STATE OF CALIFORNIA • DEPARTMENT OF TRANSPORTATION

TECHNICAL REPORT DOCUMENTATION PAGE

TR0003 (REV 10/98)

1. REPORT NUMBER	2. GOVERNMENT ASSOCIATION NUMBER	3. RECIPIENT'S CATALOG NUMBER
CA14-2272		

4. TITLE AND SUBTITLE

Field Testing the Effectiveness of Adaptive Traffic Control for Arterial Signal Management

5. REPORT DATE

03/28/2014

6. PERFORMING ORGANIZATION CODE

7. AUTHOR

Robert Campbell and Alexander Skabardonis

8. PERFORMING ORGANIZATION REPORT NO.

UCB-ITS-PRR-2014-03

9. PERFORMING ORGANIZATION NAME AND ADDRESS

California PATH Program
Institute of Transportation Studies
University of California, Berkeley
109 McLaughlin Hall
Berkeley, CA 94720

10. WORK UNIT NUMBER

11. CONTRACT OR GRANT NUMBER

IA 65A0430

12. SPONSORING AGENCY AND ADDRESS

California Department of Transportation
Division of Research Innovation, and System Information MS-83
1227 O Street
Sacramento CA 95814

13. TYPE OF REPORT AND PERIOD COVERED

final report
October 18, 2011 to August 31, 2013

14. SPONSORING AGENCY CODE

15. SUPPLEMENTARY NOTES

16. ABSTRACT

The report describes the methodology and findings of the evaluation of adaptive signal control in a real life corridor. The study section was a five mile segment of the Pacific Coast Highway in Los Angeles with nine signalized intersections operating under adaptive control using the Los Angeles Department of Transportation (LADOT) Adaptive Traffic Control System (ATCS). Optimal fixed time time-of-day (TOD) plans were developed and implemented at the test site. The performance of the ATCS and the fixed TOD plans was evaluated using extensive field data on travel times and queue lengths collected through probe vehicles, Bluetooth sensors and video cameras. The findings indicate that ATCS performed better than the fixed-time plans during the time of peak direction in the arterial through traffic. All strategies had similar performance in the midday time period. A number of limitations was identified for ATCS under oversaturated conditions, including under allocating green time to the critical approach at the bottleneck intersection, allocating more green time than necessary at intersections upstream of the bottleneck, and inappropriate setting of offsets at intersections downstream of the bottleneck resulting in additional delays for traffic departing the bottleneck and creating the potential for queue spill-backs to the bottleneck itself. Possible remedial actions for these issues are discussed.

17. KEY WORDS	18. DISTRIBUTION STATEMENT
adaptive signal control, signal coordination, Bluetooth detector, probe vehicle, offsets, splits, cycle length, signal timing, signal phase, occupancy, level of service, platoon progression, peak period, green bandwidth, queue length	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161

19. SECURITY CLASSIFICATION *(of this report)*	20. NUMBER OF PAGES	21. COST OF REPORT CHARGED
Unclassified	84	

DISCLAIMER STATEMENT

CALIFORNIA PATH PROGRAM
INSTITUTE OF TRANSPORTATION STUDIES
UNIVERSITY OF CALIFORNIA, BERKELEY

Field Testing the Effectiveness of Adaptive Traffic Control for Arterial Signal Management

Robert Campbell
Alexander Skabardonis

California PATH Research Report
UCB-ITS-PRR-2014-03

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 Agreement 65A0430

March 2014

CALIFORNIA PARTNERS FOR ADVANCED TRANSIT AND HIGHWAYS

ABSTRACT

The report describes the methodology and findings of the evaluation of adaptive signal control in a real-life corridor. The study section was a five mile section of the Pacific Coast Highway in Los Angeles with nine signalized intersections operating under adaptive control using the Los Angeles DOT ATCS system. Optimal fixed time time-of-day plans were developed and implemented at the test site. The performance of the ATCS system and the fixed-time plans was evaluated using extensive field data on travel times and queue lengths collected through probe vehicles, Bluetooth sensors and video cameras. The findings indicate that ATCS performed better than the fixed-time plans during the time of peak direction in the arterial through traffic. All strategies had similar performance in the midday time period. A number of limitations was identified for ATCS under oversaturated conditions, including under allocating green time to the critical approach at the bottleneck intersection, allocating more green time than necessary at intersections upstream of the bottleneck, and inappropriate setting offsets at intersections downstream of the bottleneck resulting in additional delays for traffic departing the bottleneck and creating the potential for queue spillbacks to the bottleneck itself. Possible remedial actions for these issues are discussed.

ACKNOWLEDGEMENTS

This work was performed by the California PATH Program at the University of California at Berkeley, in cooperation with the State of California Business, Transportation and Housing Agency, Department of Transportation (Caltrans), Division of Research, Innovation and Systems Integration (DRISI) under the Interagency Agreement #65A0430. The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the State of California.

The authors thank John Slonaker of Caltrans DRISI for his support and advice during the project. We are grateful to Ajaykumar Shah, Dean De Leon and Lap Nguyen of Caltrans District 7 signal operations for their assistance throughout the project. We also thank the project technical advisory committee members Ted Lombardi, Kai Leung, and Jorge Fuentes of Caltrans for their comments and suggestions throughout the study.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

Several adaptive systems have been developed and implemented for control of signalized intersections along arterials and networks [1]. These systems adjust the settings at traffic signals (cycle length, green times, offsets) based on real-time data on traffic conditions, and also can respond to unexpected or unplanned events, such as incidents, special events, and adverse weather. Reported benefits of adaptive systems include travel time and stop savings, smoother traffic flows and fuel and air pollutant reductions [2]. Limitations for their wide spread deployment include capital, operational and maintenance costs for the system hardware and software, and complexity of underlying control algorithms.

In a recent PATH study [3], a comprehensive literature review on signal control strategies for arterials was undertaken. A total of twelve control strategies were tested through the PARAMICS microscopic simulation model [4] on a real-life arterial site. The control strategies tested included the RHODES and TUC adaptive strategies, plus traffic responsive plan selection, isolated and coordinated actuated control, and the fixed-time plans operating at the site. Additional model runs were also performed to evaluate the effectiveness of strategies by testing the impacts of 5% and 10% uniform increases in traffic demands. The simulation results showed that the RHODES adaptive signal control strategy is the best strategy in terms of both the overall system and arterial only traffic performance.

The objective of this follow-up study is to conduct a field test of an adaptive signal control strategy to determine its effectiveness under real-world operating conditions. The findings of the evaluation will assist the California Department of Transportation (Caltrans) in selecting signal control strategies on arterial highways, and in developing a systematic integrated control of freeways and arterials for corridor management.

This document is the final report for the project. The next Chapter presents the study methodology, and the characteristics of the selected study site. Chapter 3 describes the field data collection and processing. Chapter 4 describes the development and implementation of optimized fixed-time time of day timing plans. Chapter 5 presents the findings from the evaluation of the control strategies based on field data. The last Chapter summarizes the study findings, and suggests future research on arterial signal control.

CHAPTER 2
METHODOLOGY

The primary emphasis on this project is to evaluate the effectiveness of an operational adaptive control algorithm against fixed-time time of day plans which is the predominant control strategy along signalized arterials. The following sections discuss key considerations and steps in the evaluation process, and the selected study section for the field tests.

2.1 Evaluation of the Alternative Control Strategies

The evaluation of adaptive signal control against the conventional fixed-time control should be based on the comparison of adaptive control against fixed-time control with optimal signal settings to estimate the true benefits of adaptive control [5]. It is also important that data on traffic demand and other operating conditions throughout the field experiments are carefully analyzed to ensure that the differences in traffic performance are due to the control strategies and not external factors. The measures of effectiveness (MOEs) for the evaluation of the alternative strategies will be the travel time on arterial links supplemented by delays and queue lengths on arterial links and cross-streets.

The field testing of the adaptive control strategy on the selected site involves the following steps:

- "Before" data collection: Field data collection on MOEs will be collected under the operation of the existing adaptive system under a range of operating conditions.
- Development of Optimized Time-of-Day Fixed-Time Plans: Optimized fixed time plans will be developed for the typical three time periods of the day (am peak, midday, pm peak) using data on traffic demand from the surveillance system of the adaptive control algorithm. The timing plans will be tested through a microscopic simulation model to identify and correct any operational issues prior to the field implementation.
- Field implementation and fine tuning: The optimized timing plans will be implemented in the field, and adjustments will be made as appropriate based on field observations prior to the collection of the traffic performance data.
- "After" data collection: Field data on the MOEs will be collected following the implementation of fixed-time plans.
- Analysis of the field measurements: The "before" data on MOEs will be used to assess the operation of the adaptive system under a range of operating conditions. Comparison of the 'before" and "after" data will determine the effectiveness of adaptive control against to the optimized fixed-time plans.

The technology for field data collection will depend on the characteristics of the test site and the surveillance system capabilities. For example we will utilize the availability of images from the video surveillance cameras at the intersections, to obtain estimates of delays and queue lengths through image processing. Floating cars equipped with GPS units along with Bluetooth sensors, will be utilized to obtain estimates of arterial link travel times and delays. The duration of the field tests will provide sufficient data to determine if statistically significant improvements have been obtained. At a minimum one week of "before" and "after" field data will be collected.

2.2 Test Site Selection

This research effort will field test and evaluate an adaptive signal control system against conventional fixed-time control on a real-world arterial. It is therefore important that a suitable test site is selected for the evaluation of the control strategies. Criteria for test site selection include:

- Traffic volumes and patterns: The selected site should have both heavy traffic volumes and highly variable traffic patterns. In general, there are no significant benefits from adaptive control if the arterial volumes follow a predictable traffic pattern throughout the day. Furthermore, if the traffic volumes are well below capacity then the variability in traffic patterns can be accommodated with conventional fixed-time or actuated control.
- Existing signal system capabilities: signal hardware and software and communications that allow central or decentralized monitoring of the signal system and remote implementation of changes in signal settings. Also, the site should already operate under adaptive signal control. Installation of a new adaptive system is not possible because of time and budget constraints of the project.
- Data availability: Surveillance system (loop detectors and video cameras) in place that provide data on traffic demands and operating conditions at each signalized intersection in the study area.
- Cooperation with Caltrans & local agency staff: Staff in charge of the system operation and maintenance willing to participate in the study, review the proposed algorithms and provide cooperation and support throughout the field test

A section of Pacific Coast highway (PCH) in Los Angeles was selected as the study site. This state highway is one of the primary traffic corridors linking the greater Los Angeles area with the cities of Malibu, Oxnard, and Ventura. The study segment begins at the connection to Interstate 10 in Santa Monica and ends at the Malibu city limit, with Topanga Canyon Blvd and California Incline serving as its boundary intersections (Figure 2-1). On this 5-mile stretch, PCH has two to three lanes in each direction and average day time volumes (i.e., 6 AM to 8 PM) of 34,000 vehicles in the north bound and 38,000 vehicles in the southbound direction. There are nine signalized intersections on the study segment, all of which are controlled by the Los Angeles DOT Adaptive Traffic Control system (thereafter called ATCS). The system is maintained and operated by the Caltrans District 7 signal operations staff.

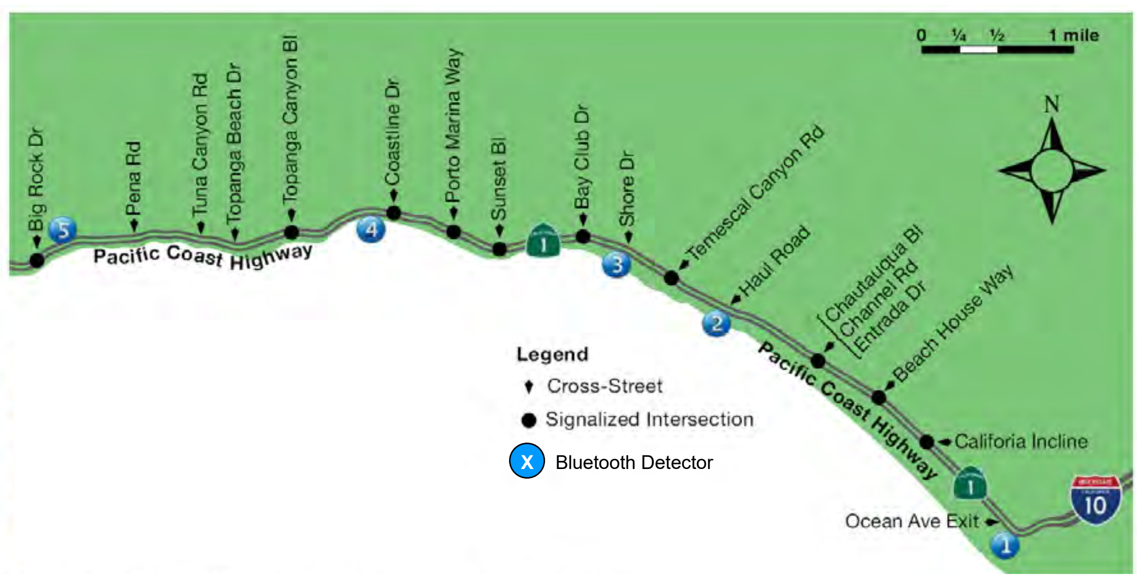


Figure 2.1 the Study Section

The ATCS system [6] updates cycle lengths, splits, and offsets at each intersection once per cycle based on prevailing traffic conditions. Changes to each parameter are incremental and are based on detector data from each intersection, although the system has the ability to vary the size of these increments up to a certain threshold depending on how quickly traffic conditions are changing. Splits are based on traffic volumes and occupancies of each approach, offsets are based on minimizing the number of stops for the approach with the highest flows (with special priority given to coordinated directions), and a section-wide cycle length is based on the minimum time needed to keep all signals in a particular section operating below saturation. Loop detectors are located on all approaches to the major phases, and are typically placed 200 to 300 feet in advance of the intersection to measure platoon arrival patterns. In the case of PCH, the cycle times were constrained to 240 seconds during the AM and PM Peak throughout the field testing of the ATCS system (March through June 2013), leaving only the splits and offsets available for adjustment at these times. There are stop bar detectors on all approaches at each intersection, with suitably placed green extension detectors on PCH and major cross-streets as well. For cross-streets, green extension was granted for vehicles crossing both upstream and stopline detectors. Figure 2-2 shows an intersection display at the Caltrans TMC with signal timing information and detector location.

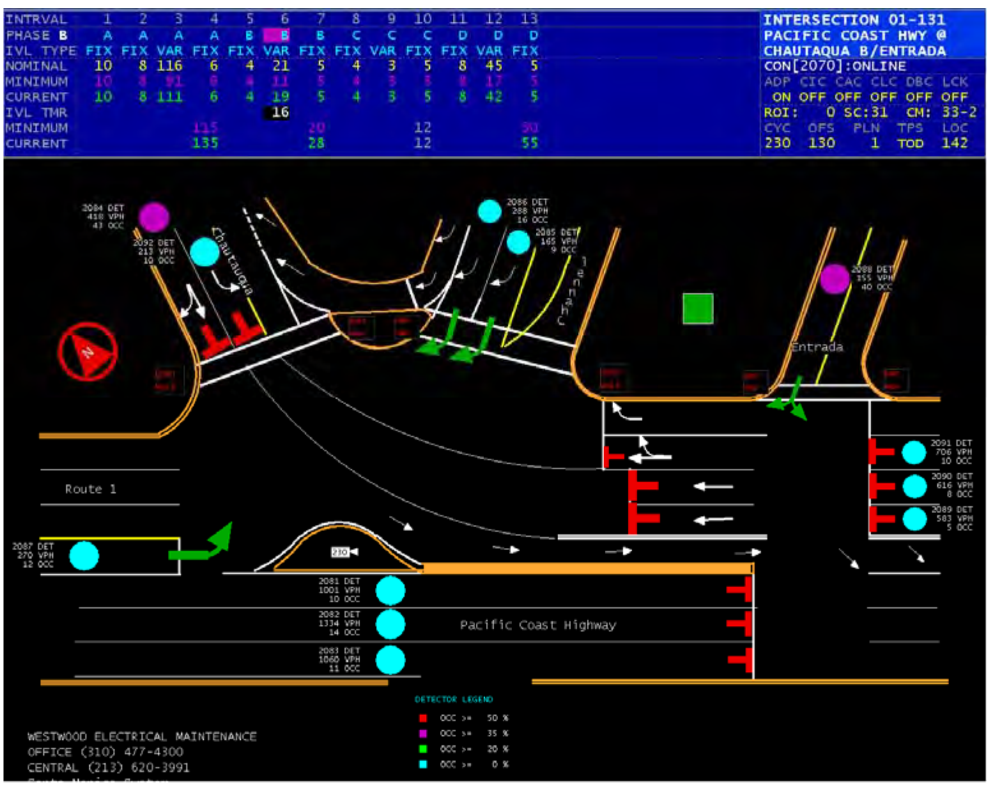


Figure 2.2 PCH/Chautaqua Blvd Intersection Display: Signal Settings and Detector Locations

Phase splits and offsets are available for each intersection on each signal cycle, and limited video data are also available at major cross streets to monitor queue lengths. Volume and occupancy data for each intersection are available in the form of 15-minute detector summaries (Figure 2.3). Travel time data are obtained using Bluetooth readers at five locations as shown in Figure 2-1. The collection and processing of the field data are described in Chapter 3.

ADAPTIVE TRAFFIC CONTROL SYSTEM
DETECTOR HISTORY REPORT
DATA FOR TUESDAY
Date: 10/25/2011
VOLUME IN VEHICLES
OCCUPANCY IN PERCENT
DETECTOR 2049

TIME	:00		:15		:30		:45		HOURLY		STATUS
	VOL	OCC	VOL	OCC	VOL	OCC	VOL	OCC	VOL	OCC	
05:00	28	0	38	1	45	1	57	1	168	1	OK
06:00	87	2	102	3	138	4	173	10	500	4	Pulse Mode
07:00	224	13	273	20	293	16	294	23	1084	18	OK
08:00	295	20	286	20	293	17	261	18	1135	19	OK
09:00	256	19	243	17	241	18	233	15	973	17	OK
10:00	241	15	220	12	189	7	217	10	867	11	OK
11:00	181	8	190	7	173	7	175	5	719	7	OK
12:00	170	5	185	11	155	5	182	8	692	7	OK
13:00	174	7	164	6	207	9	164	7	709	7	OK
14:00	177	11	222	14	218	12	180	5	797	11	OK
15:00	189	17	219	17	225	13	226	16	859	15	OK
16:00	183	10	202	12	192	10	189	14	766	12	OK
17:00	201	7	159	9	153	4	169	6	682	7	OK
18:00	138	4	148	8	131	4	111	3	528	5	OK
19:00	133	4	103	3	114	4	88	2	438	3	Pulse Mode
20:00	79	2	100	5	89	2	73	2	341	3	OK
21:00	83	2	77	2	69	1	67	2	296	2	OK
22:00	64	1	68	2	58	1	51	1	241	1	Pulse Mode
23:00	47	1	28	0	38	1	21	0	134	1	OK

TOTAL VOLUME 11929
AVERAGE OCCUPANCY 8
08:00 AM PEAK OCCUPANCY 19
08:00 AM PEAK VOLUME 1135
15:00 PM PEAK OCCUPANCY 15
15:00 PM PEAK VOLUME 859

Figure 2.3 Detector Report PCH ATCS System

CHAPTER 3

DATA COLLECTION AND PROCESSING

This Chapter describes the collection and analysis of field data for the development and evaluation of alternative control strategies along the PCH study corridor.

3.1 Volume Data

Detector count data are provided by Caltrans for each intersection and approach, disaggregated by detector and by 15-minute period between 6 AM and midnight. Figure 3.1 shows the dates of available data between March and June 2013. Additional data were also provided for several days in June 2012, to coincide with the preliminary period of travel time data collection. The data are provided in the form of Detector History Report files (text files), which are then imported and processed within Microsoft Excel using a spreadsheet designed specifically to analyze and summarize these reports.

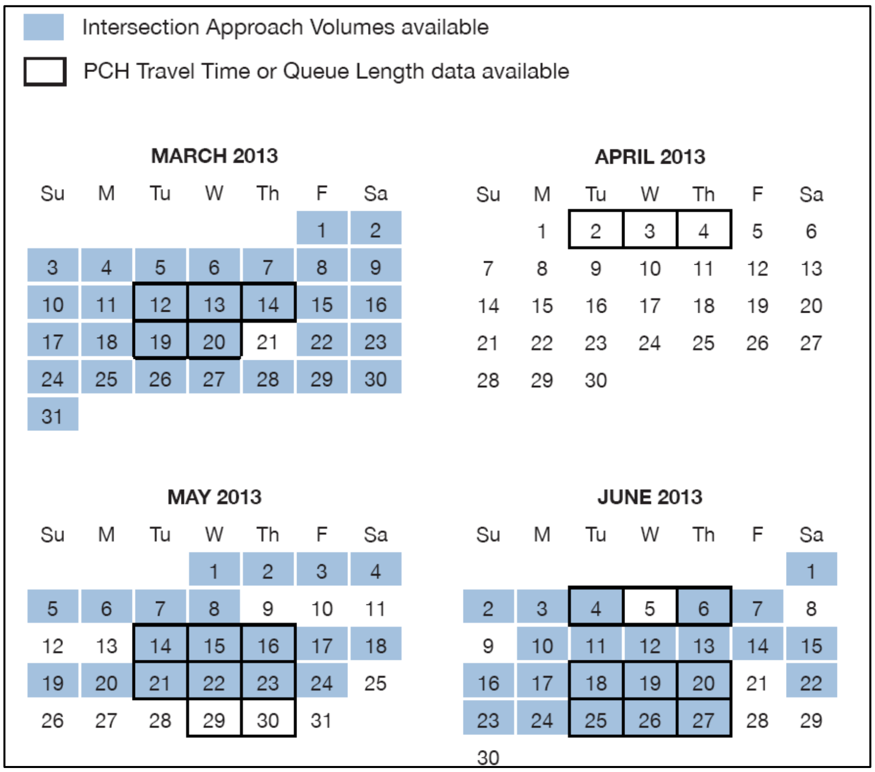


Figure 3.1 Dates of Available Volume Data

Missing days of data shown in Figure 3.1 are generally indicative of data that Caltrans was unable to provide. This was a consequence of the fact that the Caltrans staff had to archive the data manually, and had only a one-week window of opportunity to do it for each date before the system would overwrite it with new data. The gap on March 21, however, was due to a landslide on PCH that blocked the northbound lanes for most of the day, such that the demand data—though available—were atypical and unreliable.

The system had the ability to perform basic detector error checking, by identifying any hour-long period where no vehicles were detected by a particular detector. However, in our analysis, we considered any 15-minute period without vehicle presence to be a sign of detector error. If such a period appears as an isolated incident, we use interpolation between the periods immediately before and after the empty interval to estimate the missing volume data. In some cases, however, missing data occurs for many hours at some detectors. For these cases, we rely on data from the same weekday in the preceding and/or following week to guide us as we fill in the gap with an appropriately-scaled demand curve. Specifically, we use the four 15-minute periods immediately before and after the data gap to evaluate the ratio between demands in the current week and the preceding/following week. We then apply this ratio to the measured volumes during the period of data dropout in the adjacent weeks to obtain rescaled demand curves for filling in the missing data of the current week.

Some assumptions regarding the volume data were required due to the limitations of detector placement. For example, turning ratios had to be assumed for shared lanes and in the case of lanes without detection at all, these volumes had to be estimated using data from neighboring detectors. We had to make the following assumptions for this analysis:

- For the left turns from Topanga Canyon Blvd to PCH, only one of the two lanes had detection. To estimate the total left turning volumes, we doubled the volumes measured by the one detector.
- For the left turns from Sunset Blvd to PCH, one of the three lanes is a shared left/through/right turn lane. For the purposes of our volume analysis, we include all traffic using this shared lane regardless of whether the vehicle ultimately turned left or made another movement instead.
- For the left turns from Chautauqua Blvd to PCH, the turn lane is a shared left/right turn lane. As with Sunset Blvd, we include all traffic using this shared lane regardless of whether the vehicle ultimately turns right or left onto PCH.

3.1.1 Determination of Analysis Periods

The objective of the analysis of the volume data was to compare traffic patterns between different periods of travel time and queue length data collection on PCH. Thus, our first step in the analysis involves splitting the volume data into appropriate groups that reflect these different data collection periods.

Also, we are only concerned with Tuesdays, Wednesdays, and Thursdays, as other weekdays are considered less consistent or predictable with respect to traffic volumes and travel patterns, and are poorly suited for comparative analyses. Furthermore, we are only interested in performing volume analyses over the days for which we have queue length or travel time data, since our goal is to assess the extent to which the performance measures gathered during the various data collection periods are comparable. Thus, the different periods investigated with respect to traffic volumes are:

- **Preliminary ATCS data collection (ATCS-2012):** Using video cameras and probe vehicles equipped with GPS tracking equipment, we measured travel times on the PCH corridor and queue lengths on major cross-streets in June 2012. Days analyzed in June 2012 were 6-7, 6-12, 6-13, and 6-14.
- **Primary ATCS data collection (ATCS):** Using Bluetooth detectors, we collected travel time data on the PCH corridor between March 12 and April 4, 2013, prior to implementing our optimized time-of-day (TOD) timing plans. Queue length videos could not be collected during this time because of an unplanned video outage caused by a contractor that accidentally cut the fiber optic data connection supplying these camera feeds from the field to the TMC. When considering the days for which we have volume data in this period, and when excluding weekends, Mondays, Fridays, and March 21 (because of the landslide that disrupted PCH traffic), we are left with the following days for this volume analysis period: 3-12, 3-13, 3-14, 3-19 3-20, 3-26, 3-27, and 3-28.
- **TOD-230 data collection (TOD-230):** The first set of TOD plans to be implemented used the same 230-second cycle length that ATCS was using at the time the plans were developed (June 2012). This was done to provide the most reasonable comparison between ATCS and TOD operation; as Caltrans was forcing ATCS to use a fixed 230 second cycle length in 2012 and a 240 second cycle length during the field test in 2013, it was appropriate to constrain our TOD optimizations in the same way. This period is referred to as the "TOD-230" implementation, and occurred between May 16 and June 14, 2013. However, queue length and travel time data could only be collected between May 16 and June 6, as a result of battery life and memory card capacity limitations. Although TOD-230 data were

also available on May 15, they were excluded because this was a transition day between ATCS and TOD-230 operation, and so we exclude it to avoid any traffic disruptions or performance irregularities associated with this shift in control. When considering the days for which we have volume data in this period, and when excluding weekends, Mondays, and Fridays, we are left with the following days for this volume analysis period: 5-16, 5-21, 5-22, 5-23, 6-4, and 6-6.

- **TOD-optimized data collection (TOD-Optimized):** In addition to the TOD-230 timing plans, we also implemented an additional set of timing plans for which the 230-second cycle length constraint was removed. These timing plans, which we will refer to as "TOD-Optimized", were deployed on PCH in June 2013. Travel time and queue length data were collected throughout this entire two-week period. The days in this volume analysis period are: 6-18, 6-19, 6-20, 6-25, 6-26, and 6-27.

3.1.2 Analysis of the Volume Data

Average Volumes

We estimated the daytime (6 AM to 8 PM) volumes measured at all major entrances to the PCH study corridor for the dates specified in the "Analysis Periods" section earlier. For cross-street approaches, the volumes are measured only for the lanes serving the critical movement. Specifically, these critical movements by cross-street are:

1. Topanga Canyon Blvd: left turns onto southbound PCH
2. Sunset Blvd: left turns onto southbound PCH
3. Temescal Canyon Rd: left turns onto southbound PCH
4. Chautauqua Blvd: left turns onto southbound PCH
5. Channel Rd: right turns onto northbound PCH

Figures 3.2 and 3.3 show volume plots for two major cross-streets: Temescal Canyon Rd and Sunset Blvd. It can be seen that traffic volumes decreased during the week of March 26, with the drop being most apparent at Temescal Canyon Rd. This may be due to the Spring Break, which occurred at Palisades Charter High School on Temescal Canyon Rd (and possibly at other schools in the area as well) during that time. Similar drop in volumes were found at the rest of the cross streets. Because we expect that this would have an impact on the performance of PCH in this area, it would be reasonable to exclude this week of data when conducting our comparative analysis between ATCS and TOD-230 control strategies.

Figure 3.2 shows a very pronounced decline in volumes at Temescal Canyon Rd in June 2013 with the drop becoming more severe toward the end of the month. The same pattern was observed in the rest of the southernmost intersections on PCH (Chautauqua Blvd, Beach House and California Incline). This drop may be related to communication problems associated with the four PCH intersections that Caltrans became aware of toward the end of June. The Detector History Reports show that starting in June 4, several gaps begin appearing intermittently in the 15-minute volumes at these four southernmost intersections, with the gaps becoming more frequent toward the end of the month. As a result, the detector data for June at these four southernmost intersections are deemed unreliable and excluded from the volume analysis.

However, although there is no other reasonable data source we can use as a substitute for the cross-street volumes at Temescal Canyon Rd, Chautauqua Blvd, and Channel Rd, we can use the northbound PCH volumes at Bay Club Drive to obtain a general idea of the northbound volumes at California Incline. While the data for the two intersections are not directly comparable due to the opportunities for vehicle ingress and egress between them (particularly at Chautauqua Blvd, where a significant volume of traffic enters northbound PCH from Channel Rd, and a significant volume exits northbound PCH for Chautauqua Blvd), we can still use the volumes at Bay Club Drive to gain insight into any shifts in traffic patterns on PCH that occurred between the TOD-230 and TOD-Optimized analysis periods.

Table 3.1 shows the average traffic volumes at all major entrances to PCH for each analysis period after the adjustments for the Spring Break period and data losses in late June 2013.

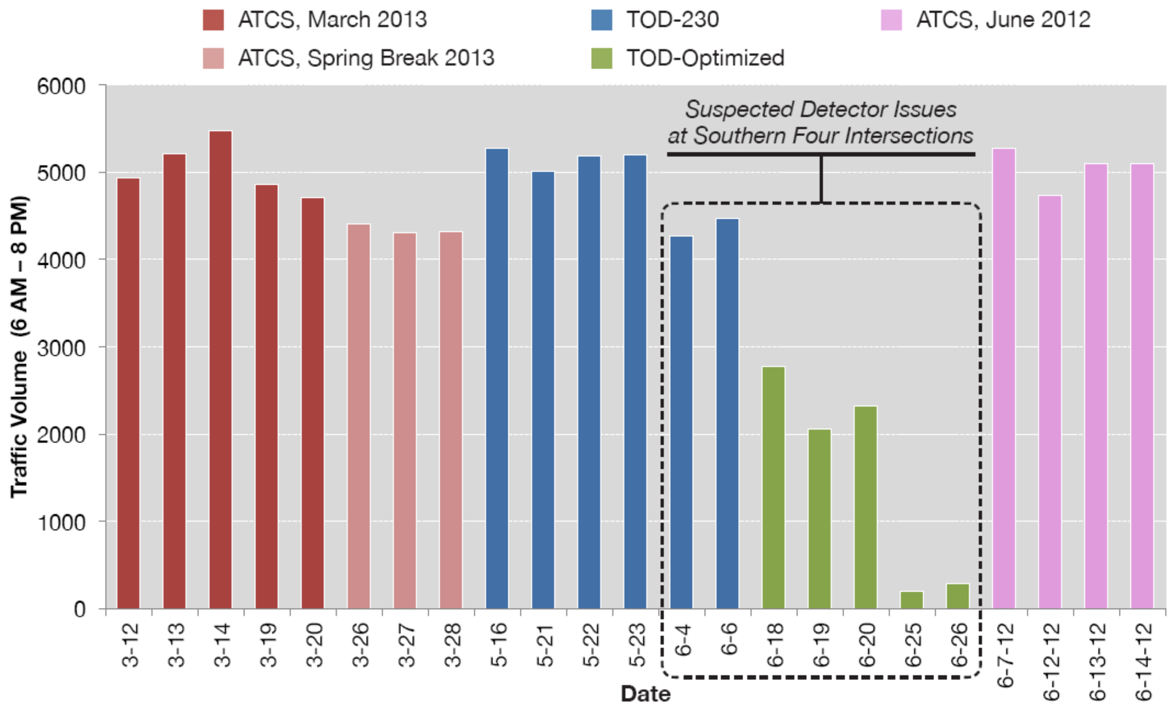


Figure 3.2 Daytime Traffic Volumes--Temescal Canyon Rd (left turns to PCH)

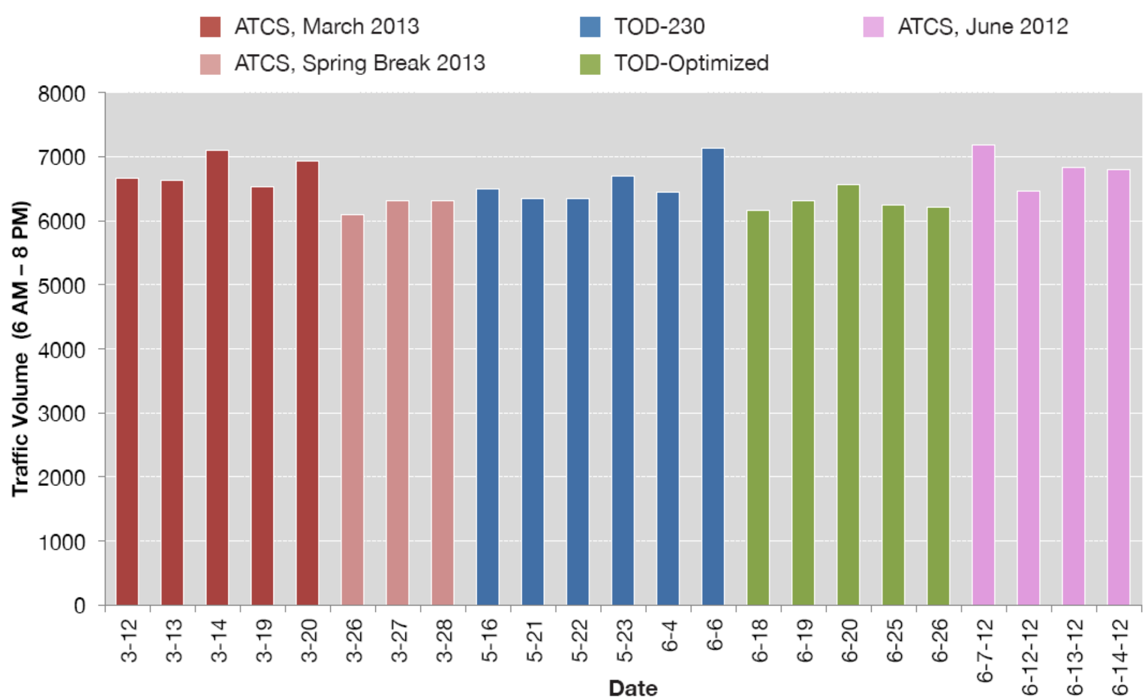


Figure 3.3 Daytime Traffic Volumes--Sunset Blvd (Left Turns To PCH)

Table 3.1 Average Daytime Volumes (6 AM To 8 PM) at All Major Entrances to the PCH Corridor

	ATCS*	Spring Break	TOD-230	TOD-Optimized	ATCS-2012
PCH southbound at Topanga Canyon Bl	18,796	19,440	19,578	19,937	19,828
PCH northbound at CA Incline	29,577	29,183	27,182	N/A	30,204
Topanga Canyon Bl (left turns to PCH)	7,543	7,145	7,700	8,195	7,925
Sunset Bl (left turns to PCH)	6,772	6,238	6,576	6,301	6,824
Temescal Canyon Rd (left turns to PCH)	5,036	4,346	4,899	N/A	5,050
Chautauqua Bl (left turns to PCH)	4,444	4,038	4,220	N/A	4,478
Channel Rd (right turns to PCH)	6,370	5,939	6,311	N/A	7,105
PCH northbound at Bay Club	—	—	28,238	28,480	—

"N/A" indicates that the average could not be calculated due to insufficient data for the TOD-Optimized period.
— indicates that the average was not necessary.
**Excludes Spring Break 2013*

We performed a series of statistical tests to evaluate whether significant differences in volume exist between any of the analysis periods to assess whether any demand effects may be influencing the differences in traffic performance (e.g., travel times) found between any analysis periods.

We found no statistically significant differences between the ATCS and TOD-230 volume on a 5% level, giving us confidence in the reliability of our assessment of ATCS operation with respect to travel time data. In only one case, on southbound PCH at Topanga Canyon Blvd, the difference was significant at a 10% level. Table 3.1 shows that the average traffic volume increased between the ATCS and the TOD-230 periods, suggesting that any measured drop in performance on southbound PCH during the TOD-230 implementation may be caused to an extent by an increase in traffic volume. However, when we examine only the volume data for the critical 6 AM to 10:30 AM period, we obtain a p-value of 0.58 for this two-tailed t-test, indicating that the volumes are still comparable in the AM Peak. A similar test on volumes for the 2–8 PM Peak period yields a p-value of 0.234 whereas a test on the midday volumes gives a p-value of 0.027, indicating that the difference in volumes on southbound PCH at Topanga Canyon Blvd is due to changes in the midday rather than differences in the AM or PM Peak periods.

Next, we investigate the relationship between ATCS-2012 volumes and TOD-230 volumes. We found that the volumes on the side street approaches and on the PCH mainline are not significantly different on a 10% level between the two analysis periods, except at Channel Rd where the p-value was 0.015. Table 3.1 reveals that the volumes at Channel Rd dropped significantly after the ATCS-2012 period, which suggests that shorter queues observed on Channel Rd during the TOD-230 period might be explained to an extent by a reduction in volume rather than the switch from ATCS to TOD timing plans. Although the ATCS volumes from 2012 more closely match the TOD-230 volumes than do the ATCS volumes from 2013, we will use the travel time measurements from 2013 because that data is much richer than the travel time data from 2012, and also because the volumes were not significantly different between ATCS (in March 2013) and TOD-230 during the AM and PM Peak periods.

The comparison of volumes during the ATCS and ATCS-2012 analysis periods shows that all the major side street approaches except Channel Rd had comparable traffic volumes, but that the PCH mainline experienced declines in volume between June 2012 and March 2013 that were significant at a 5% level. However, when looking only at the PM Peak, the volumes were not significantly different—we obtain a p-value of 0.115 for PCH in the southbound direction at Topanga Canyon Blvd, and a p-value of 0.579 for PCH in the northbound (peak travel) direction at California Incline. For the AM Peak, our p-values for the PCH mainline are 0.483 on the southbound side at Topanga Canyon Blvd and 0.009 on the north bound side at California Incline. Thus, the mainline volumes are comparable between the ATCS and ATCS-2012 data collection periods for the southbound direction during the AM Peak and for both directions during the PM Peak.

Regarding the cross-street volumes in this ATCS/ATCS-2012 comparison, our results support those obtained earlier in our comparison of ATCS-2012 and TOD-230, which indicated that no significant changes in demand occurred between the ATCS and TOD-230 analysis periods on any of the major side street approaches apart from Channel Rd. With respect to travel time data, our findings in comparing ATCS and ATCS-2012 reveal that our PM Peak volumes did not change between the two analysis periods, as did our AM Peak volumes in the critical southbound direction. Although we will not be using the travel time data from the ATCS-2012 period for comparison to TOD-230 travel times, we will use the 2012 probe vehicle data to obtain insight into the disaggregate travel times on individual links of PCH, which is not possible with the Bluetooth data from 2013. Based on these volume analysis results, our disaggregate travel times by link from the probe vehicle runs may also be used to represent the travel time characteristics of each link in 2013, with the exception of the northbound side of PCH in the AM Peak.

In our analysis of the traffic volumes during the TOD-230 and TOD-Optimized data collection periods, we find that the northbound and southbound PCH volumes are not significantly different, whereas the volumes at Sunset Blvd and Topanga Canyon Blvd are significantly different at a 10% level. Note, however, that the t-test for northbound PCH volumes is done using the measurements at Bay Club Drive, since the southernmost four intersections provided unreliable data during the TOD-Optimized analysis period. Despite being different at a 10% significance level, the volumes at Topanga Canyon Blvd and Sunset Blvd were not significantly different during the PM Peak, with p-values of 0.219 for Topanga and 0.183 for Sunset Bl. However, during the AM Peak, a two-tailed t-test yielded p-values of 0.001 for Sunset Blvd and 0.008 for Topanga Canyon Blvd, with the volumes increasing at Topanga Canyon Blvd and decreasing at Sunset Blvd from the TOD-230 analysis period to the TOD-Optimized period. This suggests that if the observed queue lengths at Topanga Canyon Blvd are longer in the morning under TOD-Optimized operation relative to TOD-230 operation, this may be explained to an extent by an increase in underlying demand on that street that occurred during the TOD-Optimized implementation. Similarly, shorter queue lengths during the AM Peak at Sunset Blvd during TOD-Optimized operation may reflect—to an extent—a reduction in volumes on that approach.

Finally, we compare ATCS volumes to the volumes in the last week of March 2013, which captures Spring Break at a major school near the middle of the study corridor. In all cases, the differences in volumes between the ATCS and Spring Break periods were significant at a 10% level, and for all major cross-streets along the study corridor, the differences were significant at a 5% level as well. This supports our decision to exclude this week from our ATCS analysis period, as it indicates that traffic patterns were significantly different (based on the averages shown in Table 3.1) on PCH that week.

Daily Demand Profiles on PCH Corridor

Figures 3.4 and 3.5 show plots of the average volumes on the PCH mainline for each 15-minute interval throughout the day, for each analysis period. In all cases, the overall shapes of the demand curves match up well: the peaks seem to occur at approximately the same time, rise to the same demand level, and persist for the same duration. Furthermore, the paces by which the demands increase before the peaks and decrease afterward are also comparable across all analysis categories. Therefore, the plots of the daily flow profiles do not suggest that any noticeable changes in traffic patterns occurred in the time between any of the four analysis periods.

No statistical tests are performed in the daily traffic demand profiles to evaluate whether the curves diverge by a statistically significant amount in any particular 15-minute window, because the different control strategies used during the TOD-230, TOD-Optimized, and ATCS analysis periods preclude any such comparisons. Specifically, the changes in traffic control between these different analysis categories may affect how soon vehicles are able to enter the PCH study corridor if the queue extends far upstream of the approach being analyzed, since the volumes are measured at inductive loops near the intersections themselves. Thus, slightly different demand profiles may be measured by the detectors as a result of the changes in traffic control even if the underlying demands have not changed, and so the plots in Figures 3.4 and 3.5) can be used for qualitative assessment of the demand profiles but not direct statistical testing within individual 15-minute bins.

PCH Directional Volume Comparison

Figure 3.6 shows the volumes per travel direction on PCH throughout the day, measured at Sunset Blvd. This intersection was selected based on its central location on the corridor and the fact that it is generally the primary bottleneck for both directions during the peak periods (in addition to Topanga Canyon Blvd in the southbound direction and Chautauqua Blvd in the northbound direction). The volume data is taken from the days of ATCS operation: all of the shaded Tuesdays, Wednesdays, and Thursdays of Figure 3.1 through May 14 (the last day ATCS was in operation) were included, except the Spring Break week of March 25-29. In the Figure, the lighter shaded areas represent standard deviations away from the mean, shown as a thick dark line.

Figure 3.6 confirms that the traffic demands on the corridor are higher flows in the peak directions during the AM and PM Peak periods. We also confirm that the morning peak is largely contained within the 6:30–9:30 AM period, while the evening peak typically falls between 2:30 and 7:30 PM. This Figure also justifies our decision to give priority to the northbound direction during the PM Peak and the southbound direction during the AM Peak when developing our signal timing offsets, as the peak directions during both times of the day have significantly more volume than the off-peak directions.

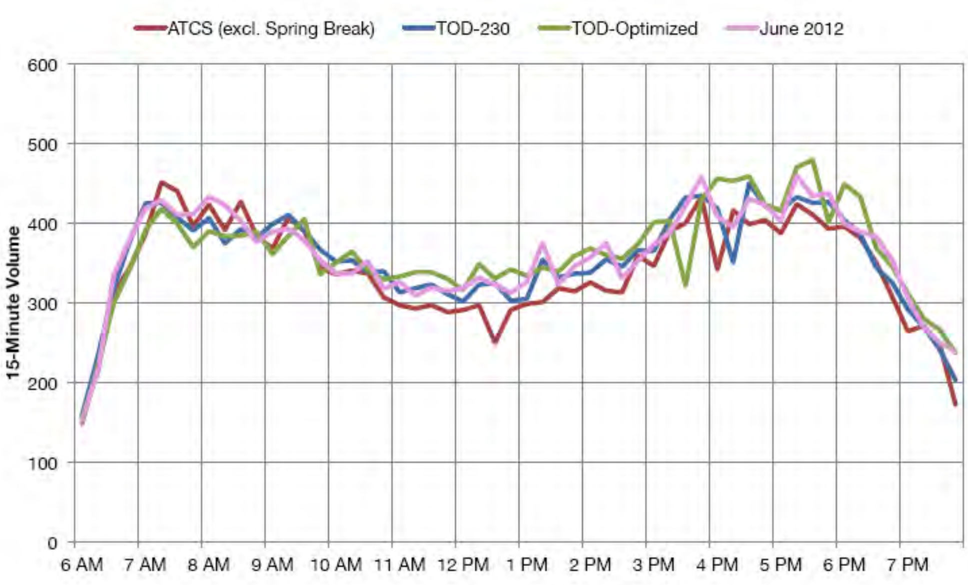


Figure 3.4 Daily Flow Profile -- SB PCH at Topanga Canyon Blvd

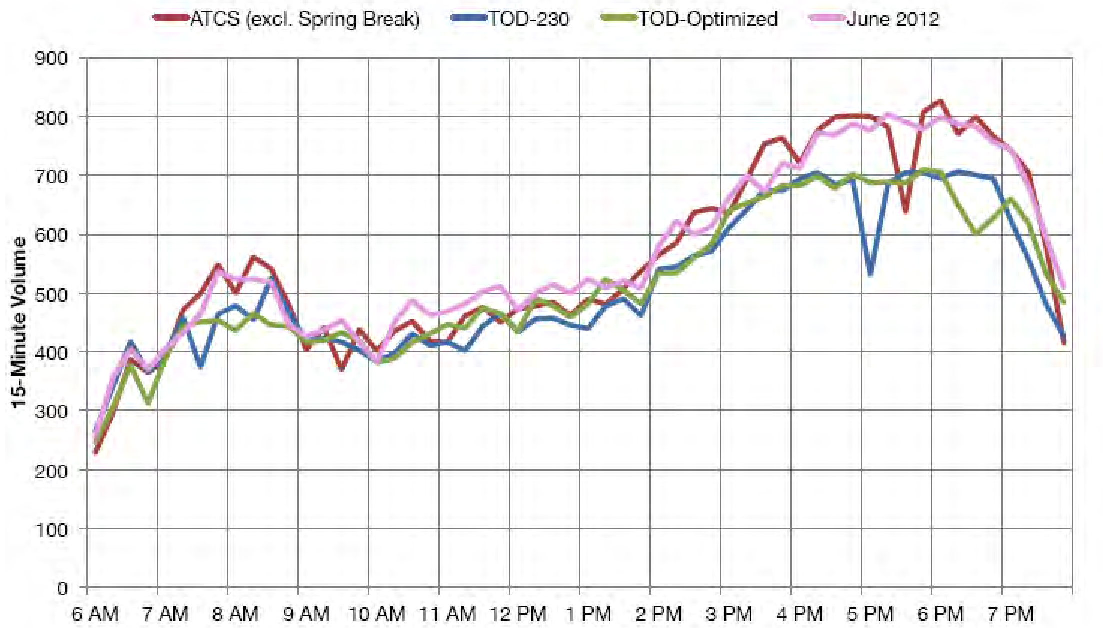


Figure 3.5 Daily Flow Profile – NB PCH at Bay Club Dr

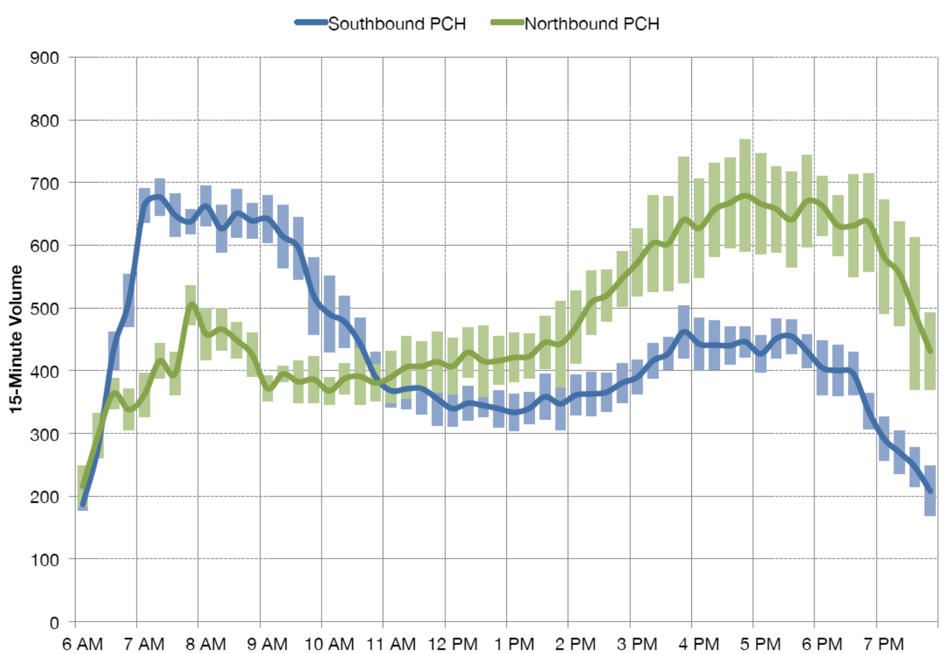


Figure 3.6 Directional Volumes on PCH at Sunset Blvd

3.2 Signal Timing Data

Signal timing data (splits and offsets) were provided by Caltrans for each intersection, disaggregated by signal cycle throughout the day. Figure 3.7 shows the dates of available data between March and May 2013. The data are provided in the form of Real-Time Split Monitor Report files (text files), which can be imported and processed within Microsoft Excel using a spreadsheet designed specifically to analyze and summarize these files.

As with the detector data, missing days of data shown in Figure 3.7 are generally indicative of data that Caltrans did not have available, often as a result of the reports not being manually archived within one week of their production (after which the system overwrites the data). The gap on March 21 is due to the landslide that blocked PCH that day, and the data between March 25 and 29 was excluded due to that week's traffic pattern irregularities caused by Spring Break.

As the Split Monitor reports are generated by ATCS, no data is available after May 14 due to the switch from ATCS to TOD timing plan operation. During the TOD-230 and TOD-Optimized implementations, ATCS continued to function in the background, but the timing parameters it produced were not communicated to the signals in the field. Thus, the Split Monitor reports do exist for the TOD-230 and TOD-Optimized analysis periods, but these reports reflect ATCS splits that were not being implemented in the field, making them unusable.

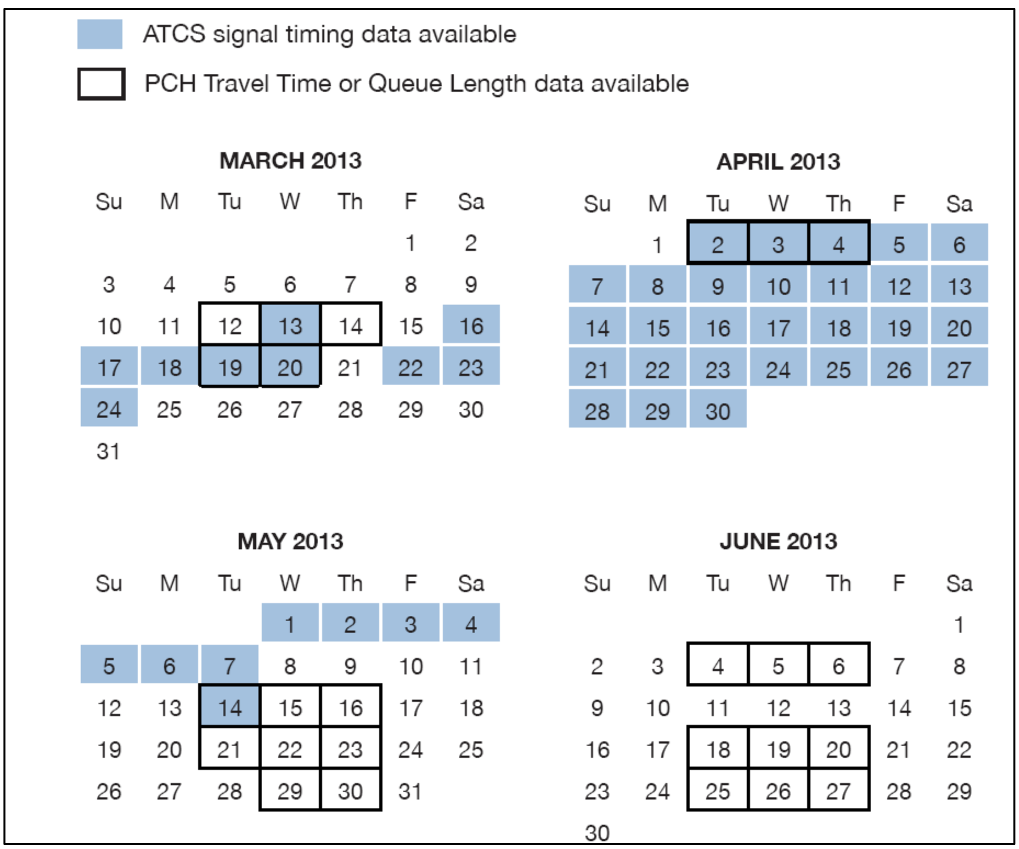


Figure 3.7 Dates of Available Signal Timing Data from ATCS System

The ATCS system updates cycle lengths, splits, and offsets at each intersection once per cycle based on prevailing traffic conditions. Changes to each parameter are incremental and are based on detector data from each intersection, although the system has the ability to vary the size of these increments up to a certain threshold depending on how quickly traffic conditions are changing. Splits are based on traffic volumes and occupancies measured by loop detectors on each approach, offsets are based on minimizing the number of stops for the approach with the highest flows (with special priority given to coordinated directions), and a system-wide cycle length is based on the minimum time needed to keep all signals in a particular section operating below saturation. In the case of PCH, the cycle times were constrained to 240 seconds during the AM and PM Peak throughout the field testing of the ATCS system (March through June 2013), leaving only the splits and offsets available for adjustment at these times.

Figure 3.8 is a sample plot of the median cycle time and phase durations under ATCS operation throughout the day, for the Chautauqua Blvd intersection. Light bands around the medians represent interquartile ranges.

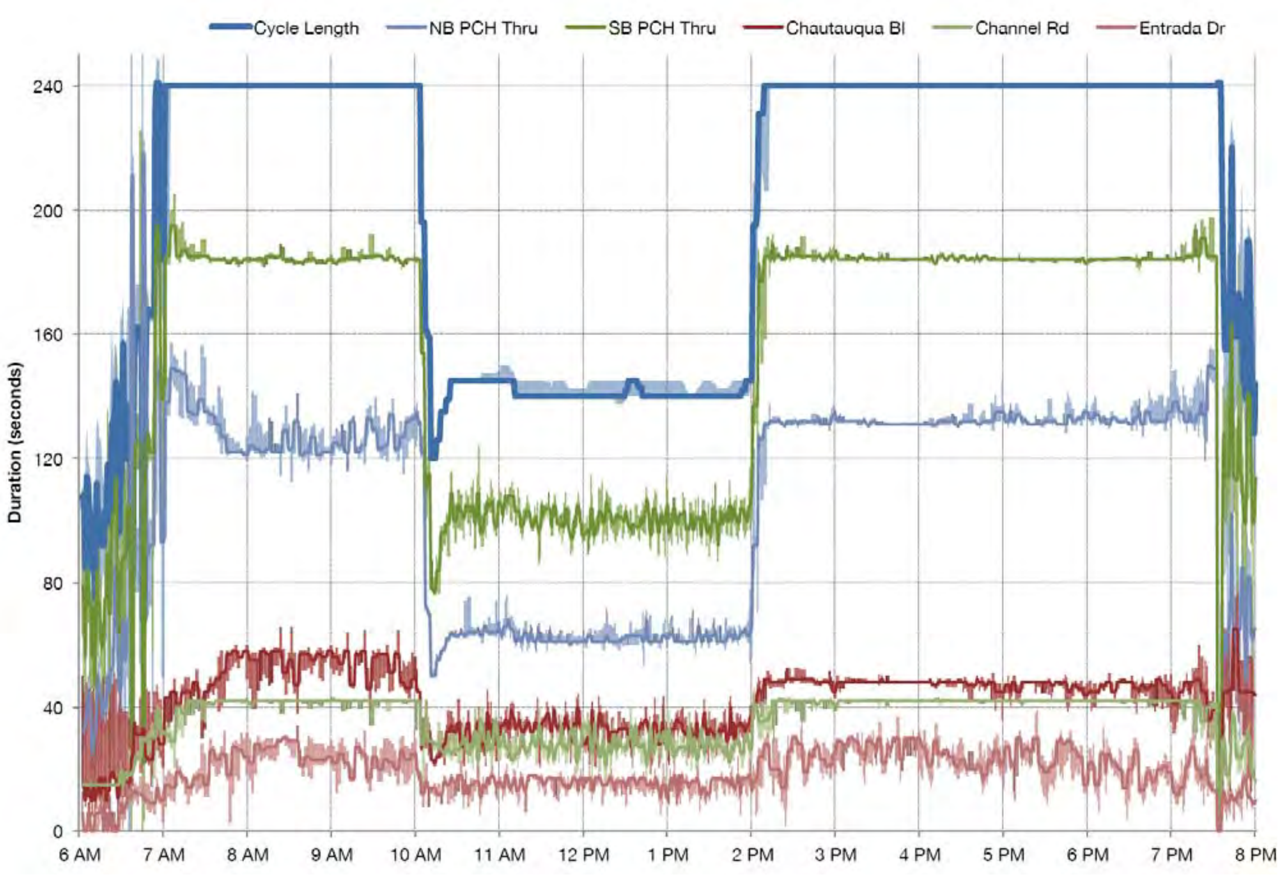


Figure 3.8 Median Cycle Time and Splits at Chautauqua Blvd under ATCS Control

3.3 Travel Times

Travel time data on the PCH corridor were collected in June 2012 using probe vehicles and in March through June 2013 using Bluetooth detectors.

3.3.1 Probe Vehicles

The Bluetooth detectors were not available to us in June 2012 when our initial round of ATCS data collection was carried out. Therefore, we relied on GPS-equipped probe vehicles to obtain vehicle trajectories along the corridor throughout the day. Although this process was much more labor-intensive than the Bluetooth detector approach, as it required a driver for each equipped vehicle the entire time, the result was a richer data set that provided detailed travel time information for each link of PCH.

The trajectories were obtained using two vehicles driven on a loop between Big Rock Drive and Ocean Avenue. Data collection occurred between 6:30–11:00 am for the AM Peak, and between 2:15 –8:00 pm for the PM Peak. The two vehicle operators drove with the overall flow of traffic during each travel time run, and contacted each other by phone approximately once an hour to ensure that they were maintaining proper temporal spacing between their runs. The operators were also responsible for noting any traffic irregularities observed during their travel time runs (e.g., lane closures, traffic collisions), so that the affected trajectories could be removed from the data set prior to analysis.

An iPhone and the iOS app Runmeter ($5) were used in each vehicle to collect the trajectory data automatically, such that the only user interaction required was resetting the app at the beginning of each new travel time run. This also afforded greater flexibility in obtaining vehicles for the travel time runs, as any standard vehicle with a 12-V auxiliary power outlet (i.e., cigarette lighter socket, to provide power to the iPhone throughout the day) could be used. Using the iPhone's GPS signal, the app recorded the vehicle's position approximately once every 10 seconds while in motion, and at lower frequencies when stationary. The results could then be exported as a text file and imported into Microsoft Excel for processing and analysis.

Figures 3.9 and 3.10 show sample individual vehicle trajectories recorded for each direction, date, and time of day. Intersection locations are superimposed on the charts to assist with interpretation of the trajectories and the observed stops (i.e., horizontal portions of each trajectory). These figures provide insight into the more detailed analyses that probe vehicle data make possible, as we can extract travel times, delays, and numbers of stops on each link from the raw data.

A new trajectory was recorded in each direction roughly once every 20 minutes, with lower frequencies during periods of heavy congestion. Over the course of three mornings and afternoons, a total of 195 trajectories were recorded. The endpoints for the north and south directions were chosen to coincide with the locations used to define the corridor during the Bluetooth data collection: the northbound corridor started at McClure Tunnel and ended at the "Signal Ahead" flashing beacon near Big Rock Drive, whereas the southbound corridor started at the "Signal Ahead" flashing beacon and ended at Haul Road.

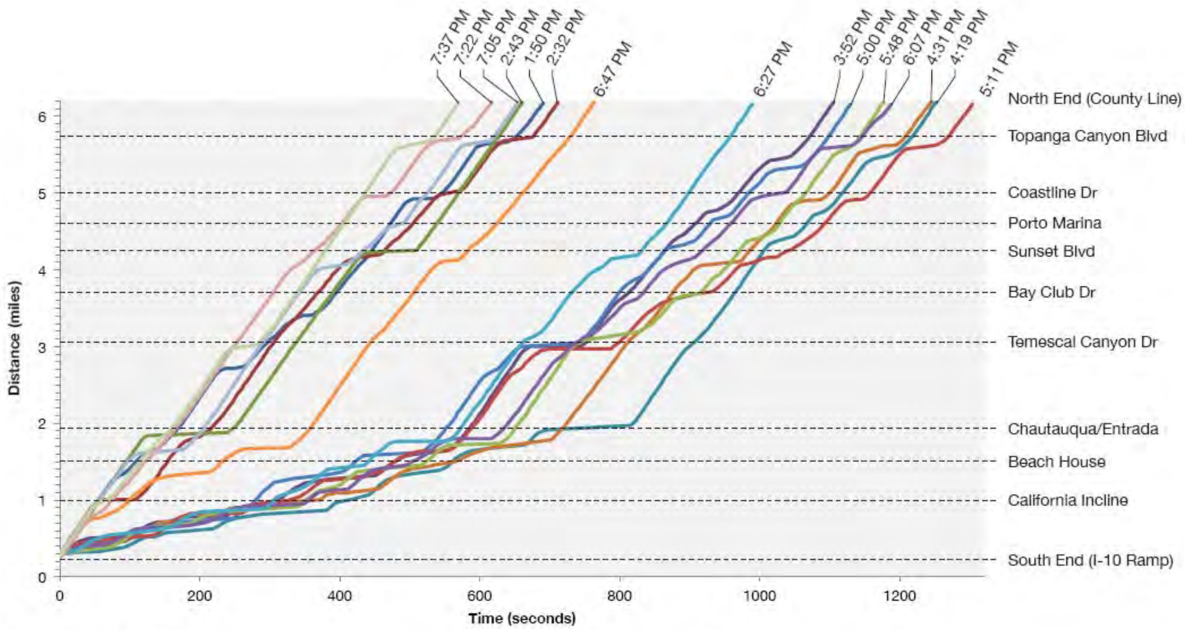


Figure 3.9 Probe Vehicle Trajectories: NB PCH June 7, 2012 PM Peak

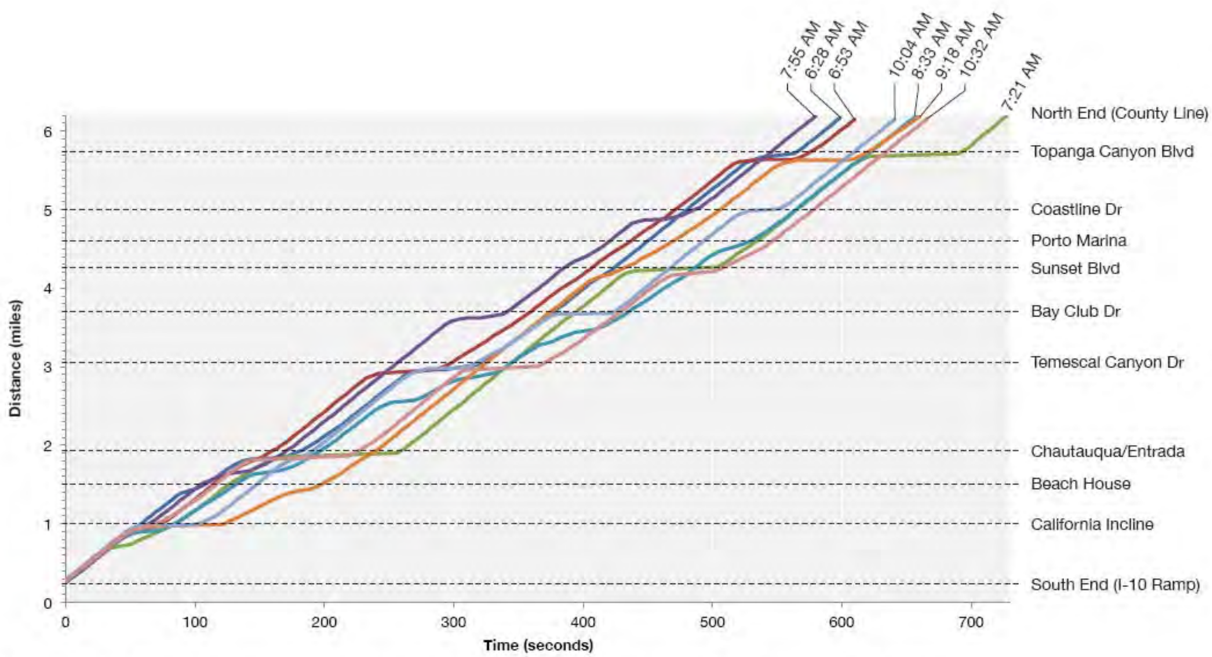


Figure 3.10 Probe Vehicle Trajectories: NB PCH June 12, 2012 AM Peak

3.3.2 Bluetooth Data

Five Bluetooth detectors were deployed along the Pacific Coast Highway project corridor on Tuesday, March 12, 2013, as indicated in Figure 3.11. The Bluetooth units anonymously collect the unique identifiers of discoverable Bluetooth devices within range (about 100 feet) every five seconds, which are then used to estimate travel times from one unit to another on the corridor at all times of the day. The distances between adjacent detector locations are:

- McClure Tunnel to Haul Road: 2.4 miles
- Haul Road to Shore Drive: 0.8 miles
- Shore Drive to Lifeguard Station: 1.8 miles
- Lifeguard Station to Big Rock Drive "Signal Ahead" warning: 2.1 miles

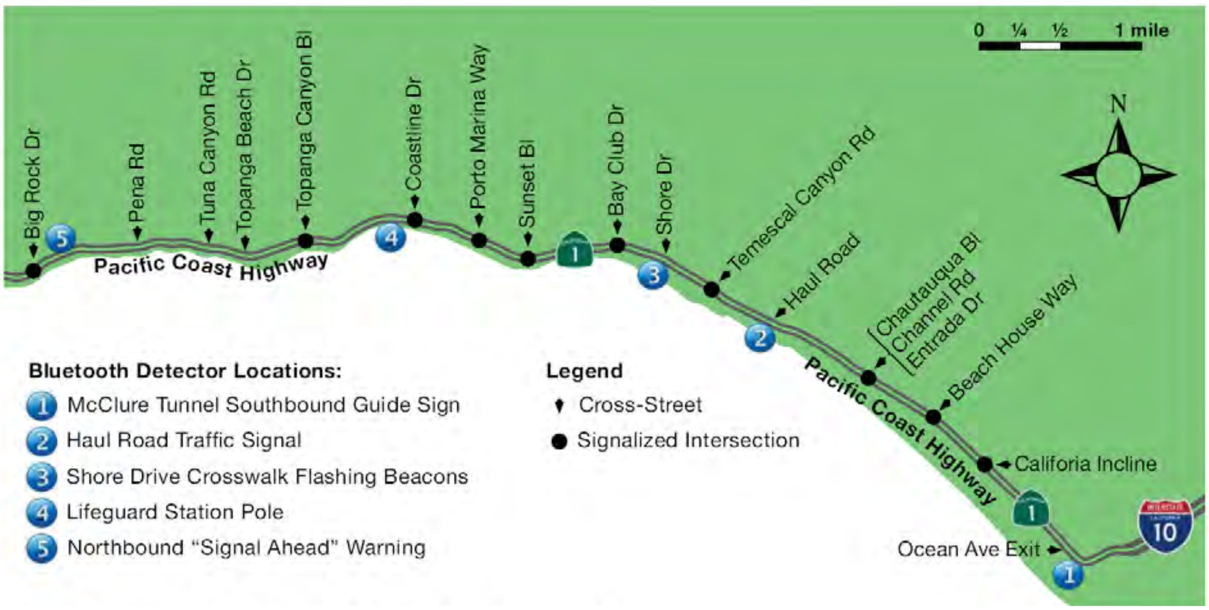


Figure 3.11 Bluetooth Detector Locations on PCH

Figure 3.12 shows the specific days of travel time data used to obtain the travel time profiles for each of the three control strategies (i.e., ATCS, TOD-230, and TOD-Optimized). Only data on typical weekdays (Tuesdays, Wednesdays, and Thursdays) were used. Additionally, the following exclusions were made:

- Morning data for March 12 was unavailable, as the Bluetooth detectors were not deployed and activated until late morning on this date.
- All data for March 21 was excluded, as a landslide blocked part of PCH that day.
- Data for April 9 through May 9 is not available because the Bluetooth units lost battery power during this time.
- Data for May 15 and 16 is limited to the midday period only, as the TOD-230 timing parameters for the AM and PM Peak periods were still being adjusted on these days in response to observed traffic behavior on PCH.
- Data for May 21 were available after 10:30 AM, as the TOD-230 timing plan for the AM Peak was still being adjusted at this time, and because long queues at Topanga Canyon Blvd prompted us to run the AM Peak plan until 10:30 to allow all lingering peak period congestion to clear.

- Data for May 28 was excluded, as the Memorial Day holiday on May 27 could have affected traffic patterns on this Tuesday.
- Data for June 11-13 is not available because the Bluetooth units lost battery power during this time.
- Data for June 18 is limited to the midday period only, as the TOD-Optimized timing parameters for the AM and PM Peak periods were still being adjusted on these days in response to observed traffic behavior on PCH.

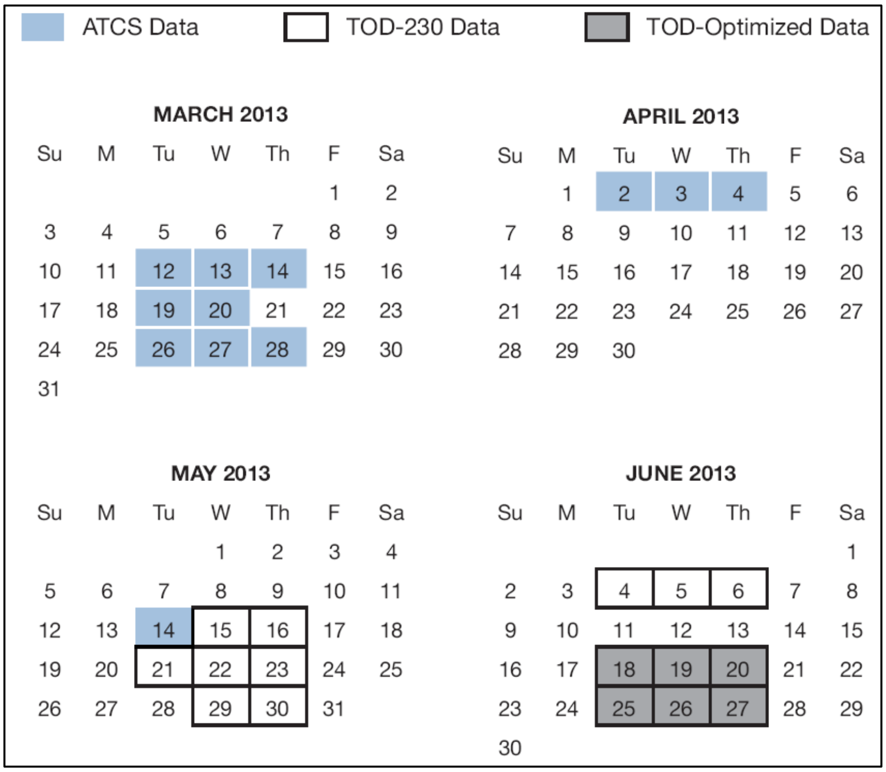


Figure 3.12 Dates of Available Bluetooth Travel Time Data Collection per Control Strategy

Detection Rates for Each Bluetooth Unit

Table 3.2 shows the number of unique Bluetooth devices detected each day for a typical week of data collection on the PCH corridor (March 14–20, 2013). The data for each detector represents vehicles crossing the detection zone in either direction. The numbers shown in this Table (for the number of *unique* Bluetooth devices detected) is different from the *total* number of detections in that a device in a vehicle may be detected several times as it passes the detector, although this only represents one unique device detection.

A threshold gap time of 240 seconds was used to distinguish between a single vehicle being detected multiple times as it passed through a detection zone, and multiple trips by the same vehicle. If the time between successive detections of the same Bluetooth ID exceeded this duration, the two detections were considered separate trips.

The drops in detection rate at Big Rock Drive on March 15 and at Shore Drive on March 19 are due to the failures of those units to upload their data successfully at the end of those days. Although rare, such events are not indicative of problems with the unit or its data, and are generally a result of poor network reception at the time the upload was scheduled to occur. Throughout the entire study period, these were the only two dates where upload failures occurred, apart from those instances where data were not uploaded because of power loss for the Bluetooth detector (i.e., depletion of the internal battery).

Table 3.2 Detection Rates at Each Bluetooth Location

	Thursday 3/14	Friday 3/15	Saturday 3/16	Sunday 3/17	Monday 3/18	Tuesday 3/19	Wednesday 3/20
McClure Tunnel	6830	6862	6675	6735	6377	6641	6608
Haul Road	6710	6849	6163	4545	6355	6680	6693
Shore Drive Crosswalk	6376	6072	5347	4113	5559	53	5826
Lifeguard Station	5427	5375	4699	3468	4812	5034	5134
Big Rock Dr	4310	52	3870	3025	3772	3920	4013

Successful Matches between Bluetooth Detectors

Table 3.3 shows the number of trips by travel direction identified between each pair of Bluetooth detectors on the corridor in a typical week (between March 14 and March 20). For a successful trip identification to occur, the same Bluetooth ID must have been detected at each of the two Bluetooth units within 60 minutes of each other. If the elapsed time between detections of a Bluetooth device at the initial and ending Bluetooth units exceeded this duration, the two detections were not considered part of a single trip (and were not used for subsequent travel time estimation). The table includes all successful matches based on a threshold of 60 minutes, regardless of whether they were considered outliers or not.

Table 3.3 Matched Bluetooth IDs between Unit Pairs—Period 3-14 through 3/20

		Ending BluFax Unit				
		McClure Tunnel	Haul Road	Shore Drive Crosswalk	Lifeguard Station	Big Rock Drive
Initial BluFax Unit	McClure Tunnel	—	13,218	9,363	8,335	5,811
	Haul Road	16,054	—	14,815	13,106	8,657
	Shore Drive Crosswalk	11,304	14,620	—	11,783	7,534
	Lifeguard Station	10,249	13,125	11,839	—	10,261
	Big Rock Dr	6,715	8,451	7,346	10,095	—

Bluetooth Penetration Rates

Figure 3.13 shows the penetration rate of discoverable Bluetooth devices in the traffic stream, assuming each unique Bluetooth ID detected belongs to a different vehicle on PCH (as opposed to one vehicle with multiple Bluetooth devices, or a Bluetooth device carried by a pedestrian). This penetration rate is calculated by taking the number of unique Bluetooth IDs detected and dividing it by the traffic volumes at McClure Tunnel. The average percentage of vehicles that had discoverable Bluetooth devices and were successfully detected at McClure Tunnel was 9.4% over the entire time period.

Figure 3.13 also shows the rate of successful Bluetooth ID matches for traffic heading in the northbound direction at McClure Tunnel. This successful match rate is calculated by taking the number of Bluetooth ID matches obtained between McClure Tunnel and Haul Road (in that order), and dividing it by the traffic volumes at McClure Tunnel in the northbound direction. The average percentage of vehicles that had discoverable Bluetooth devices and were successfully matched between McClure Tunnel and Haul Road on the northbound side was 5.3% over the entire time period plotted in Figure 3.13. In other words, approximately one out of every 20 vehicles on the northbound side of PCH was used to generate a travel time estimate between McClure Tunnel and Haul Road on average.

Sample Segment Travel Times

Figure 3.14 shows travel time data throughout a typical day (Thursday, March 14), based on individual Bluetooth IDs matched between adjacent detectors on the corridor. The plots shown in Figure 3.14 are for the northbound direction of the two southern-most segments: McClure Tunnel to Haul Road, and Haul Road to Shore Drive.

Five-minute bins are used for the average calculations in Figure 3.14 and symbols/colors are used to differentiate outliers from other data points. To identify travel time outliers, such as drivers who made stops at businesses between the two Bluetooth detectors, each data point is compared to its 30 adjacent travel times. From these 30 neighboring points, an average travel time and an Inter-Quartile Range (IQR) is calculated. If the subject point is more than four IQRs higher than the mean travel time for its 30 nearest neighbors, it is considered an outlier. The threshold time used for vehicle matching was 60 minutes (see the description of Table 3.3 for an explanation of this parameter).

PCH Corridor Travel Times

Figures 3.15 and 3.16 show the mean travel time, mean speed, and sample size (number of matched Bluetooth IDs between the BluFax units at the north and south ends of the corridor) for five-minute bins throughout the day in each direction on a typical day (Thursday, March 14, 2013). Data shown in the figures is for only those vehicles that were matched between the detectors at McClure Tunnel and Big Rock Drive (i.e., the travel times are not simply estimated based on the sum of travel times between adjacent detectors on the corridor). As before, the threshold time used for vehicle matching was 60 minutes. Outliers were excluded from the plots below and from mean travel time calculations.

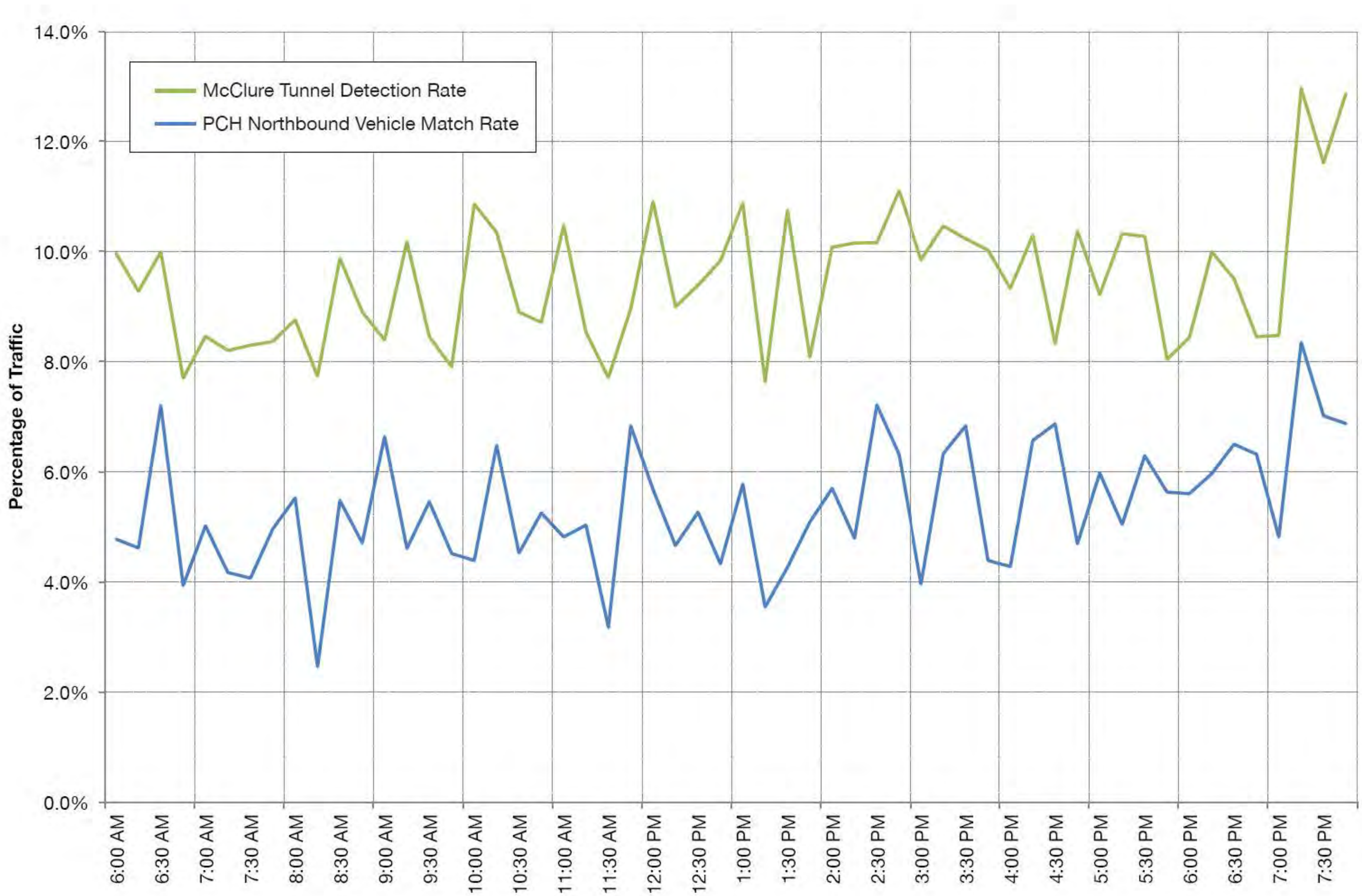


Figure 3.13 Detection Rates, Traffic Volumes, And Estimated Penetration Rate For Discoverable Bluetooth Devices In Traffic Passing The Mcclure Tunnel Blufax Unit, In 15-Minute Bins For The Week Of March 12 To March 19, 2013

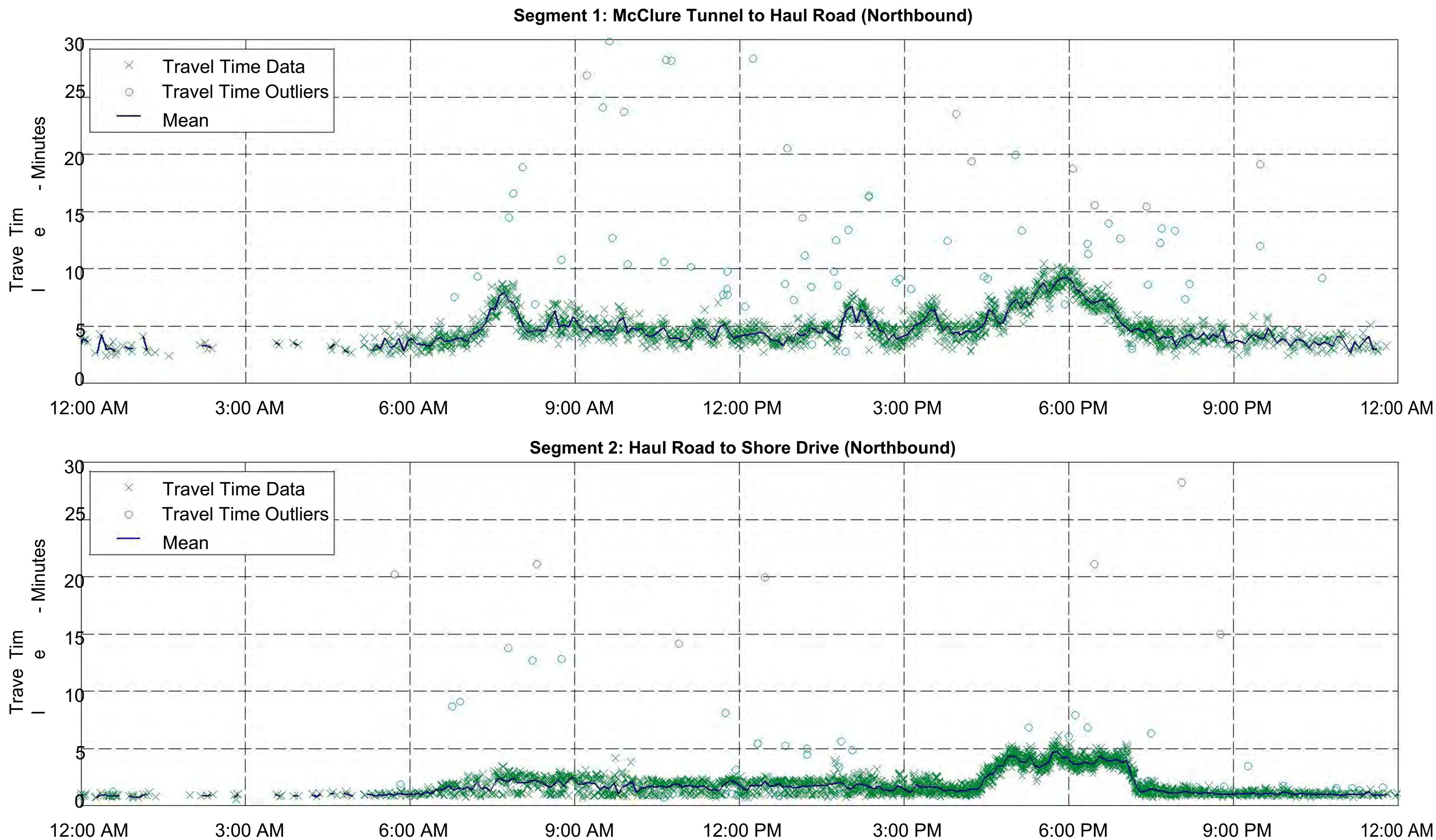


Figure 3.14 Travel Times NB PCH by Segment --Thursday, March 14, 2013

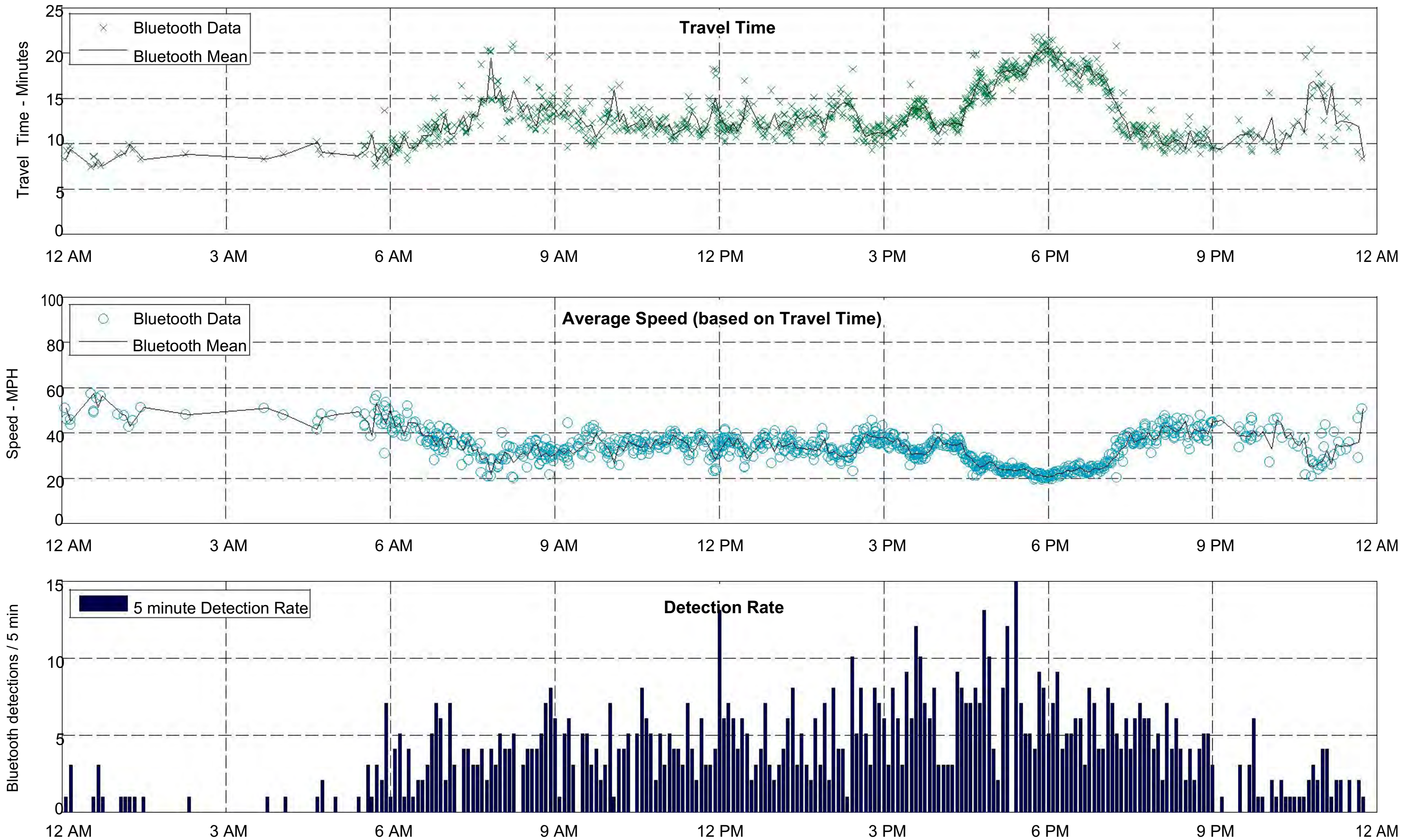


Figure 3.15 NB PCH Corridor-wide Bluetooth Data --Thursday, March 14, 2013

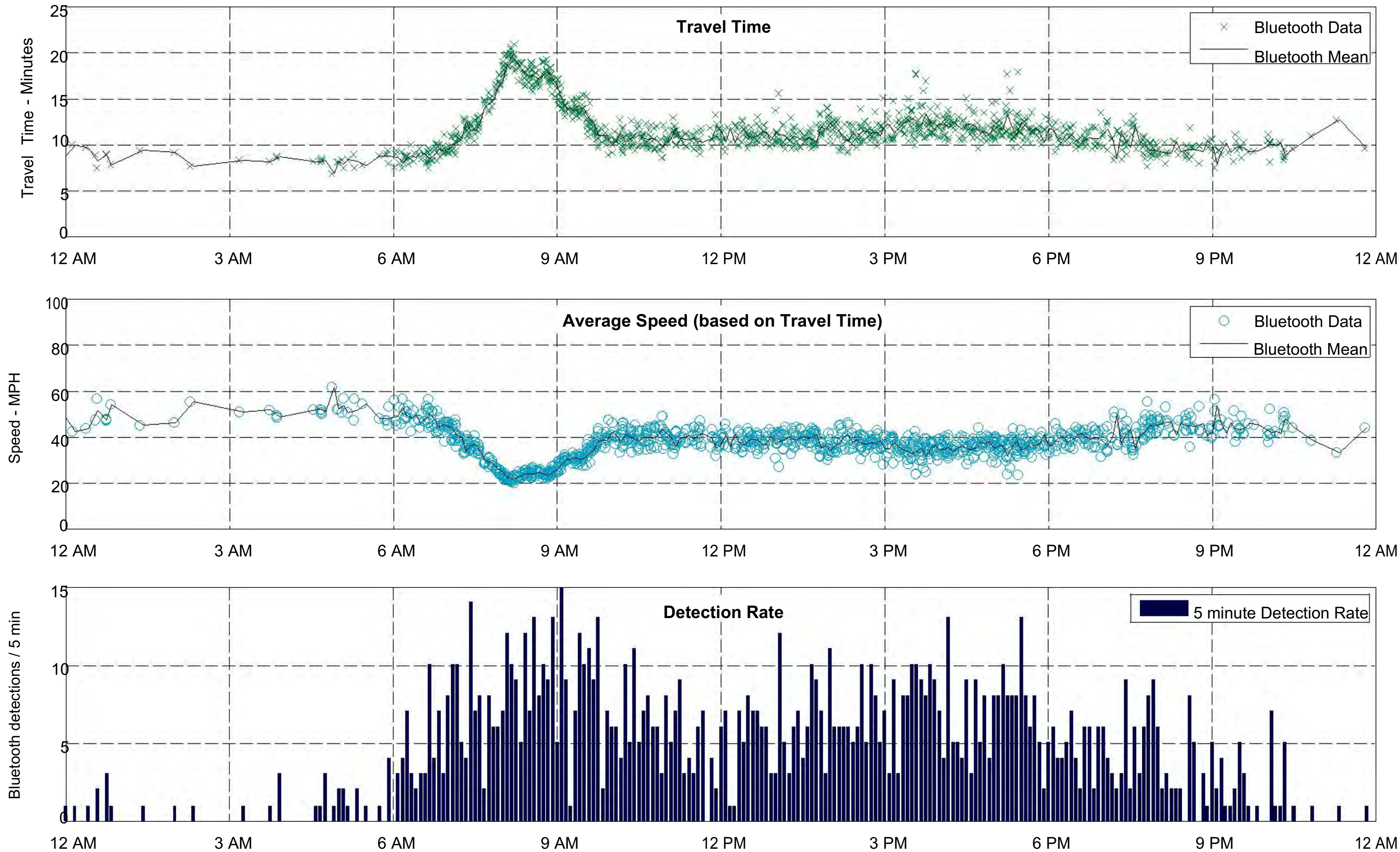


Figure 3.16 SB PCH Corridor-wide Bluetooth Data--Thursday, March 14, 2013

CHAPTER 4

DEVELOPMENT OF FIXED-TIME TIME-OF-DAY PLANS

This Chapter describes the development and implementation of fixed-time time-of-day (TOD) timing plans for the PCH corridor as an alternative control strategy to the adaptive traffic control system (ATCS) currently in place. These TOD plans may be used as replacements for ATCS if needed, and analogous sets of plans developed for other corridors according to the same process may be used similarly.

4.1 Development of Timing Plans with SYNCHRO

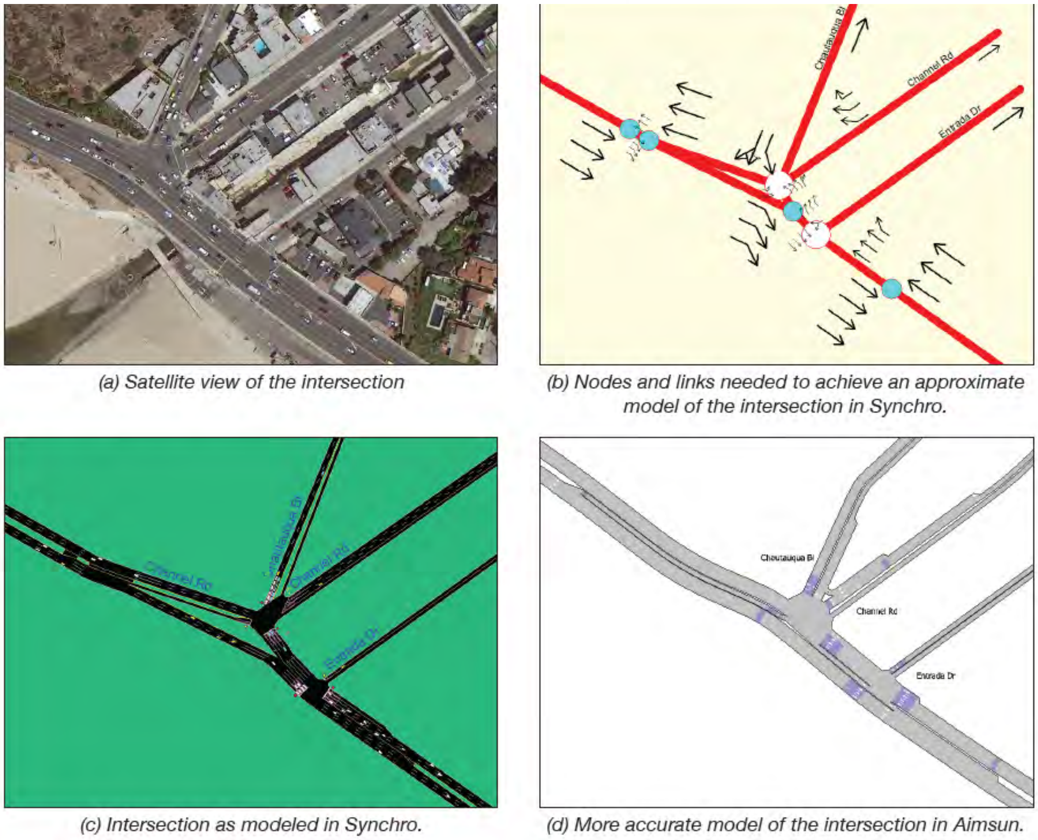
4.1.1 Creating the SYNCHRO model

As the SYNCHRO model will be the foundation of the TOD timing plans, it must be built as accurately as possible with respect to roadway geometry, signal timing parameters, and volume data. In particular, the following values had to be defined for our model before we could use it to generate our TOD timing plans:

- **Intersection/link geometrics:** link speed, distances between intersections, number of lanes on each link, lane widths, grade (by movement), turn bay lengths, lane configuration.
- **Saturation flows:** in place of the SYNCHRO defaults, we used 1,700 vphpl for left turns and 1,800 vphpl for through movements.
- **Signal phasing:** pedestrian Walk/Flashing-Don't-Walk durations, assignment of movements to phases, minimum splits, yellow time, all-red time, recall modes.
- **Signal settings:** In SYNCHRO, splits and offsets were defined based on the values currently being used in the field. To obtain these, we used the ATCS Real-Time Split Monitor Reports for typical weekdays (Tuesdays, Wednesdays, and Thursdays) and computed the average splits and offsets for each intersection across all available days of data. It was not critical that these splits and offsets in SYNCHRO be highly accurate or representative of the current signal operation in the field, since the optimizations will largely replace these values with new ones. Rather, these inputs are only necessary as starting points for the SYNCHRO optimizations.
- **Traffic volumes:** hourly volumes were input into SYNCHRO using the UTDF importer, which allowed us to import several days of data and automatically calculate average peak-hour volumes and peak-hour-factor values for each intersection. A separate copy of the model was saved for each hour of volume data imported, covering all hours between 6 am and midnight.

  To automatically convert the detector volumes into the UTDF format for SYNCHRO, we used a specially constructed Excel spreadsheet that automatically retrieved detector data from the ATCS Detector History reports and prepared it as a UTDF file, while making some assumptions about detector volumes as needed.

SYNCHRO is a signal timing optimization tool designed for typical intersection configurations. Therefore it is difficult to accurately recreate complex or unusual intersection geometry in the model (see Figure 4.1a). In these situations, we used novel link and node arrangements to get the SYNCHRO model to represent the real-world conditions as accurately as possible (see Figures 4.1b and 4.1c), with the understanding that the AIMSUN microsimulation model would provide a better representation of how traffic might be expected to behave in response to the SYNCHRO timing plans (see Figure 4.1d).



(a) Satellite view of the intersection

(b) Nodes and links needed to achieve an approximate model of the intersection in Synchro.

(c) Intersection as modeled in Synchro.

(d) More accurate model of the intersection in Aimsun.

Figure 4.1 Modeling the Intersection of Chautauqua Blvd, Channel Rd, and Entrada Dr.

4.1.2 Signal Timing Optimization

We developed optimal timing plans for each hour of the day. From these hourly optimized timing plans and the hourly volume data, we searched for appropriate break points for our TOD periods and used these to produce optimized timing plans per time period (AM peak, midday and PM peak). The development of the hourly optimized timing plans consisted of the following steps:

1. **For each intersection, optimize the cycle lengths and the intersection splits:** For the complex intersection at Chautauqua, which had to be modeled as two separate but linked intersections in SYNCHRO, the cycle lengths and splits had to be optimized at each intersection twice in an alternating order (e.g., cycle lengths and splits at one intersection, then at the other, then at the original intersection a second time, and finally at the other intersection a second time).
2. **Optimize the network-wide cycle length:** We imposed an upper limit of 240 seconds, though this was a non-binding constraint on the optimization. We used increments of 10 seconds, allowed half-cycle lengths, enabled the “extensive” offset optimization method, and did not allow uncoordinated intersections. All intersections were placed in the same “zone” in the model, so that there would be one common cycle length throughout the entire corridor.
3. **Optimize the network offsets:** We kept the phase sequence fixed (by disabling the lead/lag optimization option in SYNCHRO). Also, because we already optimized the splits for each intersection, we opted to “Use Existing Splits” in the options screen.

By default, SYNCHRO uses a “performance index” as its objective function for signal timing optimizations. Although this can be manually overridden in the software in favor of total delay or another custom performance metric, we kept the default objective function in place.

Figure 4.2 shows the optimized cycle lengths for the corridor and splits for major cross-streets by hour. The splits are measured as fractions of their respective cycles instead of absolute durations, so that the trends observed in the splits for each intersection throughout the day reflect the changes in demand, without the influence of changes in cycle length.

The demand by hour for each cross-street is shown in Figure 4.3. The Figure includes only the critical movement (e.g., the left turns for the Sunset Blvd approach, not the right turns or through traffic destined for the Gladstone parking lot), as this was the determining factor in the split duration for the approach. The northbound and southbound PCH volumes are from the entrances to the corridor at the south and north ends, so that we exclude the influence of any large turning traffic volumes from the cross-streets. The hourly volumes are averaged across all Tuesdays, Wednesdays, and Thursdays of data available.

The determination of the TOD break points was based on identification of major changes occurring in traffic demand, cycle length, or cross-street splits. We also chose our boundaries conservatively for the rush periods, so that the peaks from one day to another could be expected to fall within the “AM Peak” and “PM Peak” TOD periods, even as the peaks shift earlier or later from day to day. The first break point at 10 am was chosen in response to the large drop in volume at Topanga Canyon Blvd that occurred at that time, bringing the corridor out of its morning rush period and ushering in the lower volumes of the midday. The second break point at 2 pm was selected in response to the large jump in cycle times and volumes observed at 3 pm, and the desire to provide a one-hour buffer to accommodate any days that the PM rush period begins any earlier than its typical 3 pm time. This type of buffer was not as crucial at the ends of the AM and PM periods, as volumes declined gradually in both instances and did not show any large jumps at the trailing ends of the periods (such that the effects of peak shifting would not be as severe in these cases). The last break point was set before the 8-9 pm hour due to the continuing trend of declining volumes seen in the 7-8 pm hour and the noticeable drop in cycle length from its 4-6 pm value of 180 seconds to 130 seconds after 8 pm—a decrease of 28%.

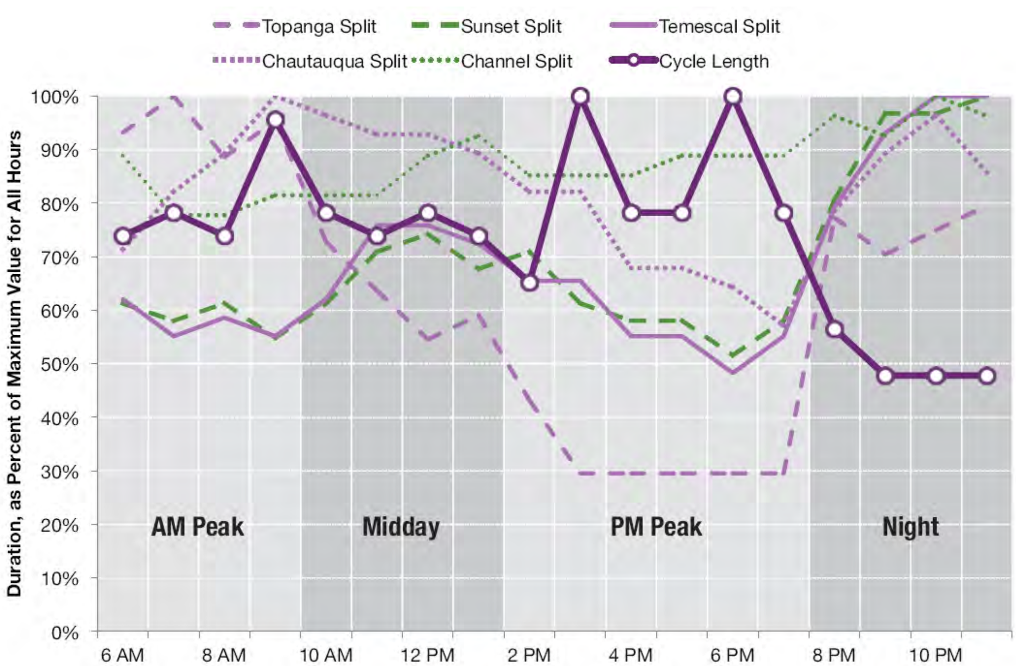


Figure 4.2 TOD Optimized System Cycle Lengths and Splits for Major Cross-Streets

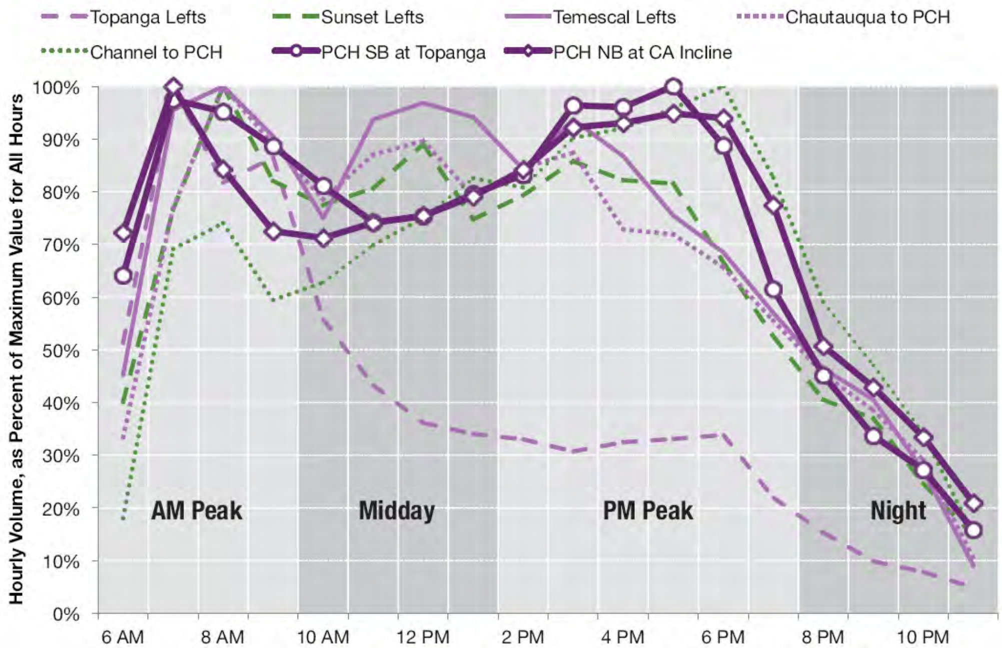


Figure 4.3 Hourly Entrance Volumes for PCH and Major Cross-Streets

With our TOD periods now established as 6–10 am, 10 am – 2 pm, 2–8 pm, and 8 pm – 6 am, we used SYNCHRO to produce a set of optimized timing plans for each of these periods following the same general procedure used for the hourly optimizations earlier. However, when importing the volume data from the UTDF file for each multi-hour period, we used the “Average Daily Peak Hours” option instead.

Two sets of timing plans were produced: one (referred as “TOD-230” plan) used a fixed cycle length of 230 seconds during the peak periods, and another (referred as “TOD-Optimized” plan) used a fully optimized and unconstrained cycle length—subject to an upper limit of 240 seconds—at all times of the day. The TOD-230 plans were developed to facilitate comparisons between ATCS and the TOD operation approach, as the signal timing data indicated that a cycle length constraint of 230 seconds was enforced on ATCS during the AM and PM peak periods. Because the midday period was not subject to a cycle length constraint of 230 seconds under ATCS, no midday TOD-230 timing plan was generated. Rather, only one midday plan was produced with an optimized cycle length determined by SYNCHRO.

The resultant splits for each of the TOD plans generated in SYNCHRO are shown in Table 4.1, with the cycle lengths given in Table 4.2 (recall that half cycles were allowed at minor cross-streets). The phase numbers used for each intersection in Table 4.1 correspond to typical NEMA/170 controller phase numbering.

Generally, a shorter cycle length results in shorter queues and lower delays for undersaturated conditions, whereas a longer cycle length results in fewer stops per vehicle. With respect to capacity, longer cycle lengths can help by reducing the fractional lost time per cycle and minimizing the number of pedestrian phases that need to be accommodated each hour, while shorter cycle lengths can help by increasing the benefit associated with auxiliary lanes at intersections and reducing the likelihood of a queue spillback from a lane drop downstream of the intersection.

On PCH, this translates into potentially favorable conditions for shorter cycle lengths at Sunset Blvd (where there is an auxiliary lane) and Temescal Canyon Rd (where there is a lane drop downstream of the intersection on the northbound side), and potentially favorable conditions for longer cycle lengths at Chautauqua Blvd (where there are oversaturated conditions and lost time makes up a larger portion of the cycle than at other intersections). Shorter cycle lengths may also benefit the Southbound PCH left turn movement at Chautauqua Blvd and Channel Rd, where the hard left turn to Chautauqua Blvd requires gaps. During oversaturated conditions when such gaps are rare, vehicles making this movement must wait until the yellow clearance interval to proceed, which occurs more frequently if the cycle length is shorter. While waiting for the clearance interval, these vehicles may obstruct the queue discharge of all other vehicles behind them in the turn pocket on southbound PCH.

Table 4.1 Original SYNCHRO Splits for all TOD Timing Plans

	CA Incline						Beach House Way				Chautauqua Bl				
Phase	1	2	4	5	6	8	1	2	6	8	2	4	5	6	7
TOD-Optimized AM Peak	6	122	25	47	80	29	6	140	151	8	112	29	39	68	29
Midday	6	52	6	27	34	10	7	54	66	8	109	32	39	65	32
TOD-Optimized PM Peak	16	127	20	51	91	24	6	60	71	8	118	33	36	77	33
TOD-230 AM Peak	6	173	34	66	112	38	13	191	209	10	163	38	54	104	38
TOD-230 PM Peak	11	176	26	70	116	30	22	174	201	18	157	44	46	106	44

	Temescal Canyon Rd						Bay Club		Sunset Bl					
Phase	1	2	3	4	5	6	2	4	1	2	3	4	5	6
TOD-Optimized AM Peak	8	96	10	34	28	76	135	24	10	101	10	28	20	91
Midday	9	95	10	34	25	79	135	24	11	99	10	29	20	90
TOD-Optimized PM Peak	8	110	10	30	13	105	145	24	10	111	10	28	12	109
TOD-230 AM Peak	8	147	8	45	38	117	196	23	11	143	10	45	32	122
TOD-230 PM Peak	10	147	8	43	26	131	196	23	14	143	12	40	24	133

	Porto Marina Way				Coastline Dr				Topanga Canyon Bl			
Phase	2	4	5	6	2	4	5	6	2	5	6	8
TOD-Optimized AM Peak	151	8	6	140	138	21	5	129	96	7	84	63
Midday	66	8	6	55	135	24	13	118	56	8	43	18
TOD-Optimized PM Peak	161	8	6	150	148	21	6	138	145	18	122	24
TOD-230 AM Peak	209	10	8	196	199	20	9	186	135	10	120	84
TOD-230 PM Peak	213	6	6	202	199	20	9	186	194	19	170	25

Table 4.2 Cycle lengths for all TOD Timing Plans

	TOD-Optimized AM Peak	Midday	TOD-Optimized PM Peak	TOD-230 AM Peak	TOD-230 PM Peak
California Incline	170	170	180	230	230
Beach House Way	170	85	90	230	230
Chautauqua Bl	170	170	180	230	230
Temescal Canyon Rd	170	170	180	230	230
Bay Club Dr	170	170	180	230	230
Sunset Bl	170	170	180	230	230
Porto Marina Way	170	85	180	230	230
Coastline Dr	170	170	180	230	230
Topanga Canyon Bl	170	85	180	230	230

4.1.3 Adjustment of Offsets for Peak Periods

When optimizing the offsets, SYNCHRO attempts to balance the progression in both directions according to the traffic demands of each direction. However, we manually calculated the offsets on PCH to give priority to traffic in the peak directions (i.e., northbound PCH in the PM Peak, and southbound PCH in the AM Peak).

The offsets are calculated based on the expected travel times between successive intersections. The travel times are calculated from the distances between each intersection and the free-flow speed (assumed 40-45 mph given the posted speed limit of 45 mph). We can then determine the appropriate offsets for each signal as follows:

1. Set the start of green for the through phase in the peak direction (i.e., the priority direction) at the farthest upstream intersection to 0.
2. At each successive downstream intersection, set the start of green for the through phase in the peak direction to the sum of the start of green for the intersection immediately upstream of it and the travel time between these two intersections.
3. At each intersection, compute the offsets (with respect to the start of yellow) by summing the duration of the through movement phase in the peak direction and the start of green established for that intersection in Step 2.

Figure 4.4 shows a time-space diagram for the new offsets for the AM peak in the southbound direction, with the green bandwidths drawn in blue. For comparison, a similar time-space diagram using SYNCHRO's original offsets is given in Figure 4.5. In these figures, both axes follow the same time scale, with distances between intersections converted into travel times on the vertical axis based on the corridor speed limit of 45 mph.

Green bandwidths per direction and time of day are shown in Table 4.3, along with the bandwidth efficiencies for both the originally SYNCHRO optimized offsets and the adjusted offsets.

Table 4.3 Green Wave Bandwidths and their Corresponding Efficiencies

	Cycle Length *(sec)*	Northbound Bandwidth *(sec)*	Southbound Bandwidth *(sec)*	Bandwidth Efficiency
AM Peak 170-cycle Offsets (sec)	170	0	98	29%
Midday 170-cycle Offsets (sec)	170	9	16	7%
PM Peak 180-cycle Offsets (sec)	180	71	11	23%
AM Peak 230-cycle Offsets (sec)	230	0	135	29%
PM Peak 230-cycle Offsets (sec)	230	109	0	24%
Original SYNCHRO TOD-Optimized Offsets, AM Peak	170	0	42	12%
Original SYNCHRO TOD-Optimized Offsets, PM Peak	180	17	10	8%

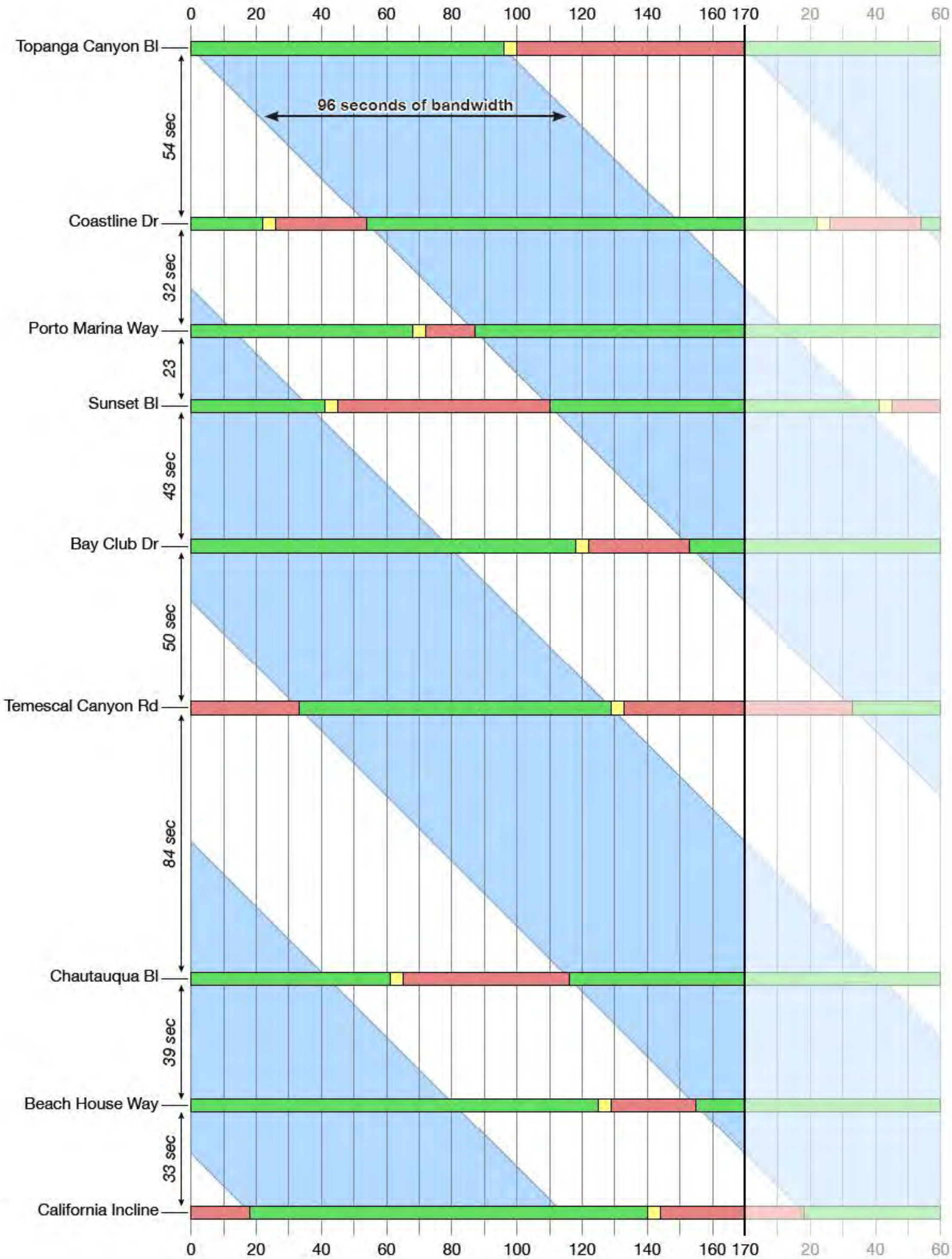


Figure 4.4 SB AM Peak TOD Plan Bandwidth—Adjusted Offsets

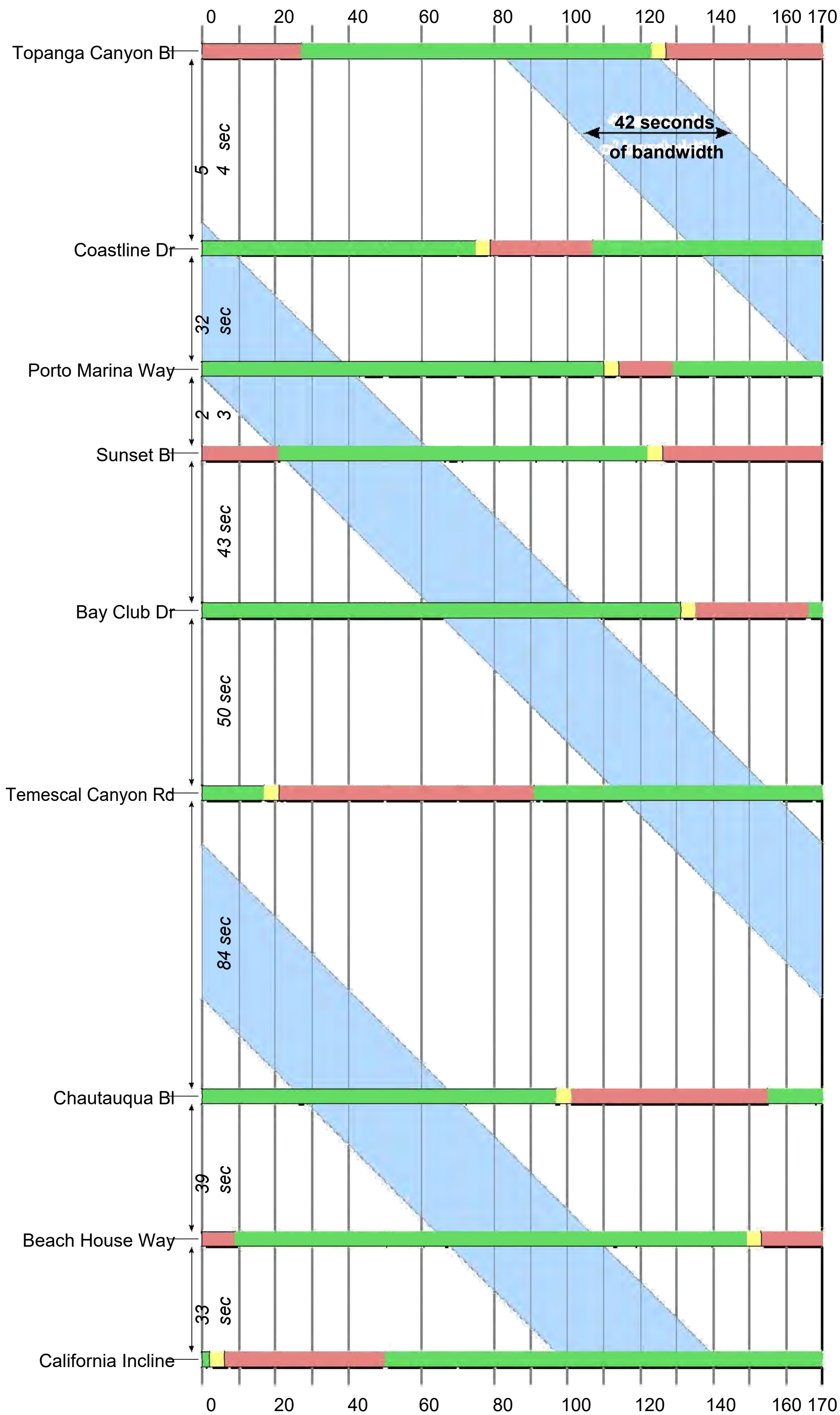


Figure 4.5 SB AM Peak TOD Plan Bandwidth—SYNCHRO Offsets

4.2 Testing the Performance of Fixed-Time Plans with Simulation

To give us a better understanding of how traffic will respond to our new fixed time TOD timing plans prior to the field implementation, we applied them to a calibrated AIMSUN [7] microscopic simulation model of the PCH corridor. From these results, we refined our timing plans to address any operational issues observed in the simulation, e.g., long queues on cross-streets, or delays on the PCH mainline.

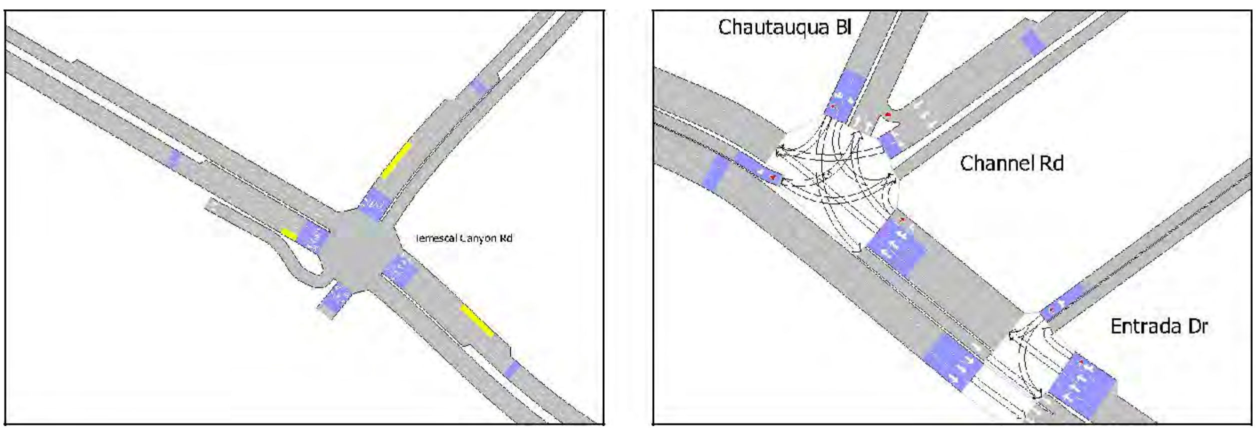
4.2.1 AIMSUN Model Development

Before we could test the new timing plans in the AIMSUN model, it had to be accurately constructed. The general steps we followed in building our model were:

Create links and nodes: Using a properly scaled, high resolution satellite image of the PCH corridor for reference, we drew the links and intersections with proper spacing and lane counts (see Figure 4.6a). We then specified the proper speed limits, lane widths, and grade for each link.

Define intersection movements: At each intersection, we defined all allowable turning movements and the yield points for permissive turns as needed (see Figure 4.6b). We also specified the acceptable origin and destination lanes for each movement (e.g., right turns were only allowed to end in the right lane of the receiving link, in accordance with California law), and created clearance requirements for each movement to avoid any unrealistic gridlock conditions from arising in the simulations (which was particularly common at the complex Chautauqua Blvd intersection, where traffic would often enter immediately after the start of green and before any vehicles currently in the intersection had sufficient time to clear). Allowable right-turn-on-red movements were also specified in the model.

Add vehicle detectors: We added inductive loop detectors (see Figure 4.6a), using roadway schematics and Street View imagery from Google Maps to properly position them in the model.



(a) Bus stops (yellow), lane drops, and detector locations (blue) at Temescal Canyon Rd.

(b) Turning movements at Chautauqua Bl, which also include yield points and clearance requirements.

Figure 4.6 Details of the PCH model in AIMSUN

Add traffic volumes: The ATCS Detector History reports provided us with the volume data. As we had done earlier with the SYNCHRO model, we limited ourselves to data from Tuesdays, Wednesdays, and Thursdays, due to the more inconsistent nature of traffic on Mondays, Fridays, and weekends. We used a specially constructed Excel spreadsheet to automatically retrieve 15-minute detector volumes from the ATCS Detector History reports and prepare them in a plain text format. These text files could then be imported into AIMSUN using a custom written Python script.

As we did earlier when inputting volume data into SYNCHRO, we had to make assumptions about turning movement volumes for lanes without detectors. For example, at the Chautauqua/Channel/Entrada intersection, the left turn capacity from southbound PCH to Chautauqua/Channel is affected by the fraction of vehicles attempting to make the hard left turn onto Chautauqua Bl, since these drivers must wait for gaps in oncoming traffic from Channel Rd. These waiting drivers can sometimes obstruct the queued traffic behind them as they search for gaps, which reduces the left turn capacity and increases the left turn queue length. If this queue becomes long enough to spill beyond the dedicated turn lane, it begins to disrupt the free flow of vehicles heading south on PCH, resulting in longer and highly-variable travel times. Since the detector data does not enable us to measure the fraction of left-turning vehicles that turn onto Chautauqua Blvd, this parameter must be assumed.

Add heavy vehicle volumes: Using the video data of the cross-street queue lengths, we counted the number of heavy vehicles entering PCH per hour over a period of several hours. Specifically, we counted all trucks, non-MTA buses, vehicles towing trailers, and any other vehicles that were significantly longer than normal or had noticeably lower acceleration than normal. Using this in conjunction with the 15-minute volume data, we approximated the flow fraction of heavy vehicles on PCH and added truck volumes to the model accordingly.

Assign signals and detectors to each intersection movement: We assigned each movement to one or more actuated phases. For each phase, we set the recall mode, minimum and maximum green times, yellow duration, and all-red time. Stop bar detectors and upstream advance detectors were assigned to the appropriate phases and given the ability to call or extend its phase as appropriate.

Add bus routes: Using MTA bus schedules for the lines that traverse the PCH corridor, we defined bus departure tables and routes in our model that vary throughout the day. Using Street View imagery from Google Maps, we then located the positions of each bus stop along the corridor and added them to the model accordingly.

4.2.2 AIMSUN Model Calibration

Once our AIMSUN model was fully defined and operational, we calibrated the vehicle, roadway, and intersection parameters so that the model behavior would reflect our real-world measurements as closely as possible. In particular, the two metrics we used for calibration were queue lengths on the cross-streets and travel times on the PCH mainline.

The parameters that we adjusted on a model-wide level to influence both travel times and queue lengths were:

Driver reaction times: By adjusting the reaction time distribution parameters, we were able to make drivers more or less responsive to events such as the initiation of a green phase (for drivers at the front of a queue) or the acceleration of a vehicle immediately ahead (for drivers in the middle of a queue). In both instances, the net effect would be a change in intersection capacity, generating different queue lengths and travel times.

Car-following model formulation: A two-lane car-following model and a one-lane car-following model were both tested. Ultimately, we found that switching between these models had only marginal effects on the travel times and queue lengths.

Driver lane-changing aggressiveness: By adjusting how aggressively drivers change lanes by accepting shorter gaps, we could influence the capacity and travel time of individual links.

Free-flow speeds: Higher free-flow speeds on PCH mainline resulted in lower travel times on each link. Reducing the free-flow speed on the cross street generally resulted in lower queue lengths, as lower cruising speeds would result in a higher density of vehicles still upstream of the queue, and consequently a lower number of vehicles queued.

In addition, the following parameter was adjusted on each cross-street to influence queue lengths:

Turning maneuver speed from the cross-street to PCH: Reducing the speed of the turning maneuvers onto PCH reduced the number of vehicles that could be accommodated during each phase, generating longer queue lengths on the cross-street.

The parameters that we adjusted on each PCH segment to influence travel times were:

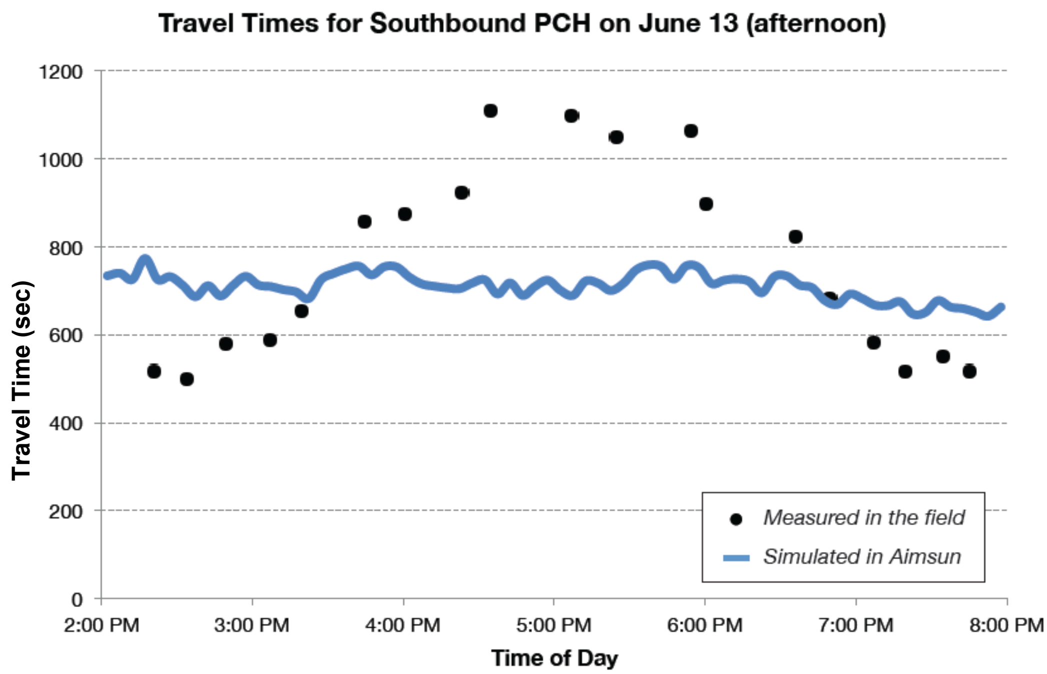
Right-lane speed: In many instances, the free-flow speed on the right lane on a particular PCH link may be lower than the rest of the through lanes due to several causes including: bicyclists using the right lane, or right shoulder to the point where drivers in the right lane do not feel comfortable passing, drivers attempting to pull into or out of parallel parking spots or driveways, drivers searching for addresses or parking, and slow drivers or heavy vehicles.

Through-movement departure speeds at intersections: Higher speeds for the through-movement at an intersection result in higher capacities and lower travel times on the PCH segments that are upstream of that intersection.

We simulated with AIMSUN all time periods for which we developed signal timing plans. For each time period, we ran multiple model replications (simulations with different random seeds) and warm-up periods of 15 minutes for each. To simulate the actual signal behavior during each of those periods, we used the ATCS Real-Time Split Monitor Reports to evaluate the average splits for each intersection and phase by hour. These average splits were automatically calculated for each day and hour from the ATCS reports using an Excel spreadsheet designed specifically for this purpose, and were saved to plain text files that could be subsequently imported into AIMSUN using a Python script designed to parse them. The signal control plans based on these average splits for each hour were then operated in AIMSUN as fixed time rather than actuated, to ensure that the cross-streets received as much green time on average in simulation as they had in reality.

Figure 4.7 shows sample comparisons between the calibrated model travel times and the field measured travel times, while Figure 4.8 shows the fit between the simulated and observed queue lengths at major cross-streets.

The calibration efforts resulted in an AIMSUN model that is much more representative of real-world conditions on the PCH corridor, but it is still not perfect (as could be said for any simulation) as revealed by the deviations between observed and simulated travel times and queue lengths in Figures 4.7 and 4.8. This is due in part to the limitations of AIMSUN, and to the assumptions inherent in the model. Identifying these issues and being aware of their potential consequences will help us properly interpret the effects we observe when our new TOD timing plans are tested in AIMSUN. For example, knowing that cross-street queue lengths in AIMSUN have a tendency to grow longer than in reality (as a result of the way we calibrated the model) suggests that long queues on PCH resulting from our TOD timing plans are a greater cause for concern than long queues on the cross-streets.



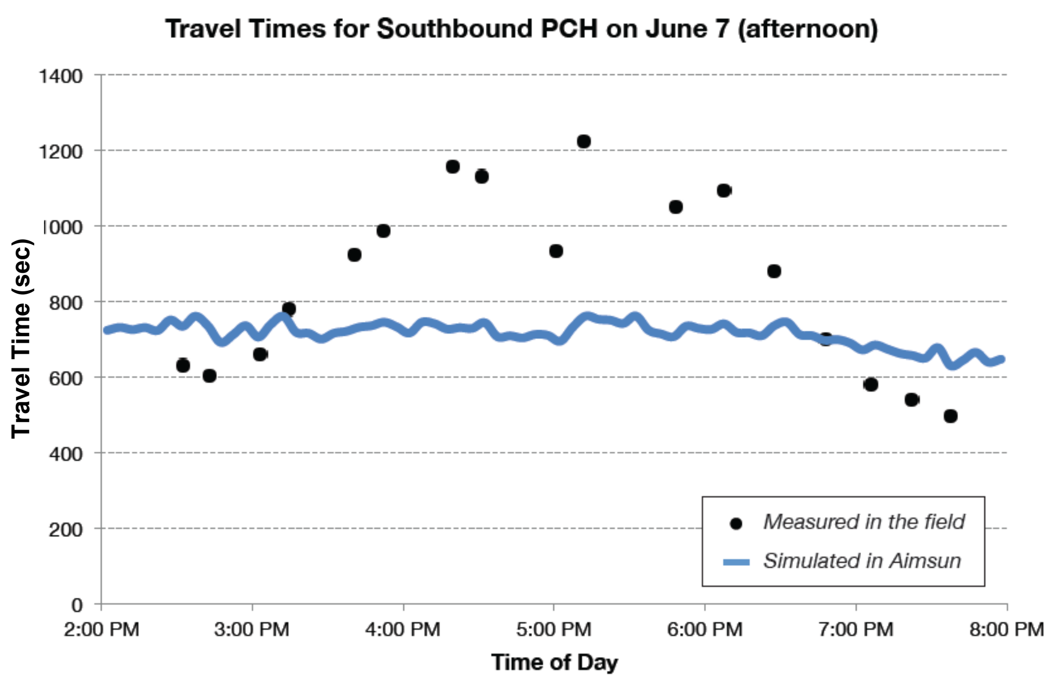


Figure 4.7 Simulated vs. Measured Travel Times on SB PCH

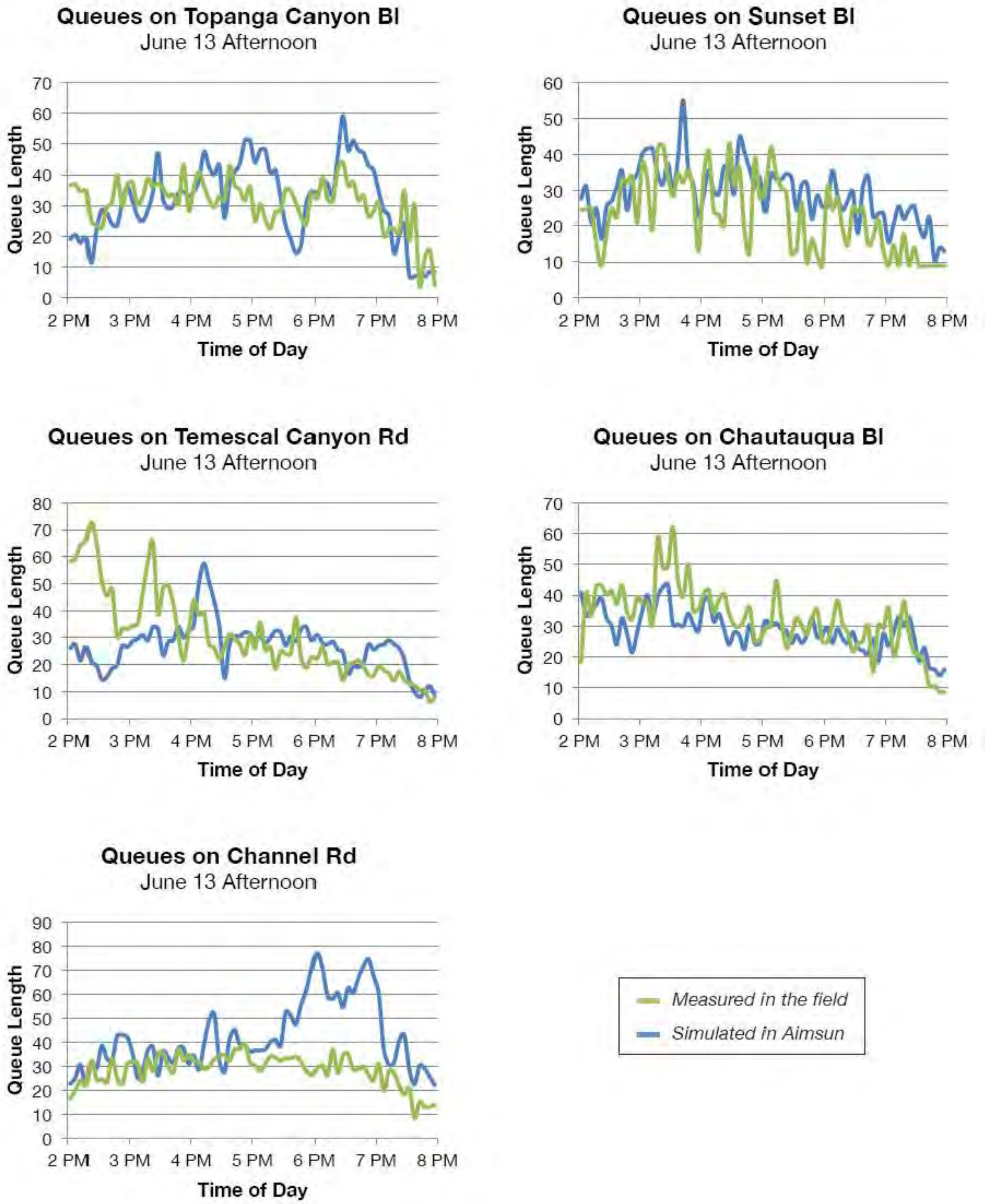


Figure 4.8 Simulated vs. Measured Queue Lengths on PCH Major Cross-Streets

Among the behaviors and situations that our model cannot accurately recreate are:

1. Parking maneuvers
2. Pedestrian activity (whether crossing at signalized intersections or mid-block).
3. People pulling into and out of driveways.
4. Bicyclists on the roadway.
5. People looking for addresses and beach parking lots, and people queuing up waiting to make turns into these driveways and lots (and impeding traffic in the left or right lane as a result).
6. Minor traffic incidents, which occurred on occasion during our travel time measurements. Any travel time runs that overlapped with an incident's presence were discarded, but the impact on travel times had a tendency to persist even long after the incident had cleared.
7. Distracted drivers.

Taken collectively, all of the above factors negatively impact the flow of traffic on PCH and its cross-streets, resulting in potentially optimistic representations of corridor performance in AIMSUN. Furthermore, there are several other unavoidable limitations of our model that impact the reliability of the simulated travel times and queue lengths:

1. The effects of the Haul Road traffic signal were excluded from the model due to lack of signal timing data for it. This signal would have had a tendency to increase the travel times on PCH.

2. Entrance volume data are obtained from detectors just upstream of the two intersections at the north and south ends of the corridor (i.e., at California Incline heading north, and at Topanga Canyon Blvd heading south). This misses most of the delays incurred by drivers waiting to get onto the corridor prior to passing through each of these intersections, which is a significant portion of the overall delay during the peak periods. The resulting entrance flows at the north and south ends of the corridor are limited by the capacity of the intersections themselves due to the positioning of the detectors at the stop bars, making it impossible to capture the queues and delays upstream of these two intersections.

3. Signal phase durations are taken as averages over hour-long periods, and are set to fixed-time operation during each of those hours rather than ATCS adaptive control, because we could not obtain access to the adaptive control algorithm used in PCH.

4.2.3 Testing the Fixed-Time TOD Plans

With our AIMSUN model calibrated to be as accurate a representation of reality as possible, we loaded the new TOD timing plans into the model using a custom-made Python script within AIMSUN and ran several simulations to test each one. For each period, we tested the corresponding timing plan on volume data from several weekdays (Tuesdays, Wednesdays, and Thursdays only), running multiple replications for each weekday. We then made adjustments to the splits at each intersection to address any long queues that formed on PCH or the cross-street in our simulation.

To assist us in evaluating the effects of our splits and cycle length adjustments, we used AIMSUN's database output option to save travel time and queue length statistics for each link in 5-minute increments of the simulation. These results were then read into Excel and aggregated to produce average travel time and queue length measures for the entire simulation period, and then aggregated across all days and replications to generate performance metrics that characterized the overall effects of our timing plan modifications. This enabled us to consistently measure and evaluate the effects of our changes in a precise, quantitative way, and freed us from a dependence on less-reliable spot checks of each link and intersection as the simulations ran—a more subjective method that is poorly suited for comparing similar timing plan alternatives.

4.3 Field Implementation

The optimized TOD signal timing plans were implemented on the PCH test corridor in the second half of May 2013. Although these TOD plans had been optimized, thoroughly tested, and fine-tuned in SYNCHRO and AIMSUN traffic simulation software before being deployed, they still required additional adjustments in the days following their field implementation to achieve the best performance in the field. This was expected, as even the best traffic simulation and analysis software cannot reproduce real-world conditions and driver behavior with complete accuracy.

ATCS control was switched off on the morning of May 15, 2013, and a set of TOD timing plans with optimized cycle lengths (the "TOD-Optimized" strategy) ran in its place. By the end of May 16, however, it was apparent that fine-tuning would be necessary to improve the performance of these shorter cycle length plans in the field. Because the ATCS system used 230-second cycle lengths in June 2012 (the time period that our timing plan development was based on), longer cycles were considered the more conservative approach, as the longer cycle lengths were known to be viable from our experience with the ATCS system, whereas the shorter cycle lengths— despite being shown to be better in our computer analysis—had no such historical justification on PCH. Therefore, the longer cycle length TOD plans (the "TOD-230" timing plans) were used during the fine-tuning period, which spanned the three weekdays between May 17 and May 21, 2013 (May 18-19 was a weekend). When these refinements were complete, the TOD-230 timing plans were then left in place to run for an additional three weeks as performance data was collected in the field.

The same adjustments from the May 17–21 period that were necessary to improve the performance of the TOD-230 timing plans were subsequently translated over to the TOD-Optimized timing plans as well. For example, splits of the TOD-Optimized timing plans were adjusted so that their proportions matched those of the TOD-230 plans as closely as possible, subject to the minimum green time constraints of each phase. However, a small refinement period was still required in the two days following the deployment of the TOD-Optimized plans in June 2013, largely to fix issues related to the acceptance of the timing plans by the field controllers.

Figure 4.9 summarizes the dates of implementation for the TOD-230 and TOD-Optimized timing plans in May and June 2013. Traffic incidents, controller errors, and other performance-affecting corridor irregularities are indicated on the relevant dates as well.

In the first days of running the new timing plans, we closely monitored the corridor through a combination of probe vehicle runs, TMC real-time monitoring software, and CCTC video feeds, and used these observations to verify that all timing plans were operating properly.

From our observations, we discovered that the signal at Chautauqua Blvd was a larger bottleneck during the PM peak than we had anticipated, with delays at Sunset Blvd dropping very low as that bottleneck became starved by the upstream flow constraint at Chautauqua Blvd. Upon deeper investigation, we determined that the controller at Chautauqua Blvd was not honoring the maximum green time for the Chautauqua Blvd signal, and was instead giving that approach an extra 15 seconds during each cycle that should have gone to northbound PCH. Once the signal timing engineers isolated and resolved the problem, the bottleneck at Sunset Blvd became active, and the rest of the corridor behaved as expected.

Figure 4.9 Timeline of TOD Plans Implementation on PCH

4.4 Field Adjustments to the Optimized Splits and Offsets

Making adjustments to the offsets based on field observations can be difficult, due to the fact that the coordinated phases (i.e., the PCH through phases) may start earlier than expected if any of the other phases terminate early due to low demand. However, the start of yellow can be used as a reliable marker of offset execution, since that event occurs consistently at the same time from one cycle to another. If the yellow phase for a particular intersection starts sooner or later than expected, the offset for that signal would need to be manually adjusted higher or lower to compensate, and a similar adjustment would need to be applied to all downstream intersections as well.

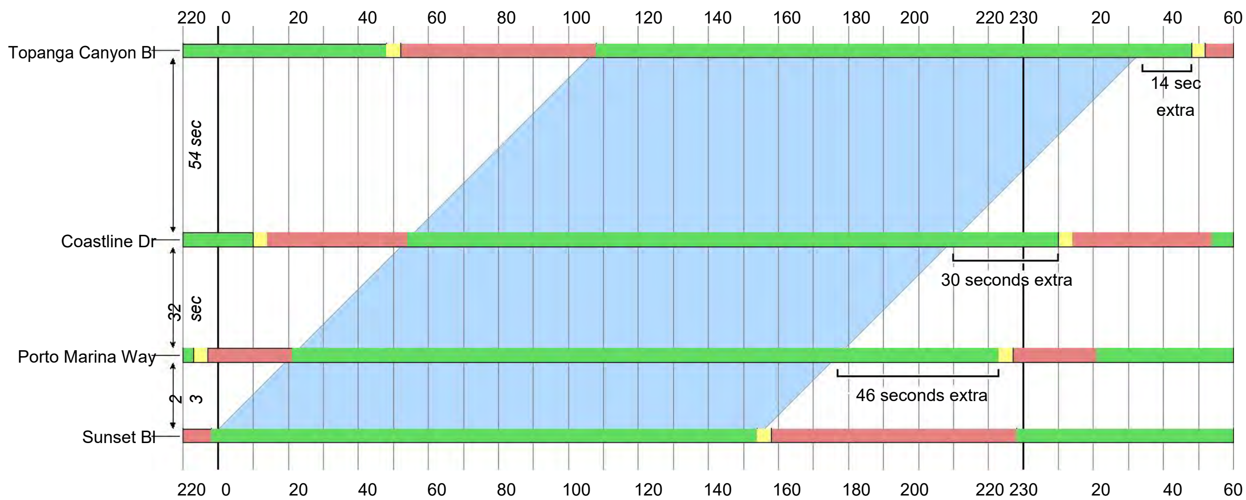
Although such an adjustment would not normally be necessary, it was warranted in our case on PCH past Chautauqua Blvd, where the use of two separate signals on the northbound side of PCH with two different yellow start times created ambiguity in defining the true “start of yellow.” As a result, the offset between Chautauqua Blvd and Temescal Canyon Rd (the intersection immediately north on PCH) during the PM Peak was several seconds longer than expected, and so we had to manually lower the offsets at Temescal Canyon Rd and every other intersection farther north to compensate.

Another necessary adjustment in the PM Peak was shifting the offsets of all signals downstream of the critical bottleneck at Sunset Blvd to provide better progression between Sunset Blvd and the north end of the corridor, thereby reducing the likelihood of a queue spilling back to Sunset Blvd from any downstream intersection—which would hurt the capacity of the one intersection that needs it most. To accomplish this, we examined the bandwidth between Sunset Blvd and Topanga Canyon Blvd (i.e., the last signal on the northbound side of PCH), and adjusted the offsets of all signals after Sunset Blvd so that the green phases for the through movement were centered on the bandwidth (Figure 4.10b), as opposed to being aligned along the left edge (Figure 4.10a).

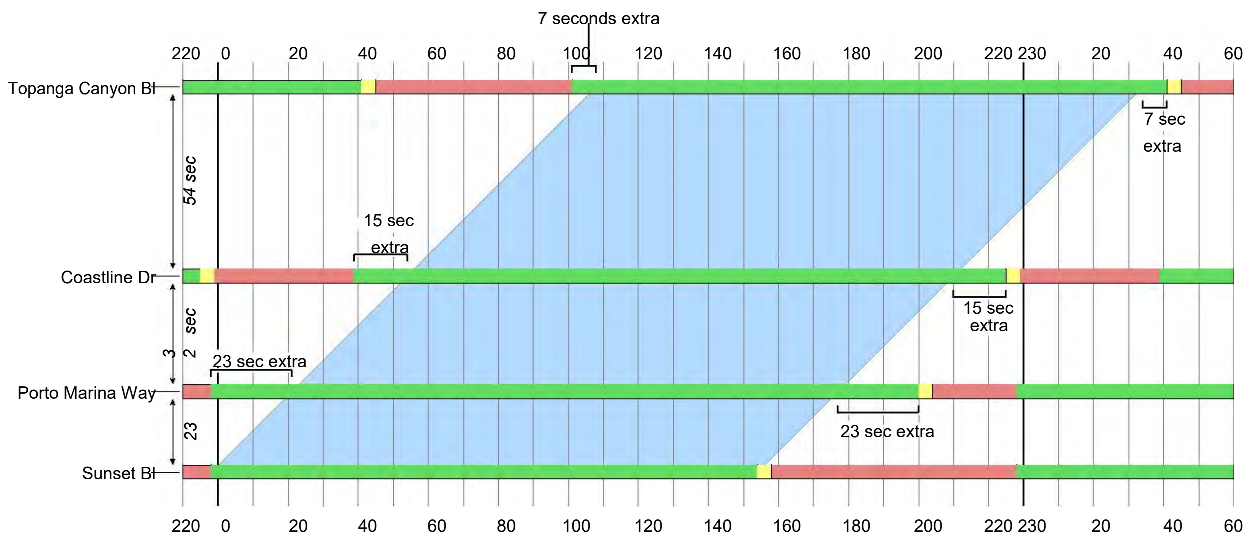
As Figure 4.10 indicates, this shift results in half of the extra green time, or the remaining green time after taking out the time used by the bandwidth, being used to clear any queues at the signal before the arrival of the platoon from Sunset Blvd. The other half of the extra green time is then used to accommodate any late arrivals from Sunset Blvd due to platoon dispersion. Initially, all of the green phases had been aligned to the bandwidth along the left edge, meaning that all extra green time was occurring after the bandwidth. This also meant that the platoon from Sunset was being stopped at every downstream intersection that had a queue present, since the green phases were starting just as the platoon was expected to arrive (without making any accommodations for vehicles that may have already been waiting at the signal).

We also adjusted the splits at several intersections in response to disproportionately long queues forming on either the cross-street or the PCH mainline. Specifically, we adjusted the splits at Sunset Blvd to give the minimum allowable green time to Sunset Blvd, thereby leaving the largest splits possible for the through movements at that intersection. The minimum green time was found to be more than sufficient to accommodate the volumes on Sunset Blvd for almost every cycle. Furthermore, the splits at Topanga Canyon Blvd in the morning and at Chautauqua Blvd in the afternoon were adjusted to balance delays between the PCH mainline and the cross-street traffic, with moderate priority going to PCH.

Table 4.4 shows the final splits for each of the TOD plans after adjustments based on field observations. Note that although Sunset is still the most constraining bottleneck location during the PM peak, it is not possible to give any more green time to it due to the minimum green time constraints on the cross-street phase due to pedestrian requirements. Furthermore, additional green time was not given to PCH at Topanga Canyon Blvd during the AM Peak despite the fact that this was the most constraining bottleneck on the southbound side, as doing so resulted in delays on the cross-street that were longer than the delays on PCH.



(a) Before adjusting offsets to center the green phases on the bandwidth from Sunset Blvd.



(b) After adjusting offsets to center the green phases on the bandwidth from Sunset Blvd.

Figure 4.10 Offsets for Intersections downstream of Sunset Blvd—TOD 230 Plan, PM Peak

Table 4.4 Final Splits for TOD Timing Plans

	CA Incline						Beach House Way				Chautauqua Bl				
Phase	1	2	4	5	6	8	1	2	6	8	2	4	5	6	7
TOD-Optimized AM Peak	6	123	24	47	81	29	9	137	151	8	118	23	38	75	23
Midday	14	115	24	59	69	24	7	54	66	8	112	29	39	68	29
TOD-Optimized PM Peak	8	135	20	53	89	24	7	59	71	8	124	27	34	85	27
TOD-230 AM Peak	6	173	34	66	112	38	13	191	209	10	167	34	54	108	34
TOD-230 PM Peak	11	176	26	70	116	30	22	174	201	18	165	36	46	114	36

	Temescal Canyon Rd						Bay Club		Sunset Bl					
Phase	1	2	3	4	5	6	2	4	1	2	3	4	5	6
TOD-Optimized AM Peak	8	103	6	31	27	84	136	23	10	109	6	24	24	95
Midday	9	95	10	34	25	79	135	24	11	99	10	29	20	90
TOD-Optimized PM Peak	8	112	6	32	19	101	146	23	10	119	6	24	10	119
TOD-230 AM Peak	8	147	8	45	38	117	196	23	11	164	10	24	32	143
TOD-230 PM Peak	10	147	8	43	26	131	196	23	14	159	12	24	18	155

	Porto Marina Way				Coastline Dr				Topanga Canyon Bl			
Phase	2	4	5	6	2	4	5	6	2	5	6	8
TOD-Optimized AM Peak	152	7	6	141	139	20	5	130	98	7	86	61
Midday	66	8	6	55	135	24	13	118	56	8	43	18
TOD-Optimized PM Peak	163	6	6	152	149	20	6	139	150	14	131	19
TOD-230 AM Peak	209	10	8	196	199	20	9	186	135	10	120	84
TOD-230 PM Peak	213	6	6	202	199	20	9	186	194	19	170	25

4.5 Comparison of ATCS, TOD-230, and TOD-Optimized Phase Durations

We compared the green splits from ATCS and from the TOD-230 and TOD-Optimized timing plans, for the peak hours of the analysis periods and for those days for which we have available travel time and volume data. This is because we use the findings from this phase duration analysis to help us interpret the results of our ATCS assessment of control strategies with respect to traffic performance.

We calculated the average ATCS phase durations for the PCH mainline in Table 4.5 and for the major cross-street approaches in Table 4.6. The tables also include the splits for the TOD-230 and TOD-Optimized timing plans. Because the cycle lengths were not constant across the different control strategies (i.e., ATCS, TOD-230, and TOD-Optimized) or across times of day, the typical phase durations in Tables 4.5 and 4.6 are also given as percentages of the cycle to facilitate comparisons.

In the AM peak period the ATCS splits are longer for the northbound PCH by about 1-10% of percent cycle length compared to the splits from TOD-230 and TOD-Optimized plans. The absolute differences between ATCS and TOD-Optimized plan splits are quite large because of the longer cycle under ATCS (240 vs. 170 sec); the additional time in the cycle under ATCS is used mostly by the phases serving the mainline PCH. The differences in the splits between TOD-230 and TOD-Optimized are small in terms of percent cycle length. The same pattern of results can be observed for the southbound PCH, although the differences between ATCS and TOD-230 are smaller in terms of % of cycle length. The findings from comparison of splits in the PM peak period were similar as in the AM peak. In the midday period, the ATCS splits are longer by 5-12 % compared to the TOD-Optimized plan for both directions of PCH in terms of percent of cycle.

Comparison of the splits for the major cross streets (Table 4.6) show that all strategies result in similar phase durations in all time periods. Most of the splits on the cross streets range between 10-20% of the cycle, except the Topanga Canyon Blvd where the green split is about 40% of the cycle because of the high volume especially in the AM peak.

Table 4.5 Splits for Northbound and Southbound PCH At Major Intersections

		California Incline	Chautauqua Bl	Temescal Canyon Rd	Sunset Bl	Topanga Canyon Bl
Northbound PCH Splits, in seconds (percent of cycle in parentheses)						
AM Peak	ATCS	131 (56%)	128 (53%)	145 (61%)	156 (65%)	127 (53%)
	TOD-230	112 (49%)	108 (47%)	117 (51%)	143 (62%)	120 (52%)
	TOD-Optimized	81 (48%)	75 (44%)	84 (49%)	95 (56%)	86 (51%)
Midday	ATCS	78 (53%)	63 (45%)	82 (57%)	87 (58%)	99 (68%)
	TOD-Optimized	69 (41%)	68 (40%)	79 (46%)	90 (53%)	43 (51%)
PM Peak	ATCS	153 (64%)	135 (56%)	163 (68%)	163 (68%)	183 (76%)
	TOD-230	116 (50%)	114 (50%)	131 (57%)	155 (67%)	170 (74%)
	TOD-Optimized	89 (49%)	85 (47%)	101 (56%)	119 (66%)	131 (73%)
Southbound PCH Splits, in seconds (percent of cycle in parentheses)						
AM Peak	ATCS	183 (79%)	187 (78%)	161 (68%)	177 (74%)	138 (58%)
	TOD-230	173 (75%)	167 (73%)	147 (64%)	164 (71%)	135 (59%)
	TOD-Optimized	123 (72%)	118 (69%)	103 (61%)	109 (64%)	98 (58%)
Midday	ATCS	109 (74%)	101 (72%)	91 (63%)	96 (64%)	112 (77%)
	TOD-Optimized	115 (68%)	112 (66%)	95 (56%)	99 (58%)	56 (66%)
PM Peak	ATCS	200 (83%)	187 (78%)	173 (72%)	177 (74%)	206 (86%)
	TOD-230	176 (77%)	165 (72%)	147 (64%)	159 (69%)	194 (84%)
	TOD-Optimized	135 (75%)	124 (69%)	112 (62%)	119 (66%)	150 (83%)

Table 4.6 Splits for the Major Cross Streets on PCH

		Channel Rd	Chautauqua Bl	Temescal Canyon Rd	Sunset Bl	Topanga Canyon Bl
AM Peak	ATCS	42 (18%)	54 (23%)	47 (20%)	34 (14%)	93 (39%)
	TOD-230	34 (15%)	54 (23%)	45 (20%)	24 (10%)	84 (37%)
	TOD-Optimized	23 (14%)	38 (22%)	31 (18%)	24 (14%)	61 (36%)
Midday	ATCS	28 (20%)	33 (24%)	25 (17%)	25 (17%)	25 (17%)
	TOD-Optimized	29 (17%)	39 (23%)	34 (20%)	29 (17%)	18 (21%)
PM Peak	ATCS	42 (18%)	47 (20%)	36 (15%)	33 (14%)	25 (10%)
	TOD-230	36 (16%)	46 (20%)	43 (19%)	24 (10%)	25 (11%)
	TOD-Optimized	27 (15%)	34 (19%)	32 (18%)	24 (13%)	19 (11%)

CHAPTER 5

EVALUATION OF CONTROL STRATEGIES

This Chapter describes the evaluation of alternative control strategies based on field data. Section 5.1 presents the findings from the comparisons of travel times on PCH under different control strategies collected with Bluetooth detectors. Section 5.2 presents a comparison of travel times collected from probe vehicles and Bluetooth detectors under ATCS control. Section 5.3 describes the findings from the arterial Level of Service analysis on PCH. The last section presents the findings from the investigation of the ATCS control under oversaturated conditions.

5.1 Arterial Travel Times—Bluetooth Data

We collected Bluetooth travel time data along PCH between March and June 2013 (Figure 3.12), and generated travel time profiles for each of the three control strategies tested:

- ATCS operation
- TOD timing plans with cycle lengths constrained to 230 seconds during the AM and PM peaks (i.e., “TOD-230”)
- TOD timing plans with SYNCHRO-optimized cycle lengths (i.e., “TOD-optimized”)

These travel time profiles and statistics are given in Figures 5.1 through 5.4 for northbound and southbound directions of PCH corridor and control strategy. Tables 5.1 and 5.2 show travel time statistics disaggregated by link separately for the northbound and southbound travel directions. In the southbound direction, the portion of PCH between Haul Road and McClure Tunnel is excluded from our analysis, because congestion from the freeway entrance had a tendency to occasionally spill back past the Bluetooth unit at the McClure Tunnel particularly in the PM Peak, which led to inappropriate inflation of southbound travel time measurements for the study corridor.

Because ATCS operation was not constrained to a cycle length of 230 seconds during the Midday period, the TOD timing plan for 10 AM to 2 PM was not constrained either. Consequently, the Midday TOD timing plan is unchanged between the TOD-230 set of plans and the TOD-Optimized set, as the Midday timing plan uses the SYNCHRO-optimized cycle length of 180 seconds in both cases. Thus, we would not expect the travel times to differ on a statistically significant level during the Midday period between those days using the TOD-230 timing plans and those days using the TOD-Optimized set. Any 5-minute window of the midday period with a statistically significant difference between the TOD-230 and TOD-Optimized operation reflects either a Type I error, or the influence of some other variable we have not explicitly controlled for (such as changes in weather patterns or traffic patterns between May and June).

As a measure of typical corridor travel time performance, we have used the median travel time rather than the mean, due to its robustness against outliers (which can inflate the expected values for travel time, since travel time distribution tends to be skewed to the right). This also protects against traffic incidents, special events, or other atypical conditions inappropriately influencing our travel time profiles. To obtain the travel time profiles in Figures 5.1-5.4, the median of all data points in each 5-minute period is computed for each day. Next, the median of these travel time estimates for a particular time of day is calculated across all days of data for a particular control strategy, and this second calculated median is used to obtain the travel time profiles shown in Figures 5.1-5.4. The figures also show the inter-quartile ranges (IQRs), which function as a measure of the travel time variability throughout the day. The lower edge of the IQR indicates the 25th percentile travel time, whereas the upper edge indicates the 75th percentile. These can roughly be interpreted as the lower and upper bounds on the expected PCH travel times on typical days.

A Wilcoxon Rank-Sum test is used to evaluate whether the travel time data for two different control strategies represent a statistically significant difference. In Figures 5.1-5.4, green boxes are used to indicate the times of day where a statistically significant difference occurs at a 5% level.

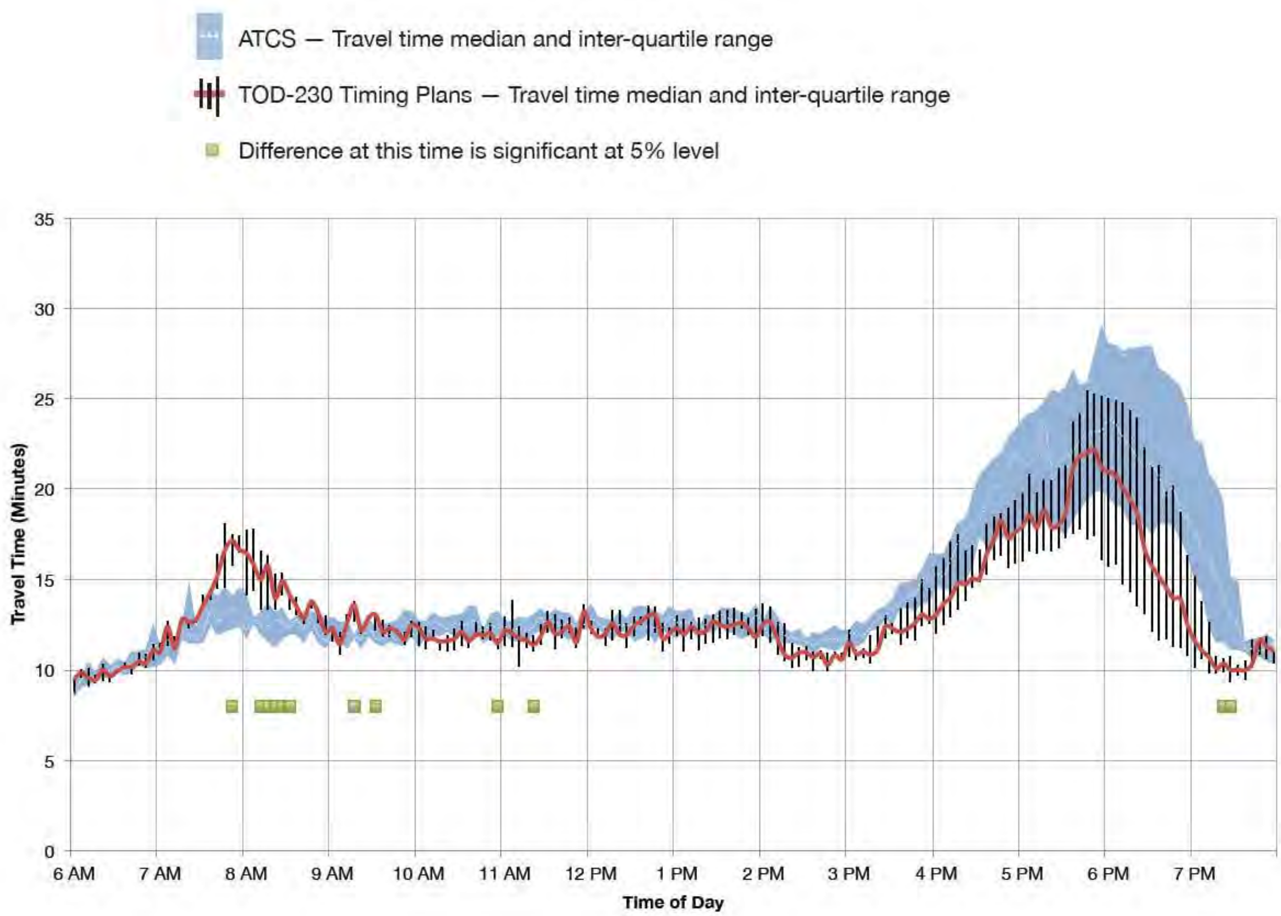
The findings can be summarized as follows:

ATCS vs. TOD-230: In the southbound direction, ATCS outperformed the TOD-230 plan in the am peak 90% of the time. The average travel time savings due to ATCS operation was 2.9 minutes. The differences in travel times were statistically significant most of the AM peak period. ATCS and TOD-230 had very similar performance in the midday and pm peak periods. In the northbound direction, ATCS outperformed the TOD-230 plan about 80% of the time, with lower travel times by an average of 1 minute. The differences in travel times were statistically significant during the 8-9 AM time interval. Both strategies had very similar performance in the midday period. TOD-230 had better performance in the pm peak; the average travel times were lower by 2.4 min compared to ATCS, but the difference was not statistically significant due to the large variations in travel times.

TOD-230 vs. TOD-Optimized: In the southbound direction, TOD optimized plan outperformed the TOD-230 plan in the am peak 60% of the time. However the differences in travel times were small and not statistically significant, except during the onset of congestion 7-8 AM time interval. In the midday, both strategies resulted in very similar performance. In the pm peak, the TOD-230 plan outperformed the TOD optimized plan most of the time; the difference in travel times was 0.8 min which was statistically significant. In the northbound direction, the TOD-Optimized plan was better 80% of the time, with average travel time savings of 1.1 minutes. The traffic performance was very similar in the midday period under both timing plans. In the pm peak, TOD-230 outperformed the TOD-optimized timing plan, and the differences were statistically significant especially at the onset and end of the peak period. The average travel time savings was 5.5 minutes with TOD-230 plan.

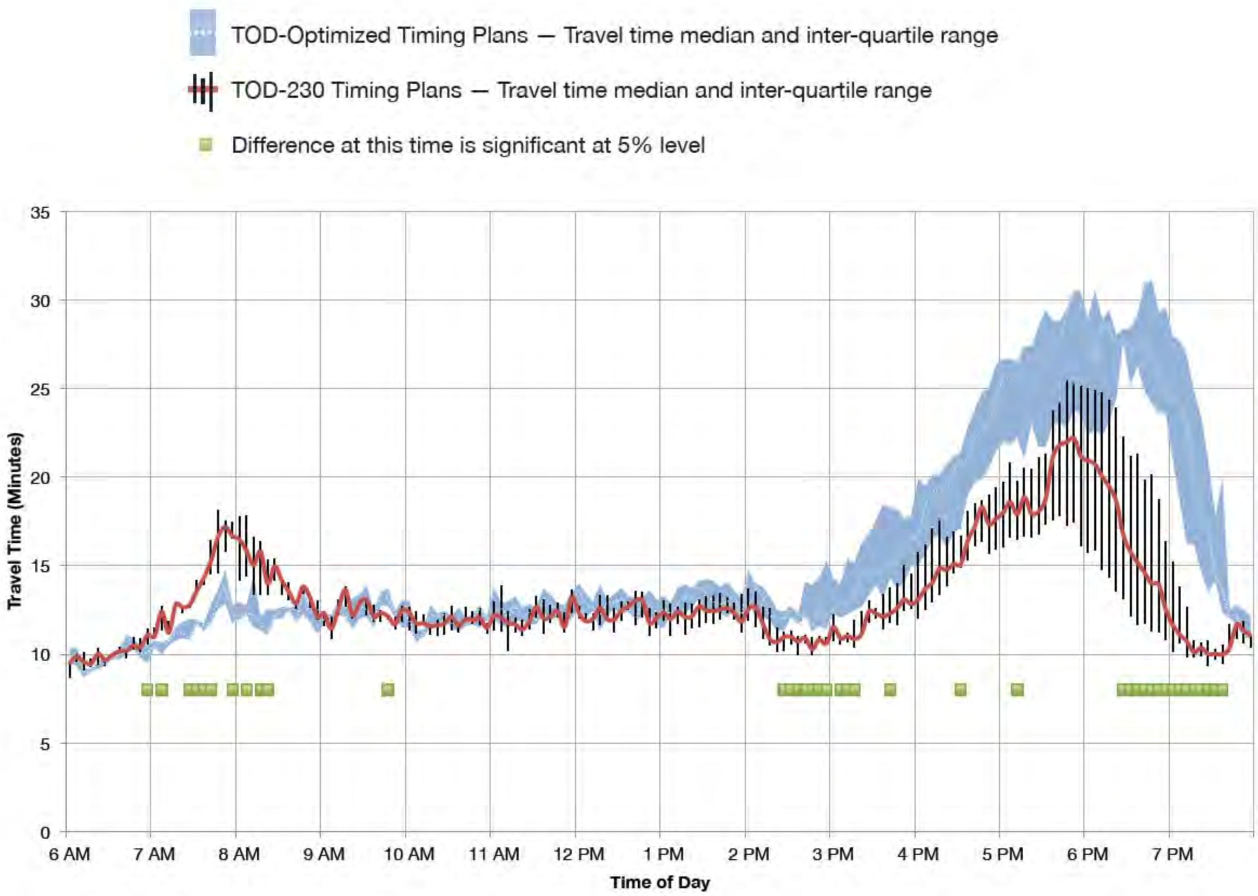
ATCS vs. TOD-Optimized: In the southbound direction, the comparison of performance measures in the same findings as ATCS vs. TOD-230: ATCS was the better control strategy in the am peak, both strategies had similar performance in the midday, and ATCS had better performance in the PM peak period. In the northbound direction, the average travel times were very similar in the am peak and midday periods. ATCS had better performance in the pm peak period.

Most of the travel time differences among the strategies in the northbound direction occurred in the first segment (McClure Tunnel to Haul Road) especially in the pm peak period as shown in Table 5.1. This segment includes the major intersections of California Incline and Chautauqua Blvd. In the southbound direction, the largest travel time differences occurred in the first section (Big Rock Drive to Coastline Drive) as shown in table 5.2. This section includes the major PCH/Topanga Blvd intersection.



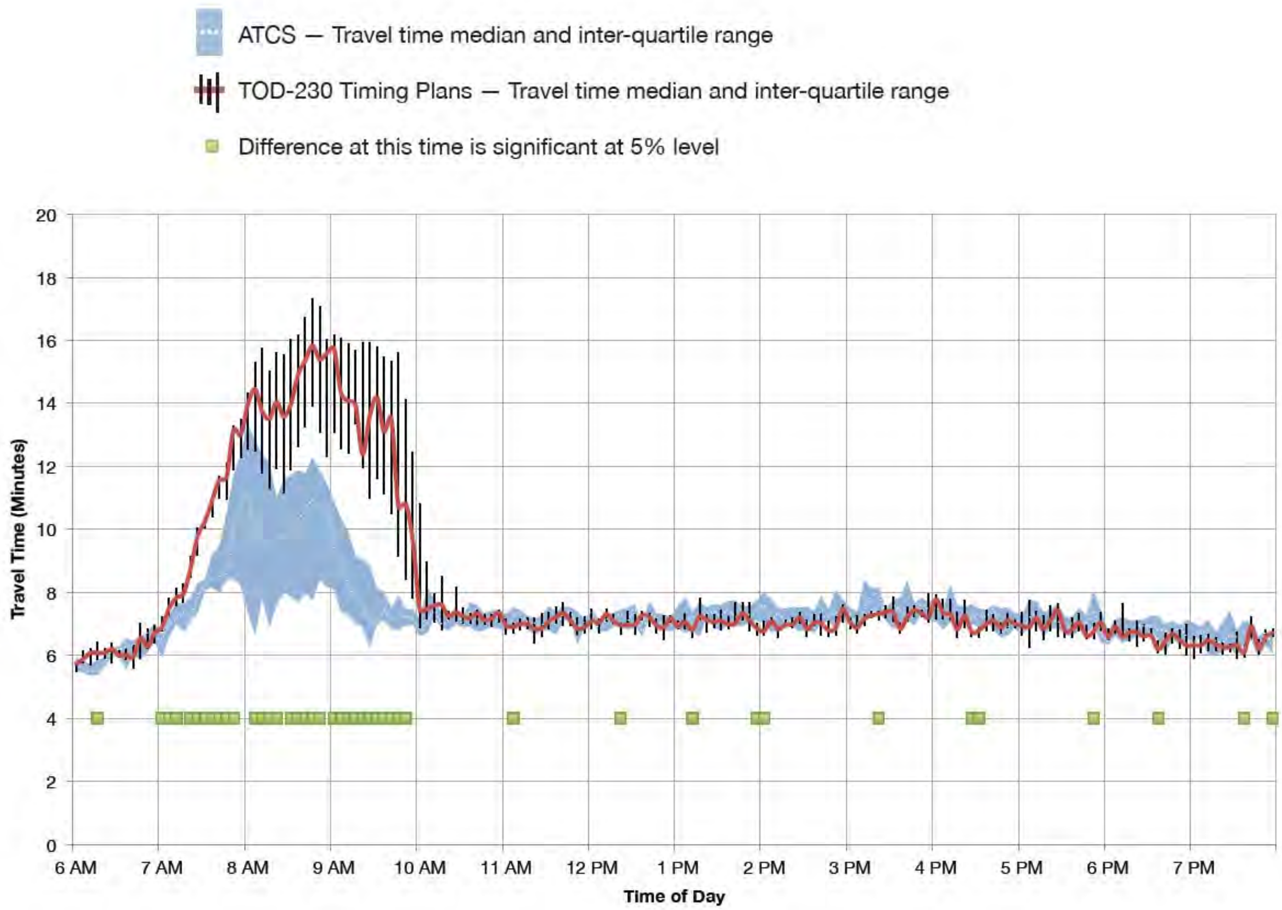
	AM PEAK 6–10 AM	MIDDAY 10 AM – 2 PM	PM PEAK 2–8 PM
Percent of time ATCS outperforms TOD-230 timing plans	79.2%	16.7%	5.6%
Average Travel Time for TOD-230 timing plans	12.66	12.14	14.33
Average Travel Time for ATCS	11.67	12.53	16.75
Average Travel Time savings with ATCS	0.99	-0.39	-2.41

Figure 5.1 NB PCH Travel Times (McClure Tunnel to Big Rock Dr), for ATCS and TOD-230 Operation



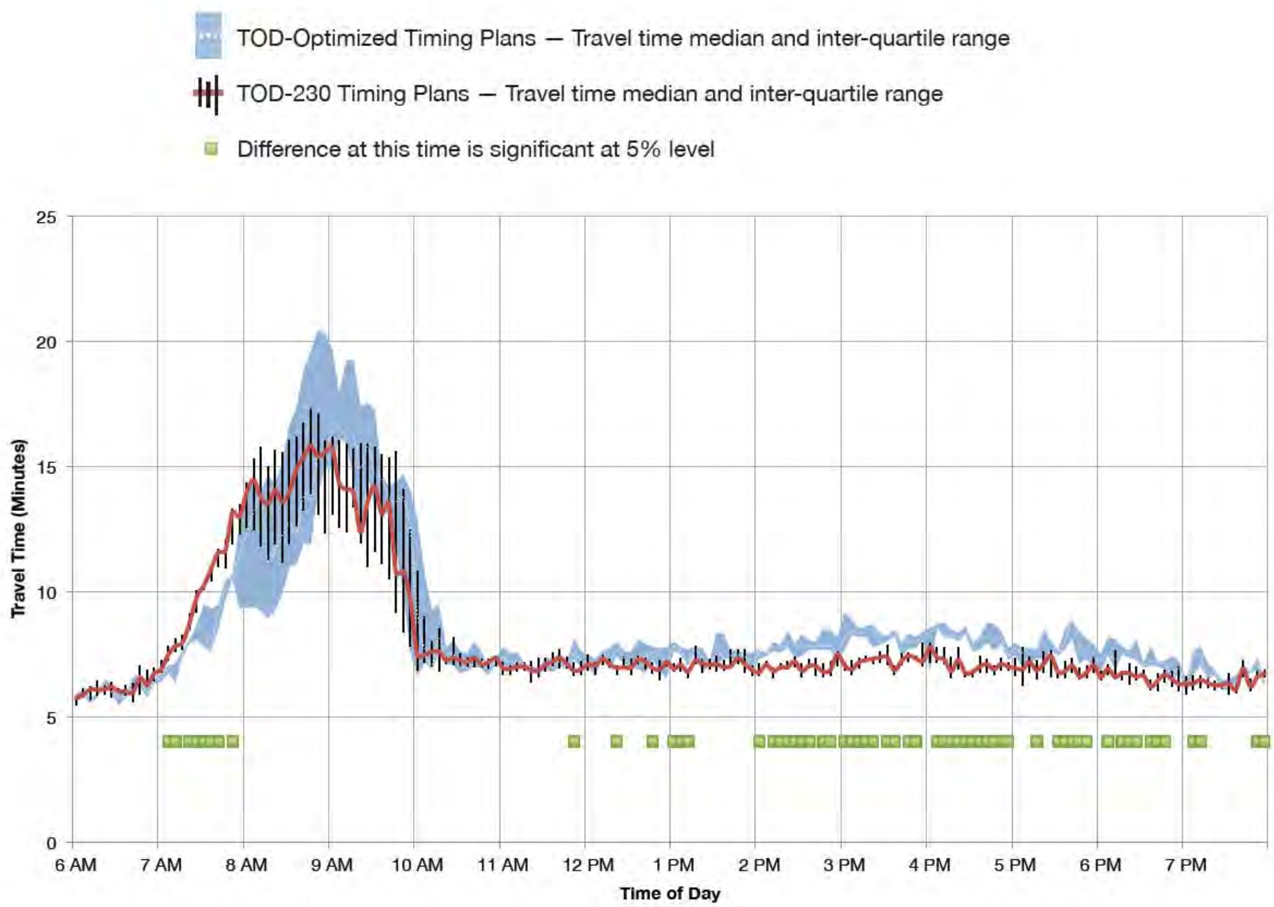
	AM PEAK 6–10 AM	MIDDAY 10 AM – 2 PM	PM PEAK 2–8 PM
Percent of time TOD-Optimized outperforms TOD-230 timing plans	81.3%	25.0%	1.4%
Average Travel Time for TOD-230 timing plans (min)	12.66	12.14	14.33
Average Travel Time for TOD-Optimized timing plans (min)	11.56	12.52	19.84
Average Travel Time savings with TOD-Optimized timing plans (min/veh)	1.10	-0.38	-5.51

Figure 5.2 NB PCH Travel Times (McClure Tunnel to Big Rock Dr), for TOD-230 and TOD-Optimized Operation



	AM PEAK 6–10 AM	MIDDAY 10 AM – 2 PM	PM PEAK 2–8 PM
Percent of time ATCS outperforms TOD-230 timing plans	89.6%	31.3%	27.8%
Average Travel Time for TOD-230 timing plans	10.90	7.12	6.87
Average Travel Time for ATCS	8.04	7.17	7.04
Average Travel Time savings with ATCS	2.87	-0.05	-0.17

Figure 5.3 SB PCH Travel Times (Big Rock Dr to Haul Road), for ATCS and TOD-230 Operation



	AM PEAK 6–10 AM	MIDDAY 10 AM – 2 PM	PM PEAK 2–8 PM
Percent of time TOD-Optimized outperforms TOD-230 timing plans	60.4%	25.0%	5.6%
Average Travel Time for TOD-230 timing plans (min)	10.90	7.12	6.87
Average Travel Time for TOD-Optimized timing plans (min)	10.65	7.33	7.65
Average Travel Time savings with TOD-Optimized timing plans (min/veh)	0.26	-0.21	-0.78

Figure 5.4 SB PCH Travel Times (Big Rock Dr to Haul Road), for TOD-230 and TOD-Optimized Operation

Table 5.1 NB PCH Travel Times by Segment —Bluetooth Data

A. Segment: McClure Tunnel and Haul Road

	AM PEAK 6–10 AM	MIDDAY 10 AM – 2 PM	PM PEAK 2–8 PM
ATCS vs. TOD-230 Operation			
Percent of time ATCS outperforms TOD-230 timing plans	79.2%	91.7%	6.9%
Average Travel Time for TOD-230 timing plans	5.33	4.90	5.79
Average Travel Time for ATCS	4.24	4.42	8.13
Average Travel Time savings with ATCS	1.10	0.49	-2.33
TOD-230 vs. TOD Optimized Operation			
Percent of time TOD-Optimized outperforms TOD-230 timing plans	93.8%	50.0%	0.0%
Average Travel Time for TOD-230 timing plans (min)	5.33	4.90	5.79
Average Travel Time for TOD-Optimized timing plans (min)	4.03	4.96	9.16
Average Travel Time savings with TOD-Optimized timing plans (min/veh)	1.31	-0.06	-3.36

B. Segment: Haul Road and Shore Drive Crosswalk

	AM PEAK	MIDDAY	PM PEAK
ATCS vs. TOD-230 Operation			
Percent of time ATCS outperforms TOD-230 timing plans	54.2%	0.0%	54.2%
Average Travel Time for TOD-230 timing plans	1.43	1.25	2.07
Average Travel Time for ATCS	1.36	1.83	2.11
Average Travel Time savings with ATCS	0.08	-0.57	-0.03
TOD-230 vs. TOD Optimized Operation			
Percent of time TOD-Optimized outperforms TOD-230 timing plans	91.7%	18.8%	8.3%
Average Travel Time for TOD-230 timing plans (min)	1.43	1.25	2.07
Average Travel Time for TOD-Optimized timing plans (min)	1.12	1.34	2.98
Average Travel Time savings with TOD-Optimized timing plans (min/veh)	0.32	-0.09	-0.91

C. Segment: Shore Drive Crosswalk and Coastline Dr

	AM PEAK	MIDDAY	PM PEAK
ATCS vs. TOD-230 Operation			
Percent of time ATCS outperforms TOD-230 timing plans	6.3%	0.0%	9.7%
Average Travel Time for TOD-230 timing plans	2.66	2.82	3.23
Average Travel Time for ATCS	2.82	3.30	3.72
Average Travel Time savings with ATCS	-0.17	-0.48	-0.49
TOD-230 vs. TOD Optimized Operation			
Percent of time TOD-Optimized outperforms TOD-230 timing plans	31.3%	27.1%	5.6%
Average Travel Time for TOD-230 timing plans (min)	2.66	2.82	3.23
Average Travel Time for TOD-Optimized timing plans (min)	2.86	2.99	4.34
Average Travel Time savings with TOD-Optimized timing plans (min/veh)	-0.21	-0.17	-1.11

D. Segment: Coastline Drive Lifeguard Station and Big Rock Drive

	AM PEAK	MIDDAY	PM PEAK
ATCS vs. TOD-230 Operation			
Percent of time ATCS outperforms TOD-230 timing plans	41.7%	27.1%	25.0%
Average Travel Time for TOD-230 timing plans	3.08	2.97	2.75
Average Travel Time for ATCS	3.19	3.03	2.82
Average Travel Time savings with ATCS	-0.11	-0.06	-0.07
TOD-230 vs. TOD Optimized Operation			
Percent of time TOD-Optimized outperforms TOD-230 timing plans	18.8%	35.4%	8.3%
Average Travel Time for TOD-230 timing plans (min)	3.08	2.97	2.75
Average Travel Time for TOD-Optimized timing plans (min)	3.35	3.01	2.97
Average Travel Time savings with TOD-Optimized timing plans (min/veh)	-0.27	-0.04	-0.22

Table 5.2 SB PCH Travel Times by Segment—Bluetooth Data

A. Segment: Big Rock Drive and Coastline Drive Lifeguard Drive

	AM PEAK	MIDDAY	PM PEAK
	6–10 AM	**10 AM – 2 PM**	**2–8 PM**
ATCS vs. TOD-230 Operation			
Percent of time ATCS outperforms TOD-230 timing plans	83.3%	56.3%	45.8%
Average Travel Time for TOD-230 timing plans	5.88	2.86	2.78
Average Travel Time for ATCS	3.84	2.84	2.78
Average Travel Time savings with ATCS	2.05	0.03	-0.01
TOD-230 vs. TOD Optimized Operation			
Percent of time TOD-Optimized outperforms TOD-230 timing plans	62.5%	35.4%	9.7%
Average Travel Time for TOD-230 timing plans (min)	5.88	2.86	2.78
Average Travel Time for TOD-Optimized timing plans (min)	5.46	2.92	2.93
Average Travel Time savings with TOD-Optimized timing plans (min/veh)	0.43	-0.06	-0.16

B. Segment: Coastline Drive Lifeguard Station and Shore Drive

	AM PEAK	MIDDAY	PM PEAK
ATCS vs. TOD-230 Operation			
Percent of time ATCS outperforms TOD-230 timing plans	93.8%	25.0%	59.7%
Average Travel Time for TOD-230 timing plans	3.81	2.92	2.83
Average Travel Time for ATCS	2.97	3.02	2.79
Average Travel Time savings with ATCS	0.83	-0.10	0.04
TOD-230 vs. TOD Optimized Operation			
Percent of time TOD-Optimized outperforms TOD-230 timing plans	62.5%	18.8%	6.9%
Average Travel Time for TOD-230 timing plans (min)	3.81	2.92	2.83
Average Travel Time for TOD-Optimized timing plans (min)	3.70	3.10	3.19
Average Travel Time savings with TOD-Optimized timing plans (min/veh)	0.11	-0.18	-0.36

C. Segment: Shore Drive and Haul Road

	AM PEAK	MIDDAY	PM PEAK
ATCS vs. TOD-230 Operation			
Percent of time ATCS outperforms TOD-230 timing plans	58.3%	50.0%	69.4%
Average Travel Time for TOD-230 timing plans	1.21	1.20	1.31
Average Travel Time for ATCS	1.17	1.17	1.20
Average Travel Time savings with ATCS	0.04	0.03	0.11
TOD-230 vs. TOD Optimized Operation			
Percent of time TOD-Optimized outperforms TOD-230 timing plans	70.8%	41.7%	41.7%
Average Travel Time for TOD-230 timing plans (min)	1.21	1.20	1.31
Average Travel Time for TOD-Optimized timing plans (min)	1.14	1.23	1.36
Average Travel Time savings with TOD-Optimized timing plans (min/veh)	0.07	-0.03	-0.05

5.2 Comparison of Bluetooth and Probe Vehicle Travel Times

This section describes the findings from the comparison of travel times collected with probe vehicles in June 2012, and the travel times collected using Bluetooth detectors in late March 2013.

Table 5.3 summarizes the corridor-wide travel times for each direction and time of day, based on the trajectories recorded using iPhone-equipped probe vehicles. As expected, the mean and standard deviation of the travel times is generally lower in each direction during its off-peak time—the AM Peak for the northbound direction and the PM Peak for the southbound direction. However, travel times in different directions are not directly comparable, as the northbound and southbound corridors are of different length. The number of trajectories collected for June 12 is approximately half of that for other days, due to only one driver being available for data collection on that date. Data was also collected on the morning of June 7, but this data was unusable due to configuration problems affecting both GPS devices.

Table 5.3 Probe Vehicle Travel Time Runs June 2012

			GPS Runs	Mean Travel Time (sec)	Standard Deviation (sec)	Minimum Travel Time (sec)	Maximum Travel Time (sec)
Thurs Jun 7	PM	North	15	970.13	262.50	617	1365
		South	19	437.03	63.22	357	562
Tue Jun 12	AM	North	8	696.66	43.92	632	773
		South	8	756.74	377.25	388	1410
Wednesday June 13	AM	North	16	677.89	98.17	498	865
		South	17	518.87	144.25	352	765
	PM	North	19	882.44	232.14	619	1292
		South	19	412.72	40.99	341	525
Thursday June 14	AM	North	17	693.11	62.00	592	819
		South	16	491.20	88.01	373	660
	PM	North	19	1044.03	344.79	611	1784
		South	18	447.38	28.23	396	489
All days combined	AM	North	41	687.87	74.54	498	865
		South	41	554.49	215.54	352	1410
	PM	North	53	965.19	288.34	611	1784
		South	56	432.11	48.15	341	562

The travel times in each direction of PCH obtained from each data collection effort are shown in Figures 5.5 and 5.6. Both the median and inter-quartile range (IQR) are shown, to give a sense of the spread of the data. The green boxes indicate when the difference between the probe vehicle travel times and the Bluetooth travel times is statistically significant on a 5% level, according to a Wilcoxon Rank-Sum test. From these Figures and the statistical test results, we conclude that the two travel time estimation methods yield comparable results, and that the travel times between the two data collection periods are not significantly different.

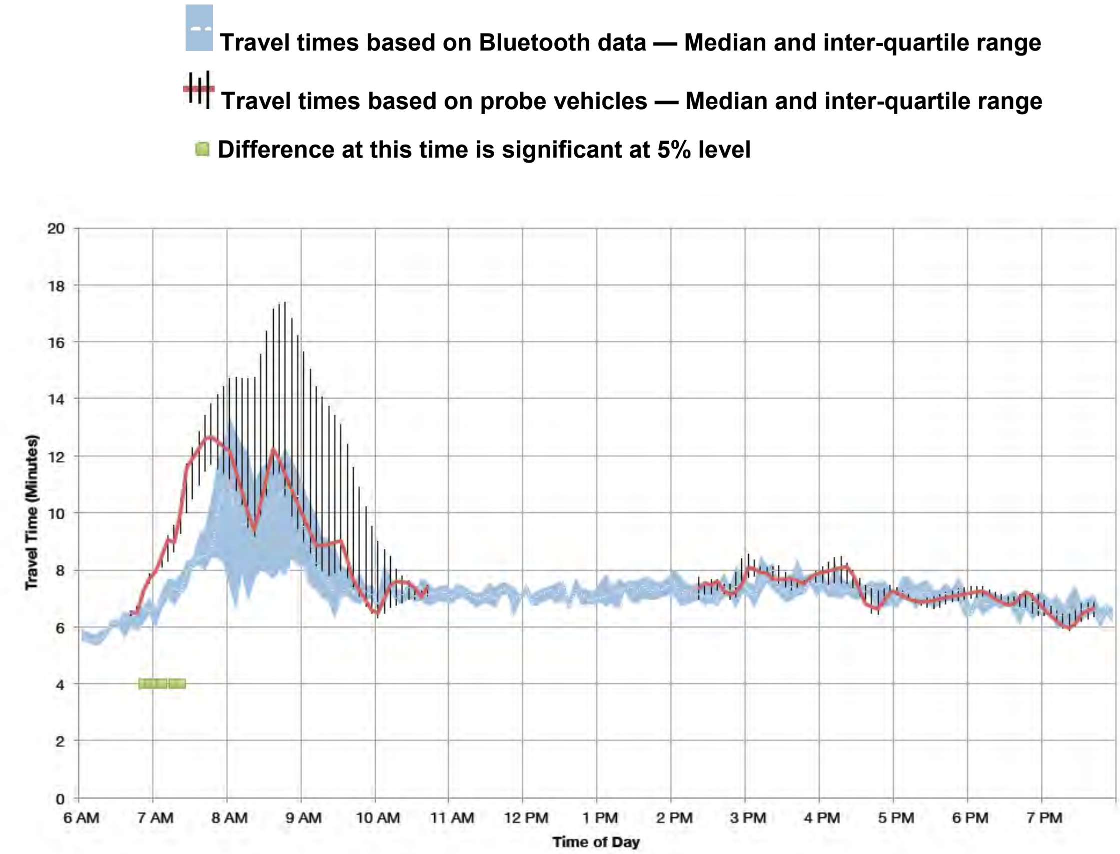


Figure 5.5 SB PCH Travel Times --Probe Vehicles vs. Bluetooth

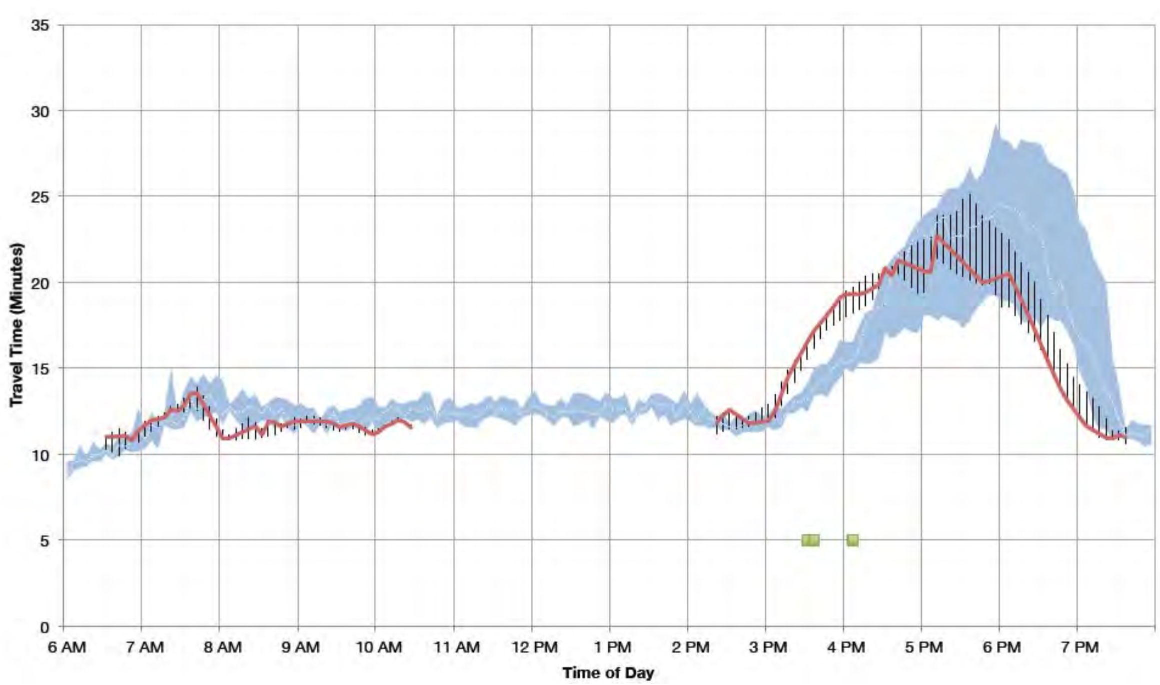


Figure 5.6 NB PCH Travel Times --Probe Vehicles vs. Bluetooth

Delays at Traffic Signals

We calculated the travel times on each link from the individual vehicle trajectories. Next, we calculated the delays on each approach (link) as the difference between the actual travel time and the free-flow travel time. The free-flow travel time was calculated from the link distance and the free-flow speed of 40 mph. The assumed free-flow speed of 40 mph may be slightly conservative given the 45 mph speed limit.

The calculated delays for each probe vehicle trip is given in Table 5.4 for a typical afternoon (Wednesday June 13, 2012) in the northbound direction of PCH. The spreading of congestion upstream of Chautauqua Blvd is clearly shown, as major delays begin occurring upstream of Beach House Way after 2:49 PM and upstream of California Incline after 3:44 PM. Similarly, the delays have largely diminished upstream of California Incline by 6:35 PM, followed by Beach House Way at 7:06 PM.

Note that based on the way the delays are calculated in Table 5.4, the delay that occurs on a particular link is attributed to the signal at the downstream end of that link. Thus, if queues on the northbound direction of PCH from Chautauqua Blvd spill back beyond Beach House Way, the delays measured on the link between California Incline and Beach House Way are attributed to the Beach House Way rather than Chautauqua Blvd.

Next we calculated the delays between major intersections only (Figures 5.7 and 5.8). Specifically, the minor cross- streets at Coastline Dr, Porto Marina Way, Bay Club Dr, and Beach House Way are disregarded, so that the delays are measured only on the following "major" PCH links:

In the northbound direction:

1. Before California Incline
2. Between California Incline and Chautauqua Blvd
3. Between Chautauqua Blvd and Temescal Canyon Rd
4. Between Temescal Canyon Rd and Sunset Blvd
5. Between Sunset Blvd and Topanga Canyon Blvd

In the southbound direction:

1. Before Topanga Canyon Blvd
2. Between Topanga Canyon Blvd and Sunset Blvd
3. Between Sunset Blvd and Temescal Canyon Rd
4. Between Temescal Canyon Rd and Chautauqua Blvd
5. Between Chautauqua Blvd and California Incline

The scales are kept consistent between each of the two Figures, to facilitate comparisons between the magnitude of northbound and southbound delays. By defining the links in this way, we obtain a more accurate picture of the delays created by each signal, with the assumption that the minor intersections we have disregarded do not contribute significantly to delays on PCH. In Figures 5.7 and 5.8 for example, all delays on the link between California Incline and Beach House Way in the northbound direction will be ascribed to the signal at Chautauqua Blvd, and not to the signal at Beach House Way. If the northbound queue at Chautauqua Blvd spills back past California Incline, however, the delays created by this spillover will still be attributed to California Incline instead of to Chautauqua Blvd.

In Figures 5.7 and 5.8, delays on some links are shown to increase to a certain threshold, and then remain approximately steady for a period of time before receding. This can be explained by the queues spilling back to the next link upstream, such that the maximum amount of queue storage for the subject link has been reached and is maintained throughout the period of steady delays. For example, in Figure 5.7, the delay on the "Before Chautauqua Blvd" link increases between 2:30 and 3:30 pm as the queue grows on this link. At 3:30 pm, the delay for "Before Chautauqua Blvd" becomes approximately constant until about 6:45 pm, indicating that the link is entirely queued and additional delays are being absorbed by the next upstream link instead: the "Before California Incline" link. Finally, at about 6:45 pm, the delays at California Incline have largely diminished and the delays on the "Before Chautauqua Blvd" link begin to shrink as well, indicating that the queue has receded downstream of California Incline.

The same kind of behavior is observed in the northbound direction upstream of Sunset Blvd. The bottleneck at Sunset Blvd is seen to activate around 4:30 pm in Figure 5.7, and delays start appearing on the "Before Temescal Canyon Rd" link soon afterward— at which point the vertical separation between the "Before Temescal Canyon Rd" link and the "Before Sunset Blvd" link becomes roughly constant due to the queue storage limit being reached on the "Before Sunset Blvd" link.

In Figure 5.8, we observe a similar effect in the southbound direction during the AM Peak. Here, delays on the "Before Topanga Canyon Blvd" link and the "Before Sunset Blvd" link start growing simultaneously, as both signals become active bottlenecks. However, the queue from Sunset Blvd spills back to Topanga Canyon Blvd at approximately 7:30 am as evidenced by the constant delay on the "Before Sunset Blvd" link between then and about 9 am.

Table 5.4 NB PCH Signal Delay -- Probe Vehicles

	Time of Day	2:21 PM	2:34 PM	2:49 PM	3:07 PM	3:19 PM	3:44 PM	4:00 PM
	Total Trip Time (sec)	543	527	605	611	677	882	898
Delays before each signal (sec)	California Incline	0	0	27	13	0	49	93
	Beach House	2	6	13	15	48	173	112
	Chautauqua/Entrada	37	26	35	40	113	110	92
	Temescal Canyon Bl	0	0	0	0	0	0	0
	Bay Club Dr	0	0	0	0	0	0	5
	Sunset Bl	41	30	45	62	62	71	34
	Porto Marina	3	2	2	0	0	0	22
	Coastline Dr	0	0	0	0	0	0	19
	Topanga Canyon Bl	0	0	0	0	0	0	22

	Time of Day	4:23 PM	4:34 PM	5:07 PM	5:24 PM	5:54 PM	6:00 PM	6:35 PM
	Total Trip Time (sec)	951	1144	1125	1077	1089	921	845
Delays before each signal (sec)	California Incline	210	220	201	196	138	77	0
	Beach House	133	163	117	51	136	38	64
	Chautauqua/Entrada	93	79	88	69	59	74	97
	Temescal Canyon Bl	0	0	24	75	37	109	43
	Bay Club Dr	0	66	55	76	107	46	36
	Sunset Bl	0	43	41	41	19	55	53
	Porto Marina	15	40	14	6	0	7	14
	Coastline Dr	1	0	27	0	45	0	1
	Topanga Canyon Bl	10	17	35	47	32	5	33

	Time of Day	6:49 PM	7:06 PM	7:19 PM	7:34 PM	7:45 PM	Mean	St. Dev.
	Total Trip Time (sec)	708	610	543	578	542	783	222
Delays before each signal (sec)	California Incline	0	0	0	47	0	67	83
	Beach House	69	18	1	0	0	61	60
	Chautauqua/Entrada	109	5	43	49	63	67	31
	Temescal Canyon Bl	0	0	0	0	32	17	31
	Bay Club Dr	7	0	0	0	0	21	33
	Sunset Bl	61	36	26	0	0	38	21
	Porto Marina	0	20	2	0	0	8	11
	Coastline Dr	0	6	0	0	0	5	12
	Topanga Canyon Bl	0	34	0	0	0	12	16

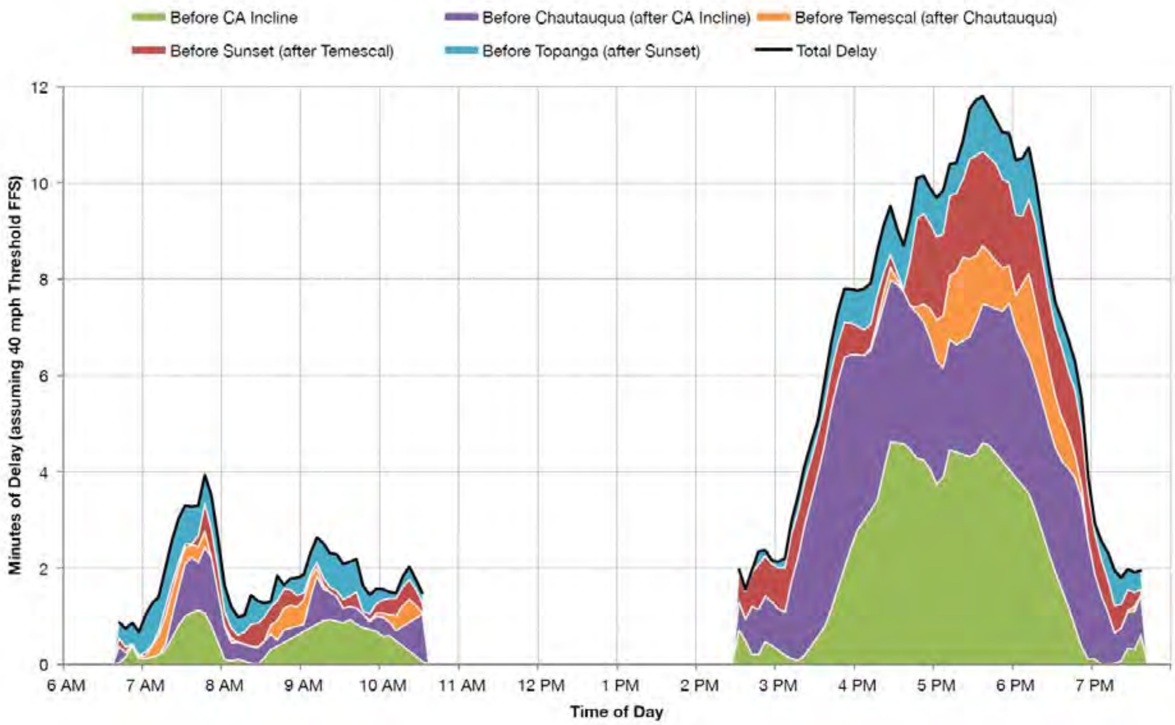


Figure 5.7 Delay at Major Intersections NB PCH-Probe Vehicle Data

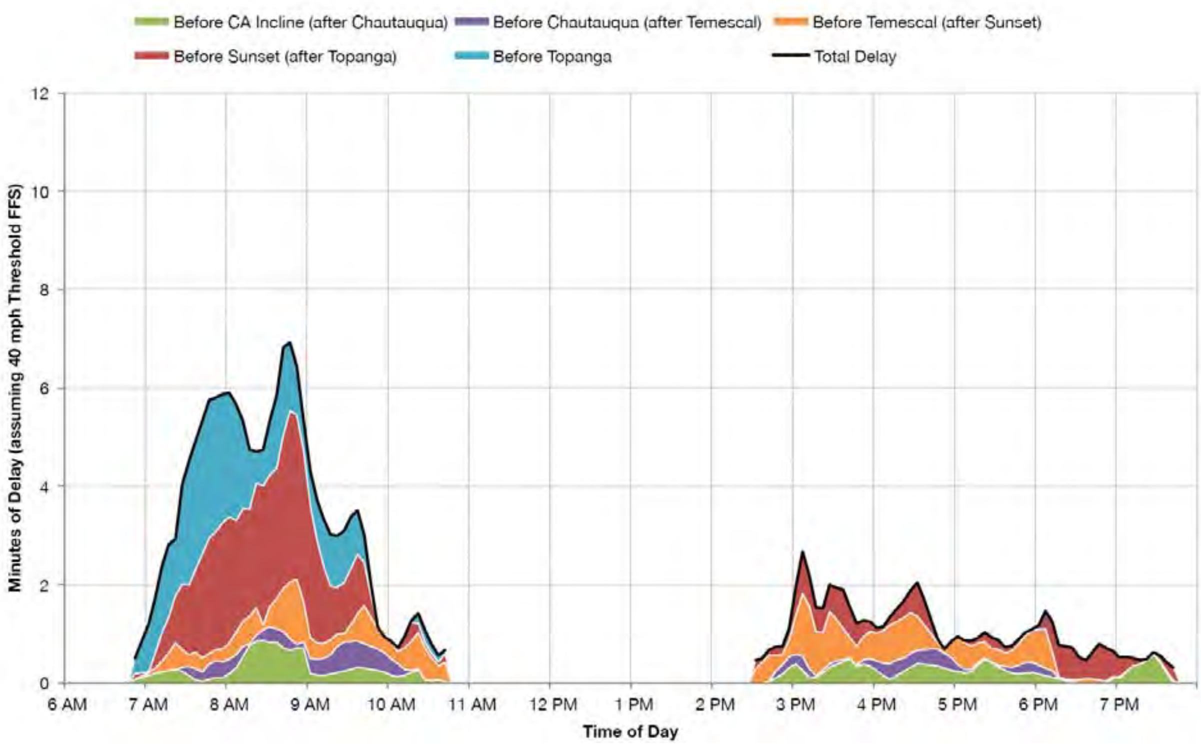


Figure 5.8 Delay at Major Intersections--SB PCH-Probe Vehicle Data

5.3 Arterial Level of Service (LOS) Analysis

Using the travel time data from the Bluetooth detectors in 2013, we can evaluate the arterial Level of Service (arterial LOS) for PCH under each control strategy on all segments bounded by the Bluetooth units. We use travel time data to estimate average speeds on each segment, and use the criteria from the 2010 Highway Capacity Manual (HCM2010) [8] to assign a level of service based on the ratio of the segment's prevailing speed at a given time to the base free flow speed for that segment.

To calculate the base free flow speed for a given analysis segment, we aggregated the median (over 5-minute intervals) travel times for that segment across all 5-minute bins between 9 PM and 6 AM and across all days of Bluetooth travel time data. The overall median of this set was then used to represent the typical travel time on the segment for an unimpeded vehicle. All days of data were included in this calculation regardless of which control strategy was in place on a particular day, as the period between 9 PM and 6 AM was a "free" (i.e., uncoordinated actuated) operation period for all of the strategies (ATCS, TOD-230, and TOD-Optimized). The base free flow speed was then computed by dividing the segment length by the typical travel time of an unimpeded vehicle on the segment.

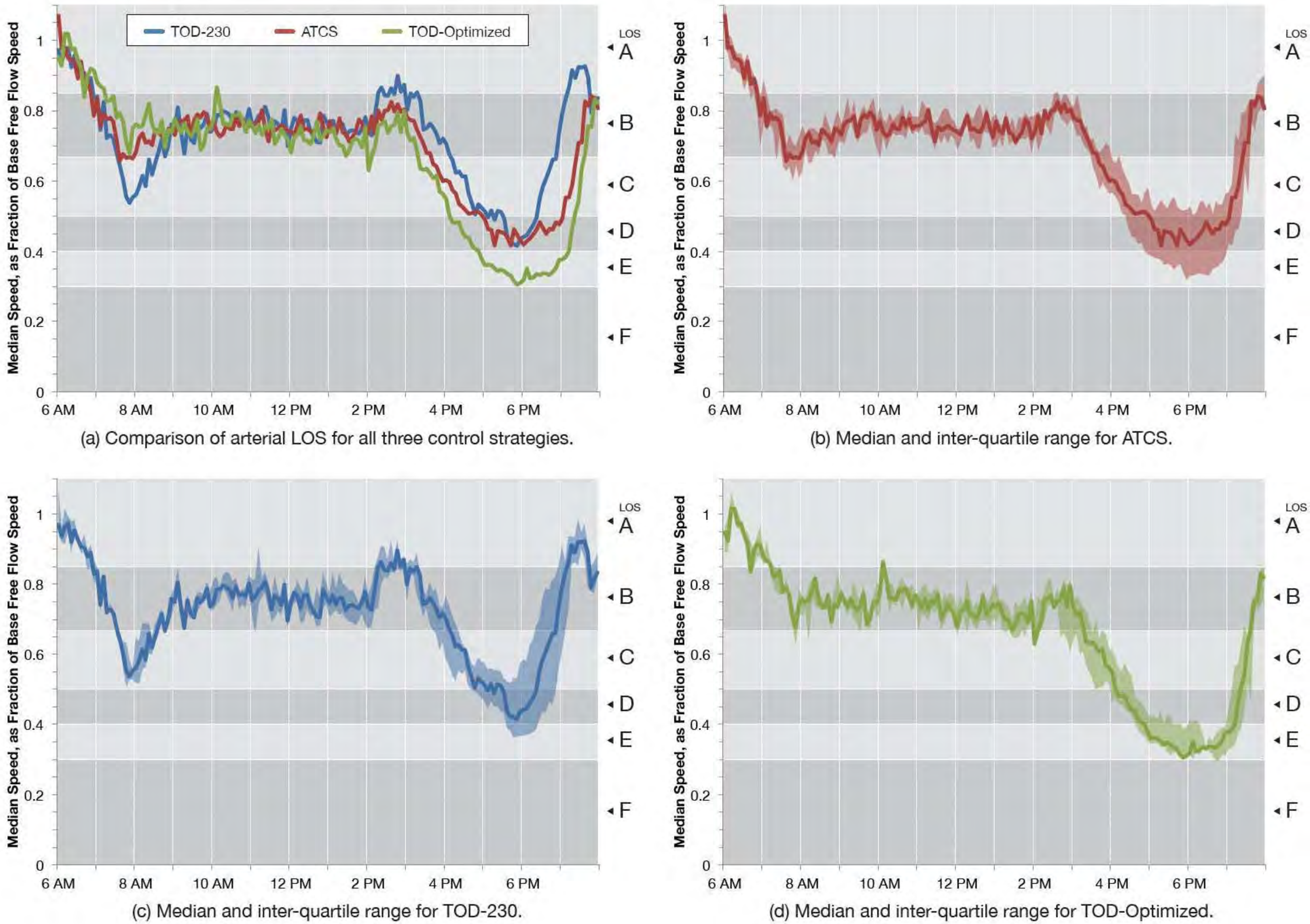
Figures 5.9 and 5.10 show the LOS estimates for the entire corridor. Part (a) of each figure can be used to compare the typical LOS resulting from each of the three control strategies, while Parts (b), (c), and (d) can be used to examine the typical spread of speeds (as a fraction of base free flow speed) at a particular time of day —and consequently the spread of LOS as well—for each of the three control strategies. Because travel times are not normally distributed (due to the presence of a lower bound but no upper bound), we would not expect the speeds to be normally distributed either, and have therefore used median and interquartile range to describe the location and scale parameters of the data in Figures 5.9 and 5.10 rather than the more commonly used mean and standard deviation.

We also determined the LOS for the following component segments, on both directions of the PCH:

- Between Big Rock Drive and Coastline Drive
- Between Coastline Drive and Shore Drive
- Between Shore Drive and Haul Road
- Between Haul Road and McClure Tunnel (northbound only)

Figure 5.3 is a sample LOS plot for a segment. Note that on the southbound direction, the portion of PCH between Haul Road and McClure Tunnel is excluded from our analysis due to the tendency of traffic on eastbound Interstate 10 to spill back beyond McClure Tunnel during the PM Peak, inflating our travel time measurements for the southbound side on this link and yielding misleading results in terms of control delay and performance on PCH

To supplement the LOS estimates generated in Figures 5.9 and 5.10 using the Bluetooth Data, we evaluated the arterial LOS for individual links between each signalized intersection on PCH using the probe vehicle data from June 2012. Although the higher-resolution nature of this data enables us to evaluate the performance of each link on PCH rather than on larger segments spanning several intersections at a time, the probe vehicle data set is limited with respect to sample size. More precisely, there are only three days of data for the AM and PM Peak periods, and the sampling frequency on each day is approximately once every 15-20 minutes, rather than several times every 5 minutes as was the case for the Bluetooth data. A sample for link LOS results on PCH are given in Figure 5.11 for the NB and SB links on Sunset Blvd.



(a) Comparison of arterial LOS for all three control strategies.

(b) Median and inter-quartile range for ATCS.

(c) Median and inter-quartile range for TOD-230.

(d) Median and inter-quartile range for TOD-Optimized.

Figure 5.9 Arterial LOS --NB PCH Corridor --McLure Tunnel and Big Rock Drive, Bluetooth Data

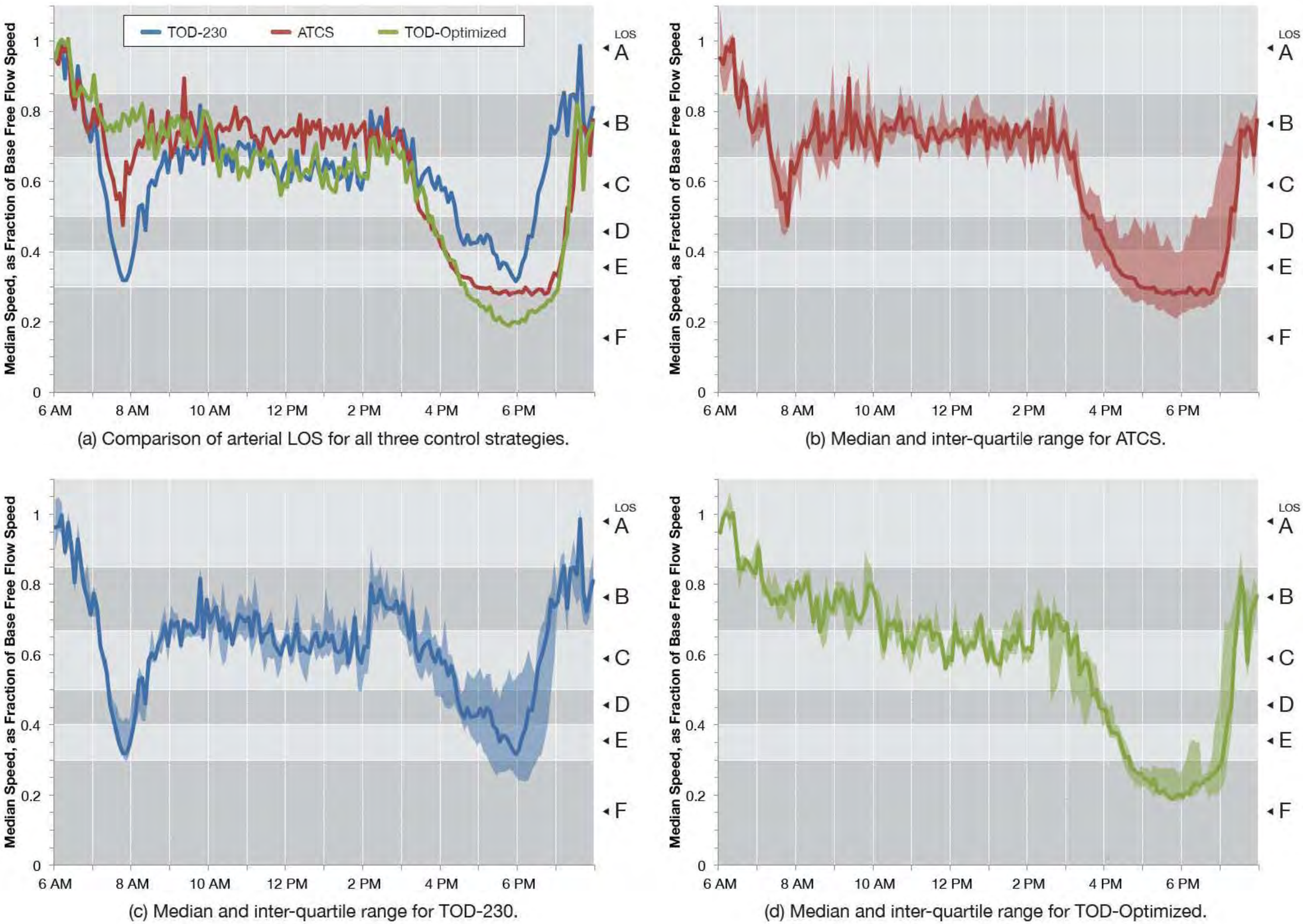
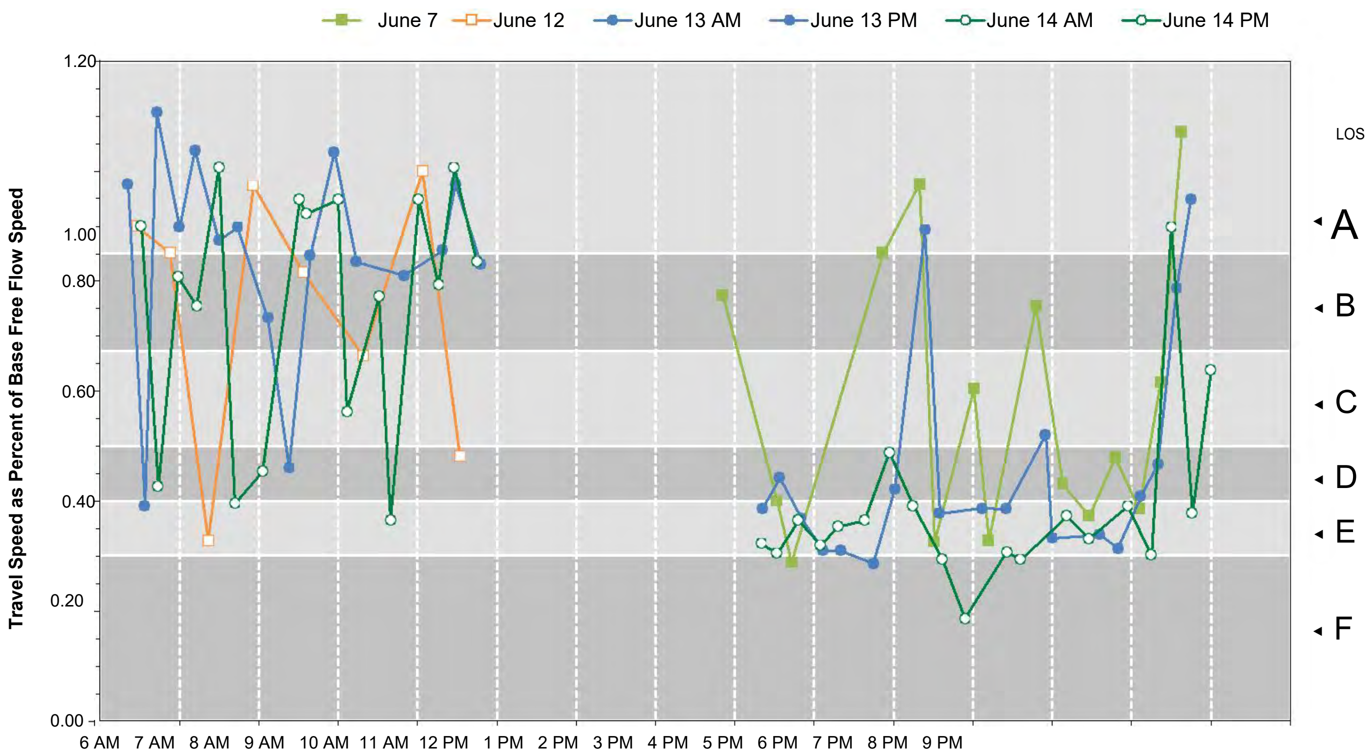
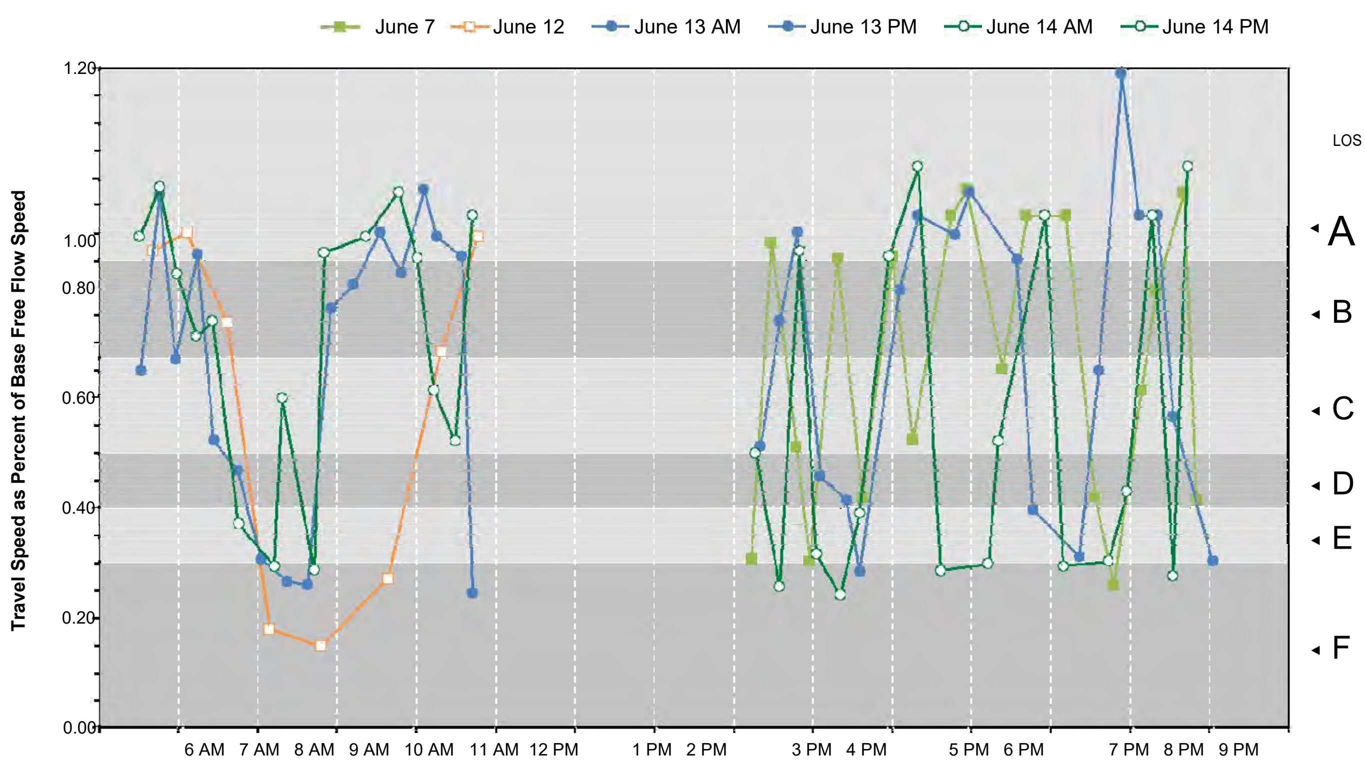


Figure 5.10 Arterial LOS –SB PCH Corridor --Big Rock Drive and Haul Road, Bluetooth Data



(a) The PCH link approaching the intersection in the northbound direction.



(a) The PCH link approaching the intersection in the southbound direction.

Figure 5.11 Arterial LOS for PCH at Sunset Blvd, Probe Vehicle Data

5.4 Operational Issues of ATCS Control[1]

In the recent NCHRP report on the state of practice for adaptive systems worldwide, it was reported that 33% of ATCS users have found adaptive systems to be counterproductive in oversaturated traffic conditions [1]. This section identifies and evaluates the impact of operational issues that may negatively affect the performance of the ATCS system on PCH when oversaturated conditions arise on at least one intersection approach. These issues include:

1. Inefficiently setting splits at the active bottleneck intersection.
2. Inefficiently setting splits and offsets at intersections upstream of the bottleneck.
3. Improperly setting offsets at intersections downstream of the bottleneck.

The analysis was performed using field data collected in March through June 2013. The analysis focused on the PM Peak (2–8 PM) for typical weekdays (Tuesdays, Wednesdays, and Thursdays) between March 5 and June 13, 2013. Data were excluded in the cases of holidays and the weekdays adjacent to them, major traffic incidents, and data quality or availability issues (e.g., days when Bluetooth units did not collect data due to power failure, or days when volume data were not collected due to field communication link failure). Additional data from June 2012 were used to supplement our analysis.

5.4.1 Inefficient splits at the Bottleneck Location

In the afternoon, the primary bottleneck or critical intersection is Sunset Blvd, with northbound traffic on PCH being the heavier oversaturated critical direction and southbound PCH traffic being the lighter non-critical direction.

One feature of adaptive control systems is their ability to allocate green time proportionally to each approach based on perceived demand obtained from detector readings. During oversaturated conditions, however, the ATCS may incorrectly believe that the queues are still clearing on all approaches each cycle, and detector readings may suggest that green time utilization is equal on all approaches when in fact it is not. This can happen if large gaps arise in discharging queues as a result of (1) heavy vehicles and buses, (2) cars caught behind bicyclists and drivers making parallel parking maneuvers, and (3) the split of mainline lanes of traffic into additional lanes (turn pockets, auxiliary lanes, etc.) just upstream of the intersection. Longer loops at the stop bar can address these issues to an extent, but may be impractical as extending the loop length may also inhibit the ability to accurately measure occupancy.

In our analysis, we found that ATCS frequently gave additional time to the cross-street at Sunset Blvd even after the startup queue cleared during periods when persistent queues of 100 vehicles or more were present on the northbound PCH approach. This may be because the adaptive system believed that the queues were clearing on both approaches each cycle, based on the gap characteristics of PCH traffic and the widening of PCH from two lanes to five (two turn pockets plus an auxiliary through lane) just before the intersection. As a consequence, travelers on the mainline incurred large delays as crucial green time was allocated instead to the side street at this critical bottleneck location.

Using video footage of the queues on Sunset Blvd for June 7, 12, 13, and 14, 2012, we investigated the relationship between occupancies and residual queues on this approach. In the four days of queue length footage, there were no cycles out of the 516 total cycles where a residual queue of more than 7 or 8 vehicles was observed behind the stop bar in any lane, and only three cycles where a short residual queue of 7-8 vehicles in one lane was seen. The 15-minute occupancy data for periods with and without these short residual queues are plotted in Figure 5.12, which reveals that 40% may be a suitable conservative threshold for identifying when residual queues start to form.

---

[1] Campbell, R., and A. Skabardonis, "Issues that Reduce the Performance of Adaptive Control Systems under Oversaturated Conditions," paper 14-5438, 93rd TRB Annual Meeting, Washington DC, January 2014 (forthcoming Transportation Research Record, Journal of the Transportation Research Board)

Given the limited cases of queue formation in our data for Sunset Blvd, we also conducted a similar analysis of Topanga Canyon Blvd to supplement our results from Sunset Blvd. Unlike Sunset Blvd, however, long residual queues were frequently observed, and in this case we were able to partition the data from 1189 cycles into well-defined “No Queue” or “Residual Queue” categories. From the occupancy distributions for Topanga Canyon Blvd, we find that 100% of the measured occupancies exceeded 0.40 when a residual queue was present, whereas only 9 out of the 215 measured occupancies exceeded 0.40 when a residual queue was not present. This suggests a threshold occupancy of about 40%, just as we obtained from the data for Sunset Blvd. Because the average green time per cycle for Topanga Canyon Blvd throughout the day is 35 seconds, whereas the average at Sunset Blvd is 27 seconds, and given that cycle length is kept the same between the two intersections, it is reasonable to expect the threshold occupancy at Sunset Blvd to be higher than the 40% threshold observed at Topanga Canyon Blvd. Thus, using a threshold occupancy of 40% is a conservative criterion to identify oversaturated conditions at Sunset Blvd.

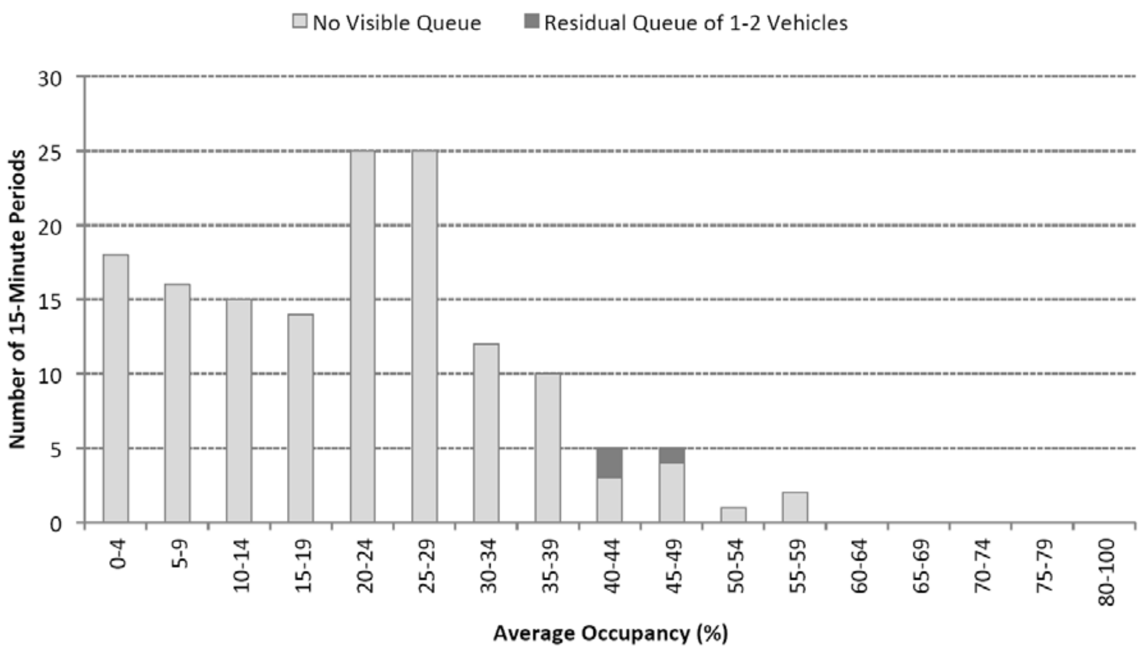


Figure 5.12 Detector Occupancy Distributions at Sunset Blvd—ATCS Operation

Next, we determined when the bottleneck is active on PCH at Sunset Blvd. We processed detector data from the northbound PCH detectors at Bay Club Drive—a low-volume cross-street 2850 feet south of Sunset Blvd. Assuming a jam density of 180 veh/mi/lane, we find that approximately 100 vehicles/lane can be stored between Bay Club Drive and Sunset Blvd. Assuming a saturation headway of 2 seconds, this is enough storage to provide queue discharge for approximately 200 seconds of green time. Given that the through phase for the northbound mainline at Sunset Blvd has a 99th percentile duration of 191 seconds, we can reasonably conclude that any northbound queue spillover from Sunset Blvd to Bay Club Drive implies that a persistent queue is present at Sunset Blvd (i.e., the northbound mainline queue at Sunset Blvd will not completely clear in the next cycle).

Using occupancy and flow time-series data, we can evaluate when the northbound queue at Sunset Blvd has spilled back to Bay Club Drive. Because the average red-time for northbound PCH at Bay Club Drive is less than 10% of the entire cycle, the detectors on the mainline behave similarly to freeway detectors, meaning that we can estimate when queue spillover occurs at Bay Club Drive by looking for large increases in occupancy without an accompanying increase in flow (see Figure 5.13, which suggests that queue spillback occurred at Bay Club Drive between 4 and 6:30 PM that day). During these periods, we reasonably conclude that the northbound approach at Sunset Blvd is oversaturated. From a data set of 16 days, we identified 9 with a well-defined period of oversaturation at Sunset Blvd based on occupancy and flow data at Bay Club Drive. After excluding 4 days for which the LA-ATCS splits data were not available, we were left with 5 days of usable data. When examining the real-time adaptive splits given to the Sunset Blvd side-street when the northbound bottleneck at Sunset Blvd was active on each of these days, we obtained an average phase duration of 30.2 seconds.

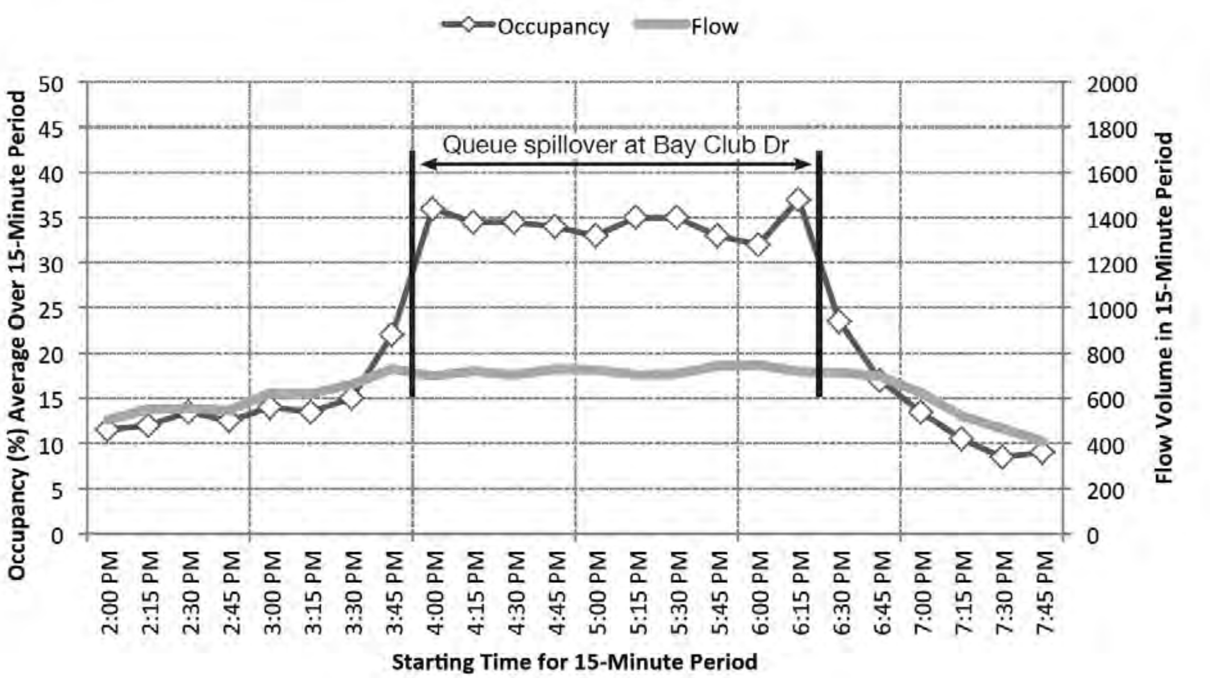


Figure 5.13 Bottleneck Identification —NB PCH at Sunset Blvd

Following the implementation of the fixed-time TOD-230 plan in May and June 2013, a maximum green time of 24 seconds was imposed on the Sunset Blvd side-street split. Also the cycle time was reduced from 240 to 230 seconds, such that the equivalent green time constraint under the original cycle length would have been a limit of approximately 25 seconds. Despite this reduction in effective green time by 16%, there was no increase in the occupancy measurements on the cross-street (Figure 5.14). In fact, the frequency of occupancies exceeding 40% dropped from 6.8% before the green time reduction to 4.5% after it. This indicates that the ATCS was giving more green time to the side-street than necessary to accommodate all its demand, meaning that it was inefficiently setting splits when considering that the mainline was experiencing long residual queues during these times. In fact, it is probable that even more green time could be reallocated from Sunset Blvd to PCH than the 5 seconds we experimented with before significant residual queues begin forming on the side-street.

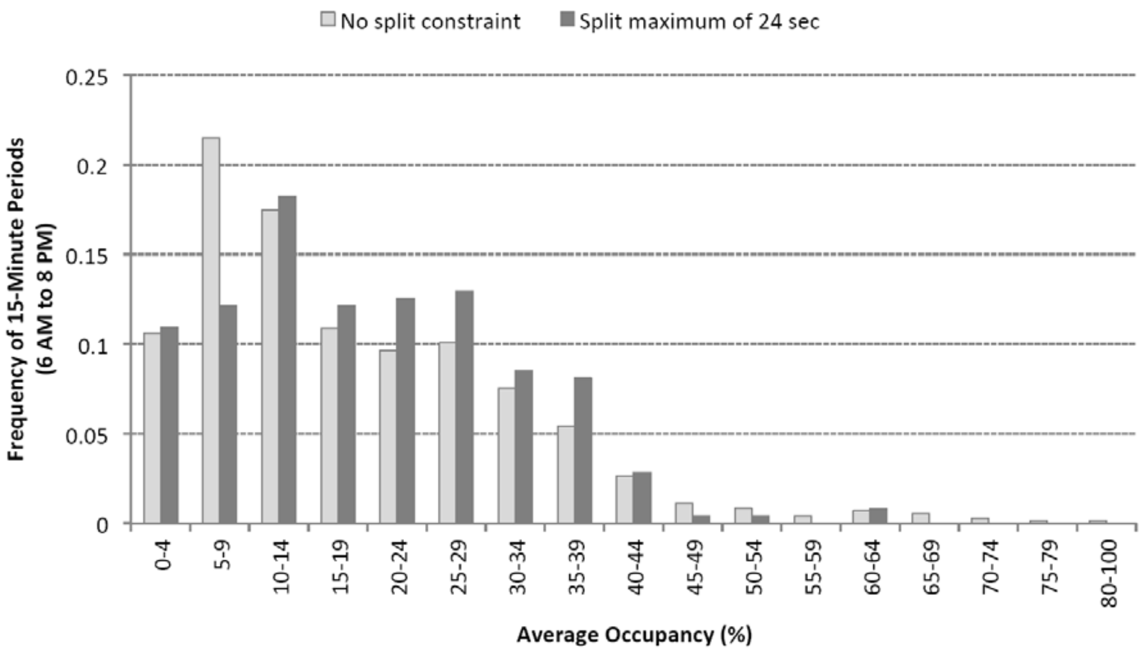


Figure 5.14 Detector Occupancy Distributions Sunset Blvd—ATCS vs. TOD-230 Operation

Comparing volume data for the days with and without the 24-second limit on the Sunset Blvd split, we find that there was no drop in demand for the days when the split was constrained, with a two-tailed Student's t-test yielding a p-value of 0.85. Thus, we can be confident that our results were not affected by a drop in traffic demand on the days with the 24-second limit.

These findings indicate that the adaptive system gives far more time than appropriate (30.2 seconds on average) to the Sunset Blvd side-street when the northbound bottleneck is active, whereas a split of less than 25 seconds was found to be sufficient to accommodate the side-street demands. Reducing the split for Sunset Blvd would increase capacity of the mainline at this critical location during those crucial periods when the northbound bottleneck is active. Even larger capacity increases could be achieved if the side-street split were reduced to the point where non-trivial residual queues began to form (i.e., queues longer than 1-2 vehicles), which may not be unreasonable given the presence of long queues on the PCH mainline.

5.4.2 Inefficient Splits & Offsets Upstream of the Bottleneck

When a corridor-oriented adaptive system does not have the ability to determine which intersection is the critical bottleneck location, it can result in sub-optimal allocation of green time at locations upstream of the bottleneck.

When the critical bottleneck becomes oversaturated, the mainline splits at each intersection upstream would ideally be adjusted to reflect the capacity constraint at the bottleneck, since any additional green time given to the mainline in the critical direction at these locations results in no travel time benefit to the extra vehicles served (assuming no major trip ends exist between the upstream intersection and the bottleneck). When an adaptive system adjusts splits without awareness of whether the critical approach is feeding an active bottleneck farther downstream, the result is an over-allocation of time to the critical direction when the extra time is of no benefit to the drivers heading toward the bottleneck. Without such a downward adjustment to the green time for the critical mainline direction, the bottleneck queue could eventually spill back past upstream intersections, further worsening delays as vehicles begin to block intersections and obstruct side-street traffic. Queue spillover at upstream intersections can also encourage the adaptive system to give more green time to the mainline at those locations (as vehicles sit on the stop bar detectors during the green phase, raising occupancy measurements for that approach and encouraging the algorithm to give even more green time to that approach), when such a decision would not increase the productivity of the oversaturated direction and would increase delays for all conflicting movements.

Instead, more efficient operation of the intersections upstream of the critical intersection would involve reducing the mainline splits for the critical direction to the minimum needed to avoid starving the bottleneck downstream, and giving the time balance to conflicting movements such as side streets or opposing left turns. The mainline offsets could also be adjusted to accommodate platoon progression in the opposite non-critical direction (i.e., heading away from the bottleneck), since the presence of a residual queue at the bottleneck implies that progression has already broken down for the peak direction. To maintain efficient operation, these split and offset adjustments at upstream intersections would be reversed once the residual queue at the critical intersection begins to dissipate.

To explore how the ATCS control responds at upstream intersections when the northbound bottleneck at Sunset Blvd becomes active, we examine the splits and offsets for northbound PCH at Temescal Canyon Rd, shown in Table 5.5. From this data, we find that the average northbound PCH split at Temescal Canyon Rd was 170 seconds when the bottleneck at Sunset Blvd was active, whereas it was only 163 seconds in the hour leading up to activation. In every case, cycle length remained unchanged at 240 seconds.

These findings indicate that 163 seconds of green time for northbound PCH at Temescal Canyon Rd is more than enough to avoid starving the downstream bottleneck at Sunset Blvd, and that the ATCS is inappropriately exceeding this threshold while the downstream bottleneck is active. This inefficient split for northbound PCH at Temescal Canyon Rd does not improve travel times on the northbound side, encourages queue spillover at additional intersections upstream of the bottleneck at Sunset Blvd, and adds to delays on conflicting movements that could instead be taking advantage of this extra time (e.g., southbound PCH left turns, side-street movements). In fact, given that 163 seconds was enough to allow a residual queue to develop at Sunset Blvd, the optimal amount of green time for the northbound movement at Temescal Canyon Rd would be even lower than this as long as the bottleneck at Sunset Blvd is active, assuming again that there are no major northbound trip ends between Temescal Canyon Rd and Sunset Blvd (and that no more time can be allocated to the critical direction at the Sunset Blvd bottleneck).

From Table 5.5, we also observe that the offsets remained unchanged throughout the entire PM Peak period in every case, regardless of whether the Sunset Blvd bottleneck was active or not. Based on this finding, we conclude that the adaptive system is not intelligently switching between an offset plan (i.e., a set of offsets for the signals on PCH) that favors the critical northbound direction when the bottleneck is inactive, and an offset plan that accommodates the opposite southbound direction when the northbound bottleneck is active and northbound progression has been disrupted by persistent queuing at Sunset Blvd. Instead, the adaptive algorithm is generating an optimal offset plan without properly taking into consideration whether the northbound bottleneck is active. This consideration is important, as the southbound direction should be given priority only if the bottleneck is active on the critical northbound side. The net effect is unnecessary delays to southbound traffic whenever the Sunset Blvd northbound approach is oversaturated, or unnecessary delays to northbound traffic if the LA-ATCS offsets are already being selected to favor the non-critical southbound direction (an unlikely, but possible, scenario).

Table 5.5 NB PCH Splits and Offsets at Temescal Canyon Rd

15-Min Period Start	3-13-13		3-20-13		3-26-13		5-1-13		5-2-13	
	Split	Offset	Split	Offset	Split	Offset	Split	Offset	Split	Offset
2:00 PM			180	160	156	160	157	230		
2:15 PM			168	160	162	160	171	230	172	230
2:30 PM			173	160	160	160	172	230	173	230
2:45 PM			171	160	157	160	168	230	172	230
3:00 PM	162	160	169	160	156	160	172	230	169	230
3:15 PM	149	160	144	160	156	160	171	230	175	230
3:30 PM	148	160	148	160	168	160	169	230	171	230
3:45 PM	154	160	166	160	159	160	170	230	162	230
4:00 PM	155	160	160	160	161	160	167	230	171	230
4:15 PM	156	160	174	160	163	160	168	230	174	230
4:30 PM	170	160	167	160	174	160	171	230	170	230
4:45 PM	166	160	170	160	171	160	173	230	174	230
5:00 PM	172	160	168	160	163	160	168	230	173	230
5:15 PM	160	160	160	160	175	160	171	230	174	230
5:30 PM	150	160	164	160	170	160	174	230	174	230
5:45 PM	166	160	166	160	176	160	173	230	173	230
6:00 PM	161	160	152	160	176	160	175	230	178	230
6:15 PM	172	160	165	160	184	160	171	230	172	230
6:30 PM	170	160	175	160	179	160	177	230	172	230
6:45 PM	170	160	166	160	180	160	184	230	171	230
7:00 PM	173	160	174	160	180	160	185	230	164	230
7:15 PM	179	160	169	160	190	160	179	230	171	230
Average for the period a persistent queue is present at the Sunset Bl bottleneck, and for the hour prior.										
Prior Hour	153	160	162	160	160	160	169	230	172	230
During	166	160	165	160	174	160	171	230	172	230

Shaded gray cells indicate the presence of a persistent queue on the northbound mainline at Sunset Blvd.

5.4.3 Improper Offsets Downstream of the Bottleneck

In addition to the problems we have already seen at the bottleneck intersection and those locations upstream of it, a third issue can occur if intersections downstream of the bottleneck are not operated properly, which can be another consequence of an ATCS not knowing which intersection is the critical one. Specifically, if the offsets are not set appropriately (i.e., they are set according to the adaptive system's default algorithm without consideration for the fact that one direction is handling departing traffic from a bottleneck intersection upstream), platoons departing from the critical bottleneck may incur additional, unnecessary delays as they get stopped at red signals farther downstream (i.e., the adaptive system does not give priority to the departing bottleneck traffic when computing offsets). Even more critical, though, is the possibility of a shock wave from such an interruption propagating back to the critical intersection while its green through phase is still active, thereby reducing the capacity of the bottleneck by preventing vehicles from proceeding through the intersection (despite the green signal) until the shock wave passes.

We examined the ATCS offsets for Sunset Blvd and the next three signals farther downstream. Although the offsets at Porto Marina Way, Coastline Dr, and Topanga Canyon Blvd were generally consistent across all days of data, the offset at Sunset was found to alternate between one of two vastly different values as shown in Figures 5.15(a) and 5.15(b). For the offsets plotted in Figure 5.15(a), which occurred 57% of the time, the northbound platoon from Sunset gets disrupted by the red signal at each downstream intersection, adding delay to most vehicles in the northbound direction. The shaded portion of Figure 5.15(a) indicates the amount of delay imposed on all but the first 26 seconds of the northbound platoon (i.e., all platoon vehicles in the shaded region and to the right of it are delayed) departing Sunset Blvd due to inefficient offset selection at the downstream intersections. The spacing of the intersections in Figure 5 is based on free flow speeds of 45 mph, which reflect the mainline speed limit.

The offset pattern of Figure 5.15(a) may also reduce the capacity of the northbound bottleneck at Sunset Blvd. With these offsets, the northbound green phase at Porto Marina Way terminates 120 seconds before it does at Sunset Blvd. Given a spacing of 1550 feet between Porto Marina Way and Sunset Blvd, if the reverse-propagating shock wave were to move at a speed of 9 mph or faster, it would reach Sunset Blvd while the through phase is still active. The result would be a reduction in bottleneck capacity, as northbound vehicles would be prevented from passing through the intersection until the shock passes.

Figure 5.15(b) illustrates the alternate offset pattern selected by the adaptive system 43% of the time, which is unchanged from Figure 5.15(a) aside from the significantly different offset at Sunset Blvd. Note that with the offsets of Figure 5.15(b), there are still large delays due to the poor offset choice at Topanga Canyon Blvd, but there is no longer a risk of shock wave propagation from Porto Marina Way back to Sunset Blvd (and consequently no risk of capacity reduction at the critical intersection).

Figure 5.15(c) shows the offsets implemented as part of the optimized fixed-time plans (TOD-230 and TOD-Optimized for PCH. The new offsets we used for the experiment were designed to minimize disruption of the north bound platoon departing Sunset Blvd, as shown in Figure 5c. Comparisons of Bluetooth data show that the new offsets improved travel times by 8.2 sec/veh in the northbound direction (an improvement of 2.4 %) and 4.0 sec/veh south bound (also an improvement of 2.4%). A Wilcoxon Rank-Sum test indicated that the improvement in travel times was significant at a 5% level in the northbound direction 21% of the time (using 5-minute bins between 2 PM and 8 PM), and 18% of the time in the southbound direction.

In comparing volume data for the days with and without the new offsets shown in Figure 5c, we find that there was no statistically significant change in demand between the two periods. A two-tailed Student's t-test yielded a p-value of 0.506 for the volumes past Topanga Canyon Blvd in the northbound direction and a p-value of 0.997 in the southbound direction. Thus, we can be confident that our travel time improvements were not the result of reductions in traffic volumes on PCH.

These findings indicate that although the offsets identified by the ATCS may be local optima, engineering judgment should be used to check that they do not have any obvious issues related to progression (particularly in the critical direction) or shock wave propagation to the critical bottleneck. In the case of PCH, the ATCS offsets resulted in unnecessary delays to traffic in both directions, and put the critical bottleneck at risk of being blocked by a reverse-propagating shock wave from an inappropriately timed red signal 1550 feet downstream. A simple alternative offset pattern (as shown in Figure 5.15(c)) reduced average travel times in both directions, and minimized the risk of shock waves from downstream intersections affecting the capacity of the bottleneck.

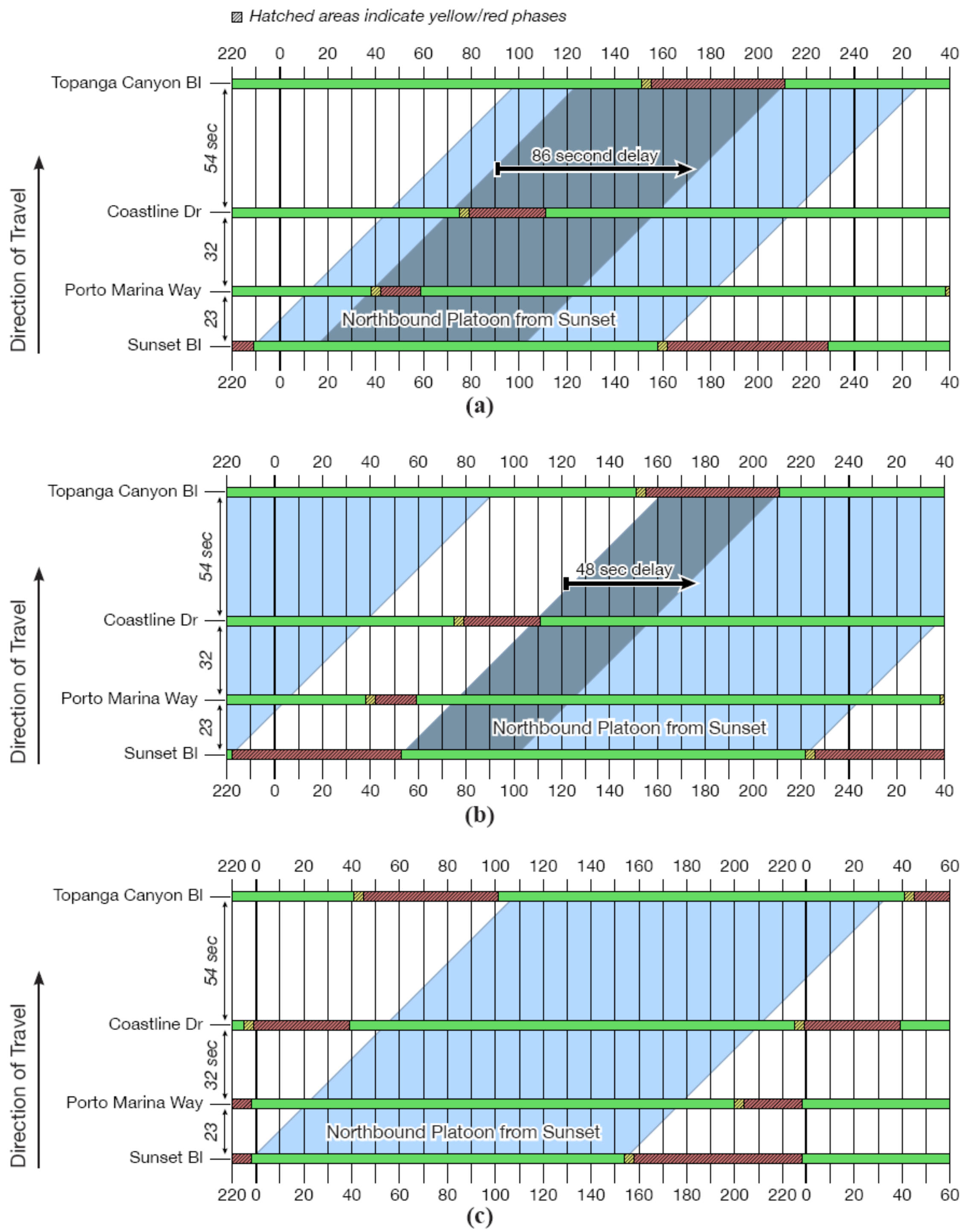


Figure 5.15 NB PCH Platoon Progression Downstream of Sunset Blvd

CHAPTER 6

CONCLUSIONS

6.1 Summary of the Study Findings

The objective of the study described in this report is to field test of an adaptive signal control strategy to determine its effectiveness under real-world operating conditions, and its strengths and weaknesses against conventional time-of day signal timing plans.

The test site is a section of the Pacific Coast Highway (PCH) in Los Angeles. It begins at the connection to Interstate 10 in Santa Monica and ends at the Malibu city limit, with Topanga Canyon Blvd and California Incline serving as its boundary intersections. There are nine signalized intersections on the study segment, all of which are controlled by the ATCS adaptive signal control system. Data on traffic volumes and signal settings were collected from the ATSAC surveillance system. Travel time data along the arterial were collected with instrumented vehicles and Bluetooth sensors. Queue length data were extracted from video recordings.

The performance of the ATCS system was compared against optimized fixed-time time-of-day plans. Two sets of timing plans were developed for the am, midday and pm peak period: the TOD-230 plan used a fixed cycle length of 230 seconds during the peak periods, and the TOD-Optimized plan used a fully optimized and unconstrained cycle length (typically 170 seconds) at all times of the day. The TOD-230 plans were developed to facilitate comparisons between ATCS and the TOD operation, because a cycle length constraint of 230 seconds was enforced on ATCS during the AM and PM peak periods. The timing plans were tested through the AIMSUN microscopic simulation model. Adjustments to the optimized settings were made as appropriate based on the simulation results to correct any operational issues prior to the field implementation. The optimized timing plans were implemented in the field, and adjustments were made as appropriate based on field observations prior to the collection of the traffic performance data.

The analysis of the performance data (travel times, queue lengths) under ATCS control shows:

- Long delays are observed at the intersections of Chautauqua Blvd, and Sunset Blvd in the pm peak in the northbound direction, and Topanga Blvd and Sunset Blvd in the am peak in the southbound direction. Often queues spillback at the upstream intersections for most of the peak periods. The level and variability of delays at these locations significantly affect the performance of control strategies tested.
- ATCS often makes inefficient choices regarding splits and offsets. At the critical intersection (system bottleneck) ATCS was giving the undersaturated side street at least 20% more green time than necessary at the critical intersection during periods when the mainline bottleneck was active. Also at the first major intersection upstream of the bottleneck, we found that at 7 seconds per cycle could be reallocated from northbound PCH to conflicting movements to reduce intersection delays without negatively affecting traffic in the critical direction. Finally, at the intersections downstream of the bottleneck, the ATCS selected offsets worsened travel times in both mainline directions by 2.4% and had the potential to reduce the capacity of the critical bottleneck as well.

The comparison of the performance of the control strategies tested is summarized in Tables 6.1 and 6.2. Table 6.1 shows the ranking of control strategies (1 being the best strategy) based on travel time data on the PCH mainline. The ranking is based on the absolute differences in the travel times, and in several cases these differences are small, as was already discussed in Chapter 5. Table 6.2 shows the Level of Service (LOS) for the arterial. The LOS is not constant in the peak times and travel directions (e.g., northbound PCH in the pm peak) because of the growth and dissipation of congestion.

- In the northbound direction, ATCS and TOD-Optimized plans had similar performance and outperformed the TOD-230 plan in the am peak period. All strategies had very similar performance in the midday period. The TOD-230 plan was the best strategy in the pm peak period.
- In the southbound direction, ATCS outperformed both fixed-time plans in the am peak period. The savings in travel times due to ATCS were statistically significant most of the AM peak period, and the LOS was better. All strategies had very similar performance and the same LOS in the midday period. The TOD-optimized plan resulted in longer travel times in the pm peak period, but the LOS was the same for all strategies tested.

Table 6.1 Ranking of Strategies Tested on PCH

A. Northbound Direction

CONTROL STRATEGY	TIME PERIOD AM	MIDDAY	PM
ATCS	2	3	2
TOD-230	3*	1	1
TOD-OPTIMIZED	1	2	3*

B. Southbound Direction

CONTROL STRATEGY	TIME PERIOD AM	MIDDAY	PM
ATCS	1*	2	2
TOD-230	3	1	1
TOD-OPTIMIZED	2	3	3*

** Differences in Performance are Statistically Significant*

Table 6.2 Level of Service on PCH

A. Northbound Direction

CONTROL STRATEGY	TIME PERIOD AM	MIDDAY	PM
ATCS	B	B	B-E
TOD-230	C	B	B-D
TOD-OPTIMIZED	B	B	B-E

B. Southbound Direction

CONTROL STRATEGY	TIME PERIOD AM	MIDDAY	PM
ATCS	B-C	B	B-A
TOD-230	B-D	B	B-A
TOD-OPTIMIZED	B-D	B	B-A

6.2 Future Research

This research project evaluated an operational adaptive signal control system against optimized time-of-day fixed-timed plans in a real-life corridor based on extensive field data. On going and future research will cover but not be limited to the following topics:

Further research is needed to address the operational issues identified for adaptive control under oversaturated conditions.

Development of an adaptive traffic control algorithm for signalized arterials that can be implemented without the extensive hardware and software required for the state-of-art adaptive systems like ATCS. The algorithm will make real-time adjustments to the splits and offsets of the baseline time-of-day plans in response to traffic demands.

REFERENCES

1. Stevanovic, A., “Adaptive Traffic Control Systems: Domestic and Foreign State of Practice,” NCHRP Synthesis 403, Washington DC, 2010.
2. Skabardonis A., “Benefits of Advanced Traffic Signal Systems,” in Measuring the Performance of ITS in Transportation Services," David Gillen Editor, Kluwer Academic Publishers, 2003.
3. Skabardonis A., and G. Gomes, “Effectiveness of Adaptive Control for Arterial Signal Management: Modeling Results,” PATH Research Report UCB-ITS-PRR-2010-33, University of California, Berkeley, August 2010.
4. Quadstone Ltd, "PARAMICS microscopic traffic simulation software,” http://Paramics-online.com.
5. Skabardonis, A., “Assessing The Benefits Of Signal Hardware Improvements,” Transportation Research Record No. 1554, 1996.
6. Skehan, S., “Adaptive Traffic Control System,” Compendium of Technical Papers, Institute of Transportation Engineers 66th Annual Meeting, pp 203-207, Minneapolis, 1996.
7. TSS Systems, “Aimsun, Microsimulator and Meso simulatorAimsun 6.1 User’s Manual,” December 2010.
8. Transportation Research Board, "Highway Capacity Manual," Washington DC, 2010.