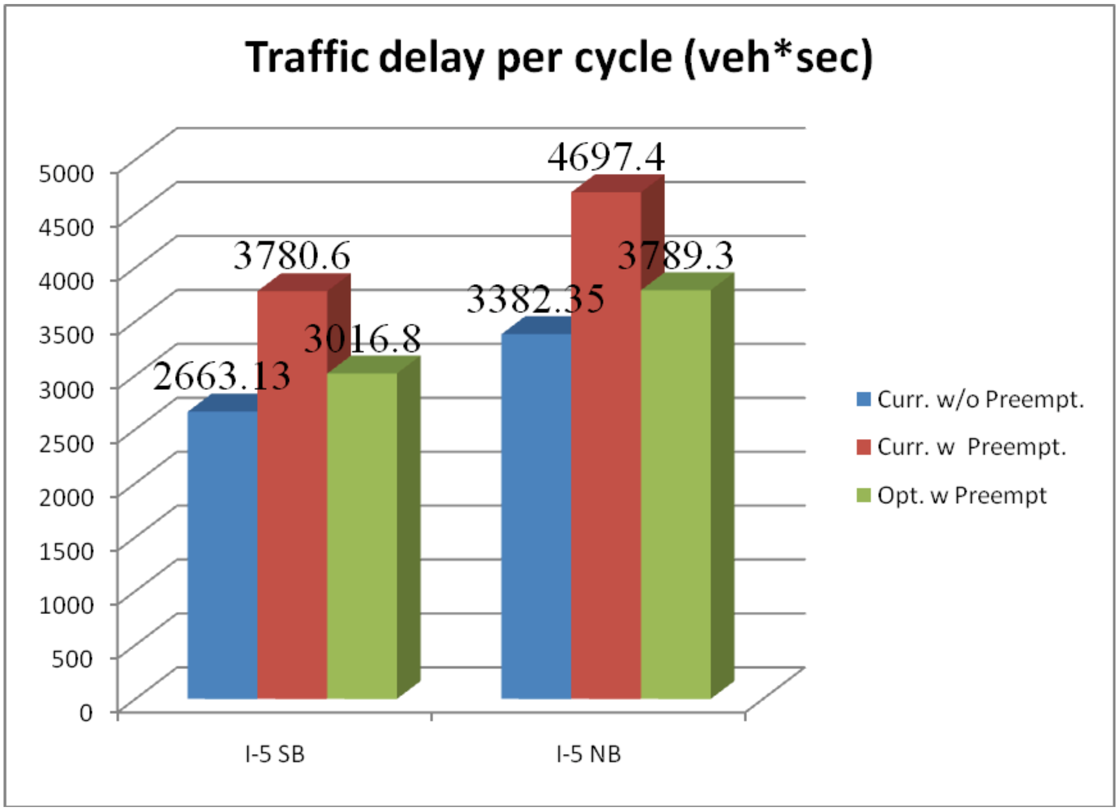



Figure 4.11 Comparison on numerical results at I-5 SB/I-5 NB @ Oceanside Blvd.

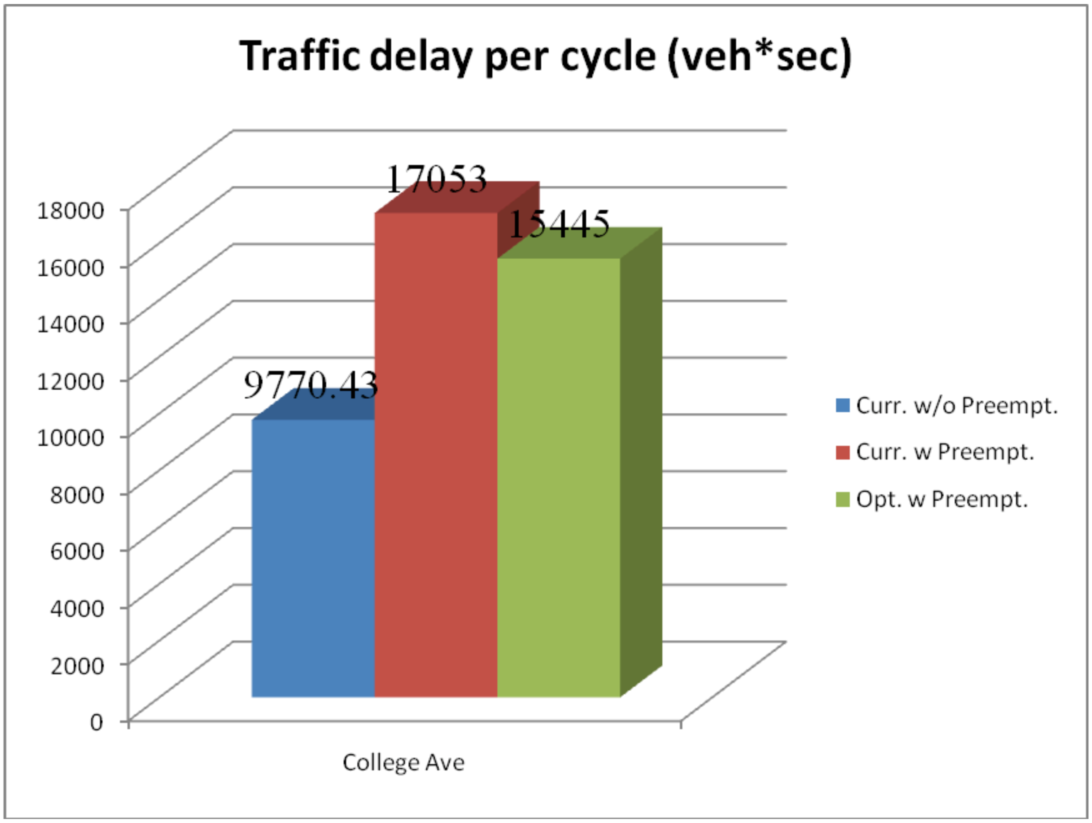



Figure 4.12 Comparison on numerical results at College Ave @ Oceanside Blvd.

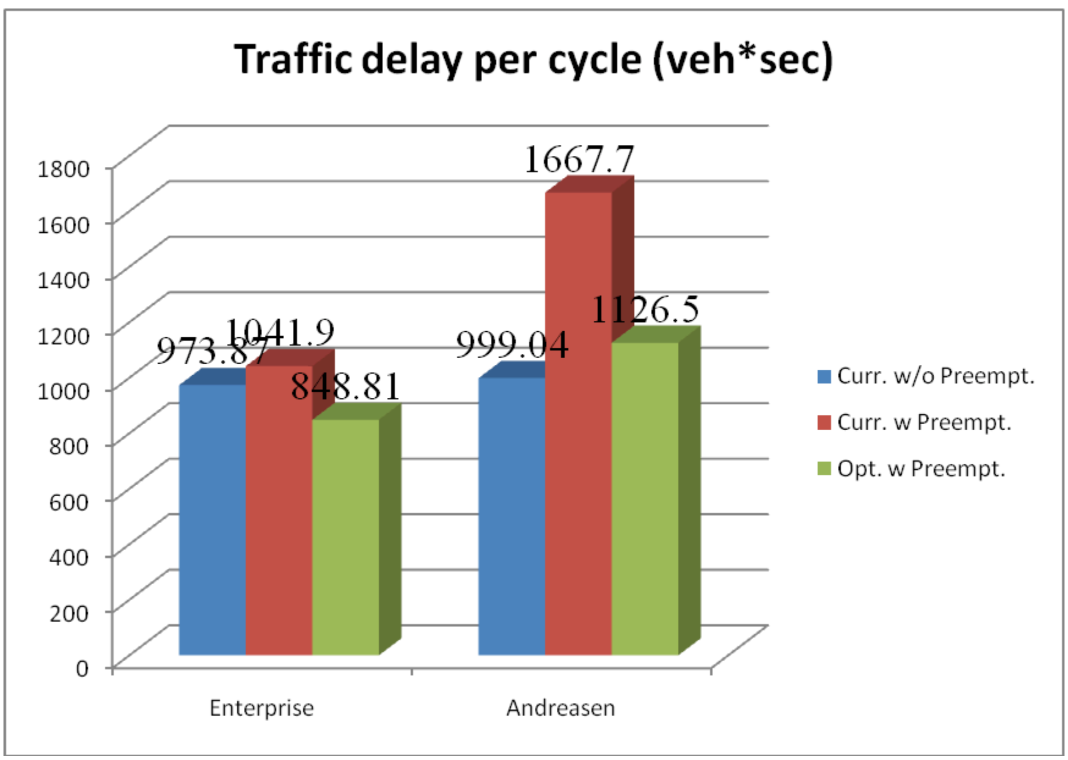



Figure 4.13 Comparison on numerical results at Enterprise/Andreasen @ Mission Rd

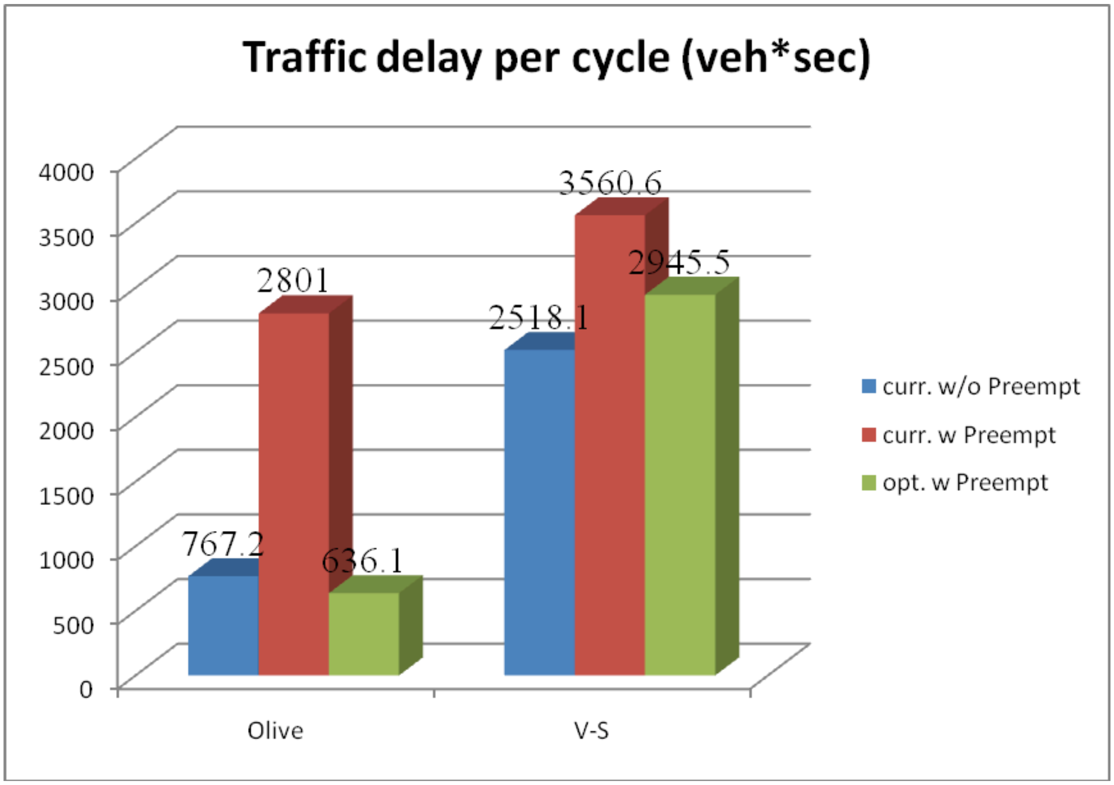



Figure 4.14 Comparison on numerical results at Olive/Santa Fe Ave. @ Vista Village Dr.

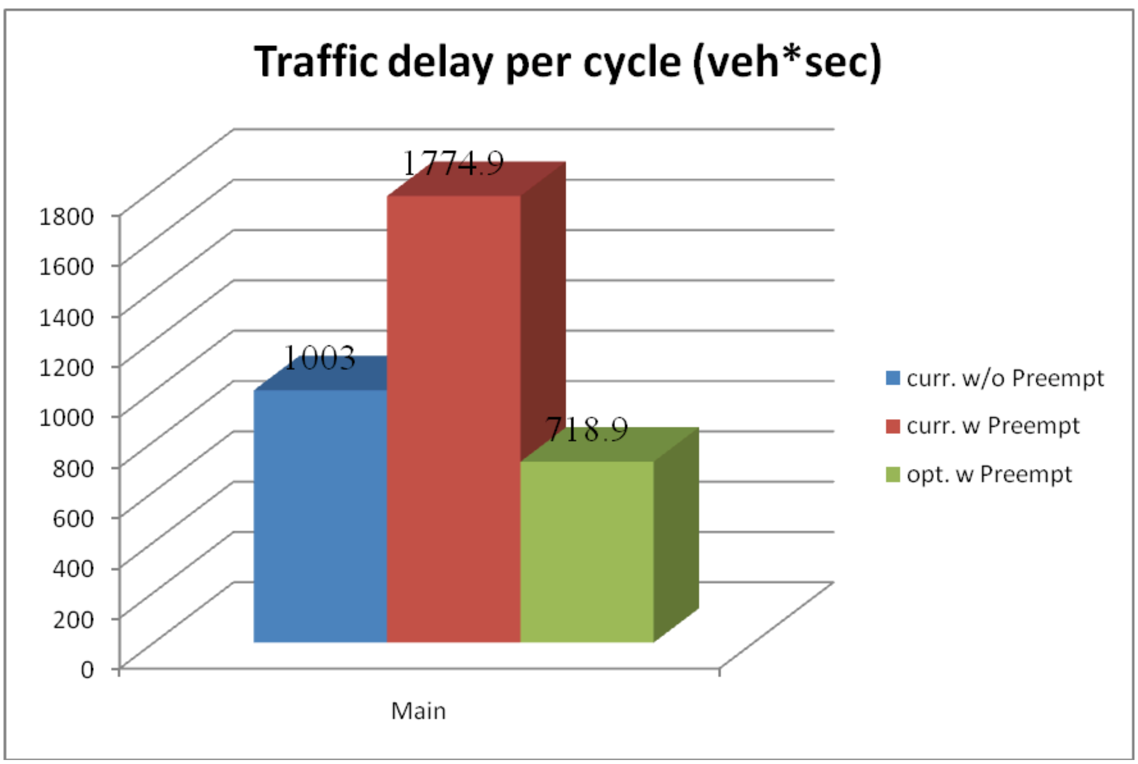



Figure 4.15 Comparison on numerical results at Olive/Santa Fe Ave. @ Vista Village Dr.

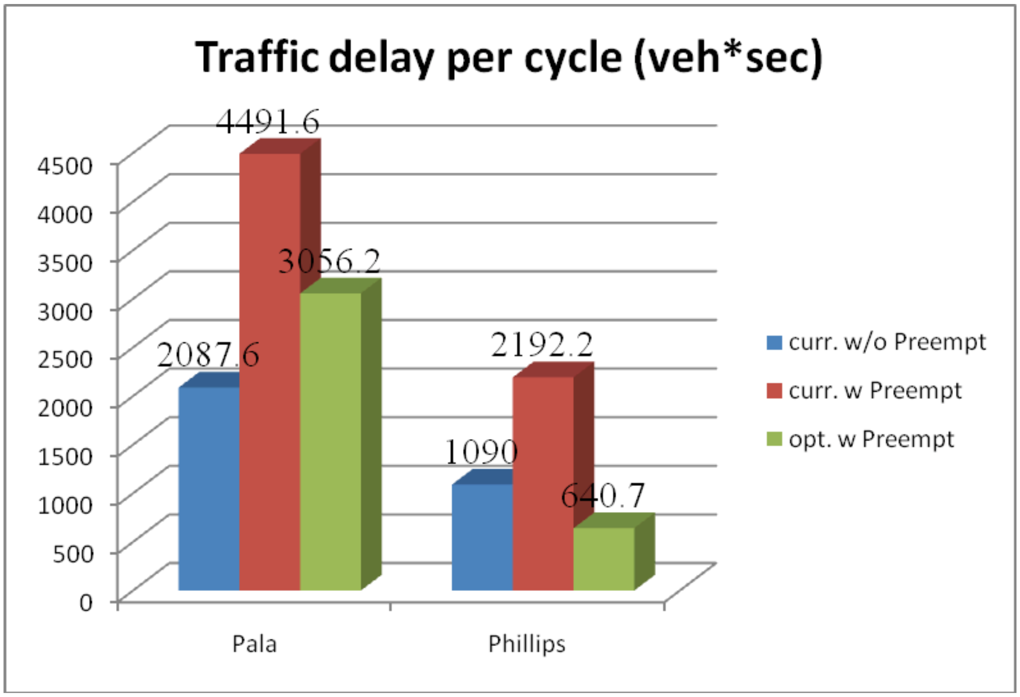



Figure 4.16 Comparison on numerical results at Pala/Phillips @ Escondido Ave.

Based on the numerical analysis results, we can make the following comments:

1. The overall intersection delay may vary w.r.t. different combination of preemption initiation time and preemption duration.
2. As is shown in figures, within one cycle after the preemption, the overall intersection delays under optimal timings are consistently less than those under current timings.
3. Note that in the 3-D diagram, the preemption initiation time varies from 0 to the cycle end (w.r.t. the Local Clock) while the preemption duration ranges from 40 sec to 90 sec. According to the preemption logic currently used in SPRINTER, such range of preemption duration is wide enough to cover most of the situation, even for those intersections around grade crossing with near-side stations.
4. At I-5 Southbound Ramp @ Oceanside Blvd and I-5 Northbound Ramp @ Oceanside Blvd, although the overall intersection delays (under either original signal timings and/or proposed ones) may vary noticeably, the normalized performance index, i.e. traffic delay per vehicle within the impacted cycle, are very close, no matter in original scenario or optimal scenario (see Figure 4.17).

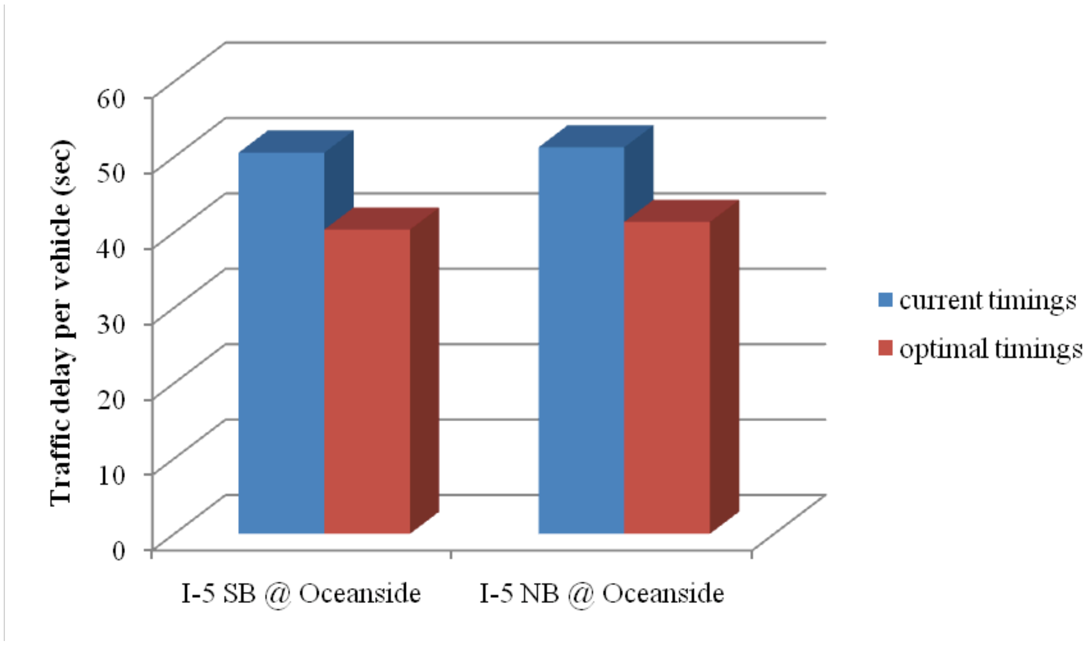



Figure 4.17 Traffic delay per vehicle at I-5 SB/I-5 NB @ Oceanside Blvd.

5. From Figure 4.6 to Figure 4.8, we can observe that for the four sites: I-5 SB/I-5 NB @ Oceanside Blvd, Andreasen @ Mission Rd, and College Ave @ Oceanside Blvd; there are non-trivial negative impacts on traffic under the original signal timings due to the interruption of SPRINTER train. However, such degradation can be greatly mitigated if the proposed signal timings are implemented, especially for the first three sites.
6. An interesting finding from some figures (e.g. at Enterprise @ Oceanside Blvd) is that we can even obtain better system performance with preemption by optimization than that without preemption if the original signal timings are not well tuned.

### *4.2. Simulation Study*

To further validate the proposed strategy as well as the numerical analysis, we built up a simulation network in PARAMICS Modeler V5.22, and simulated two scenarios: one is the scenario under original signal timings while the other is the scenario under proposed signal timings. Figure 4.18 shows the whole SPRINTER railroad about 22 miles long, along the Highway 78 corridor. This railroad traverses through City of Oceanside, City of Vista, City of San Marcos and City of Escondido in San Diego's North County region. The snapshot of I-5 Southbound Ramp @ Oceanside Blvd. and I-5 Northbound Ramp @ Oceanside Blvd., Enterprise @ Mission Rd., Andreasen @ Mission Rd., Olive/Santa Fe Ave. @ Vista Village Dr./MainSt. and Pala/Phillips @ Escondido Ave. in the simulation model are shown in Figure 4.19, through Figure 4.23.

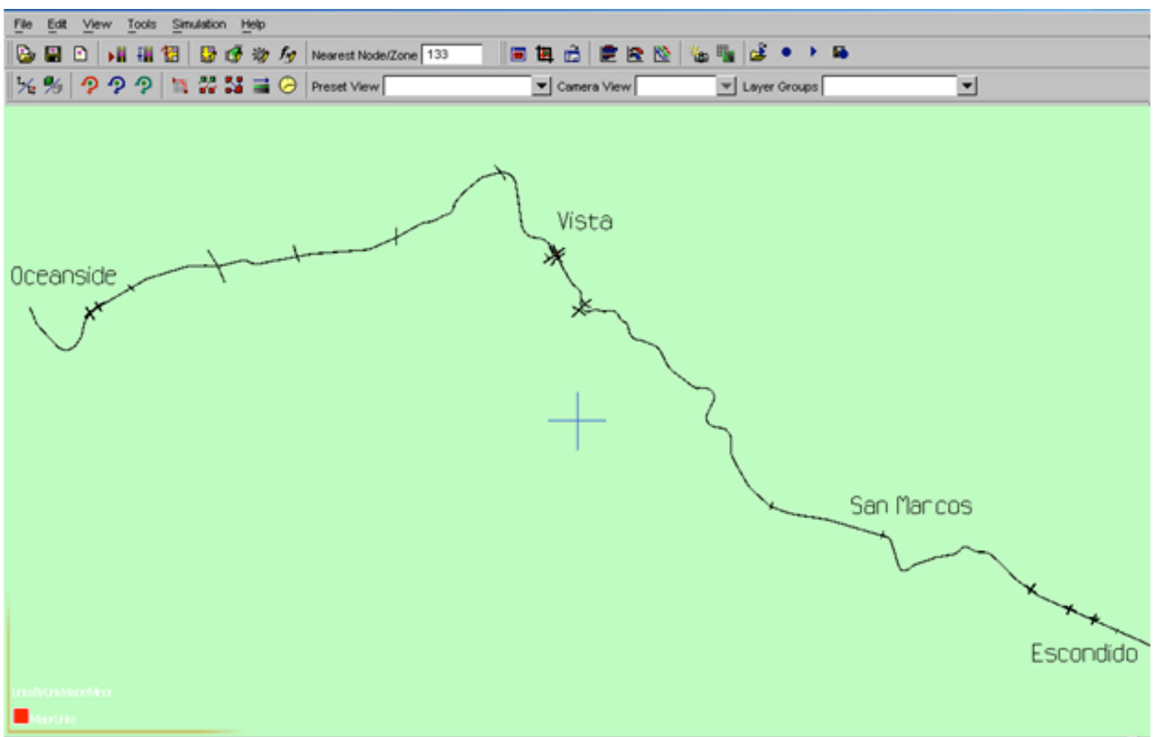



Figure 4.18 Overview of Network in PARAMICS

### 4.2.1 Setups of Simulation Model

**Simulation Network:** By manually capturing the coordinates of nodes from Google Earth, we linked these nodes to build up the SPRINTER railroad and roadways of study sites. According to documents from local jurisdiction, roadway characteristics and locations of stations can then be determined. Based on the traffic volume/ratio data and signal timing tables provided by each city, study sites in the simulation network are signalized and the Origin-Destination (O-D) matrix is reconstructed. Parameters for the operation of SPRINTER Train are input into the simulation model based on the information available from SPRINTER webpage, including physical/dynamical parameters of trains and SPRINTER online schedule.

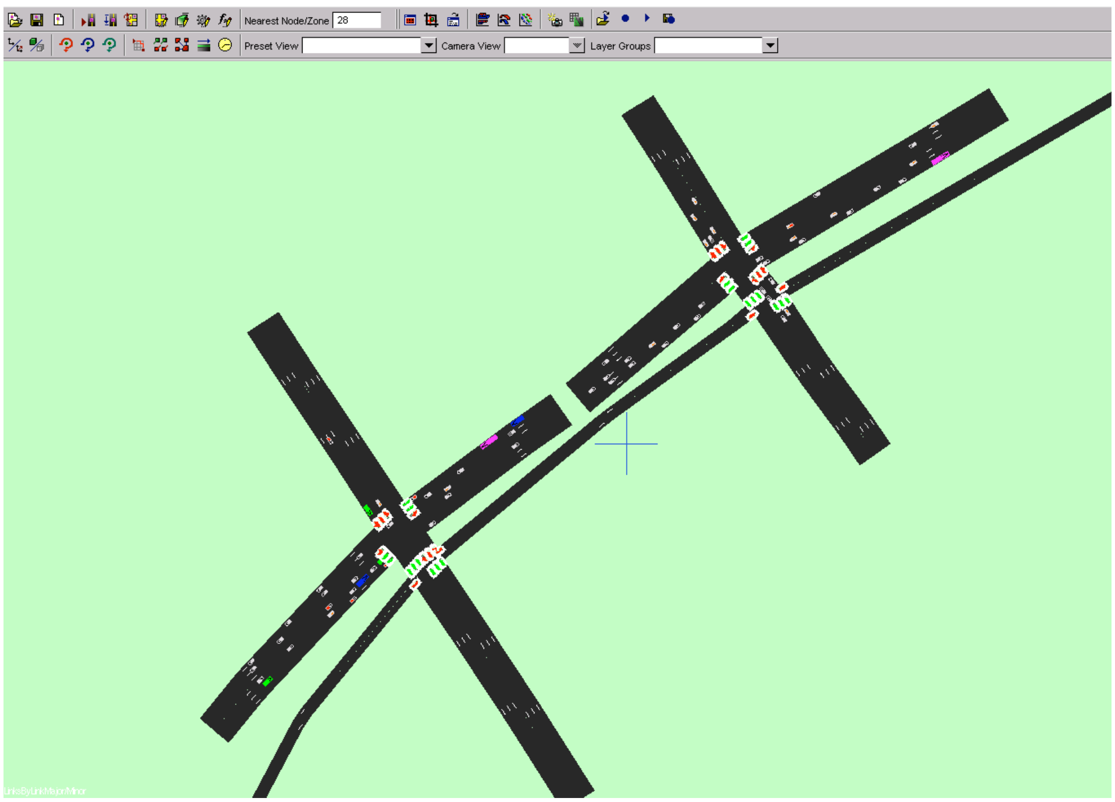



Figure 4.19 Snapshot of I-5 Ramp @ Oceanside Blvd. in PARAMICS

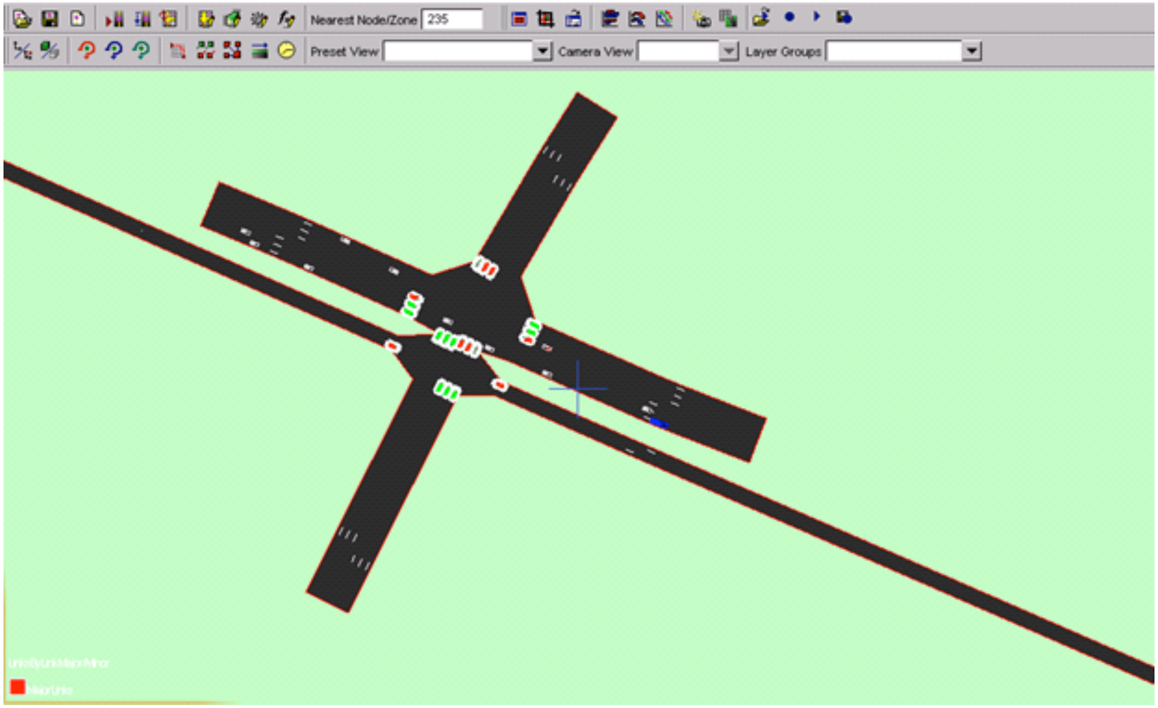



Figure 4.20 Snapshot of Enterprise @ Mission Rd. in PARAMICS

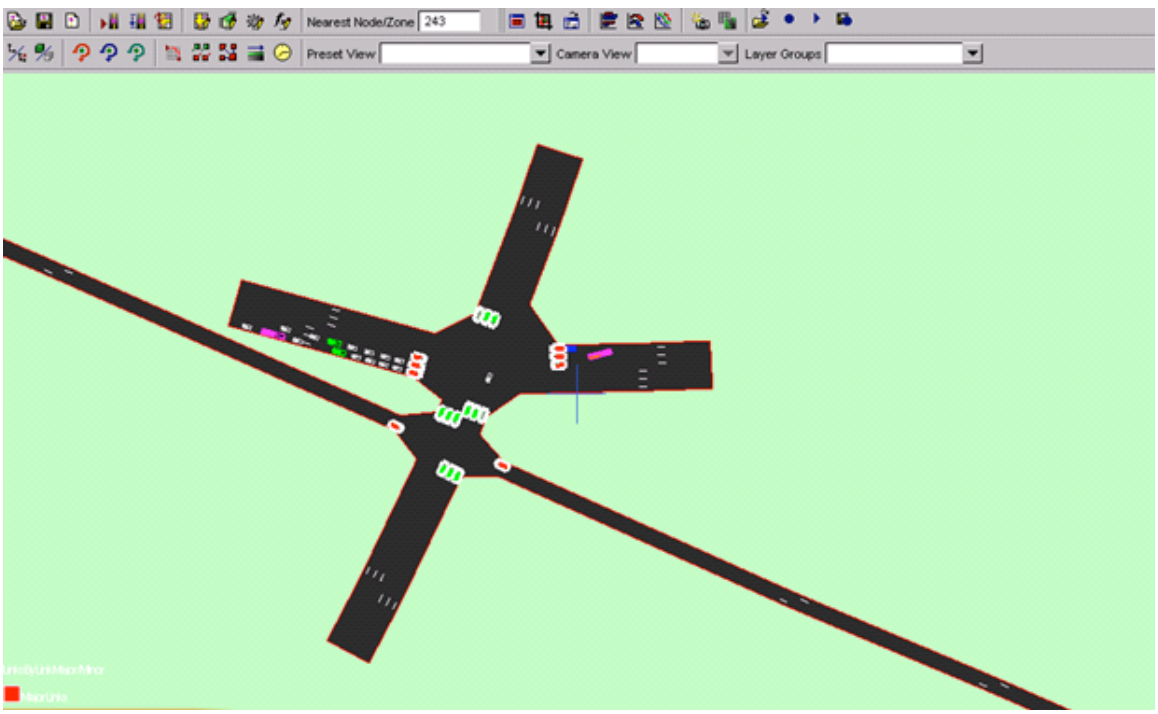



Figure 4.21 Snapshot of Andreasen @ Mission Rd. in PARAMICS

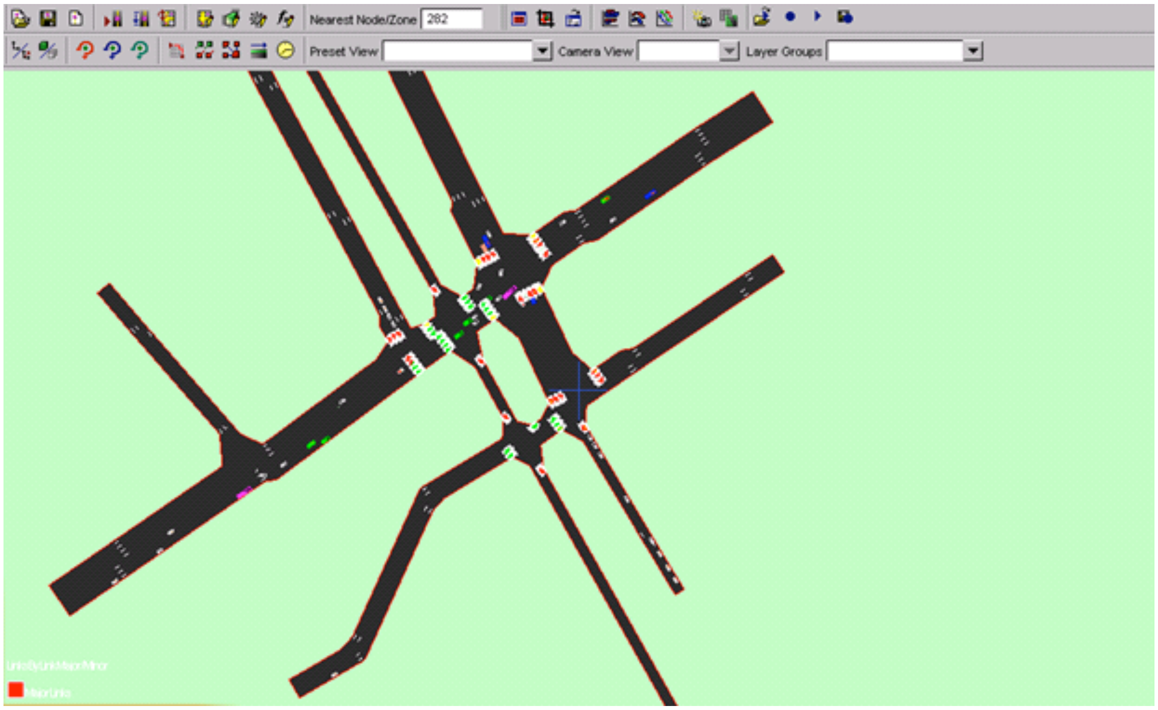


Figure 4.22 Snapshot of Olive/Santa Fe Ave. @ Vista Village Dr./Main in PARAMICS

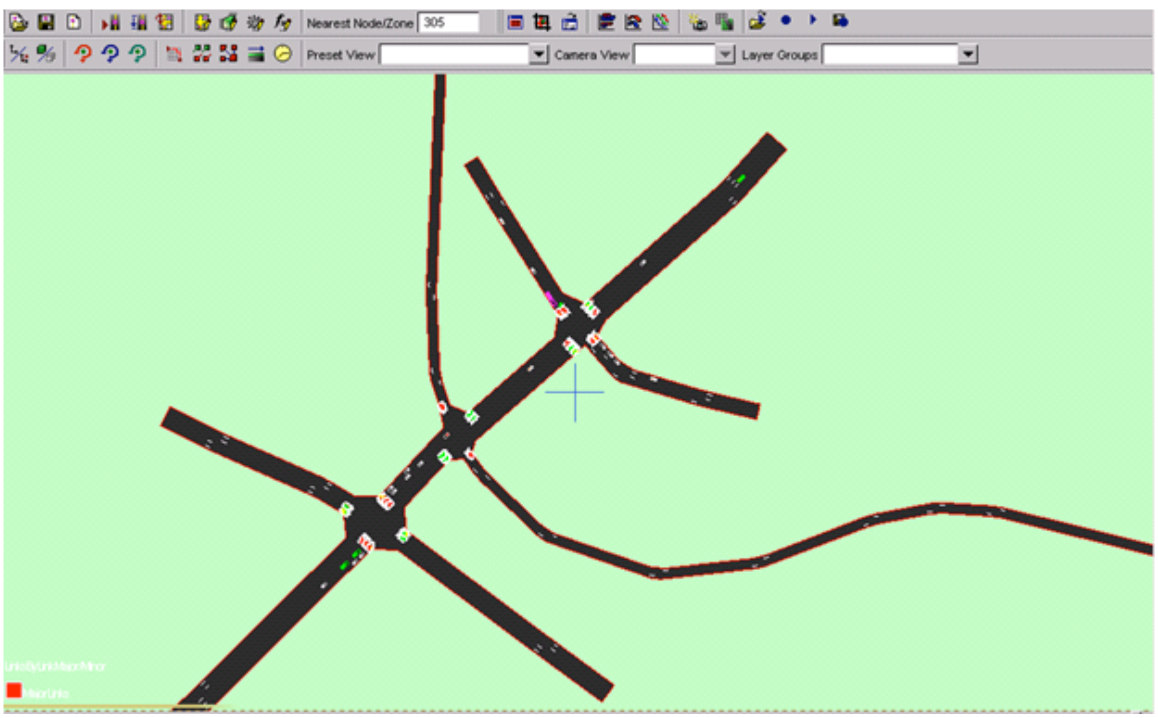


Figure 4.23 Snapshot of Pala/Phillips @ Escondido Ave. in PARAMICS

**Simulation Time Settings:** In the simulation, we selected the morning peak hour of a typical weekday, i.e. from 06:00 a.m. to 09:00 a.m., as the simulated time interval. Within this simulation time period, the first Eastbound SPRINTER train starts from

Oceanside Transit Center at 06:03 a.m. while the first Westbound SPRINTER train starts from Escondido Transit Center at exactly the same time. The average trip time is about 53 minutes and the headway is 30 minutes along each bound. Therefore, there are totally 12 trips (6 trips along each bound) during 3 hours in simulation time. In addition, the step length of simulation time is set as 0.1 second.

**Codes in the Simulation Model:** Besides the setups of simulation network, it is necessary for us to program, such as APIs, to implement the proposed strategy in PARAMICS. Basically speaking, there are three types of codes:

(a) *Preemption Logic Related*. PARAMICS Modeler itself does not provide the dedicated public transit (PT) preemption logic function. However, by additionally coding '*plans*' and '*phases*' files to call the Vehicle Actuated Signal (VAS) function which is self-contained in PARAMICS suite, SPRINTER train preemption can be fulfilled with a certain degree of flexibility, such as the preemption duration. For example, at I-5 Southbound Ramp @ Oceanside Blvd. and I-5 Northbound Ramp @ Oceanside Blvd, we coded a fixed preemption duration time, 50 sec, due to the fact that the train speed almost keeps constant in simulation and there is no near-side station around these two grade crossings.

(b) *Adaptive Signal Timing Related*. At our study sites, traffic signal controllers are running fixed timings during the simulation in the original scenario. However, in the proposed scenario, traffic signal timings adapt to not only when the train initiates the preemption, but also how long the preemption lasts, which requires actuated signals API functions to implement. Relying on the detection of the train's movement, the optimization strategy is triggered right after the train clears the grade crossing.

(c) *Data Collection Related*. To obtain MOEs and compare overall intersection delays between original scenario and proposed scenario at those study sites, we also coded "virtual" detector loops and data collection API functions to calculate the delay for each vehicle passing the controlled (signalized) intersection. Simply speaking, for vehicle *i*,

$$\text{Delay}_i = T_{actual} - D/V_{ff}$$

where $T_{actual}$ is the actual travel time for vehicle *i* between the upstream "virtual" loop and the downstream one; D is the distance between the upstream "virtual" loop and the downstream one; and $V_{ff}$ is the free flow speed or link speed limit. Obvious, the delay we calculated is the total controlled delay, including the vehicle acceleration/deceleration time, start-up loss time and stop time at signal. On the other hand, we also collect SPRINTER train's location data and "virtual" loop detector data for the purpose of calibration and future use.

### 4.2.2 Simulation Results

By analyzing total 12 trips (6 trips each bound), or say 12 preemption impacted cycles, we obtained the following simulation results under two different scenarios --- original traffic signal timings and proposed traffic signal timings.

#### 4.2.2.1 Impacts of Preemption under Original Signal Timings

As is mentioned in Chapter 2, the impacts of preemption under original signal timings are noticeable at our studied sites: I-5 Southbound Ramp @ Oceanside Blvd., I-5 Northbound Ramp @ Oceanside Blvd., Enterprise @ Oceanside Blvd. and Andreasen Dr. @ Oceanside Blvd, based on the observation from simulation. Table 4.1 summarizes the simulation results on the preemption impacts (delay per vehicle) at each intersection. Note that the preemption duration in the simulation is 50 seconds.

Table 4.1 Delay per Vehicle with and without Impacts of Preemption at Studied Sites

| | **Traffic Delay per Vehicle (sec)** | |
|---|---|---|
| | Without Preemption | With Preemption |
| **I-5 Southbound** | 34.7 | 57.3 |
| **I-5 Northbound** | 33.6 | 56.8 |
| **Enterprise** | 11.2 | 13.8 |
| **Andreasen** | 15.9 | 23.7 |

### 4.2.2.2 Comparison between Two Scenarios

**I-5 Southbound Ramp @ Oceanside Blvd.**

Based on the simulation results, the average delay per vehicle is **57.3** sec and **44.3** sec under original signal timings and proposed ones, respectively. We further validated numerical analysis against the simulation test, and realized that the difference between the numerical results and simulation ones under different timings are **7.7 percent** and **11.3 percent**, respectively. As is shown in Figure 4.24, the improvement shown in simulation is **22.7 percent** by using optimal timings, while the average vehicle delay decreases as much as **20.1 percent** in numerical analysis.

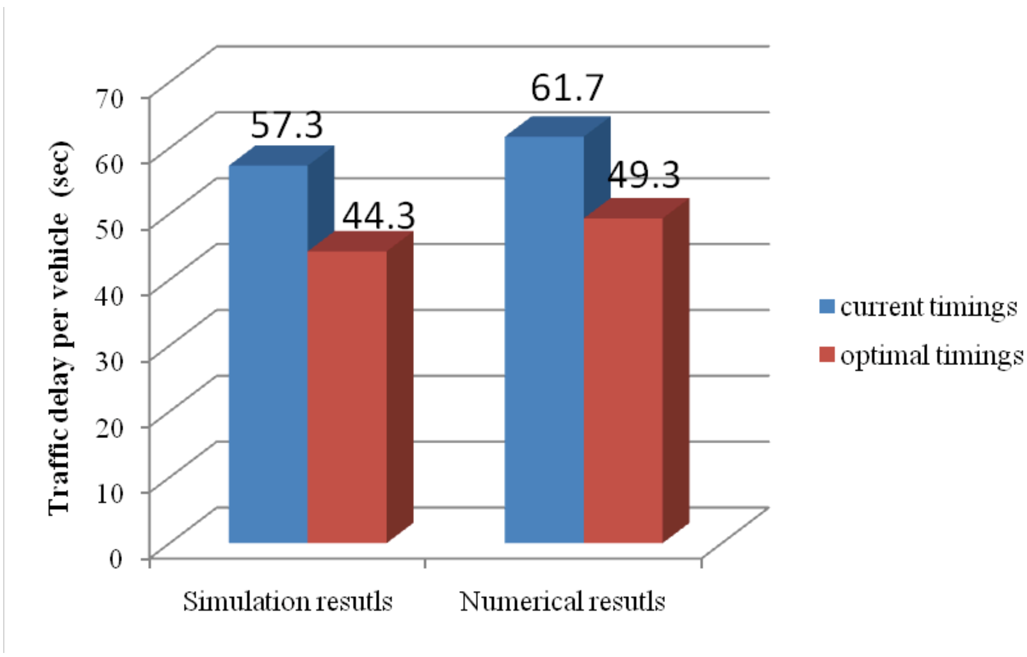



Figure 4.24 Comparison Results on Simulation and Numerical Analysis at I-5 Southbound Ramp @ Oceanside Blvd where Preemption Duration is 50 seconds.

**I-5 Northbound Ramp @ Oceanside Blvd.**

Simulation results shows that the average delays per vehicle are **56.8** sec and **50.5** sec under original signal timings and proposed ones, respectively. And the difference between the numerical results and simulation ones under different timings are **10.2 percent** and **0.0 percent**, respectively. As can be seen in Figure 4.25, the improvement is as low as **11.1 percent** by using optimal timings in simulation, but the average vehicle delay decreases as much as **19.3 percent** in numerical analysis.

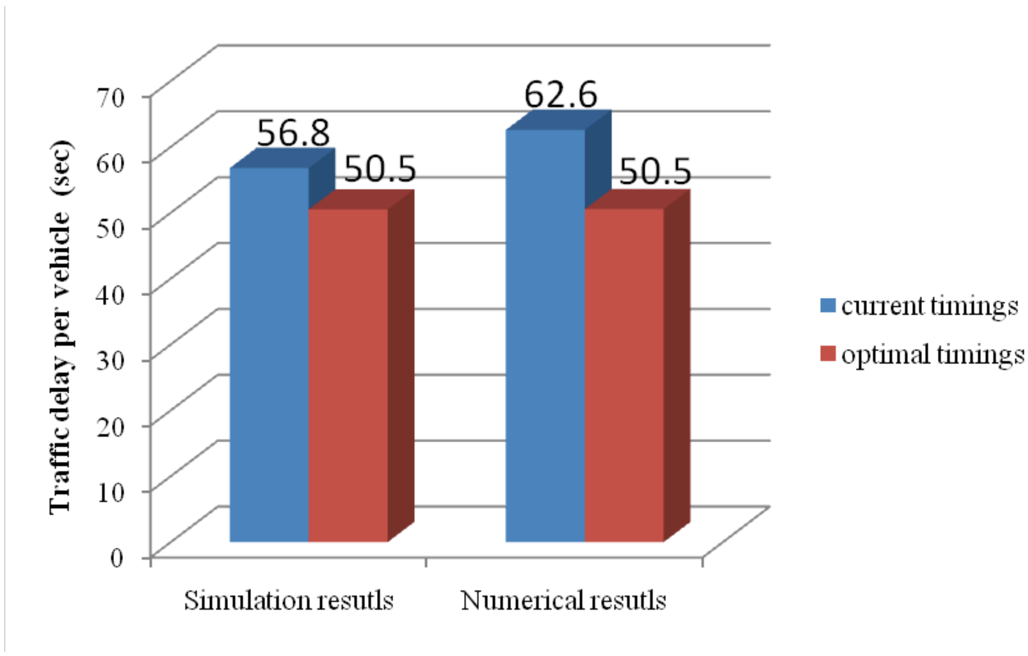



Figure 4.25 Comparison Results on Simulation and Numerical Analysis at I-5 Northbound Ramp @ Oceanside Blvd where the Preemption Duration is 50 seconds.

**Enterprise @ Mission Rd**

According to the simulation results, we can observe that the average delays per vehicle are **13.8** sec and **11.5** sec under original signal timings and optimal signal timings, respectively. We further validated numerical analysis against the simulation test, and realized that the difference between the numerical results and simulation ones under different timings are **11.3 percent** and **12.9 percent**, respectively. As is shown in Figure 4.26, the improvement shown in simulation is **16.4 percent** by using optimal timings, while the average vehicle delay decreases by **15.1 percent** in numerical analysis.

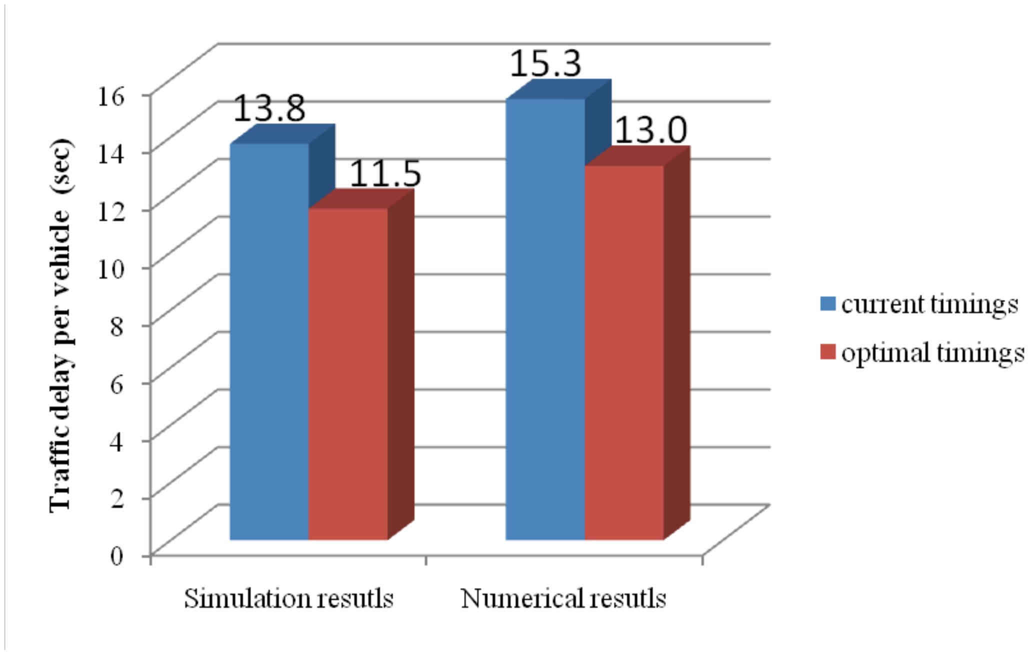



Figure 4.26 Comparison Results on Simulation and Numerical Analysis at Enterprise @ Mission Rd. where Preemption Duration is 50 seconds.

**Andreasen Dr @ Mission Rd**

As can be seen from Figure 4.27, the average delays per vehicle obtained from simulation are **23.7** sec and **21.3** sec under original signal timings and proposed ones, respectively. And the difference between the numerical results and simulation ones under different timings are **18.8 percent** and **10.5 percent**, respectively. Also, the improvement is as low as **10.1 percent** by using optimal timings in simulation, while the average vehicle delay decreases as much as **32.3 percent** in numerical analysis.

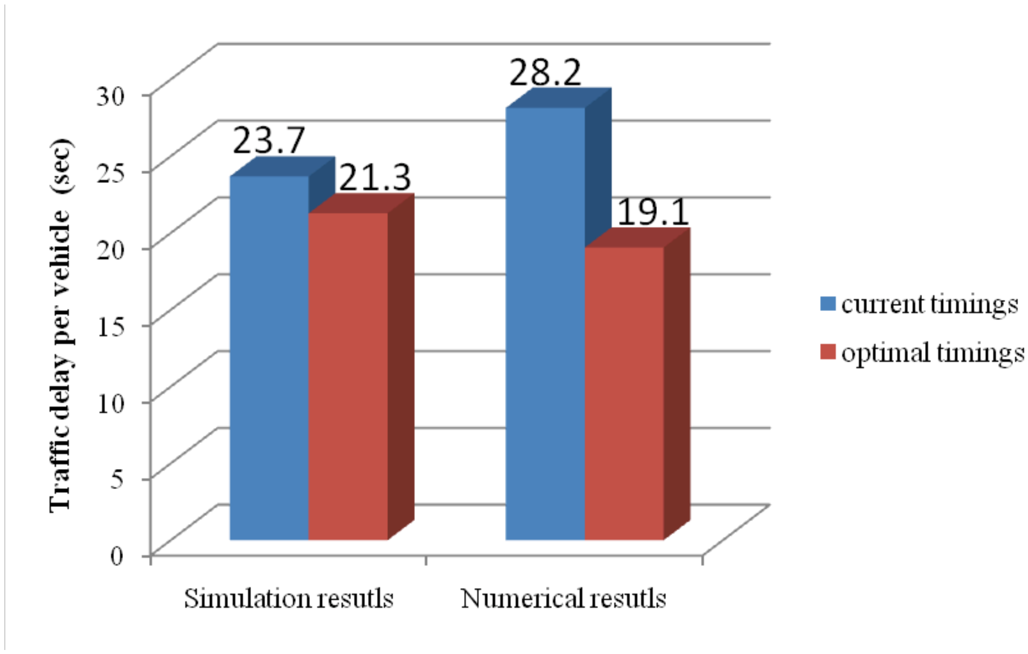



Figure 4.27 Comparison Results on Simulation and Numerical Analysis at Andreasen Dr. @ Mission Rd. where the Preemption Duration is 50 seconds.

**Olive @ Vista Village Dr.**

Based on the simulation results, the average delay per vehicle is **38.9** sec and **6.2** sec under original signal timings and proposed ones, respectively. We further validated numerical analysis against the simulation test, and realized that the difference between the numerical results and simulation ones under different timings are **23.6 percent** and **44.6 percent**, respectively. As is shown in Figure 4.28, the improvement shown in simulation is **84.1 percent** by using optimal timings, while the average vehicle delay decreases as much as **80.0 percent** in numerical analysis.

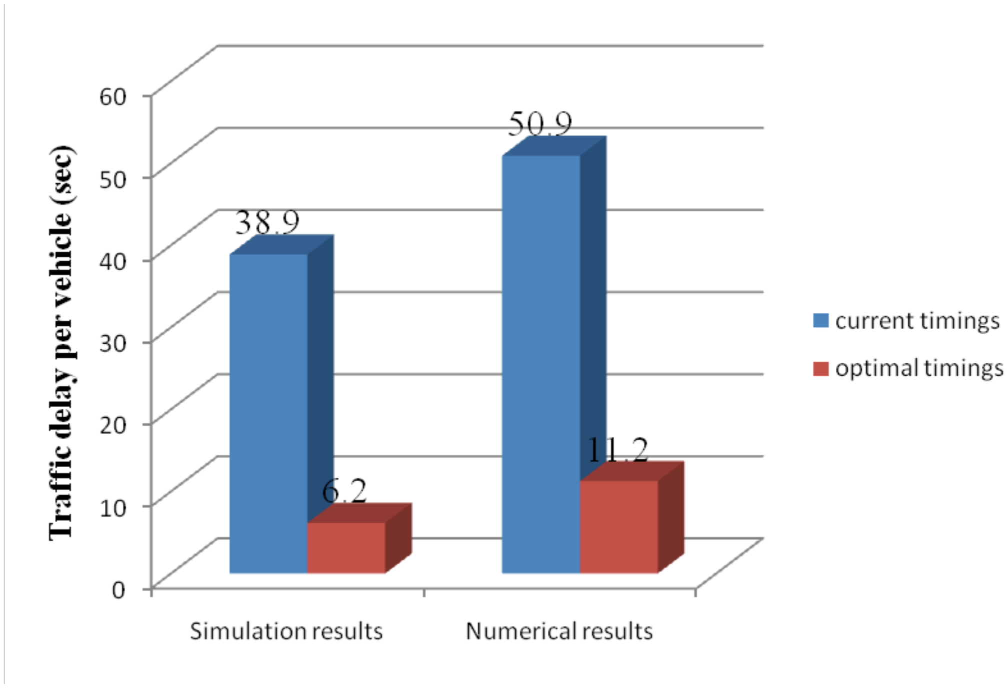



Figure 4.28 Comparison Results on Simulation and Numerical Analysis at Olive @ Vista Village Dr. where the Preemption Duration is 50 seconds.

**Santa Fe Ave. @ Main**

Simulation results shows that the average delays per vehicle are **30.8** sec and **11.7** sec under original signal timings and proposed ones, respectively. And the difference between the numerical results and simulation ones under different timings are 39**.3 percent** and **42.9 percent**, respectively. As can be seen in Figure 4.29, the improvement is as high as **62.0 percent** by using optimal timings in simulation, while the average vehicle delay decreases as much as **59.6 percent** in numerical analysis.

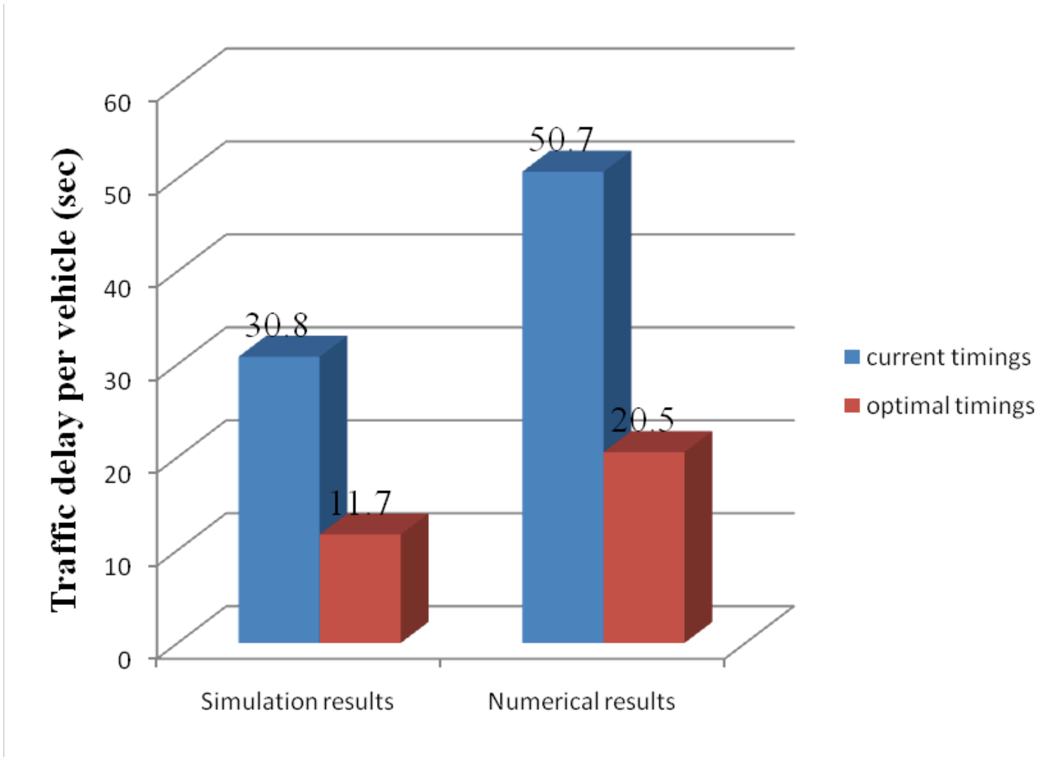



Figure 4.29 Comparison Results on Simulation and Numerical Analysis at Santa Fe Ave. @ Main where the Preemption Duration is 50 seconds.

**Santa Fe Ave. @ Vista Village Dr.**

According to the simulation results, we can observe that the average delays per vehicle are **41.6** sec and **12.3** sec under original signal timings and optimal signal timings, respectively. We further validated numerical analysis against the simulation test, and realized that the difference between the numerical results and simulation ones are **13.7 percent** under original timings, but **68.6 percent** under optimal timings, respectively. As is shown in Figure 4.30, the improvement shown in simulation is **70.4 percent** by using optimal timings, but the average vehicle delay decreases by **18.7 percent** in numerical analysis.

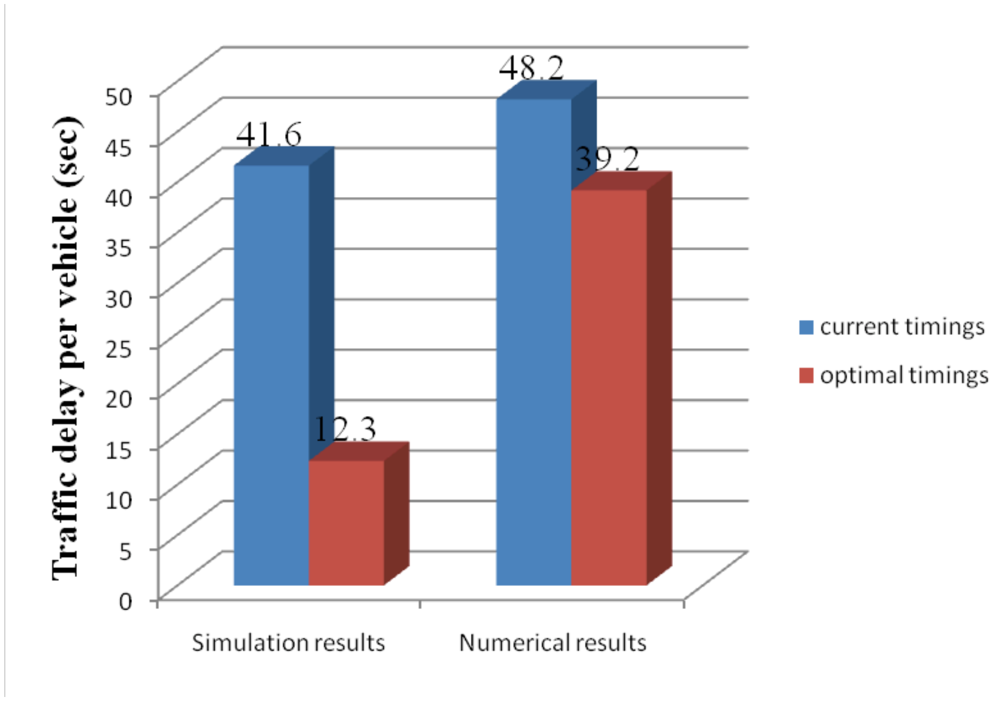



Figure 4.30 Comparison Results on Simulation and Numerical Analysis at Santa Fe Ave. @ Vista Village Dr. where the Preemption Duration is 50 seconds.

**Pala @ Escondido Ave.**

As can be seen from Figure 4.31, the average delays per vehicle obtained from simulation are **39.3** sec and **22.4** sec under original signal timings and proposed ones, respectively. And the difference between the numerical results and simulation ones under different timings are **42.0 percent** and **50.6 percent**, respectively. Also, the improvement is as high as **43.0 percent** by using optimal timings in simulation, while the average vehicle delay decreases as much as **33.2 percent** in numerical analysis.

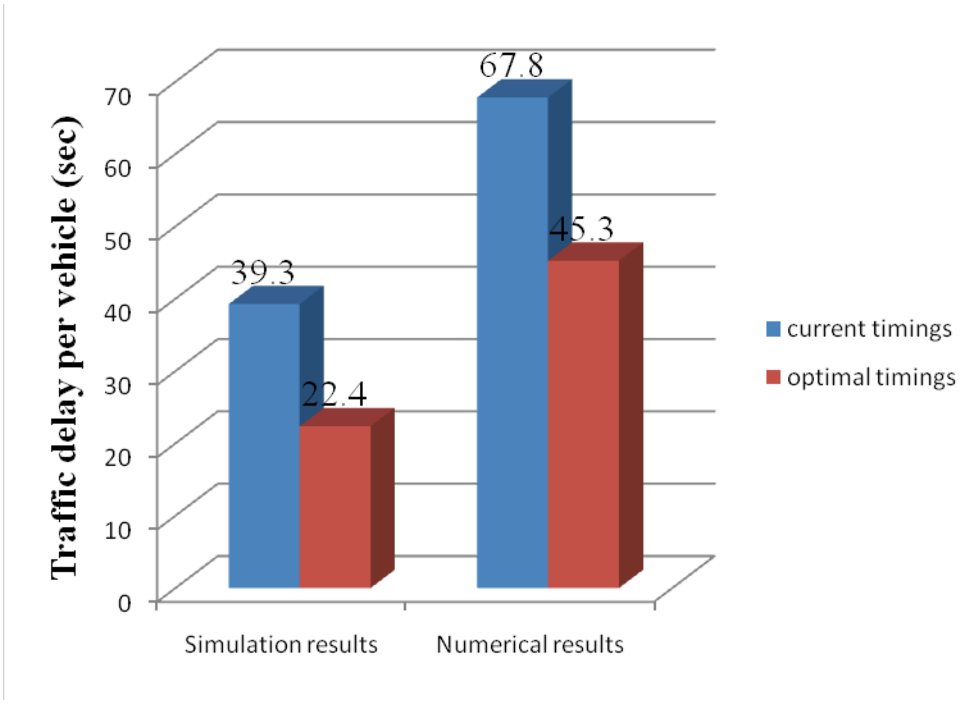



Figure 4.31 Comparison Results on Simulation and Numerical Analysis at Pala @ Escondido Ave. where the Preemption Duration is 50 seconds.

**Phillips @ Escondido Ave.**

Based on the simulation results, the average delay per vehicle is **57.4** sec and **18.6** sec under original signal timings and proposed ones, respectively. We further validated numerical analysis against the simulation test, and realized that the difference between the numerical results and simulation ones under different timings are **10.3 percent** and **16.2 percent**, respectively. As is shown in Figure 4.32, the improvement shown in simulation is **67.6 percent** by using optimal timings, while the average vehicle delay decreases as much as **65.3 percent** in numerical analysis.

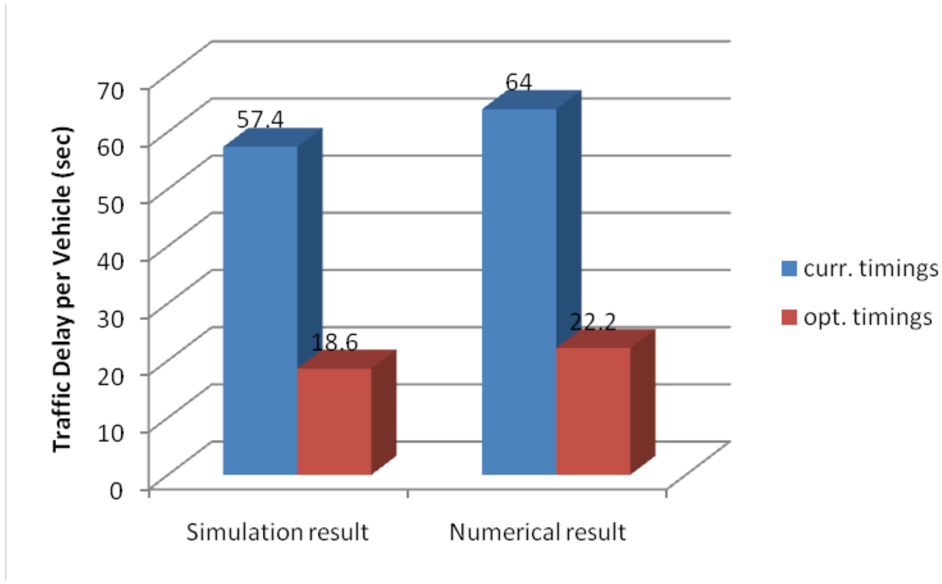



Figure 4.32 Comparison Results on Simulation and Numerical Analysis at Phillips @ Escondido Ave. where the Preemption Duration is 50 seconds.

## *5. Conclusion and Next Steps*

The SPRINTER Rail Transit Project is located in northern San Diego County. The rail line parallels the heavily-congested SR 78 corridor and is currently used in temporal and special separation by both freight and passenger transportation. It extends nearly 22 miles and connects the four North County cities - Oceanside, Vista, San Marcos, and Escondido, and unincorporated areas of San Diego County. The SPRINTER line started revenue service in March 2008.

In the project area, traffic congestion is already prevalent and traffic demand is increasing dramatically. With the traffic signal preemption provided to the new train service, the traffic congestion problem will further deteriorate if traffic signal control is not optimized.

The research team has installed GPS loggers on the trains as a means of advanced train detection. Data sent from the GPS loggers have been analyzed to study the train trajectories, station dwell time and other operating characteristics to help understand in detail the daily train operation.

This project identifies countermeasures that would minimize the impact of SPRINTER operations on local traffic operations. More specifically, it takes a two-step approach: the first step is to perform a basic signal timing optimization based on the train detection that is currently in place; the second step will use information from more advanced detection technologies and time-to-arrival prediction algorithms, so that the signal control can be further improved.

The research team worked closely with NTCD as well as the local jurisdictions to gather traffic control, traffic volume, and intersection-focused geometric information. Optimization models have been developed to minimize overall traffic delay at an intersection after the train preemption operation ends. Based on the time point of the train arrival during a traffic signal cycle, the duration of the preemption and intersection-specific information (lane geometry, traffic volume, etc.), the models estimate the waiting queue at the end of preemption, and try to clear the queue within as few cycle(s) as possible while at the same time, attempting to minimize overall traffic delay. The data collected from Caltrans and the corridor cities are fed into the model to develop sets of optimized signal timing plans for the intersections under consideration. Simulation models are developed using PARAMICS for evaluating these optimized plans. In the next phase, the optimization model will be calibrated using more updated and reliable information (for instance, traffic volume data), so that the optimized signal timing plans are more reliable and ready for field testing.

In the next phase of the project, a corresponding arrival time prediction algorithm will be proposed. In addition to more advanced train arrival prediction, an optimized signal timing and control algorithm will also be explored. The algorithm will use the more advanced information about train arrival and updated road traffic volume, as well as the preemption and safety requirements to optimize the street signal timing. This would allow timing adjustments to begin before the preemption is initiated and thereby provide better mitigation of the traffic impacts.