Development of Near Real Time Performance Measurements for Closed-Loop Signal Systems (CLS) Using Historical Traffic Data from Existing Loop Detectors and Signal Timing Data

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16. Abstract

The overarching goal of this research project was to investigate the potential for the NCDOT Central Office Signal Timing (COST) Section to monitor and assess the quality of field deployed closed-loop signal system plans using the data inherent in the systems. The project is complete and has produced recommendations and deliverables that should enhance the COST Section's ability to achieve its mission of developing and maintaining quality signal coordination plans across the state of North Carolina.

Key findings and conclusions include the identification of a series of monitoring and analysis elements that can be implemented using the OASIS software detector and split monitor logs. In order to analyze dynamic, cycle-by-cycle bandwidth, a tool entitled the Dynamic Bandwidth Analysis Tool (DBAT) was developed and provided as a project deliverable. The DBAT tool was enhanced to provide exhaustive search optimization that identifies offset combinations that maximize dynamic bandwidth for a given set of split monitor log cycle-by-cycle signal indications. DBAT optimization is feasible for systems up to four or five intersections in length. An LP formulation was developed and tested that overcomes the system size limitation. The LP formulation can serve as the basis for future development of an implementable dynamic bandwidth optimization tool.

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EXECUTIVE SUMMARY

The overarching goal of this research project was to investigate the potential for the NCDOT Central Office Signal Timing (COST) Section to monitor and assess the quality of field deployed closed-loop signal system plans using the data inherent in the systems. The project is complete and has produced recommendations and deliverables that should enhance the COST Section's ability to achieve its mission of developing and maintaining quality signal coordination plans across the state.

In reviewing the state of the practice and emerging methodologies, the project team looked closely at the SMART-SIGNAL system and the Purdue University family of signal performance evaluation methods. While the recommendations in this report do glean some features from these two cutting edge efforts, the project team does not recommend full implementation of either of these systems due to the significant changes in field equipment installation specifications and in turn significant increased costs that such full implementation would entail.

The project team recommends a series of monitoring and assessment elements that can be implemented with the current OASIS detector and split monitor logs. The recommended monitoring and assessment program elements that are based on the OASIS Detector Log are –

- Creating and analyzing the coordinated movements flow plot (see Figure 5.12)
- Analyzing the coordinated movements flow plot for assessing time of day plan suitability (see Figure 5.12)

The recommend elements that are based on the OASIS Split Monitor Log are –

- Monitoring cycle-by-cycle coordinated movements capacity (see Figure 5.13 to Figure 5.16)

- Monitoring early return to green and green extension for coordinated phases (Figure 5.19 to Figure 5.22)
- Monitoring non-coordinated phase displayed green distribution (Figure 5.23 to Figure 5.25)

A final recommend monitoring and assessment program element that is based on both the OASIS Detector Log and the Split Monitor Log is monitoring flow to capacity (see Figure 5.17 and Figure 5.18).

In addition to this Excel-based analysis of OASIS log data, the project team, in recognition of the importance of the dynamic variability of cycle-by-cycle coordinated phase green times, created a tool called the Dynamic Bandwidth Analysis Tool (DBAT). DBAT was developed in as an easy to use, standalone tool and will be provided as a project deliverable. DBAT reads in OASIS split monitor data and will allow the COST Section to continually monitor dynamic system bandwidth.

Moving beyond dynamic bandwidth monitoring, the project team enhanced DBAT to provide an exhaustive search routine that identifies the set of offsets that would have maximized dynamic bandwidth for the archived cycle-by-cycle signal indication data. This optimization capability is ready to implement for systems with up to four or five intersections. This limit in number of intersection arises from the fact that the number of offset combinations that must be analyzed in the exhaustive search grows exponentially as the number of intersections increases. A linear program formulation was developed and tested that overcomes this limitation. However, creation of an implementation-ready LP tool was beyond the project scope.

Finally, the project team collected high resolution, vehicle activation level detector data at one of the field study sites, and develop methods for analyzing this data alone and conjointly with dynamic bandwidth information. The high resolution data analyses presented in Chapter 9 hold the promise of greatly enhancing the information available for COST Section closed-loop system performance assessment. Therefore, the project team recommends that NCDOT move toward collecting and analyzing high resolution detector data. Implementing this recommendation will require additional memory storage at each controller and a modification to the OASIS software to allow archiving of the raw detector inputs. This recommended OASIS software modification is in addition to a list of additional desired OASIS software enhancements given in Chapter 10, section 10.5.2.

Table of Contents

Chapter 1. Introduction ... 1

1.1 Project Motivation ... 1

1.2 Project Objective ... 2

1.3 Summary ... 3

Chapter 2. Literature Review (TASK 1) ... 4

2.1 Signal System ... 4

2.1.1 Closed Loop System ... 4

2.1.2 Traffic Responsive System and Adaptive Traffic Control System ... 7

2.2 Timing Plan Development Process for Arterials (State of Practice) ... 18

2.3 Performance Monitoring Systems ... 21

2.3.1 SMART-SIGNAL ... 21

2.3.2 Purdue Arterial Monitoring Methods ... 29

2.4 Arterial Performance Measures ... 36

2.4.1 Number of Stops ... 36

2.4.2 Travel Speed ... 37

2.4.3 Bandwidth ... 38

2.5 Summary ... 44

Chapter 3. Select Study Locations (TASK 2) ... 46

3.1 Study Site Selection Criteria 46

3.2 NCDOT Closed Loop System 46

3.3 Selected Study Sites 47

3.4 Summary 49

Chapter 4. Data Collection Plan and Field Study (TASKS 3 AND 4) 50

4.1 Available Data Sources 50

4.2 Testing Data Sources 50

4.2.1 OASIS Detector Event Data 51

4.2.2 OASIS Split Monitor Data 56

4.3 Data Collection Plan 59

4.3.1 OASIS Log Data 60

4.3.2 Arterial Travel Time Data 60

4.3.3 High Resolution Vehicle Data 65

4.4 Study Data Collection Sites: 72

4.4.1 Pilot Site: US 70 Arterial in Garner 72

4.4.2 Other Data Collection Sites 74

4.5 Summary 74

Chapter 5. Investigate Relationships and Develop Candidate Models (TASK 5) 76

5.1 Current Signal Timing Plan 76

5.2 Travel Time Studies 78

5.2.1 Floating Car 78

5.2.2 Bluetooth 85

5.2.3 Travel Time Comparison 87

5.3 OASIS Detector Log Monitoring 88

5.3.1 Detector Log Data Quality 88

5.3.2 RTMS Unit 91

5.3.3 Volume Profile with Time of Day Plan 92

5.4 OASIS Split Monitor Log Monitoring 93

5.4.1 Coordinated Phase g/C Profile 93

5.4.2 Flow Rate over Capacity 98

5.4.3 Early Return to Green Distribution 99

5.4.4 Non-Coordinated Phase Used Green Distribution 102

5.5 Bandwidth Monitoring 105

5.5.1 Conventional Bandwidth 106

5.5.2 Dynamic Bandwidth 106

5.6 Dynamic Bandwidth Analysis Tool 109

5.6.1 Processing OASIS Split Monitor Raw Data 109

5.6.2 Development of DBAT 111

5.6.3 DBAT Evaluation 115

5.7 Summary 120

Chapter 6. Estimate Model Parameters, Test Model Accuracy, and Investigate Adaptive Implementations (TASK 6).. 123

6.1 Dynamic Bandwidth Analysis 123

6.2 Comparison of Dynamic and Programmed Bandwidth Efficiency 126

6.3 Field Dynamic Bandwidth Distribution... 127

6.4 Summary 129

Chapter 7. Perform Rigorous Comparative Assessment of Model Performance versus Conventional Plan Evaluation Methods (TASK 7) 131

7.1 DBAT Exhaustive Search Method.. 131

7.2 DBAT Exhaustive Search Results 132

7.3 Evaluation of DBAT Solution 137

7.4 ACTRA 144

7.4.1 ACTRA Reports.. 144

7.4.2 ACTRA Split Monitoring 146

7.4.3 DBAT Process Result 147

7.5 Summary 152

Chapter 8. Develop Recommendations for Implementation of Best Performing Models (TASK 8) 153

8.1 Linear Programming for Dynamic Bandwidth Optimization 155

8.2 LP Model Test.. 159

8.3 Comparison of BluFax and BlueMAC Travel Times 162

8.3.1 Data Process 163

8.3.2 MAC Address Archive 166

8.4 Summary 168

Chapter 9. Develop Recommendations for Implementation of Other Uses of Model Outputs (TASK 9) 169

9.1 INRIX and Bluetooth Travel Time Comparison 169

9.1.1 NC 55 Arterial Comparison Result.. 170

9.1.2 Western Blvd Arterial Comparison Result 173

9.2 High Resolution Detector Data 175

9.2.1 Data Quality Test 175

9.2.2 Segment Travel Speed Distribution 178

9.2.3 Combine Dynamic Bands and High Resolution Detector Data.......................... 180

9.3 Summary 186

Chapter 10. Summary of Findings, Conclusions, And Recommendations.......................... 188

10.1 Task 1 - Literature Review – Findings and Conclusions 188

10.2 Task 2 - Select Study Location – Findings and Conclusions.................................. 190

10.3 Task 3 - Design Data Collection Plan – Findings and Conclusions 190

10.4 Task 4 - Conduct Field Studies – Findings and Conclusions 191

10.5 Task 5 - Investigate Relationships and Develop Candidate Models......................... 191

10.5.1 Findings and Conclusions 191

10.5.2 Recommendations 192

10.6 Task 6 - Estimate Model Parameters, Test Model Accuracy, and Investigate Adaptive Implementations 193

10.6.1 Findings and Conclusions 193

10.6.2 Recommendations 194

10.7 Task 7 - Perform Rigorous Comparative Assessment of Model Performance versus Conventional Plan Evaluation Methods 195

10.7.1 Findings and Conclusions 195

10.7.2 Recommendations 196

10.8 Task 8 - Develop Recommendations for Implementation of Best Performing Model 196

10.8.1 Findings and Conclusions 196

10.8.2 Recommendations 197

10.9 Task 9 - Develop Recommendations for Implementation of Other Uses of Model Outputs 198

10.9.1 Findings and Conclusions 198

10.9.2 Recommendations 199

Chapter 11. Implementation and Technology Transfer Plan 201

11.1 Introduction and Deliverables 201

11.2 OASIS Log-Based Analysis and Monitoring 202

11.3 Dynamic Bandwidth Analysis Tool (DBAT) 202

11.4 Dynamic Bandwidth Linear Program 203

11.5 High Resolution Detector Data Collection and Analysis 203

References 204

LIST OF FIGURES

Figure 2-1 Open and Closed Loop System Process 4

Figure 2-2 SCOOT Operation Diagram.. 12

Figure 2-3 Dynamic Map Function in ATSAC 13

Figure 2-4 Functional Diagram of the RHODES Real-Time Traffic Control System (28).... 15

Figure 2-5 ACS-Lite Architecture (31)... 16

Figure 2-6 Classical Approach to Signal Timing 19

Figure 2-7 SMART-SIGNAL System Architecture (33).. 22

Figure 2-8 Traffic Data Collection Flow in SMART-SIGNAL (34)...................................... 23

Figure 2-9 SMART-SIGNAL Data Process Flow Chart (34) 25

Figure 2-10 SMART-SIGNAL Detector Location Configuration (34).................................. 27

Figure 2-11 SMART-SIGNAL Virtual Vehicle Maneuver Decision Tree (34)..................... 28

Figure 2-12 Virtual Vehicle Trajectory (34)... 29

Figure 2-13 Flowchart for Purdue Signal Monitoring System 30

Figure 2-14 System Log Data Sample 31

Figure 2-15 Observed Green Time 32

Figure 2-16 Estimated Capacity.. 33

Figure 2-17 Volume to Capacity Ratio Monitoring Results... 34

Figure 2-18 PCD over Several Cycles 35

Figure 2-19 PCD Over 24 hours 35

Figure 2-20 Time-Space Diagram for MAXBAND Model.. 41

Figure 3-1 Summary of Wake County Closed Loop Systems 2011 47

Figure 3-2 Study Sites... 48

Figure 4-1 OASIS System Overview... 51

Figure 4-2 Configurations of Lane-based and Link-based Detectors.................................... 53

Figure 4-3 NEMA Phase with Phase Sequence.. 57

Figure 4-4 Programmed Intersection Signal Phases... 58

Figure 4-5 Dynamic Intersection Phases... 58

Figure 4-6 Lead-Lag phase.. 59

Figure 4-7 Translink32 Display... 60

Figure 4-8 BluFAX Unit... 61

Figure 4-9 Gen 5 BlueMAC Unit... 62

Figure 4-10 Digiwest User Customized Online Website.. 63

Figure 4-11 BlueMAC Device Location and Status Map.. 63

Figure 4-12 RTMS Microwave and Beam Footprint.. 66

Figure 4-13 RTMS Field Installation.. 66

Figure 4-14 Sensys Devices.. 67

Figure 4-15 Sensys Sensor Installation.. 68

Figure 4-16 TrafficDOT 2 Sensor Status... 69

Figure 4-17 Sensys Sensors and Each Device Locations with IDs....................................... 70

Figure 4-18 Video Camera Installation.. 71

Figure 4-19 Pilot Study Site in Garner, NC.. 72

Figure 4-20 US 70 Arterial Intersection Geometry with Detector Location........................... 73

Figure 4-21 Bluetooth Locations on US 70 Arterial in Garner... 74

Figure 5-1 Travel Time Distribution by Number of Stops on EB... 80

Figure 5-2 Travel Time Distribution by Number of Stops on WB 81

Figure 5-3 Travel Time Collection and Trajectory in Time-Space Diagram 82

Figure 5-4 U.S. 70 Eastbound Corridor Space/Time Trajectories... 83

Figure 5-5 U.S. 70 Westbound Corridor Space/Time Trajectories 84

Figure 5-6 U.S. 70 Eastbound Travel Time Distribution (7AM to 8AM)............................. 85

Figure 5-7 U.S. 70 Eastbound Travel Time Distribution (8AM to 9AM)............................. 86

Figure 5-8 U.S. 70 Westbound Travel Time Distribution (7AM to 8AM)............................ 86

Figure 5-9 U.S. 70 Westbound Travel Time Distribution (8AM to 9AM)............................ 87

Figure 5-10 Detector configuration of Timber Dr. Intersection on US 70 89

Figure 5-11 RTMS Unit Location Map 91

Figure 5-12 US 70 Volume Profile.. 92

Figure 5-13 Phase 2 g/C Profile for Timber Dr. Intersection 95

Figure 5-14 Phase 6 g/C Profile for Timber Dr. Intersection 95

Figure 5-15 Phase 2 g/C Profile for Jessup Dr. Intersection... 97

Figure 5-16 Phase 2 g/C Profile for Jessup Dr. Intersection... 97

Figure 5-17 Phase 2 g/C Profile for Timber Dr. Intersection 98

Figure 5-18 Phase 6 g/C Profile for Timber Dr. Intersection 99

Figure 5-19 Phase 2 Early Return to Green Distribution for Timber Dr. Intersection 100

Figure 5-20 Phase 6 Early Return to Green Distribution for Timber Dr. Intersection 100

Figure 5-21 Phase 2 g/C Profile for Jessup Dr. Intersection... 101

Figure 5-22 Phase 6 g/C Profile for Jessup Dr. Intersection... 102

Figure 5-23 Phase 1 and Phase 5 Displayed Green Distribution on Timber Dr. Intersection

.... 103

Figure 5-24 Displayed Green Distribution on Jessup Dr. Intersection for AM Plan............ 104

Figure 5-25 Displayed Green Distribution on Jessup Dr. Intersection for PM Plan 105

Figure 5-26 AM Plan Programmed Bandwidth for US 70 Arterial in Garner, NC 106

Figure 5-27 Dynamic Bandwidth on US 70 Arterial in Garner, NC 107

Figure 5-28 Dynamic Bandwidth on US 70 Arterial in Clayton, NC.................................. 108

Figure 5-29 Dynamic Bandwidth Example 109

Figure 5-30 Split Monitor Example 110

Figure 5-31 DBAT Interface.. 113

Figure 5-32 DBAT Test Result of Alternate Progression Case 117

Figure 5-33 DBAT Test Result of Double Alternate System Case 119

Figure 5-34 DBAT Test Result of Simultaneous System Case 120

Figure 6-1 US 70 Arterial in Clayton Primary Band with Secondary Band......................... 125

Figure 6-2 Site "A" Outbound Bandwidth PDF and CDF... 128

Figure 6-3 Site "A" Inbound Bandwidth PDF and CDF 128

Figure 7-1 Site "B" Day 1 AM Peak Plan Exhaustive Search Result 134

Figure 7-2 Site "A" Day 1 AM Peak Plan Exhaustive Search Result 135

Figure 7-3 Site "C" Day 1 AM Peak Plan Last 3 Intersection Exhaustive Search Result.... 135

Figure 7-4 DBAT Exhaustive Search Result in Plan 5 139

Figure 7-5 Coordinated Phase Used Green Time Comparison (Shotwell Dr. Intersection). 141

Figure 7-6 Directional Before and After Travel Time CDF's for 1:30 pm Plan at Site A ... 142

Figure 7-7 ACTRA Split Monitoring Data Display Window... 146

Figure 7-8 Offset Reference Point 147

Figure 7-9 Western Blvd Study Sites.. 148

Figure 7-10 Exhaustive Search Result by DBAT 150

Figure 7-11 Final Exhaustive Search Result 151

Figure 8-1 Bandwidth Optimization Basic Geometry 155

Figure 8-2 Dynamic Bandwidth LP Optimization Result 160

Figure 8-3 Dynamic Bandwidth DBAT Exhaustive Search Result 161

Figure 8-4 User Interface of BluSTATs Software 163

Figure 8-5 BlueMAC Project Website 165

Figure 8-6 BlueMAC Travel Time Monitoring Example 166

Figure 9-1 INRIX TMC Segment 125+06184 171

Figure 9-2 Six Weekdays INRIX and Bluetooth Travel Time Comparison 172

Figure 9-3 INRIX TMC Segment 125-14767 174

Figure 9-4 EB Travel Time Comparison (TMC 125-14767) 175

Figure 9-5 WB Travel Time Comparison (TMC 125+14768) 175

Figure 9-6 OASIS and Sensys Detector Call Comparison 177

Figure 9-7 Segment Travel Speed Distributions 179

Figure 9-8 Dynamic Bandwidth and High resolution Detector Data (EB) 184

Figure 9-9 Dynamic Bandwidth and High resolution Detector Data (WB) 185

LIST OF TABLES

Table 2-1 Comparison of Key Features of Three Generation of Control System 9

Table 2-2 HCM 2010 LOS Criteria 38

Table 2-3 Guidelines for Bandwidth Efficiency 39

Table 2-4 Guidelines for Bandwidth Attainability 39

Table 3-1 Summary of NCDOT Closed Loop System 46

Table 4-1 Available OASIS Log Files 51

Table 4-2 OASIS Detector Occupancy Display Test Result 52

Table 4-3 Displayed Occupancy and Possible Speed Rage by Displayed Occupancy........... 55

Table 4-4 Boundary Condition Test Result 56

Table 5-1 US 70 Garner Arterial Time of Day Plan 76

Table 5-2 Jessup Dr. Intersection Programmed Signal Timing 77

Table 5-3 Timber Dr. Intersection Programmed Signal Timing 77

Table 5-4 Garner Towne Square Intersection Programmed Signal Timing 77

Table 5-5 Yeargan Rd. Intersection Programmed Signal Timing 78

Table 5-6 Floating Car Travel Time for Full Segment 79

Table 5-7 Floating Car Travel Time for Each Section 81

Table 5-8 Number of Stops and Percentage from Floating Car............................ 85

Table 5-9 Floating Car vs Bluetooth Travel Time 87

Table 5-10 OASIS Detector Log with Video Counts Comparison 89

Table 5-11 Timber Dr. Intersection Programmed g/C and Field g/C Comparison 94

Table 5-12 Jessup Dr. Intersection Programmed g/C and Field g/C Comparison................. 96

Table 5-13 Timber Intersection Phase 1 and 5 Displayed Green Time 103

Table 5-14 Cycle-by-Cycle Dynamic Bandwidth on US 70 Arterial in Garner 107

Table 5-15 Cycle-by-Cycle Dynamic Bandwidth on US 70 Arterial in Clayton 108

Table 5-16 Input Data for Alternate Progression Scenario Test 116

Table 5-17 DBAT test results of alternate progression 117

Table 5-18 DBAT test results of double alternate progression... 118

Table 5-19 DBAT test results of double alternate progression... 120

Table 6-1 DBAT Arterial Dynamic Bandwidth Analysis Results....................................... 124

Table 6-2 Dynamic Bandwidth Computation Result of Site A from DBAT 125

Table 6-3 Bandwidth Efficiency Comparisons 126

Table 6-4 Bandwidth Comparisons 129

Table 7-1 Programmed Dynamic Bandwidth with Un-weighted Optimal Solutions 136

Table 7-2 Site “A” April Two weeks Split Monitor data DBAT Process Result 138

Table 7-3 Test Results for Demand Variation Before and After Offset Change 140

Table 7-4 K-S Test Result for Before and After green used time of Coordinated Phase 141

Table 7-5 Before and After Travel Time Observation at Site A... 143

Table 7-6 Current Offset DBAT Analysis Result.. 149

Table 7-7 Comparison of Current field Offset to Optimal Offset 151

Table 8-1 Site A OASIS Split Monitor Log 159

Table 8-2 LP Optimization Result 160

Table 8-3 DBAT Exhaustive Search Result Including Secondary Bands 161

Table 8-4 DBAT Exhaustive Search Result Excluding Secondary Bands 162

Table 8-5 DBAT Exhaustive Search Result Excluding Secondary Bands 167

Table 8-6 Matching Number Comparison 167

Table 9-1 TMC 125+06184 Segment Information 171

Table 9-2 Average Travel Time Comparisons (NC 55) 173

Table 9-3 TMC 125+06184 (125-14767) Segment Information 173

Table 9-4 Average Travel Time Comparisons (Western Blvd) 174

Table 9-5 Sensys High Resolution Data Example 176

Table 9-6 Detector Detection Differences 176

Table 9-7 Speed Trap Speed Summary 178

Table 9-8 US 70 Arterial in Clayton EB during Midday 1:30 PM Plan 181

Table 9-9 US 70 Arterial in Clayton WB during Midday 1:30 PM Plan 182

Table 9-10 Dynamic Bandwidth Optimization Results 183

CHAPTER 1. INTRODUCTION

1.1 Project Motivation

According to the Texas Transportation Institute's 2012 Annual Urban Mobility Report, 1982 congestion costs were an estimated $24 billion (2011 dollars) resulting from 1.1 billion delay hours and 0.5 billion gallons of wasted fuel. The report further illustrates that these costs increased dramatically through 2011, when total congestion costs were an estimated $124 billion resulting from 5.5 billion delay hours and 2.9 billion gallons of wasted fuel (1). Furthermore, the 2007 Traffic Signal Operation Self-Assessment Survey reported that Signal Operation in Coordinated Systems was given a "D-" grade, indicating that many signalized urban streets experience heavy congestion in the peak periods (2). While the 2012 survey did not report a separate grade for operation in coordinated systems, the grade for Signal Timing Practices only improved from a "C-" to a "C" between the 2007 and 2012 surveys (3).

Unlike freeway facilities where delay results primarily from capacity constrained bottlenecks, delay along signalized arterials (whose demand does not exceed capacity) results from deceleration and stops as vehicles interact with signal control. Therefore, mitigating delay along signalized arterials through improved signal timing is a cost effective congestion management approach that can be accomplished without adding physical roadway capacity.

Coordinated signalized arterials represent a vital component of North Carolina's roadway network. The Central Office System Timing (COST) Section within the Traffic Systems Operations Unit of the Transportation Mobility and Safety Branch is responsible for developing, evaluating, and maintaining the signal timing plans for hundreds of closed loop signal systems across the state. The legacy method for evaluating signal plan performance is based on field observation and before and after travel time runs. These field observations are melded with engineering experience and judgment and input from the traveling public to inform decisions regarding timing plan modifications. This field observation based approach is time consuming and yields decisions that are founded on very limited sampling of operational conditions.

A promising answer to the shortcomings of the current evaluation methods lies in the fact that the signal systems include extensive traffic detection subsystems. The traffic stream data from signal system detectors have been traditionally exploited only for tactical use as inputs to the control system logic. However, this detector data represents a valuable resource for the dual purpose of system performance evaluation. Development of online, near real-time performance evaluation models and methods will provide a significant enhancement to the COST section's ability to efficiently and effectively deploy and maintain signal plans that deliver near optimal quality of service to the traveling public. Near-real time arterial performance evaluation will also be beneficial to NCDOT's ongoing efforts to monitor system mobility and reliability and provide timely and useful traveler information. Therefore, there is clear motivation for developing performance evaluation methods that are cost effective and comprehensive.

1.2 Project Objective

The primary objective of this research project is to enable NCDOT to implement an efficient and sustainable methodology that the COST section can use for ongoing evaluation and maintenance of closed-loop signal system timing plans. This primary objective will yield the dual benefits of increasing NCDOT staff productivity and providing greatly enhanced operational performance along North Carolina's signalized arterials. Although this primary objective is tied to the OASIS control system that is being rolled out across the state for all signal systems under NCDOT operation control, the proposed project includes a secondary objective of assessing the effectiveness of the developed methodologies within at least one major non-OASIS municipal signal system. Finally, the project includes an objective of developing recommendations for broader use of the outputs from the arterial performance models, such as use within the NCDOT's emerging mobility and reliability monitoring system.

1.3 Summary

In this chapter, the motivations and purposes of this project are introduced. The detail task lists of this project are:

1. Investigate existing OASIS log data
2. Test the quality of OASIS log data
3. Find the relationship between log data and arterial signal system performance
4. Develop an available performance monitoring model (system)
5. Develop methodology to improve arterial performance using developed monitoring model
6. Field test for evaluating developed methodology
7. Recommendations for improving arterial monitoring system

The report chapters that follow document the research methodologies employed and the results, findings, conclusions, and recommendations arising from execution of the project tasks.

CHAPTER 2. LITERATURE REVIEW (TASK 1)

For more than 100 years, traffic signals have served urban and suburban streets. Before traffic signals were developed, public officers such as policeman manually managed traffic movements. During past decades, many different computer based signal operation systems were developed for isolated intersection and coordinated arterials. This chapter presents a literature review for signal system for NCDOT project 2012-12. The literature review is a resource document for generating the models and analytical frameworks for the methodologies to be developed in subsequent research tasks.

2.1 Signal System

2.1.1 Closed Loop System

The difference between open and closed loop control systems is that an open loop system refers to a system where the communication between the controller system and the output is one way. A closed loop system has a feedback system to monitor the output of system and it corrects the errors. An example would be as clock time drift between master intersection clock and local intersection clock. Figure 2.1 shows simplified flow of two systems.

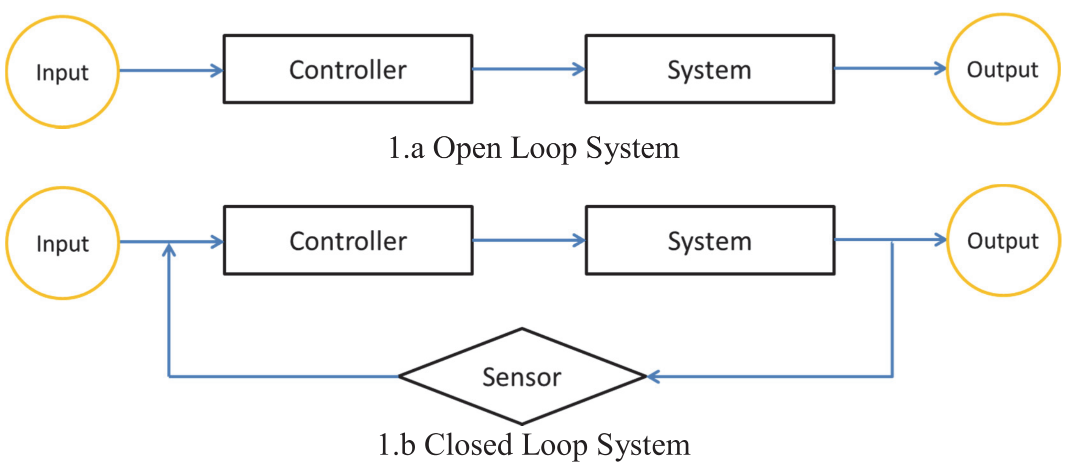


Figure 2-1 Open and Closed Loop System Process

Old forms of signal controllers are called electro-mechanical signal controllers, which are mainly composed of movable parts (cams, dials, and shafts) that control the passage of green, yellow and red lights in a predetermined sequence. Timings were controlled by mechanical tabs on a dial that were manually adjusted in the field by traffic engineers. These old systems do not provide any feedback loop in the system. They also do not allow communication between the master signal and local signal controller. Any set of controllers that does not communicate with each other is “open loop systems”.

The closed-loop system is a distributed processor traffic-control system with control logic distributed among three levels (4).

- The local controller
- The on-street master controller
- The office computer

These systems provide two-way communication between the local controllers, on-street master, and between the on-street master and the office computer. Typically, the local controller receives information from field detectors. The master controller receives information such as the status, time, and traffic volume from the local controllers. The office computer enables the system operator to monitor and control the system’s operations.

Three control modes are typically found with most closed-loop systems (4).

- Manual mode
- Time of day mode
- Traffic responsive mode.

Under the Manual Mode, the operator specifies the pattern number of the desired traffic-signal timing plans and sequence via computer console. The time of day mode allows the controller unit to automatically select and implement a predetermined traffic-signal timing plan such as cycle, offset and split based on the time of day. With the traffic responsive mode, the computer automatically selects the predefined traffic-signal timing plan. This is the best fit to accommodate the current traffic flow conditions in the signal network. The pattern selection and implementation is accomplished through a traffic flow data matching technique executed every five minutes (4).

There are four traffic signal controller types for operating traffic signal systems. These consist of pre-time (fixed time), fully actuated with non-coordinated controllers, semi-actuated, and fully-actuated with coordinated controllers. Fixed-time signal controllers are still widely used in CBD areas or similar environments where demand is over capacity. This type of controller provides exactly the same amount of green time to phases during each cycle. Fully actuated control is most often applied to non-coordinated signal controllers at isolated intersections. All phases are actuated so that the green duration of each phases is decided by minimum and maximum green and passage time, and, if necessary, pedestrian phase settings. Each phase can be extended, gapped out, or even skipped depending on demand and system settings. Therefore, in fully-actuated control there is no fixed cycle length. Volume-density represents the most complex legacy implementation of fully-actuated control with variable initial green based on arrivals on red and passage (gap) time that is shortened during the phase extension.

In semi-actuated signal control, only non-coordinated phases are actuated. All of the unused green time for non-coordinated phases reverts to the coordinated phases. This signal control scheme guarantees a programmed bandwidth (minimum bandwidth) for coordinated movements. Fully-actuated coordinated systems are similar to semi-actuated signal controllers. They can allocate a portion of coordinated phases to non-coordinated phases. This means a portion of the coordinated phases are actuated (5).

A closed loop system coordinated system consists of six main components (4).

- System detectors
- Local control equipment
- Controller master communications
- On street master
- Master central communications
- Office computer.

Among each component, the office computer allows the operator (traffic engineer) to set the time and date, display intersection, modify the master database, modify controller and coordination settings, modify system parameters, and monitor the system.

2.1.2 Traffic Responsive System and Adaptive Traffic Control System

Traffic responsive systems manage local controllers by updating cycles, offsets, and splits based on network level sensing. For example, traffic responsive plan selection (TRPS) involve matching defined plans to current traffic levels with this evaluation and selection normally taking place either in a field master controller or a central computer (6). When the master or computer selects a new timing plan due to demand changes, it sends a command to all local signal controllers in a coordinated group instructing them to change to the new plan simultaneously. The master or central computer monitors multiple traffic condition data from an array of detectors, including data such as volume and occupancy. The detector data is processed to calculate values for a few key parameters that are compared to predetermined thresholds. When the thresholds are crossed, the most applicable timing plans from within the predetermined plans is implemented for the conditions represented by the threshold categories selected.

In contrast to traffic responsive plan selection, which is by design reactive to measured traffic conditions, real-time adaptive traffic signal control system is a concept where vehicular traffic in a network is detected at system detectors, and then various algorithms are used to predict when and where the traffic will be in the future. This predictive feature allows the control system to make signal adjustments at appropriate intersections to serve the predicted traffic flows. The signal controller utilizes prediction algorithms to compute optimal signal timings based on detected traffic volume and simultaneously implement the timings in real-time.

In 1963, Miller introduced the principle of adaptive control for an online traffic modeling strategy (7). The model calculates time wins and losses and combines these criteria for the different stages in the performance measures to be optimized. The first adaptive control system (PLIDENT), was implemented in Glasgow, United Kingdom (UK) in the 1960s. However, the system did not operated effectively. (8). The second adaptive control system field trial was in Canada (9), but that trial also failed due to an inaccurate demand prediction algorithm for a longer time period, slowing the reaction of transition programs.

In the 1967, US Federal Highway Administration (FHWA) launched the Urban Traffic Control System (UTCS) project (10). The stated objectives of the UTCS projects were (11):

- To develop and test, in the real world, new computer based control strategies that would improve traffic flow.
- To document System planning, design, installation, operation, and maintenance to assist traffic engineers with installing their own systems.
- To stimulate modernization of traffic control equipment.

The UTCS project identified three generations of adaptive control systems and the plan was to demonstrate and evaluate each generation of controls. The three generations are (11):

- The first generation (UTCS-1) uses a library of predetermined timing plan, each developed with off-line optimization programs. The plan selected can be based on time of day, measured traffic pattern, or operator specification. The update period is 15 minute intervals. First generation allows critical intersection control and has a bus priority system (12, 13).
- The second generation (UTCS-2) uses timing plans computed in real time, based on predicted traffic conditions, using detector observations input into a prediction algorithm.
- The third generation was conceived as a highly responsive control with a much shorter control period than second generation and without the restriction of a cycle based system. Third generation system included a queue management control at critical intersections.

Table 2-1 Comparison of Key Features of Three Generation of Control System

Feature	First Generation	Second Generation	Third Generation
Optimization Frequency of Update	Off-line 15 minutes	On-line 5 minutes	On-line 3-6 minutes
No. of Timing Pattern	Up to 40	Unlimited	Unlimited
Traffic Prediction Critical Intersection Control	No Adjusts split	Adjusts split and offset	Adjusts split, offset and cycle
Hierarchies of Control	Pattern selection	Pattern computation	Congested and medium flow
Fixed Cycle Length	Within each section	Within variable groups of intersection	No fixed cycle length

Source: Traffic Engineering (14)

The UTCS-1 and UTCS-2 systems were installed in Washington, D.C. to develop, test and evaluate advanced traffic control strategies (15). The first generation UTCS-1 system was applied in New Orleans, controlling 60 intersections in an arterial environment. A time of day plan test resulted in an 8.8% reduction in travel time while the second generation UTCS-2 system test resulted in a comparable 8.5% reduction in travel time (16). In 1979, UTCS-1.5 was developed as an upgrade version of UTCS-1 (performance was not better than UTCS-1) (17). Like UTCS-1, timing plans were implemented from a library of pre-determined plans. However, UTCS-1.5 including the capability to generate new plans based on observed traffic data. These automatically generated plans would need to be reviewed and approved by a system operator before they could be added to the plan library. The UTCS-1.5 was tested in Broward County, Florida (1982), and Birmingham, Alabama (1984). In May of 1985, The UTCS project concluded and the policy statements were distributed on the support for the UTCS-1.5. The UTCS policy statement indicated that FHWA would not further enhance the software or documentation and that the private sector will likely develop and maintain their own system.

NCHRP 403 describes readily available adaptive control systems (18). Several adaptive traffic control systems (ATCS) are currently deployed in the United States.

- Sydney Coordinated Adaptive Traffic System (SCATS)
- Split, Cycle, Offset Optimization Technique (SCOOT)
- Automatic Traffic Surveillance and Control (ATSAC)
- Optimized Policies for Adaptive Control (OPAC)
- Real-Time Hierarchical Optimization Distributed Effective System (RHODES)
- Adaptive Control Software-Lite (ACS-Lite)
- InSync

2.1.2.1 SCATS

In the late 1970s, the Road and Traffic Authority in New South Wales, Australia developed SCATS (19). SCATS generate cycles, offsets and splits in three separate heuristic processes using calculated Degrees of Saturation (DSs) and link flows (LFs) from detector data (20, 21). Cycle length generated depends on two scenarios called low volume scenarios and high volume scenarios. Under the low volume scenarios, cycle length is determined from LFs. The high volume scenarios provided cycle length is computed using DSs. SCATS does not use common cycle lengths for coordinated signals. Instead of using common cycle length, it considers quality of progression and selectively joins together intersections that have good progression. For offset adjustments, SCATS uses a number of predetermined offset plans and seeks the best offset for particular flow patterns. However, SCATS uses only stop bar detectors, so it cannot create real-time volume (demand) profiles. On the other hand, SCATS' offset would not be sensitive against volume (demand) fluctuation.

2.1.2.2 SCOOT

The SCOOT is the most widely deployed adaptive system in existence. Figure 2.2 provides the SCOOT operation diagram. The system was developed in the United Kingdom (U.K.). SCOOT uses upstream system detectors to collect and create real-time "Cycle flow profiles" for each link. The SCOOT system detector location is different than the detector location for typical actuated control detection. Typical actuated detectors are located nearby the associated intersections. However, SCOOT system detectors are located at the upstream end of system approach links (22).

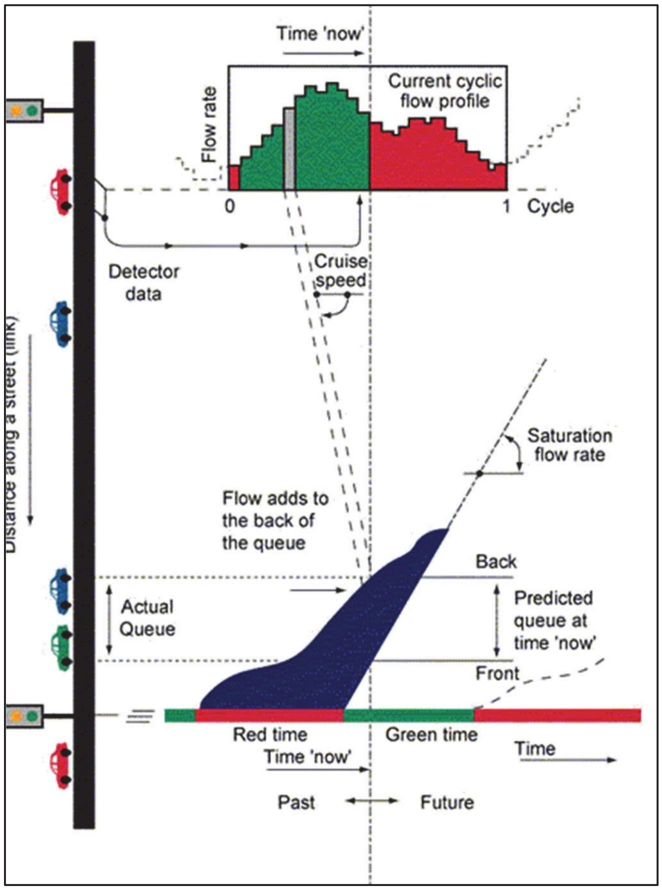


Figure 2-2 SCOOT Operation Diagram
Source: http://www.scoot-utc.com/DetailedHowSCOOTWorks.php

The upstream system detector data are processed by the SCOOT system predictive algorithms to create arrival profiles at the system intersections. SCOOT has three embedded optimization algorithms, one each for splits, offsets and cycles. SCOOT uses a common cycle length for coordinated arterial systems. These system cycle lengths are set at a length that maintains a degree of saturation less than predetermined value for all intersection within a given upper and lower boundary.

2.1.2.3 ATSAC

ATSAC was developed by Los Angeles Department of Transportation (LA DOT). ATSAC does not have any formal optimization logic (algorithms) for adjusting signal timing. It instead applies heuristic formulas based on extensive systems operation experience (23). The adaptive adjustment of signal timings are based on second-by-second fluctuation of volumes and occupancies measured at system detectors. Cycle lengths are adaptively updated within predetermined upper and low boundaries. For a given system, splits are adjusted under minimum green time consideration after the current cycle length is set. ATSAC does not provide alternate phase sequences.

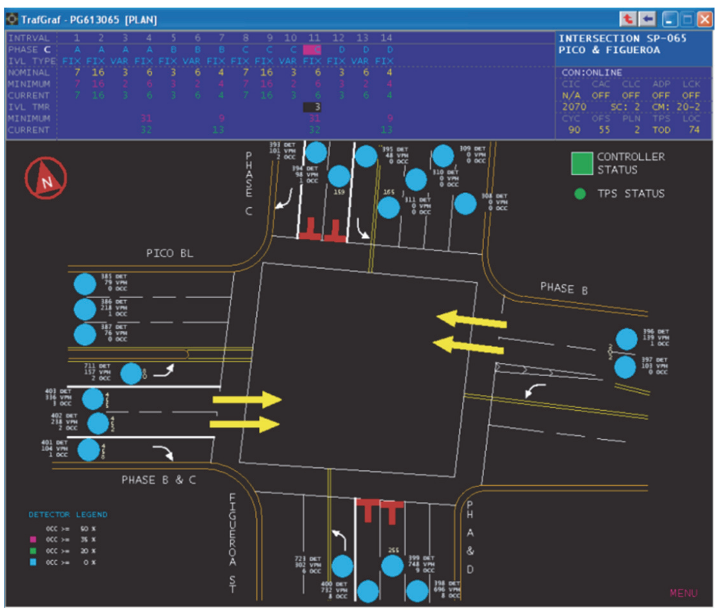


Figure 2-3 Dynamic Map Function in ATSAC
Source: NCHRP 403 (18)

2.1.2.4 OPAC

The Optimized Policies for Adaptive Control (OPAC) strategy utilizes a real-time signal timing optimization algorithm developed at the University of Massachusetts at Lowell (24). OPAC is a fully-adaptive, proactive, and distributed real time traffic control system. The system was developed as part of the FHWA Real-Time Traffic Adaptive Control System (RT-TRACS) program (25). The fundamental features of OPAC system are:

- Optimization of any or all phases splits designed to minimize total intersection delay and/or stops
- Support for phase skipping in the absence of demand
- Multiple sets of configuration parameters for customizing the resulting timing to weight certain movements for special circumstance or by time of day
- Configurable to respond to changes in left turn lead/lag phasing by time of day
- Special considerations for phase timing in the presence of congestion

OPAC is different from traditional cycle-split signal control strategies in that it drops the concept of system cycle lengths (26). In OPAC, the signal control algorithm consists of a sequence of switching decisions made at fixed time intervals. A decision is made at each decision point on whether to extend or terminate a current phase. Dynamic programming techniques are used to calculate optimal solutions.

2.1.2.5 RHODES

In 1992, the Real-Time Hierarchical Optimization Distributed Effective System (RHODES) was developed by the University of Arizona (27). RHODES is a real-time traffic adaptive control system. It has a three-level hierarchical structure for characterizing and managing traffic and predicts traffic at these levels utilizing detector and other sensor information (27, 28). RHODES can receive and consider input from different types of detectors. Based on predicted future traffic conditions, RHODES generates optimized signal control plans. Figure 2.4 illustrates the hierarchy of the RHODES system.

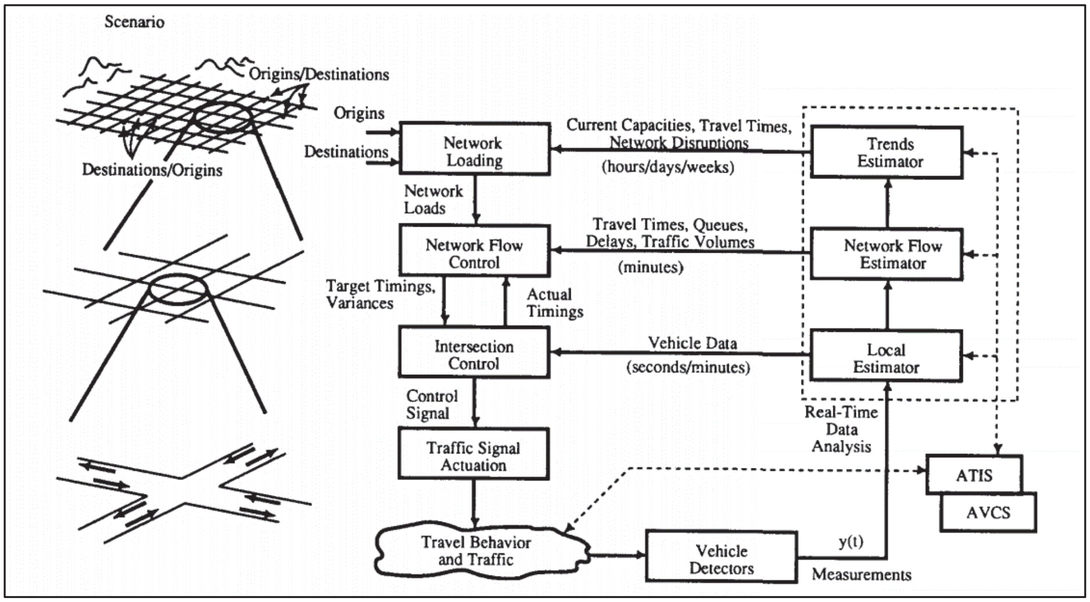


Figure 2-4 Functional Diagram of the RHODES Real-Time Traffic Control System (28)

RHODES uses a dynamic programming (DP) based real-time signal control systems similar to OPAC. However, RHODES uses signal phases as stages, the amount of green-time as decision variables, and the total number of time-steps as state variables. The RHODES DP formulation requires a fixed sequence of phases and a longer forecast horizon. Since the RHODES DP formulation requires a fixed sequence of phases, it cannot optimize phase sequences. RHODES uses the REALBAND algorithm for its signal coordination (29). REALBAND constructs a decision tree which contains all the possible decisions from the identified conflict movements. Each path in the decision tree represents a set of conflict resolutions that can be made within the system. The system calculates each path's performance such as delay and uses the calculated performance with path combination as constraints in the optimization algorithm.

2.1.2.6 ACS-Lite

ACS-Lite was designed for closed loop system's operation in the late 1990s (30). The system was developed by the University of Arizona, Purdue University, and private vendors such as Siemens and Econolite (31). This system is a reduced-scale version of the FHWA's adaptive control software. It offers small and medium-size communities a low-cost traffic control system that operates in real time, adjusting signal timing to accommodate changing traffic patterns and ease traffic congestion. Changes to cycle time are handled on a time of day plan like traditional traffic control systems. At each optimization step (which occurs approximately every 10 minutes), the system changes the splits and offsets a small amount to react traffic flow fluctuation.

ACS-Lite provides adaptive control within the industry standard context of cycle, splits, and offsets utilizing three control algorithms. Figure 2.5 shows that how the algorithms work in tandem to update traffic signal timing.

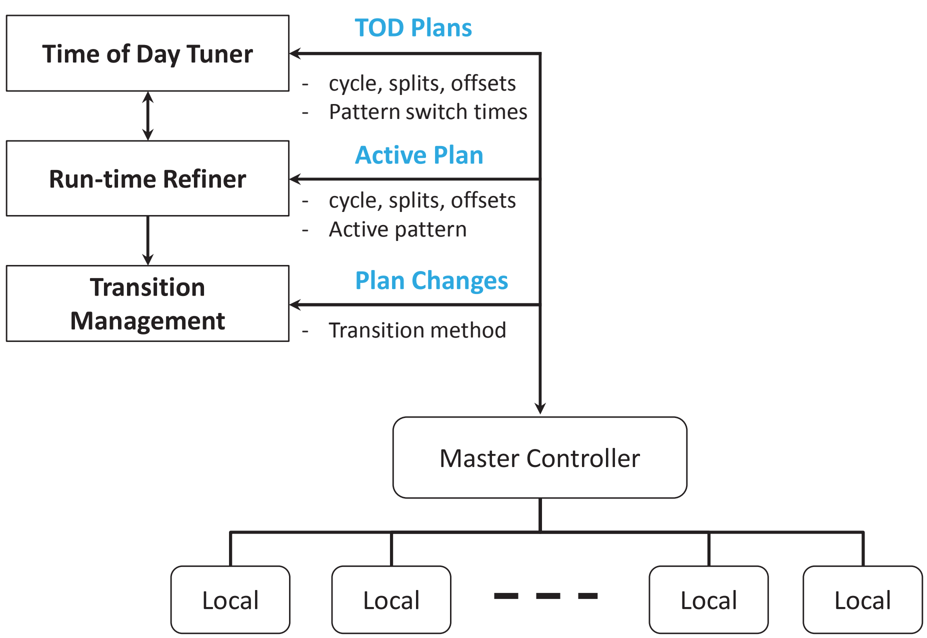


Figure 2-5 ACS-Lite Architecture (31)

2.1.2.7 InSync

The InSync system is an adaptive traffic signal system developed by Rhythm Engineering that uses advanced sensor technology, image processing, and artificial intelligence. The system uses a fundamentally different system of controlling and optimizing signal phases and timings in real-time (no cycle length and phase sequence). InSync signal timing methodology includes three major components (18).

- Digital architecture
- Global optimization
- Local optimization

The "digital architecture" term refers to the concept of a "finite state machine". In other words, InSync considers all possible non-conflict movement pairs and creates a maximum of "x" possible sequences of phase pairs at all intersections. Through the finite state machine framework, the InSync system can call any non-conflicting movement pair at any time. Thus, there is no predetermined phase sequence. The controller transitions signal indications from one state to the next based on the InSync logic, encapsulated by the local and global optimization algorithms.

This local and global optimization framework defines a two level optimization process. For global optimization, the InSync system focuses on time dependent platoon movements. The global optimizer in the system works to group platoons and optimizes their progression by maximizes the likelihood that each intersection's coordinated phase will be green at that time each "time tunnel" (which has similar concept to "green band") reaches the intersection. Conventional arterial coordination requires plan-based system cycle lengths for all coordinated signals. However, Insync does not require common cycle length for coordination. There are also no intersection timing plans for phase sequencing, and therefore there is no transition time required between the "time tunnels."

In essence, outside of the time tunnels, each intersection runs its own local optimization (i.e. the "local optimizer"). The local optimizer allows each signal in the arterial to operate in an intelligent, fully actuated mode.

2.2 Timing Plan Development Process for Arterials (State of Practice)

Signal coordination to support platoon progression is a key focus of arterial signal timing plans. The decision of whether or not to coordinate adjacent signals is evaluated in different ways. The general consideration is the space in between consecutive intersections. MUTCD provides the guidance that traffic signals within 800 meters (0.5 miles) of each other along a corridor should be coordinated unless operating on different cycle lengths.

The purpose of signal coordination is to provide smooth flow of traffic along streets and highways in order to reduce travel times, number of stops, and delays. A well-designed signal system allows platoons to travel along an arterial or throughout a network of major streets with minimum stops and delays. Designating traffic movement with the high peak hour demand as the coordinated phase is the most common practice to achieve these goals. The coordination logic (semi-actuated) reserves unused green time for the coordinated phase when non-coordinated phases have low demand. In general, this logic more stable capacity on coordinated phases and results in fewer stops for the high demand arterial traffic movements.

Figure 2.6 provides a conceptual flow chart for the current state of practice in arterial traffic signal timing development. Most of NCDOT signalized arterials have existing signal timing plan so a majority of the COST section's signal timing development efforts are involved with signal re-timing. Arterial signal re-timing can be conducted by regular schedule or on a more ad hoc basis in response to complaint calls or known traffic environment changes.

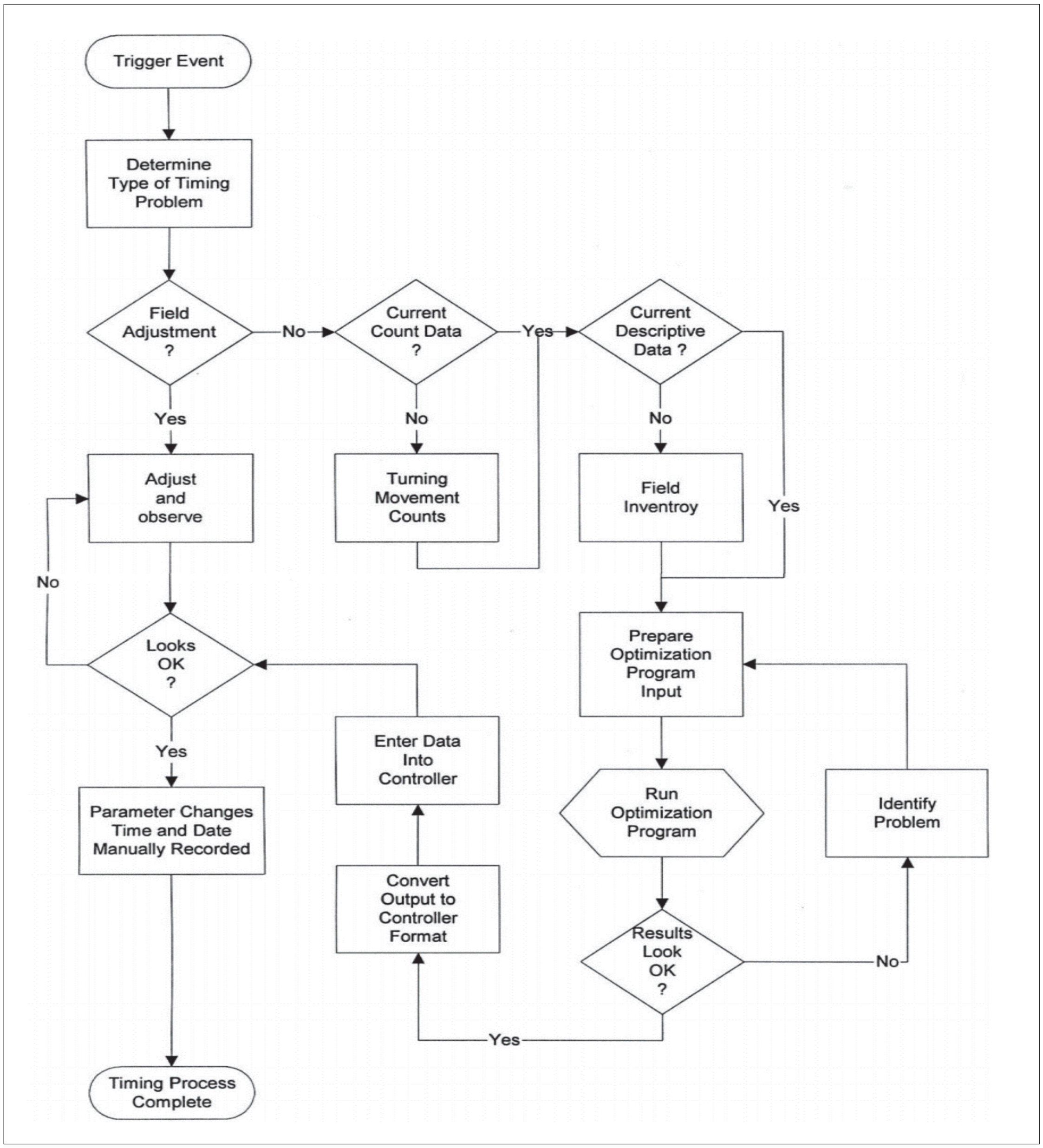


Figure 2-6 Classical Approach to Signal Timing
Source: NCHRP 409 (47)

Signal re-timing is a process that optimizes the operation of signalized intersections through a variety of low-cost improvements, including the development and implementation of new signal timing parameters, phasing sequences, and improved control strategies. When arterial signal re-timing is decided to be necessary, the signal timing engineer conducts a travel time study to evaluating current conditions and progression quality. NCDOT uses the Tru-Traffic software for analyzing arterial travel time. Tru-Traffic allows the field travel time runs to be analyzed in relation to current timing plans. After conducting field travel time surveys, intersection turning movement data are collected. Computer based simulation and traffic analysis tools such as Synchro and Vistro are normally used for signal timing development. The collected turning movement counts for each intersection are essential input data for analysis and simulation. The computer analysis provides various signal timing options with an expected performance for each option. The signal timing engineer usually selects one of these near optimal timing plan options based on experience, expected performance, and engineering judgment for each of the time of day, day of week periods that will be served by a unique timing plan.

The next step is implementation of the selected signal timing plans in the field. After implementing the selected timing plan, the signal plan engineer conducts field fine-tuning of the coordination offsets. For the offset fine-tuning, the engineer visits the site to observe existing traffic conditions, paying special attention to operational characteristics such as each intersection's initial queue length, early return to green, queue spill back, etc. The signal plan engineer must rely solely on experience and his/her engineering judgment for field offset fine-tuning. After completion of field fine-tuning, another travel time survey will be conducted for before and after comparison.

During signal timing development, the signal plan engineer must use his/her experience and engineering judgment when making decisions because computer based analysis tool can never provide an exact representation of the prevailing traffic conditions. In reality, many of the signal timing variables, such as minimum and maximum green time, green extension and etc., are given by or determined directly from policy. For example, NCDOT Traffic Management & Signal

Systems Unit Design Manual provides guidelines for typical minimum green value (7 seconds), extension value (2 second for stretch detection, 3 seconds for low speed detection).

NCHRP 409 summarizes the current signal timing state of the practice by stating that:

- Many agencies do not review field performance data to determine the adequacy of signal timing at intervals less than three years.
- Many agencies do not review signal design, operations, maintenance, and training practices annually.
- Many agencies do not have precise and clearly stated policies that support detailed objectives.

2.3 Performance Monitoring Systems

2.3.1 SMART-SIGNAL

In 2007, the University of Minnesota developed the Systematic Monitoring of Arterial Road Traffic and Signals (SMART-SIGNAL) system for monitoring arterial signal operation performance (32, 33).

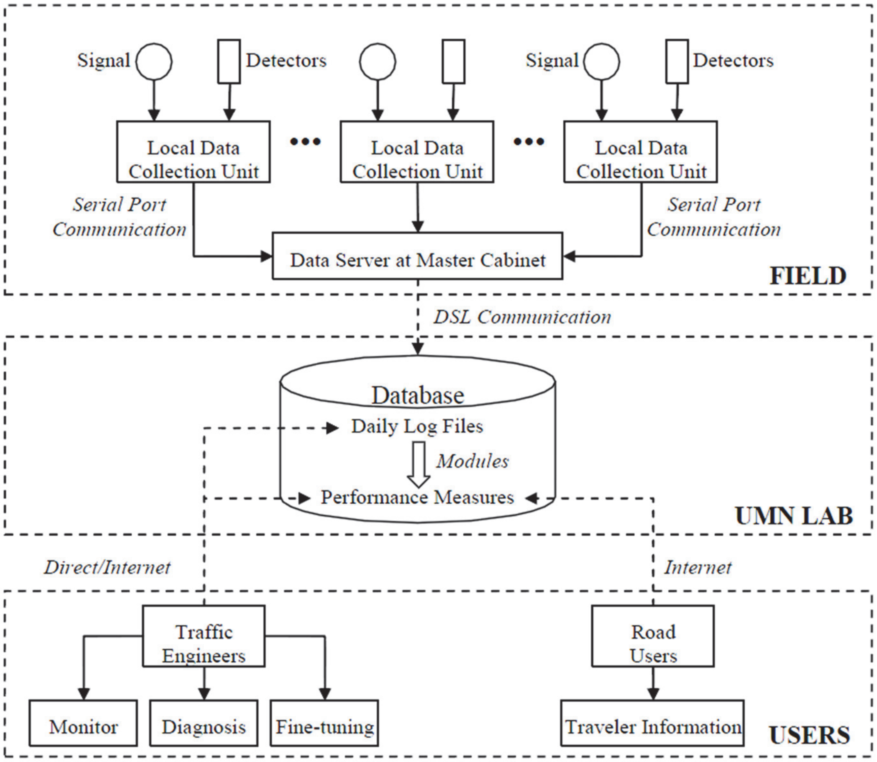


Figure 2-7 SMART-SIGNAL System Architecture (33)

The SMART-SIGNAL system collects two kinds of event signal data from “DATA Collection System”, which are vehicle actuation events and signal phase change events. Collected high resolution vehicle events data are used for estimating turning movement percentages and queue length. The results of dynamic queue length estimation and signal status date are processed for measuring each intersection's performance. Furthermore, the system generates “virtual prove car” to estimate arterial travel time to measure arterial performance. Figure 2.7 shows overall architecture of SMART-SIGNAL.

2.3.1.1 Data Collection System

General actuated signal control systems are operated by detector call and the operation results are displayed as signal phases. SMART-SIGNAL archives these two event data which are vehicle actuation events and signal phase change events. Those data sets are acquired separately from the data collection unit located in the traffic signal cabinet. SMART-SIGNAL uses Traffic Signal Timing Performance Measurement System (TSPMS) developed by TTI, as the data collection component at signal intersection (34). An industrial PC and a data acquisition card are deployed in each intersection's traffic signal cabinet to archive both events data. Collected data is transmitted to the data server in the master controller cabinet through the existing communication line. Figure 2.8 shows the structure of data collection flow in SMART-SIGNAL. The SMART-SIGNAL traffic data collection flow shows two data categories which are the existing traffic signal data and the additional data collection process. Detector calls and signal status data are existing data groups and data is processed through a Traffic Controller Interface Device (CID) and a Traffic Event Recorder software program. The processed data is archived in Traffic Log Data Server.

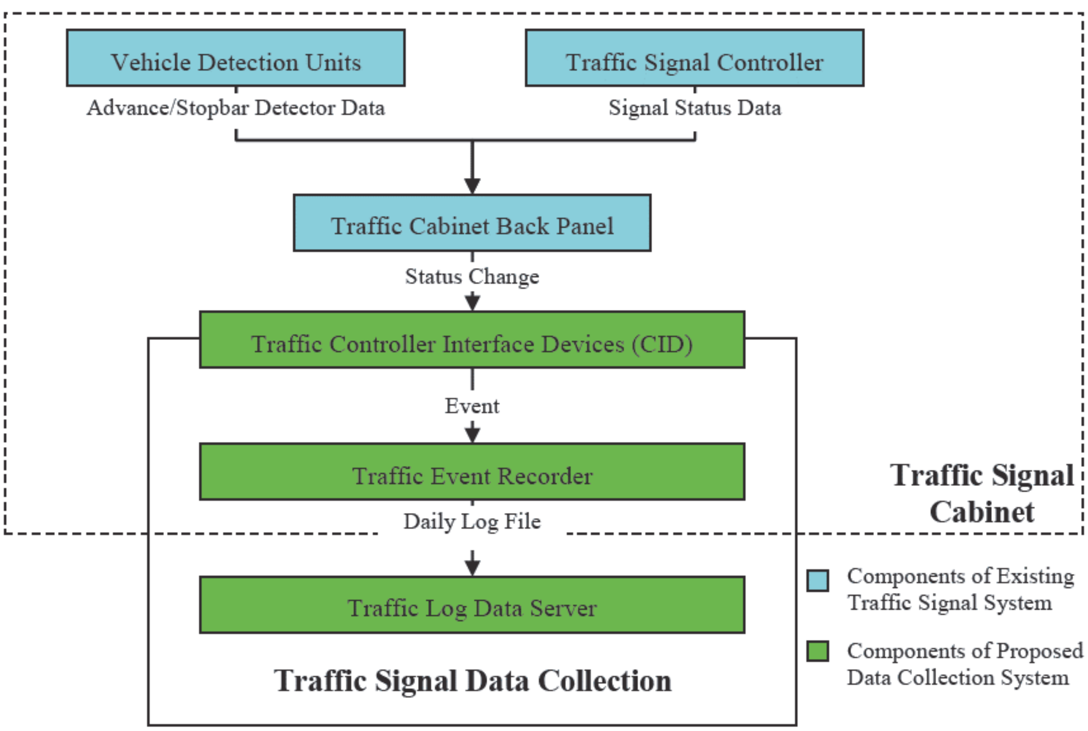


Figure 2-8 Traffic Data Collection Flow in SMART-SIGNAL (34)

2.3.1.2 Data Processing

The archived raw data is processed and converted to an easy to read format for measuring intersection and arterial performance. The data processing procedure provides high resolution detector actuation data (second-by-second) such as volume and occupancy. It also provides cycle-by-cycle signal timing with status data, indicating each phases green start and end time. Figure 2.9 shows the SMART-SIGNAL data processing flow chart. The data process begins after the raw data is transmitted back to server. The data processing flow includes four steps.

- Data Verification
- Preprocessing module
- Performance measure calculation
- Visualization

The data verification step tests the collected data quality and filters out wrong data. The preprocessing module generates some basic measures from the raw data. The performance measure calculation creates aggregated volume, delay, queue size, queue length, travel time etc. Finally the visualization step shows the results of different types of performance measures and diagnosis' fine tuning of traffics signal system.

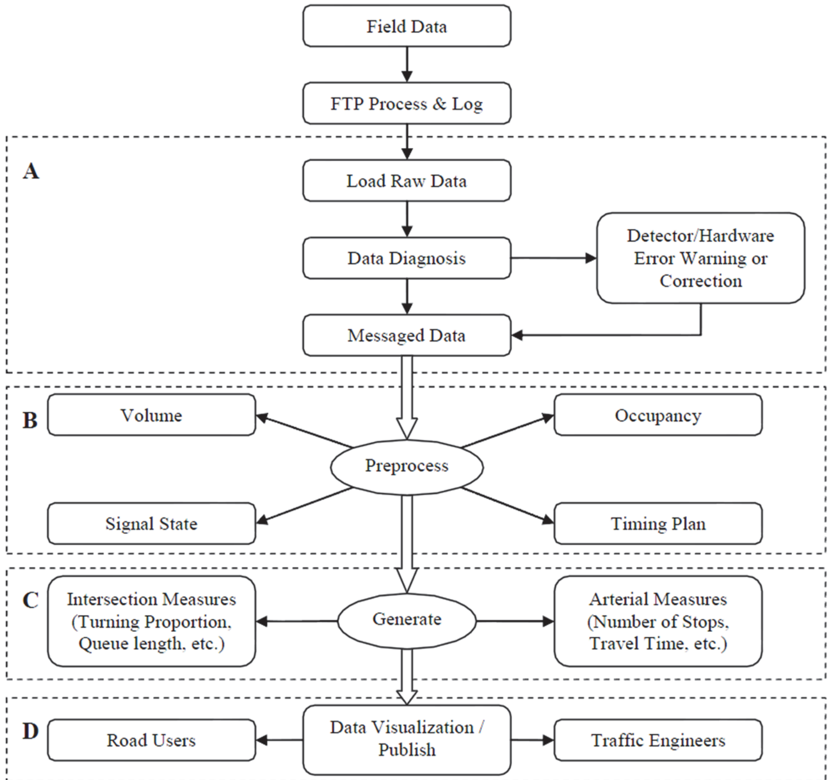


Figure 2-9 SMART-SIGNAL Data Process Flow Chart (34)

2.3.1.3 Intersection Performance Measurement

Many adaptive control systems use intersection delay and level of service as intersection performance measurements. In the SMART-SIGNAL system, queue length and turning movement proportions (TMP) are used for measuring intersection performance.

For the queue estimation, SMART-SIGNAL uses a dynamic queue length estimation method. The model calculates difference of the arrival and departure rate and provides a queue length over time. The SMART-SIGNAL queuing model defines a number of event times that describe the dynamics of queue interaction with the signal status. The queuing model includes two separate estimation models which include “Short Queue Estimation Model” and “Long Queue Estimation Model”. If vehicle arrivals can be measured from advance detectors and queue length is less than distance between stop line and detector location, it is defined as “short queue”. Otherwise, it is defined as “long queue”. Short queue can be estimated according to the queue development using advance detector calls, which can provide vehicle headway. When queue spills over the advance detector, the advance detector will be occupied by a car and it will provide hi-occupancy. SMART-SIGNAL developed the relationship of queue development and occupancy profile at a signalized intersection. Using that relationship, maximum queue length and time-dependent queue length curve are estimated. SMART-SIGNAL’s queue estimation procedure was compared to field data for 70 samples and was able to predict actual queue lengths with an average error of 7.5%, and queue sizes with an error of approximately 9.4%.

Turning movements’ proportion or counts are important information to analyze an intersection's performance, so it is often used as input data for simulation. However, it is difficult to measure directly from detector calls since full-set detector configuration is rare in the field. Right-turn detectors are usually not deployed because a protect phase is absent at the majority of the intersections in the United States. Furthermore, shared lanes (through and right, or through and left) and exclusive left turn lanes with long loop detectors also make it difficult to measure turning movement proportions directly from detector calls.

SMART-SIGNAL proposed a simple turning movement proportion estimation model using advance detectors and left turn stop bar detectors for the major approach. Left-turn stop bar detectors were used for the minor approach. Figure 2.10 shows SMART-SIGNAL detector configuration.

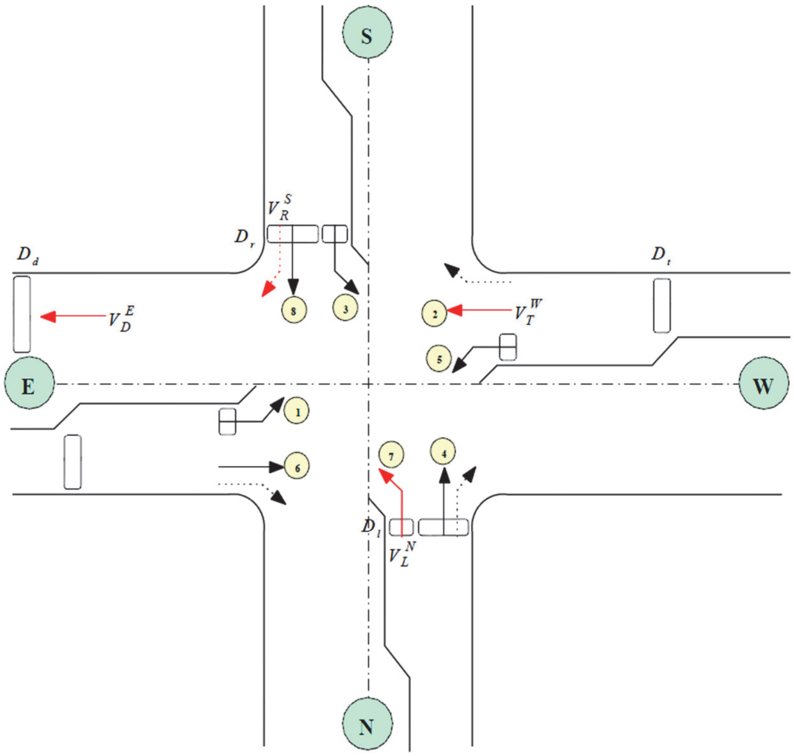


Figure 2-10 SMART-SIGNAL Detector Location Configuration (34)

To solve the turning movement proportion, two assumptions are made.

1. The travel time between each detector is known value and stable.
2. The right-turn movement traffic in a cycle is continuous and uniform.

Based on two assumptions, SMART-SIGNAL estimates the short time intersection turning movement proportion. The suggested model was tested, a total of 56 sample cycles and 85 percentiles have errors of less than 15% and the average error was 8.9%. However, the error rate can vary site-by-site due to the second assumption.

2.3.1.4 Virtual Probe Vehicle Approach for Arterial Performance Measurement

SMART-SIGNAL uses a virtual probe vehicle to estimate time-dependent arterial travel time, utilizing high resolution vehicle actuation data and signal status data. The manually generated virtual prove vehicle has three possible maneuvers including acceleration, deceleration and no-speed-change. Virtual probe decides its acceleration dependent on given traffic states, such as queue and signal status step-by-step. The step-by-step maneuver selection will be continued until its destination and the time different between start and end time will be an arterial travel time. Figure 2.11 shows step-by-step virtual prove maneuver decision tree.

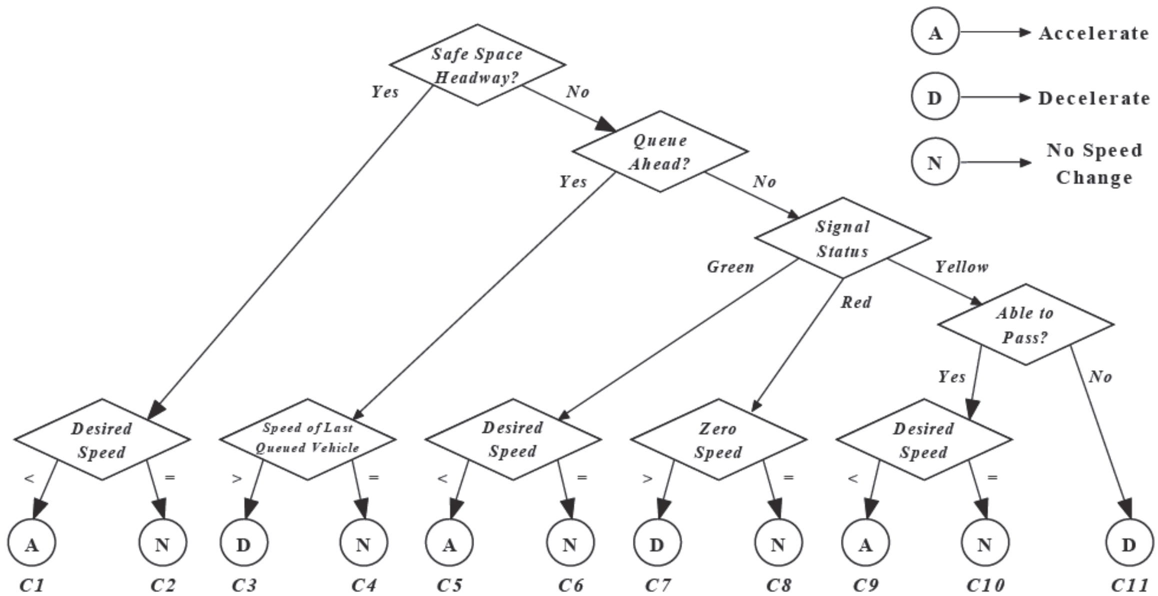


Figure 2-11 SMART-SIGNAL Virtual Vehicle Maneuver Decision Tree (34)

The suggested model is tested on a 1.83 mile long major arterial on France Avenue in Minneapolis, MN. This arterial includes 11 signalized intersections with a coordinated actuation signal controller. Figure 2.12 shows the virtual probe vehicle's trajectory with real floating car location.

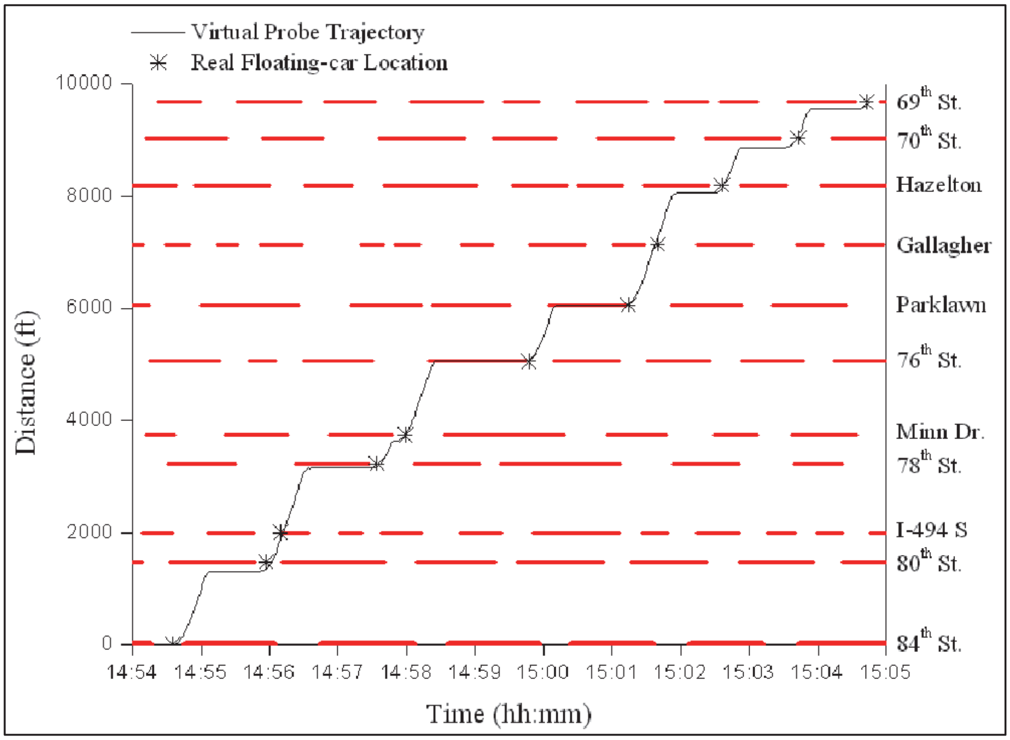


Figure 2-12 Virtual Vehicle Trajectory (34)

The study uses Root Mean Squared Percent Error (RMSP) as degree of model fitness. The reported estimation RMSP error is 0.0325, but the report did not mention test sample sizes.

2.3.2 Purdue Arterial Monitoring Methods

The Purdue Coordination Diagram (PCD) was developed by Purdue University and Indiana DOT. It uses phase status log and high resolution detector data for monitoring an intersection's level of performance and system level (arterial) performance. The Purdue system collects signal status data for monitoring cycle-by-cycle signal status. They developed a new analytical method to define dynamic cycle length. The created dynamic cycle length and each phase will have a unique ID. In addition, each intersections detector high resolution data is also archived in the system for identifying vehicle arriving and vehicle location under the signal status. The Purdue signal monitoring system creates both intersection and arterial level performance measured by combining signal status data and high resolution detector data. Figure 2.13 shows the Purdue monitoring systems flow.

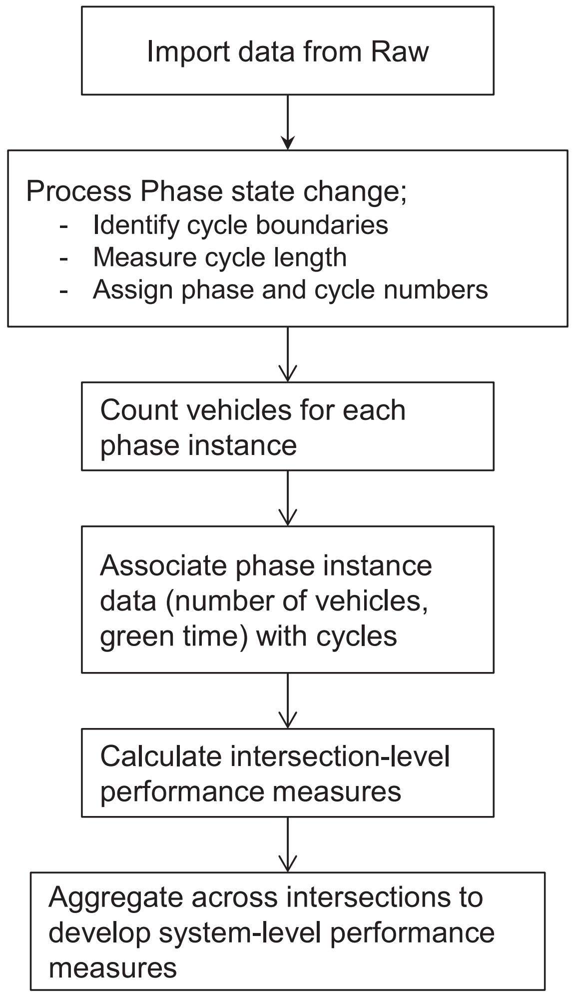


Figure 2-13 Flowchart for Purdue Signal Monitoring System
Source: NCHRP Project 3-79a

The system log event data has three elements:

- A timestamp containing the data and time of the event, with resolution of 0.1 seconds.
- A number representing event type (Phase green, Phase yellow, Detector on, Detector off, etc.)

- A number representing the event channel. For phase information, this was the number of the phase for which the event was relevant.

Figure 2.14 shows controller log data sample.

Timestamp	Parameter	Channel	Explanation
04/08/09 14:10:49.6	8	22	Detector 22 off
04/08/09 14:10:49.9	9	7	Detector 7 on
04/08/09 14:10:50.1	8	7	Detector 7 on
04/08/09 14:10:51.1	63	2	**Phase 2 yield point**
04/08/09 14:10:51.3	2	2	**Phase 2 yellow state**
04/08/09 14:10:51.3	33	2	**Phase 2 termination: gap out**
04/08/09 14:10:51.8	9	10	Detector 10 off
04/08/09 14:10:51.9	9	28	Detector 28 off
04/08/09 14:10:52.0	8	10	Detector 10 on
04/08/09 14:10:52.0	9	12	Detector 12 off
04/08/09 14:10:52.0	8	28	Detector 28 on
04/08/09 14:10:52.1	8	12	Detector 12 on
04/08/09 14:10:52.4	9	17	Detector 17 off
04/08/09 14:10:52.8	9	19	Detector 19 off
04/08/09 14:10:53.0	8	19	Detector 19 on
04/08/09 14:10:56.1	9	21	Detector 21 off
04/08/09 14:10:56.4	3	2	**Phase 2 red clearance state**

Figure 2-14 System Log Data Sample
Source: NCHRP Project 3-79a

2.3.2.1 Split Failure Monitoring

As mentioned earlier, the Purdue monitoring system archives signal status log. The total green time for a phase in a defined cycle is found by summing over all instances of the phase that occur within cycle. Therefore, capacity can be estimated under assumed, observed or estimated saturation flow rates, such as the following equation.

$$C_{\emptyset,a} = g_{\emptyset,a} * \frac{S_{\emptyset}}{3600}$$

Where, $C_{\emptyset,a}$: capacity provide to phase Ø during cycle "a"

$g_{\emptyset,a}$: the amount of effective green time for cycle "a"

$S_{\emptyset}$: saturation flow rate of phase Ø

The equation represents the total number of vehicles that can be expected to be served at the saturation flow rate. However, cycle lengths will be changed by time of day plan, so capacity per cycle in units of vehicles becomes difficult to compare between different cycle lengths. Therefore, the equation needs to be normalized as:

$$C_{\emptyset,a} = \frac{g_{\emptyset,a}}{C_a} * S_{\emptyset}$$

Where, $C_{\emptyset,a}$: capacity provide to phase Ø during cycle “a”

$g_{\emptyset,a}$: the amount of effective green time for cycle “a”

$S_{\emptyset}$: saturation flow rate of phase Ø

C_a : cycle length of “a”

Figure 2.15 shows a cycle-by-cycle effective green plot, and Figure 2.16 shows an estimated capacity result plot for a given phase.

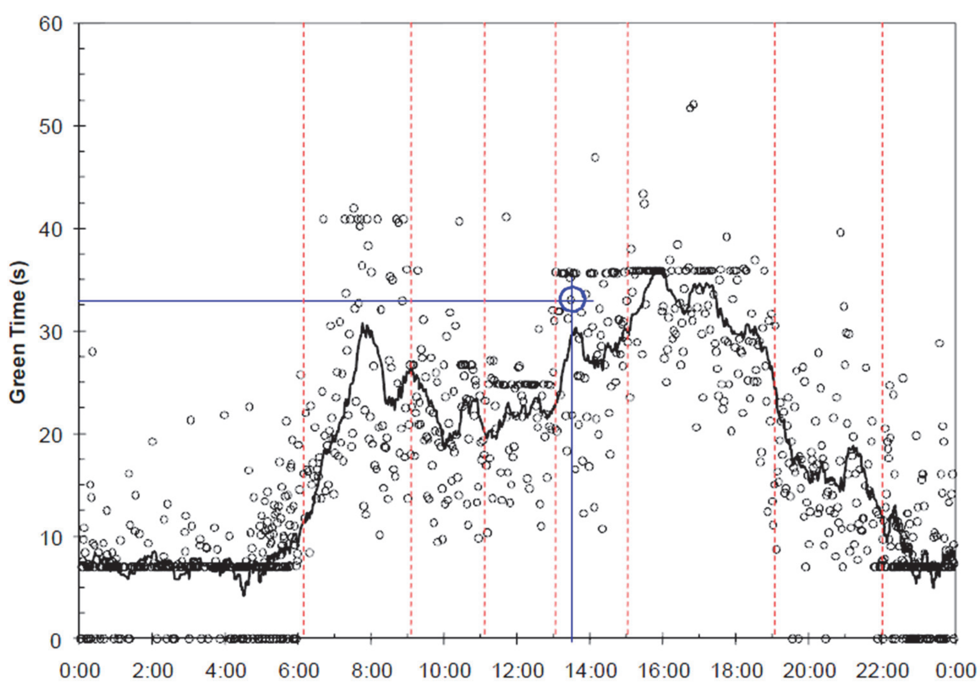


Figure 2-15 Observed Green Time
Source: NCHRP Project 3-79a

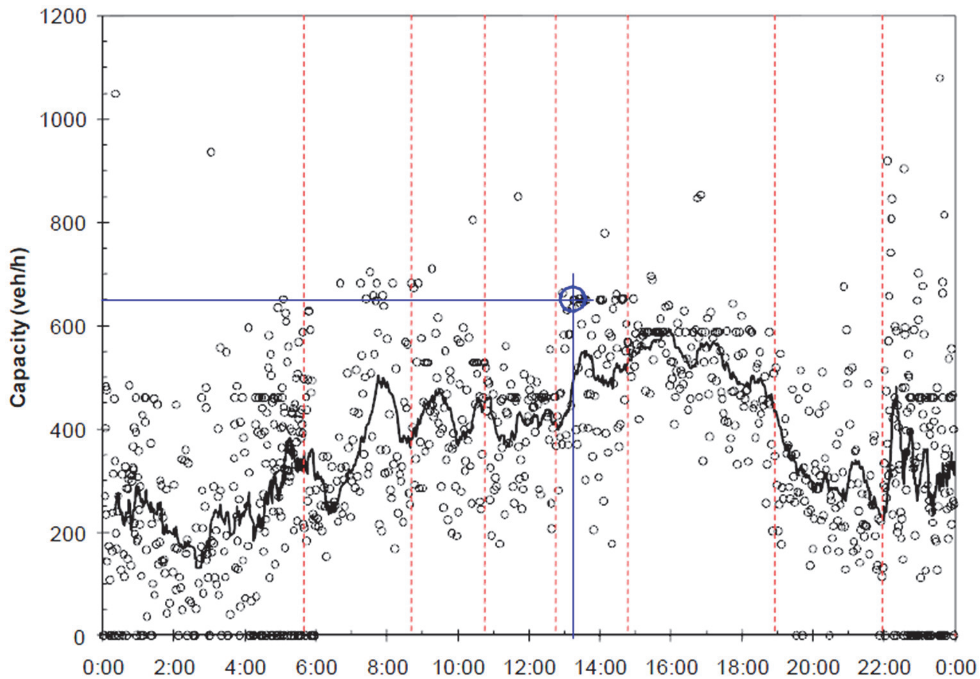


Figure 2-16 Estimated Capacity
Source: NCHRP Project 3-79a

High resolution detector data is archived in the system and provides a vehicle count during each cycle. The cycle-by-cycle vehicle counts are normalized by the following equation and it can be directly compared with Figure 2.16 estimated capacity.

$$V_{\emptyset,a} = 3600\frac{N_{\emptyset,a}}{C_a}$$

Where, $V_{\emptyset,a}$: hourly flow rate for phase $\emptyset$ during cycle “a”

$N_{\emptyset,a}$: the number of vehicle arriving during phase $\emptyset$ in cycle “a”

C_a : cycle length of “a”

Combing the normalized hourly flow rate and capacity allows the degree of saturation or volume to capacity ratio.

$$X_{\emptyset,a} = \frac{V_{\emptyset,a}}{C_{\emptyset,a}}$$

Where, $X_{\emptyset,a}$: normalized degree of saturation of phase $\emptyset$ during cycle "a"

$V_{\emptyset,a}$: hourly flow rate for phase $\emptyset$ during cycle "a"

$C_{\emptyset,a}$: capacity provide to phase $\emptyset$ during cycle "a"

The degree of saturation gives a measure that quantifies how much the provided green time is utilized by vehicles. Figure 2.17 shows the volume to capacity ratio monitoring results. The number of dots above the red line indicates signal failure. This method allows monitoring the frequency of signal failure per time of day plan for each phase.

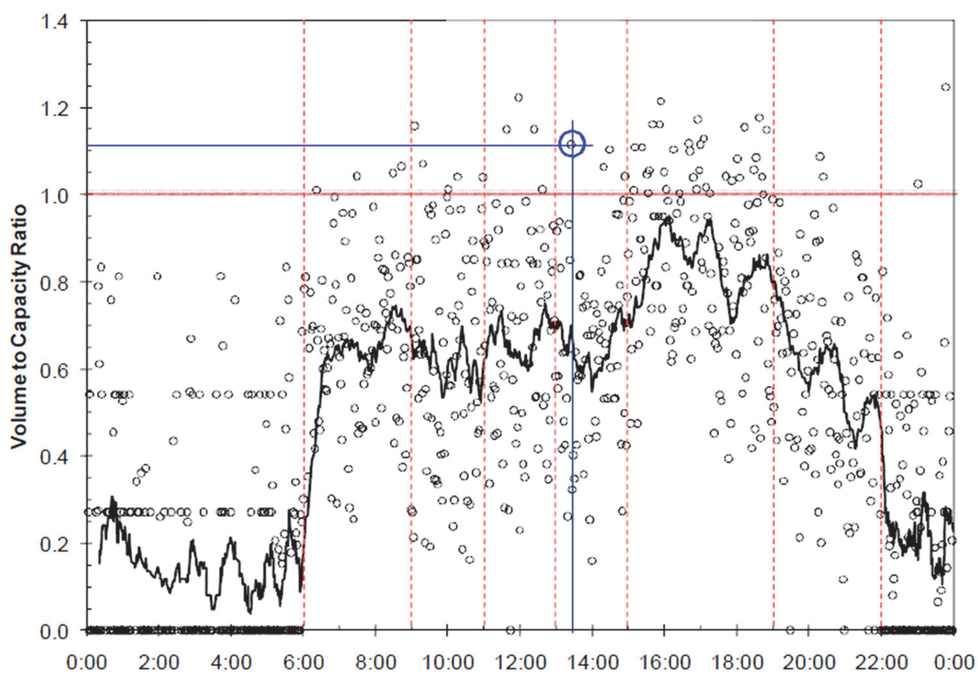


Figure 2-17 Volume to Capacity Ratio Monitoring Results
Source: NCHRP Project 3-79a

2.3.2.2 Purdue Coordination Diagram

The Purdue Coordination Diagram (PCD) is a visualization tool for evaluating the quality of progression. Figure 2.18 shows the result of high resolution detector data and phase status data for over several cycles. The green and orange lines indicate start and end time of green for each cycle.

The black dots represent each vehicle's location. The PCD directly provides the arrival on green percentage for each cycle.

Figure 2-18 PCD over Several Cycles
Source: NCHRP Project 3-79a

The black dots (vehicle location) are derived from advance detectors so the PCDs reflect actual vehicle behavior on the corridor. Figure 2.19 shows the 24 hour extended PCD.

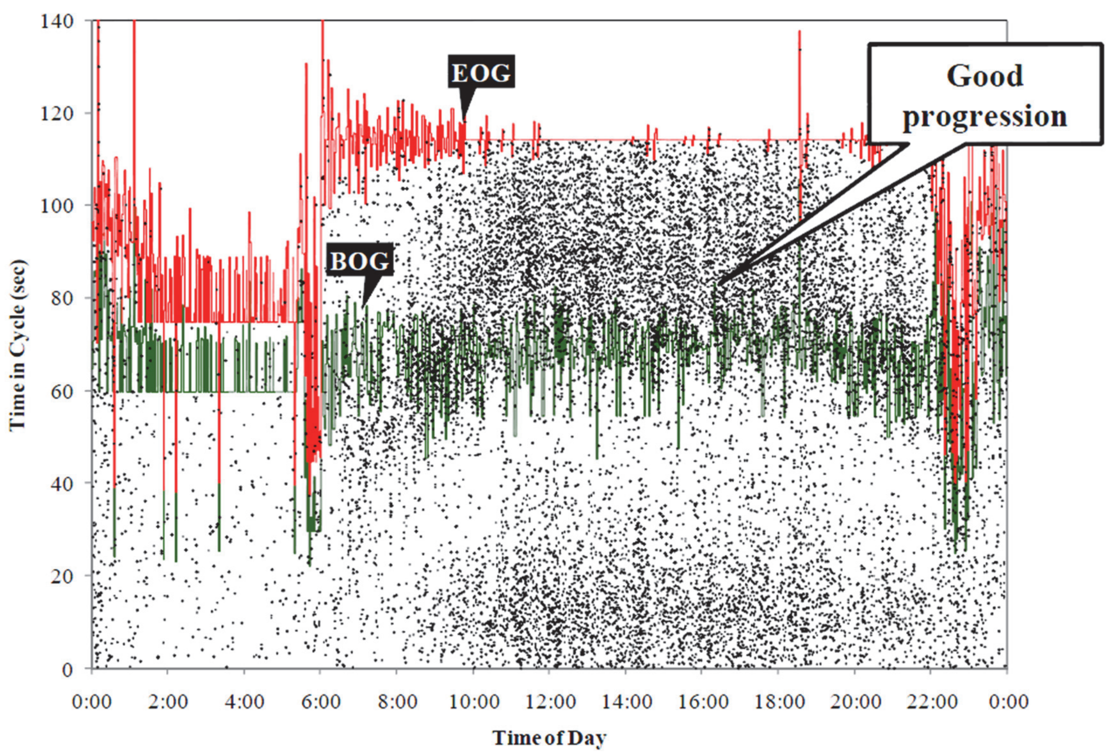


Figure 2-19 PCD Over 24 hours
Source: NCHRP Project 3-79a

The PCD describes vehicle arrival with coordinated phase status, so this plot gives information of the arterial coordination performance and provides a qualitative picture.

2.4 Arterial Performance Measures

Performance measurement is the process of collecting, analyzing and reporting information regarding the performance of an individual or system. When considering signal timing among a series of signalized intersections (as for coordinated signal operation), performance measures that account for the relative intersection of adjacent intersections becomes important. In case of arterial performance measures at each intersection along an arterial or within a network, a number of performance measures are used to assess how well the intersections fit together in terms of signal timing. The performance measures include number of stops, travel speed, and bandwidth.

2.4.1 Number of Stops

The number of stops is used frequently to measure an arterial or network's signal system effectiveness. Motor vehicle stops can often play a larger role than delay in the perception of the effectiveness of a signal timing plan a long an arterial street or a network. The number of stops or average numbers of stops per vehicle tends to be used more frequently in arterial applications where progression between intersections is a desired objective. The number of stops has not been identified as a candidate for standardization nationally. In addition, this measure is difficult to collect directly on the field. Therefore, computer based simulation tools are normally used for estimating and optimizing the number of stops. Stops are highly correlated with amount of emission and the quality of progression along arterials. FHWA's Traffic Signal Timing Manual states that the number of stops is an important measure because acceleration from stops is a major source of vehicle pollutants and surveys reveal that multiple stops along an arterial is more highly correlated with driver frustration than is delay.

2.4.2 Travel Speed

Travel speed (time) is one of the most popular measures used to assess for arterial progression. Arterial travel speeds account for both the delay at intersections and the travel time in between intersections. In the *Highway Capacity Manual* (*HCM) 2010*, the through vehicle travel speed in between two adjacent intersections is used for measuring an arterial's level of service (LOS). The HCM defines arterial LOS as a function of the class of arterial under the study and the travel speed along the arterial. This speed is based on intersection spacing, the running time between intersections, and the control delay to through vehicles at each signalized intersection. Since arterial travel time (space mean speed) in HCM method is calculated segment by segment regardless of origin or destination, the resulting speed estimates may be different corresponding to speed (travel time) measurements made from end to end travel time runs. Data was collected by a GPS enabled floating car that measured a small subset of the possible origin-destination combinations along an arterial.

In *HCM 2010*, two performance measures are used to characterize LOS for a given direction of travel along an urban street segment. One measure is travel speed for through vehicles and the other measure is the volume-to-capacity ratio for the through movement at the downstream boundary intersection. Table 2.2 HCM 2010 LOS Criteria shows the *HCM 2010* level of service thresholds established for the automobile mode on urban streets.

Table 2-2 HCM 2010 LOS Criteria

Travel Speed as a percentage of Base Free Flow Speed (%)	LOS by Volume-to-Capacity Ratio	
	≤ 1.0	> 1.0
> 85	A	F
> 67 – 85	B	F
> 50 -67	C	F
> 40 – 50	D	F
> 30 – 40	E	F
≤ 30	F	F

2.4.3 Bandwidth

2.4.3.1 FHWA Bandwidth Guidelines

The Federal Highway Administration Traffic Signal Timing Manual defines Bandwidth as the total amount of time available for vehicles to travel through a system of coordinated intersections at the progression speed, i.e. the time difference between the first and last hypothetical trajectory that can travel through the entire arterial at the progression speed without stopping. Bandwidth is an outcome of the signal timing that is determined by the offsets between intersections and the allotted green time for the coordinated phase at each intersection. Bandwidth is a parameter that is commonly used to describe capacity or maximized vehicle throughput. Bandwidth can be confirmed by the time-space diagram which is visual toll for engineers to analyze a coordination strategy and modify timing plans. Bandwidth, along with its associated measures of efficiency and attainability, are measures that are sometimes used to assess the effectiveness of a coordinated signal timing plan.

Bandwidth efficiency is calculated by:

$$B_e = \frac{(BW_1 + BW_2)}{2C}$$

Where, B_e: Two-way bandwidth efficiency

BW_1: Outbound bandwidth

BW_2: Inbound Bandwidth

C : Cycle Length

Bandwidth attainability is calculated by:

$$B_a = \frac{(BW_1 + BW_2)}{(g_{min1} + g_{min2})}$$

Where, B_a: Two-way bandwidth attainability

g_{min1}: Outbound direction minimum green along the arterial

g_{min2}: Inbound direction minimum green along the arterial

Table 2.3 and Table 2.4 show the FHWA Traffic Signal Timing Manual's two Guidelines for bandwidth efficiency and bandwidth attainability.

Table 2-3 Guidelines for Bandwidth Efficiency

Efficiency Range	Passer II Assessment
0.00 - 0.12	Poor Progression
0.13- 0.24	Fair Progression
0.25 – 0.36	Good Progression
0.37- 1.00	Great Progression

Table 2-4 Guidelines for Bandwidth Attainability

Attainability Range	Passer II Guidance
1.00 – 0.99	Increase minimum through phase
0.99 – 0.70	Fine-tuning needed
0.69 – 0.00	Major changes needed

2.4.3.2 Bandwidth Optimization

There are two categories of bandwidth optimization. The first one produces uniform bandwidths, while the other provides variable bandwidths. There are two well-known programs used for uniform bandwidths, including MAXBAND (35) and PASSER II (36).

Messer et al (36) developed Progression Analysis and Signal System Evaluation Routine (PASSER II) in 1973. PASSER II is a macroscopic deterministic optimization model. An iterative gradient search method is used to determine the phase sequence and cycle length which can provide the maximum two-way progression for a certain arterial signal system. Book's Inference Algorithm and Little's Optimized Unequal Bandwidth Equation are used in PASSER II. In PASSER II, cycle length, phase sequence and offset are varied to determine the optimal set of signal control settings, which minimize the total interference to the progression.

Morgan and Little (37) introduce a machine computation method to maximize arterial signal bandwidth. The widely used program of Little, Martin and Morgan (38) efficiently finds offsets for maximum bandwidth given cycle time, red times, intersection spacing and progression speed. Directional bandwidth can be adjusted by the target value which represents the directional flow ratio. Using mixed-integer linear programming (MILP), Little, Kelson and Gartner (35) developed the MAXBAND program in 1981. The purpose of MAXBAND is to provide the maximum bandwidth setting for coordinated arterials. This program has the following capabilities:

- Finds the best system cycle length within a given range
- Allows different progression speeds for each link within a given range
- Provides optimal major street left-turn phase sequencing
- Considers user specified queue clearance times
- Allows for directional weight factors for two-way bandwidth optimization

MAXBAND considers a two-way arterial with "n" intersections and specifies the corresponding offsets so as to maximize the number of vehicles that can travel within a given speed range without stopping throughout the arterial. In MAXBAND, phase splits at each intersection are assigned according to Webster's theory, and all signals are constrained to share a common cycle length. MAXBAND includes a queue clearance time in order to allow secondary flows which have accumulated during the red time to discharge before the platoon arrives. Figure 2.20 shows the basic geometry defined for the MAXBAND mixed-integer linear program.

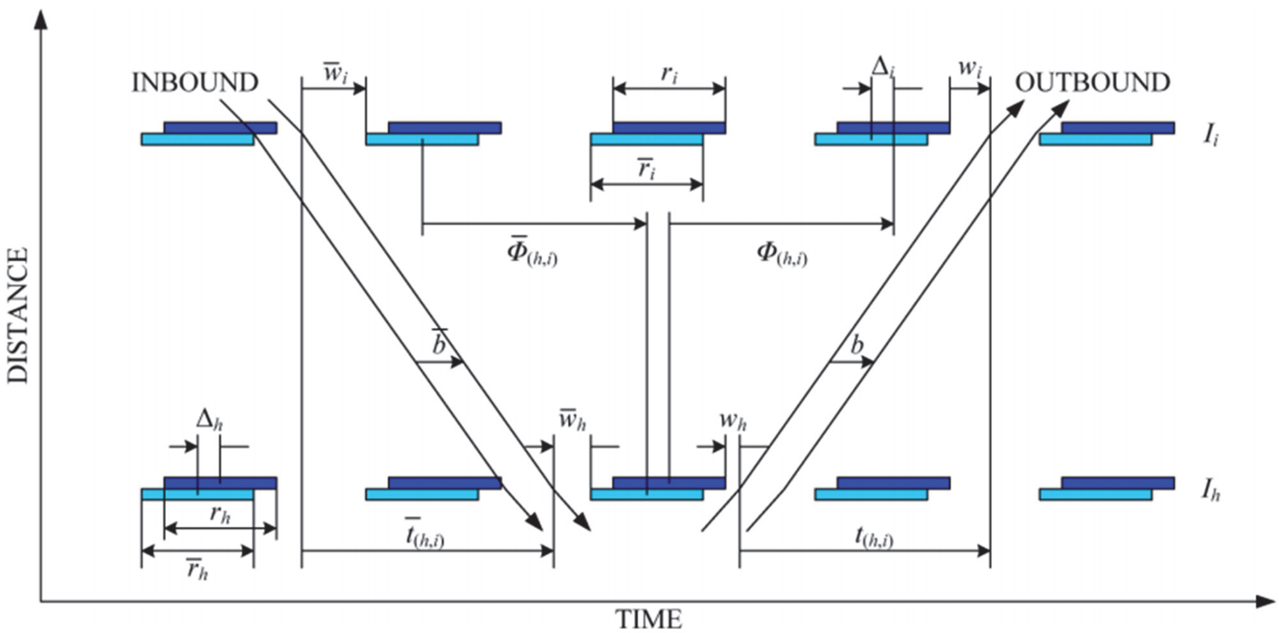


Figure 2-20 Time-Space Diagram for MAXBAND Model
(Source: Kai Lu, 2012)

The MILP form of MAXBAND is:

$$Max\ b + K\bar{b}$$

subject to

$$\begin{Bmatrix} \bar{b} \ \geq Kb\ if\ K < 1 \\ \bar{b} \ \leq Kb\ if\ K > 1 \\ \bar{b} = b \quad if\ \ K = 1 \end{Bmatrix}$$

$w_i + b \le 1 - r_i \quad \forall\, i = 1,2,\cdots,n$

$\bar{w}_i + \bar{b} \le 1 - \bar{r}_i \quad \forall\, i = 1,2,\cdots,n$

$$\left(t_{i,i+1} + \bar{t}_{i,i+1}\right) + (w_i + \bar{w}_i) - (w_{i+1} + \bar{w}_{i+1}) + (\Delta_i - \Delta_{i+1}) =$$

$$-\frac{1}{2} * (r_i + \bar{r}_i) + \frac{1}{2} * (r_{i+1} + \bar{r}_{i+1}) + (\bar{\tau}_i + \bar{\tau}_{i+1}) + m_{i,i+1} \quad \forall\, i = 1,2,\cdots,n$$

$\Delta_i = \left(\frac{1}{2}\right)[(2\delta_i - 1)l_i - (2\bar{\delta}_i - 1)\bar{l}_i$

$m_{i,i+1} = integer \quad \forall\, i = 1,2,\cdots,n-1$

$b, \bar{b}, w_i, \bar{w}_i > 0 \quad \forall\, i = 1,2,\cdots,n$

$\delta_i(\bar{\delta}_i) = binary$

Where, $b\left(\bar{b}\right) = outbound\ (inbound) bandwidth$;

$K = target\ value\ (\frac{\bar{b}}{b})$;

$n = number\ of\ intersection$;

$I_i = signal\ intersection\ i$;

$r_i(\bar{r}_i) = outbound (inbound) red\ time\ at\ I_i$;

$w_i(\bar{w}_i) = time\ from\ right\ (left)\ side\ of\ red\ phase\ at\ I_i\ to$

$left\ (right) edge\ of\ outbound\ (inbound) green\ band$;

$t_{(h,i)}\left[\bar{t}_{(h,i)}\right] = travel\ time\ from\ I_h\ to\ I_i\ outbound(inbound)$;

$\Delta_i =$ time from center of $\bar{r}_i$ $to\ nearest\ center\ of\ r_i$;

$m_{i,i+1} = loop\ interger\ variable\ related\ with\ I_i and\ I_{i+1}$;

$l_i\left(\bar{l}_i\right) = time\ allocated\ for\ outbound(inbound)\ left\ turn\ green\ at\ I_i$;

$\delta_i(\bar{\delta}_i) = left\ turn\ phase\ sequence$;

Tsay and Lin (39) introduced BANDTOP, which can obtain a saw-toothed bandwidth pattern, rather than parallel and uniform pattern. In this paper, it is evident that MAXBAND may not find maximum bandwidth for certain signal-timings. Gartner et al. (40) proposed MULTIBAND, which can consider a variable bandwidth arterial progression, in 1991. MAXBAND cannot consider actual traffic flow on the arterial link, so it is insensitive to variation in such flows. MULTIBAND used a multi-band/multi-weight concept to consider actual arterial link flow. It provides a capability to adapt the progression scheme to the specific traffic flow pattern that exists on the links of the arterial. MULTIBAND can provide a global optimal solution that calculates cycle length, offsets, progression speeds and phase sequences to maximize a combination of the individually weighted bandwidths in each directional arterial segment. Stamatiadis and Gartner (41) developed the MULTIBAND-96 program in 1996. The MULTIBAND-96 model optimizes all the signal control variables and generates variable bandwidth progressions on each arterial in the network. The MINOS mathematical programming package was used for optimization. Gartner and Stamatiadis (42) applied the MULTIBAND method to solve for an urban grid network. In this paper, the efficiency of MILP was improved by a heuristic network decomposition procedure. Tian and Urbanik (43) proposed a heuristic approach to a bandwidth oriented signal timing method based on a system partition technique. A large signalized arterial was divided into subsystems of three to five signals and then each subsystem is optimized to achieve the maximum bandwidth efficiency. From each subsystem solution, a large system offset was adjusted. This method provides maximum progression for the peak direction while maintaining partial progression for the off-peak direction. Lin et al. (44) proposed a new mixed integer nonlinear programming model for an optimal arterial-based progression algorithm. This model was designed to optimize the bandwidth while maximizing the number of non-stop vehicles through downstream intersections.

Lu et al. (45) introduced a two-way bandwidth maximization model with a proration impact factor. Under a certain weighting factor, MAXBAND and MULTIBAND may not find the maximum bandwidth solution. Therefore, in this paper, authors introduced bandwidth proration impact factors which indicate the target bandwidth demand ratio.

2.5 Summary

This chapter examined a variety of topics in signal operation including closed loop signal system features, adaptive control systems, signal timing design, measures of arterial performance, and bandwidth optimization methods.

Many adaptive control systems are used for advanced traffic management in United States. These systems require installing more detectors as well as parameters, which needs calibration. It increases installation and maintaining cost, as well as engineers retraining cost and time. For these reasons, ACS-Lite was developed. ACS-Lite is a reduced-scale version of the FHWA adaptive control software. It offers small and medium-size communities, and a low-cost traffic control system that operates in real time.

While there are many advanced traffic signal systems, very few arterial monitoring systems are available. Most signal operation firmware collects detector and signal status log data, but does not permanently archives the data for monitoring arterial signal system performance. In this chapter, two of the most advanced arterial monitoring systems are reviewed. SMART-SIGNAL uses queue estimation models and vehicle acceleration/deceleration models in order to estimate arterial travel time using the virtual probe vehicle generation method. Purdue University created several useful monitoring methodologies to monitor both intersection and arterial level of signal operating performance. Both new arterial monitoring systems are theoretically robust and show strong confidence of monitoring results in their literatures. However, both systems also require extra detectors, as well as high resolution detector data collecting systems compared to NCDOT general arterial signal system. There are limitations to apply both SMART-SIGNAL and the Purdue Arterial Monitoring Method under the NCDOT signal system. First, the NCDOT signal system does not provide high resolution detector data, so a cycle-by-cycle queue estimation method for SMART-SIGNAL method cannot be applied. Secondly, the NCDOT signal system normally does not have a stop bar detector and upstream detector for non-coordinated movements, so SMART-SIGNAL's turning movement estimation method cannot be used. In conjunction, the Purdue intersection level monitoring method for non-coordinated movements cannot be processed.

There are three generally used arterial performance measures, namely number of stops, travel speed, and bandwidth. None of these three arterial performance measures can be monitored or measured directly by the detection resident in currently deployed signal systems. Because of this, SMART-SIGNAL develops a virtual vehicle generation method to estimate arterial travel time. Purdue proposed a new arterial performance measure using the Purdue Coordination Diagram.

SMART-SIGNAL focused on estimating arterial travel time, while the Purdue system searches for arriving on green percentage (for measuring arterial progression quality). However, neither of these monitoring systems provide any information regarding bandwidth, and bandwidth is a critically important signal coordination characteristic for engineers responsible for the design and assessment of signal timing plans.

CHAPTER 3. SELECT STUDY LOCATIONS (TASK 2)

3.1 Study Site Selection Criteria

The research team selected several closed loop systems in North Carolina based on the following criteria:

- The system is controlled by NC Department of Transportation (NCDOT)
- The signal control system is closed loop
- The site covers a wide range of traffic conditions
- The corridor has at least three coordinated signalized intersections
- The corridor contains a few driveways and un-signalized intersections

3.2 NCDOT Closed Loop System

NCDOT has 422 closed loop signal systems that are controlled and maintained by 14 divisions. Among all, 62 closed loop systems belong to division 5. Division 5 includes Person, Granville, Vance, Warren, Durham, Franklin and Wake County. Table 3.1 presents the number of closed loop systems in each division.

Table 3-1 Summary of NCDOT Closed Loop System

Division	1	2	3	4	5	6	7	8	9	10	11	12	13	14	Total
Number of Systems	10	22	29	35	62	28	25	29	27	49	26	28	30	22	422

Figure 3.1 shows the distribution of closed loop systems in Wake County based on the number of intersections in the system. Wake County has 46 closed loop signal systems and 15 systems include two signalized intersections and 11 systems include three signalized intersections. More than 80% of Wake County closed loop signal systems have less than seven signalized intersections.

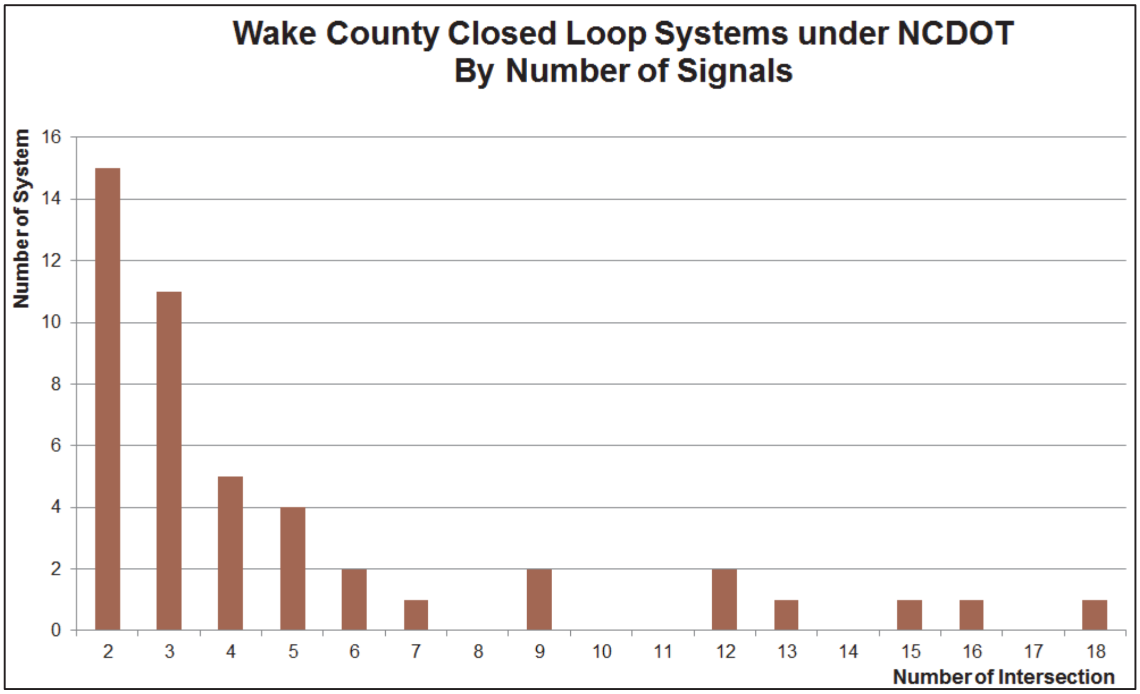


Figure 3-1 Summary of Wake County Closed Loop Systems 2011

3.3 Selected Study Sites

Three signalized coordinated arterials were selected from NCDOTs closed loop system. The first study site is on US 70 in Clayton, NC. This system includes three signalized intersections. The second site is on US 70 in Garner, NC with four signalized intersections. The final site is on NC 55 arterial in Apex, NC with seven signalized intersections. These study sites are shown in Figure 3.2. The arterials in Garner and Apex are in Division 5, Wake County, and the arterial in Clayton is in Division 4, Johnston County.

Site A
- Arterial on US 70 in Clayton, NC
- 3 signalized intersections
- No un-signalized intersections
- 22 driveways
- 3,540 ft. total route length
- Selected plan: AM peak plan
- Cycle Length: 170 seconds
- Intersection 1 offset: 34 seconds
- Intersection 2 offset: 37 seconds
- Intersection 3 offset: 120 seconds

Site B
- Arterial on US 70 in Garner, NC
- 4 signalized intersections
- No un-signalized intersections
- 15 driveways
- 3,840 ft. total route length
- Selected plan: AM peak plan
- Cycle Length: 120 seconds
- Intersection 1 offset: 62 seconds
- Intersection 2 offset: 55 seconds
- Intersection 3 offset: 45 seconds
- Intersection 4 offset: 115 seconds

Site C
- Arterial on NC 55 in Apex, NC
- 7 signalized intersections
- One un-signalized intersection
- 12 driveways
- 7,225 ft. total route length
- Selected plan: AM peak plan
- Cycle Length: 160 seconds
- Intersection 1 offset: 95 seconds
- Intersection 2 offset: 86 seconds
- Intersection 3 offset: 72 seconds
- Intersection 4 offset: 51 seconds
- Intersection 5 offset: 56 seconds
- Intersection 6 offset: 15 seconds
- Intersection 7 offset: 0 seconds

Figure 3-2 Study Sites

3.4 Summary

Project team selected three study sites for field data collection and evaluation of the proposed models. The selected study sites include:

- US 70 arterial in Clayton (including 3 signalized intersections)
- US 70 arterial in Garner (including 4 signalized intersections)
- NC 55 arterial in Apex (including 7 signalized intersections)

CHAPTER 4. DATA COLLECTION PLAN AND FIELD STUDY (TASKS 3 AND 4)

The NCDOT closed-loop signal systems for the most part utilize OASIS^{TM} for closed loop signal systems operations. This system provides log data for monitoring the system's operational status. The research team focused on these log files to develop a signal system performance evaluation method.

OASIS is a traffic control firmware developed by Econolite for implementation in an Advanced Transportation Controller (ATC) Type 2070 published by AASHTO, ITE, NEMA, and CALTRANS. NCDOT's effort to transition all state maintained systems to OASIS system should enable the streamlining of access to data for state maintained signals and systems.

4.1 Available Data Sources

OASIS provides seven system event log file histories, which are shown in Table 4.1. These include: system alarms, special events, front panel data entry, coordination plans, implemented functions, split monitoring, and detector count station data. These system logs are stored in the non-volatile RAM memory and can be cleared upon upload from a central computer. Among those seven logs, the log data of interest for this research are detector event data and split monitoring log data, both of which will be used to measure arterial operational performance. Figure 4.1 show the OASIS system overall structure.

4.2 Testing Data Sources

As mentioned earlier, OASIS provides log files which include important signal operation and arterial demand information. Prior to using it, however, a quality test on the log data is required before generating any data collection plan. The log data quality test was conducted at NCDOT's signal lab with assistance from an NCDOT signal engineer.

Table 4-1 Available OASIS Log Files

Logs	Data Type	
System Alarms Log	• Detector Failures • Hardware Failures	• Phase Conflict • Logs Full
Special Events Log	• Stop Time • Police Switch	• Preemptions
Front Panel Entries Log	• Data Element modified • Old data value • New data value	• Current user • Timestamp
Coordination Plans Log	• Source of plan implementation • Plan implemented	• Offset • Timestamp
Implemented functions Log	• Source of function implementation • Function implemented	• Timestamp
Split Monitor Log	• Active Vehicle Phases • Active Vehicle Phases State • Active Pedestrian Phases • Active Pedestrian Phases State • Active Overlaps • Active Pedestrian Overlaps • Coordination Plan	• Local Clock • Offset • Preemptions • Vehicle Calls • Pedestrian Calls • Status Response Packet
Detector Data Log	• Detector Reference • Detector Status • Average Wait • Volume	• Occupancy • Average Speed • Average Gap

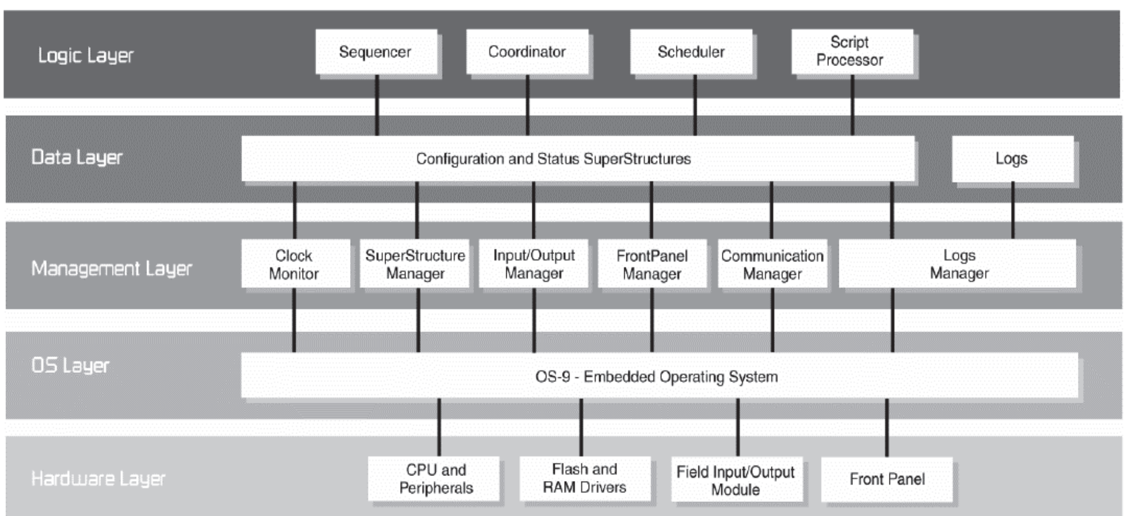


Figure 4-1 OASIS System Overview

4.2.1 OASIS Detector Event Data

Detectors play a critical role in field traffic data collection. All vehicle actuations data are directly obtained from such detectors. Their functionality, configuration, and location determine not only the quantity and quality of traffic data, but also the applicability and validity of resulting performance measurement or models.

OASIS provides detector count station data which consists of three attributes. The first is the detected vehicle volume, the second is the detected occupancy (in %) during a fixed time interval, and the last is the calculated space mean speed over the detector. The project team tested each attribute' resolution to produce an initial assessment of the OASIS logs data quality.

4.2.1.1 Occupancy

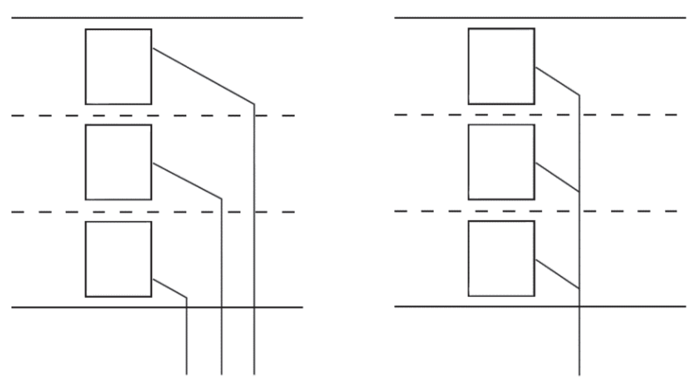
In the OASIS Detector Event log file, the reported occupancy is given in integers. It is necessary to figure out whether the system uses real numbers or integers for occupancy display. From test results, it was confirmed that the system rounds down (or truncates) the true occupancy values. Table 4.2 shows the results of this test based on a one minute occupancy reporting period.

Table 4-2 OASIS Detector Occupancy Display Test Result

Manual input occupancy time entered for detector (Sec)	Reported Occupancy in System (%)	Actual Occupancy (%)
0.9	1	1.50
1	1	1.67
1	1	1.67
1.1	1	1.83
1.2	2	2.00
2.1	3	3.50
2.2	3	3.67
2	3	3.33
2.2	3	3.67
2.2	3	3.67
3.1	5	5.17
3.2	5	5.33
3.2	5	5.33
3	5	5.00
3	5	5.00
0.2	0	0.33
0.5	0	0.83
0.5	0	0.83
0.4	0	0.67
0.5	0	0.83

4.2.1.2 Detector Calls (Volume)

For volume logs, the minimum occupancy time for a vehicle to be detected is 0.1 seconds. Any vehicles which occupy the detector for less than 0.1 seconds will go undetected. In the test, it was confirmed that at less than 0.1 seconds the system would report 0 volume, 0 occupancy and 0 speed. Therefore, for accurate OASIS vehicle volume count, the vehicle must occupy the detector for at least 0.1 seconds. In addition, NCDOT uses two different detector configurations, as shown in Figure 4.2. Configuration will significantly impact the accuracy of the data collected from the field. Lane-based detectors are installed in each lane. Every lane detector has its own wire connected to the back panel of the traffic cabinet, and responds to vehicle actuations separately. Link-based detectors are also installed in individual lanes, however, they are wired together to the traffic cabinet. They can only generate one response and the traffic cabinet cannot distinguish the source of the actuation, i.e. which lane. Link-based detectors can only indicate the presence of the vehicle, so they are also called presence detectors. The detectors installed at the pilot study site (i.e., Timber Drive) are link-based detectors.



A) Lane-based Detector B) Link-based Detector

Figure 4-2 Configurations of Lane-based and Link-based Detectors

The research team compared counts taken at the Timber Dr. intersection on US70 upstream system detectors data with downstream detector counts. From about 6:30 to 9:30 am on November 18, the Timber Dr. WB system detector (a link based detector across 3 lanes) yielded 2,477 vehicles, compared to the downstream detector which counted 4,119 vehicles, about a 70% increase which could not be simply attributed to the neglect of counting side street turn movements. The essence of the undercount is that if a vehicle occupies a linked detector in any lane, any arriving vehicle on the same or another linked lane will not trigger a new detection. Numerical tests using a simple spreadsheet simulation indicated that a 42% undercount would occur when each of the 3 lane average hourly volume is 450 vph. When the hourly volume is increased to 700 vph per lane, the possible undercounting loss goes up to 58%.

4.2.1.3 Speed

Speed is calculated from occupancy, loop detector length and an assumed average vehicle length. For occupancy calculation, the detector system must save the total occupied time or the precise occupancy (not rounded value). The mathematical formula is;

$$\bar{u} = \frac{Total\ travel\ distance}{Total\ travel\ time} = \frac{\#\ of\ vehicles * (detector\ length + vehicle\ length)}{Total\ occupied\ time}$$

$$= \frac{100 * (detector\ length + vehicle\ length) * v_i}{Occupancy * T_i}$$

Where: $\bar{u}$: space mean speed

v_i: vehicle count in time interval i

T_i: data collection time interval i (e.g., 1 minute or 15 minutes)

OASIS rounds down the displayed vehicle occupancy and calculated vehicle speed. This creates a range of possible reported speeds and occupancies that are sorted by the corresponding displayed occupancy. Table 4.3 shows the speed ranges for each occupancy range for a data log based on a one minute data collection using one vehicle.

Table 4-3 Displayed Occupancy and Possible Speed Rage by Displayed Occupancy

Input Time (sec)	Displayed Occupancy (%)	Possible Maximum displayed speed (mph)	Possible Minimum displayed speed (mph)
0.1 to 0.5	0	177	35
0.6 to 1.1	1	29	16
1.2 to 1.7	2	14	10
1.8 to 2.3	3	9	7
2.4 to 2.9	4	7	6
3.0 to 3.5	5	5	5
3.6 to 4.1	6	4	4
4.2 to 4.7	7	4	3
4.8 to 5.3	8	3	3
5.4 to 5.9	9	3	3
6.0 to 6.5	10	2	2
6.6 to 7.1	11	2	2
7.2 to 7.7	12	2	2
7.8 to 8.3	13	2	2
8.4 to 8.9	14	2	1
9.0 to 9.5	15	1	1
⋮	⋮	⋮	⋮
17.4 to 17.9	29	1	0

A range of speeds varying from 35 to 177mph may be displayed for a 0% occupancy on the system, under the detected single vehicle volume, assuming a 20 feet default vehicle length with a 6ft loop detector length. For more accurate speed estimation from occupancy, the occupancy should include at least one decimal value, and not rounded down to an integer.

4.2.1.4 *Vehicle occupancy in boundary between time intervals*

When a vehicle occupies the detector both before and after a data collection interval, OASIS reports a zero volume before and a volume of one in the after time interval (see Table 4.4). It then calculates both time periods' occupancies independently. For speed calculations, OASIS returns a zero speed in the before interval and a calculated speed from after occupancy time.

Table 4-4 Boundary Condition Test Result

Clock	Approximate input signal (sec)		Volume	Occupancy (%)	Speed (mph)
18:22	2	0.5	0	0	0
18:23		1.5	1	2	11
18:27	4	1.5	0	2	0
18:28		2.5	1	4	7
19:15	10	4.7	0	7	0
19:16		5.3	1	8	3

4.2.2 OASIS Split Monitor Data

The TransLink 32 software is required to download the OASIS log file. It is designed to monitor the 170 and 2070 master and local controllers by Econolite. Through TransLink 32, the OASIS log file can be manually or automatically downloaded and archived into the Access database. The OASIS split monitor reports the following: "Time Stamp", "Cycle", "Offset", "Plan", "CoordPhases", "ExtraTimeCP", "UsedPhase" and "AllottedPhase" as explained next.

- SMTimeStamp: Start time of each cycle (reference point)
- Cycle: Coordinated cycle length

- Offset: A time relation (local clock) with regard to the system time reference (master clock) to indicate where, in that cycle, the intersection begins / ends its coordinated phase(s) Green (main street Green)
- CoordPhases: Coordinated phase number (e.g., CoordPhases2, CoordPhases6)
- ExtraTimeCP: Remaining time in the current cycle allocated to the coordinated phase
- UsedPhase: Actual green time used in the current cycle (After reference Point)
- AllottedPhase: Maximum phase time, which includes yellow and red times

OASIS use standard National Electrical Manufacturing Agency (NEMA) phase numbers. Figure 4.3 shows a sample intersection phase sequence with each movement number.

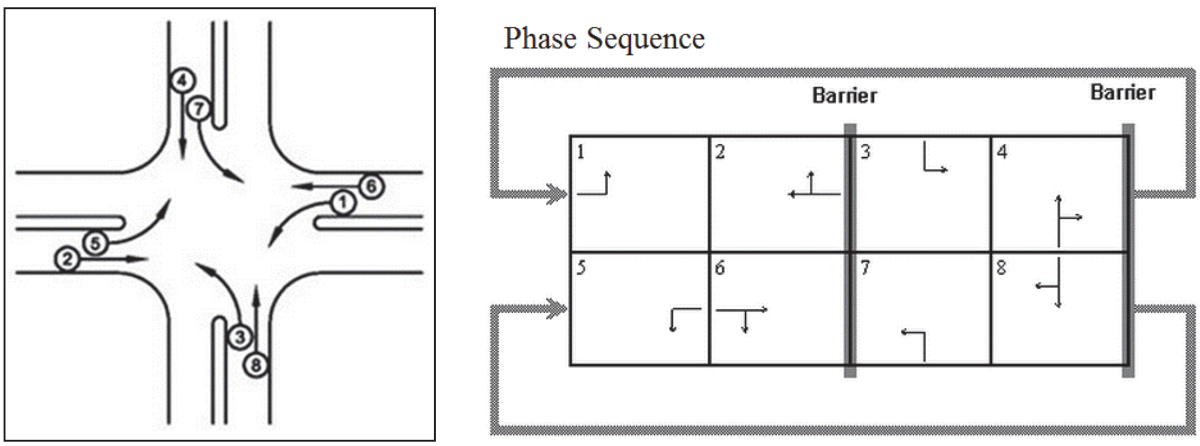


Figure 4-3 NEMA Phase with Phase Sequence

It is possible to build programmed signal phases and cycle by cycle real dynamic phases from the split monitor data. Those results are shown in Figure 4.4 and Figure 4.5.

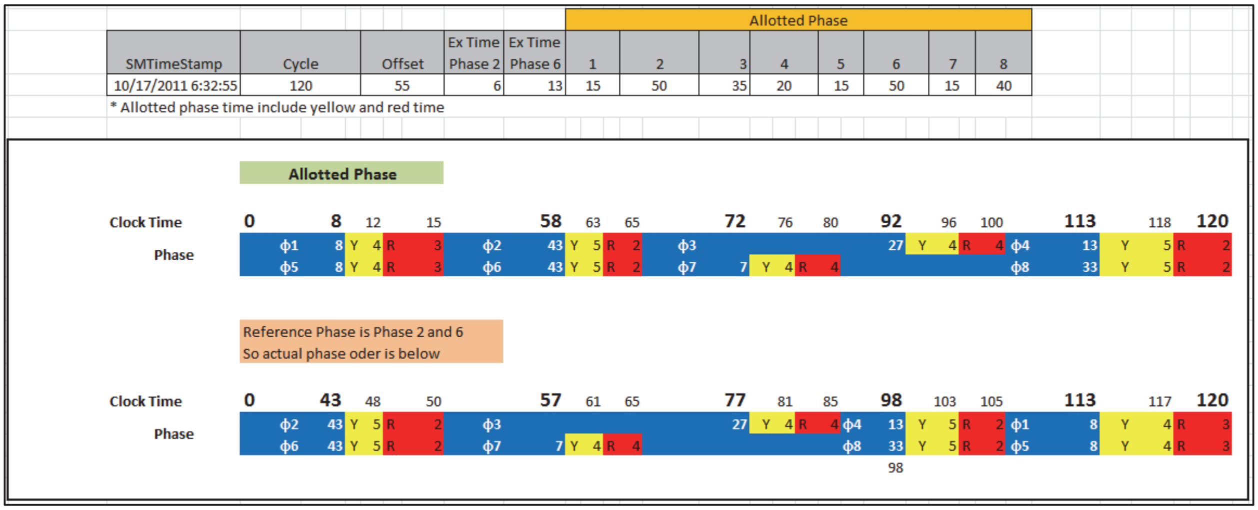


Figure 4-4 Programmed Intersection Signal Phases

Dynamic green can be generated from the “Time Stamp” (reference point) and “UsedPhase” with “ExtraTimeCP” which indicates how early the green is returned, compared to the reference point. As an example, Figure 4.5 shows the Split Monitor log table with dynamic phase sequences which are created from the Split Monitor log.

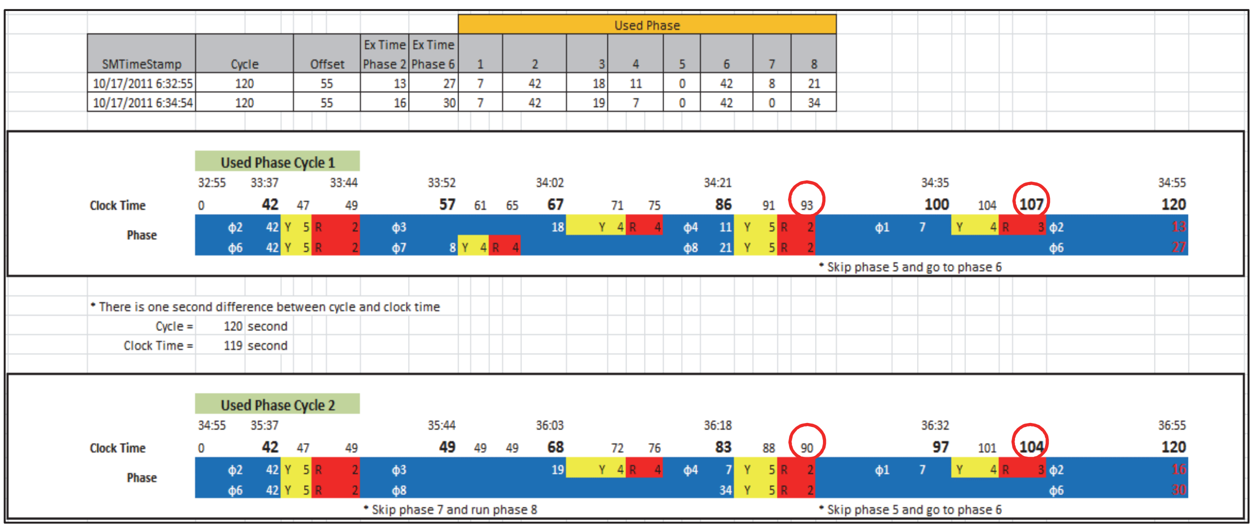


Figure 4-5 Dynamic Intersection Phases

The coordinated phases are phases 2 and 6. During the first cycle length that is 120 sec long, there is unused time that goes to ExtraTimeCP. This includes 13 seconds from phase 1 (which terminates at 107 seconds) and 27 seconds from phase 5 (which is skipped) and phase 8 is terminated at 93 seconds (with 27 seconds remaining). Phases 2 and 6 greens will then have early returns by those amounts of time.

However, it should be notes that the ExtraTimeCP is not exactly the same as early return to green when lead-lag phasing is used or with lead-lead phasing with different greens for phase 1 and phase 5. Phase 1 time will go to "ExtraTimeCP6" for phase 6 and the remaining time which is phase 6 green time minus phase 1 will go to phase 6 "UsedPhase" (see Figure 4.6).

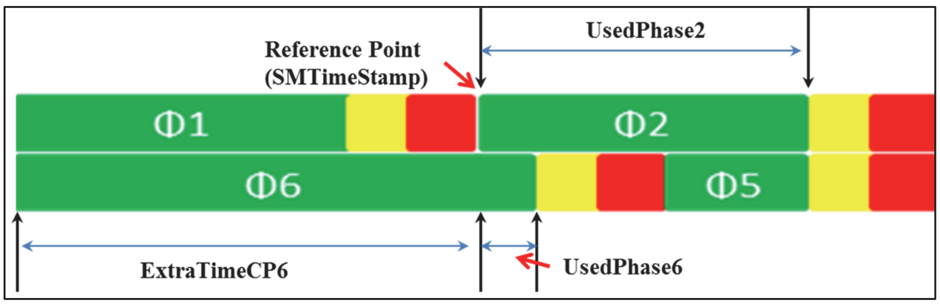


Figure 4-6 Lead-Lag phase

The coordinated phase green start clock time is the reference point time minus the previous cycle's "ExtraTimeCP"; the green terminated clock time is the reference point time plus the current cycle's "UsedPhase". This rule is not affected by the left turn phase sequence.

At that point, the research team was sufficiently familiar with both the capabilities and limitations of the OASIS log data files to begin planning for a pilot data collection, as described in the next section.

4.3 Data Collection Plan

The data collection plan involves data from a variety of sources and methods. Those included OASIS log data as discussed earlier, through travel times on the arterial, and vehicle-level high resolution speed and count data. For that purpose, the team relied on the Translink32 software, Bluetooth units, Tru-Traffic software with GeoStats devices, video cameras with recorders, RTMS unit and Sensys wireless devices for collecting those data. We describe each data source in some detail in the next sections.

4.3.1 OASIS Log Data

OASIS log data provides signal status and demand information. The OASIS split monitor and detector log data were collected by the TransLink 32 software for a period exceeding two weeks, then downloaded and archived in a Microsoft Access database. Figure 4.7 shows a screenshot of the TransLink32 interface with the scheduler.

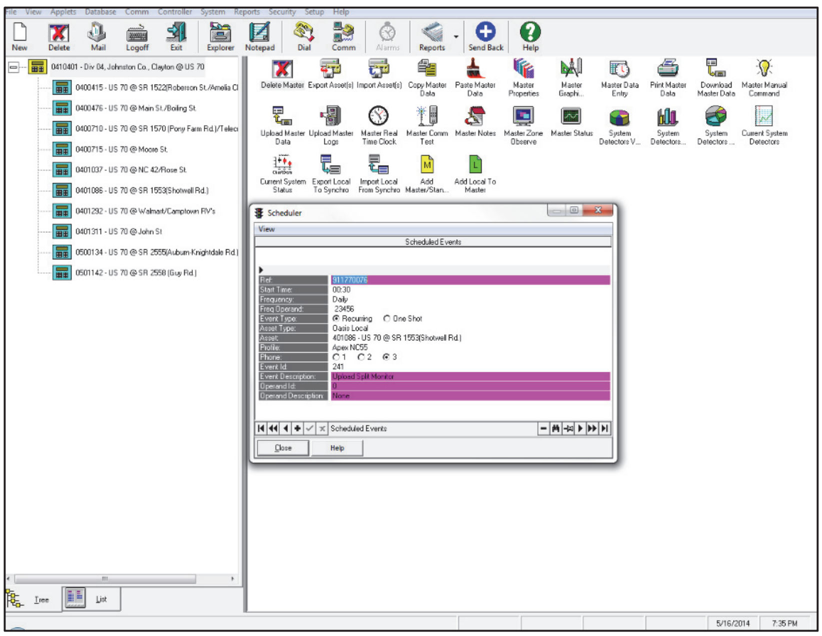


Figure 4-7 Translink32 Display

4.3.2 Arterial Travel Time Data

4.3.2.1 Bluetooth Devices

Two types of Bluetooth units, including BluFax and BlueMAC were used in this project. Both collect media access control (MAC) addresses from Bluetooth-enabled devices such as cell phones, GPS-based navigation systems, car radios and personal digital devices. MAC addresses are unique to a particular Bluetooth device consisting of 12 characters (48-bit address). The arterial travel time and space mean speed are calculated by detecting the signals of the same MAC address at two locations and subtracting the times of detection.

4.3.2.1.1 BluFAX Model

BluFAX is a portable Bluetooth traffic data collection device developed by Traffax (http://www.traffaxinc.com/) and is shown in Figure 4.8. The portable unit is powered by a battery and last 12 to 17 days without recharging according to test result by project team. However, Traffax also develops permanently mounted products. It has a GPS module used to keep record of the location where the unit is installed along the road and also to synchronize the clock time of the unit. BluFAX uses a Class 1 power transmission omnidirectional antenna, and Traffax recommends that the antenna be placed three meters above the ground. The unit archives 12 characters of all MAC addresses in a removable micro SD card placed in control box. Traffax provides the BluSTATS software to match, filter and display travel time and speeds derived from the BluFAX collected MAC addresses. BluFAX units have been used to collect travel time data at freeway as well as arterial. Traffax reports that a 2 to 3% sample of the total volume of traffic on a road (roughly corresponding to three samples MAC addresses per five minutes) should be sufficient to obtain meaningful travel time estimates.

Figure 4-8 BluFAX Unit

The BluFAX Units can be installed either in the median or on the roadside, as their detection radius is typically rage 150 feet to 200 feet. In order to capture the back of the queue for a more accurate travel time measurement, the units are put generally 1,000 to 2,000 feet away from the intersection. The devices should be attached to a permanent pole with a chain and lock. The installation of each device takes less than ten minutes, and most of the time is consumed carrying the units from the parking lot to the desired location. After placement, the technician should make sure that all four indicator lights (Power, GPS, Bluetooth, and SD Card) turn on for about 5 minutes after turning on the Bluetooth switch.

4.3.2.1.2 BlueMAC Models

BlueMAC is developed by Digiwest (http://www.mybluemac.com/). A BlueMAC unit consists of a solar panel, battery, and main control box including an Ethernet port. It collects MAC addresses just like the BluFAX unit, but truncates the first 5 digits and the last digit of the MAC address to ensure privacy. Figure 4.9 shows BlueMAC unit.

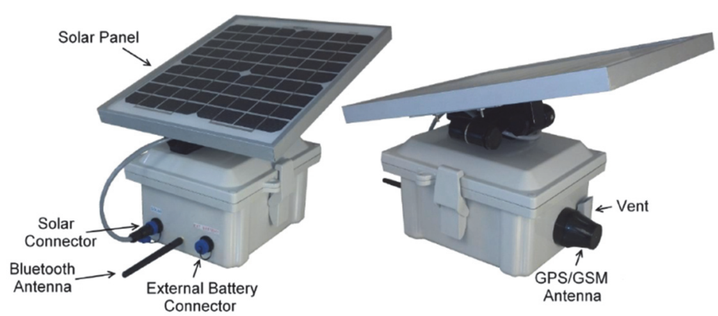


Figure 4-9 Gen 5 BlueMAC Unit

The unit is operated by a stand-alone battery and solar panel so it is suitable for permanent travel time data collection. Digiwest provides online data web-services. They also support user specific project websites as shown in Figure 4.10. The listed projects are arterial and freeway travel time data collection activities conducted by the project team.

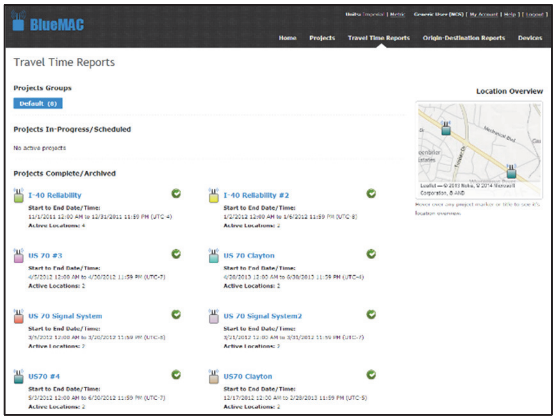


Figure 4-10 Digiwest User Customized Online Website

The user can monitor each unit's activation status as shown in Figure 4.11. The collected data is routed to a server which automatically processes the travel time (and space mean speed) and creates an origin-destination report.

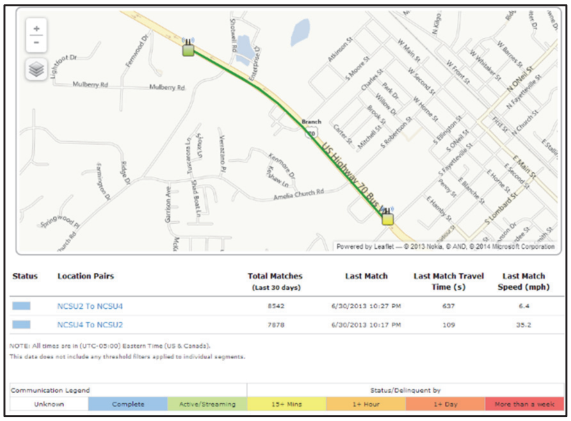


Figure 4-11 BlueMAC Device Location and Status Map

4.3.2.2 Floating Car Data Collection

Floating car is a true and tried method to collect travel time (or speed) on the road network. It gathers instantaneous speed, direction of travel and clock time information from a Global

Positioning System (GPS) enabled device in the vehicle. Equipment needed to carry out this study includes a reliable test vehicle, one GeoStats, Geologger, TravTime software, and a computer with USB connectivity. With regard to personnel, a minimum of two people are needed to properly carry out the study. One person drives the test vehicle according to the method outlined later in the methodology, and the other person will ensure that the Geologger is functioning properly and that no biases are introduced to the study. Biases could include construction in the area, lane closures, accidents, or biases caused by the test vehicle driver. The other role of the second personnel is to ensure that the study starts at the time that has been determined. In addition to these roles, a third person will hand-record event times using a stopwatch to check the data that the Geologger outputs after the study is completed. This measure ensures that there was no malfunction or misuse of the Geologger during the study.

The test vehicle technique used for this travel time study is the average vehicle method. The vehicle travels according to the driver's perception of the average speed of the traffic stream on study site arterial. The data from the Geologger will be extracted into a computer using a universal serial bus (USB) port and analyzed. The Geologger will store data from the runs such as: GPS position, date, time, latitude, longitude, and speed. This data will then be loaded into a software program called TravTime for further analysis. In TravTime, it is possible to upload shape files and maps of the study location and match the data from the Geologger to the map. The road network can be either link based (segment by segment) or point based. In TravTime, total route analysis can be performed in addition to segment by segment analysis. These analyses will generate performance measures such as: average travel time, average travel speed, average number of stops, and average stopped time. The average travel time, and average travel speed can then be checked using the hand recorded times to ensure the general accuracy of the values. With the data collected and verified, proper analysis of the performance of the study area, with regard to travel time, can be done.

4.3.3 High Resolution Vehicle Data

As mentioned earlier, OASIS split monitor log provides detailed information of signal status while the detector log, at its highest resolution, produces only one minute aggregated count data. High resolution data is needed to verify the level of vehicle platooning, and headway times to assess the data loss on link based detectors. These types of data can only be collected with specialized equipment, as explained next.

4.3.3.1 RTMS Side Fire Radar Data

The RTMS (Remote Traffic Microwave Sensor) measures the distance to objects in the path of its microwave beam. A single sensor can monitor traffic in up to 12 lanes. The sensor can be mounted on road-side poles and aimed at a right angle to the road; this is referred to as the side-fired configuration. The internal processor calculates volume, occupancy, average speed, and vehicle classifications for each lane and transmits the information using its data ports and communication interfaces. Vehicles are detected when their reflected signal exceeds the background level in their micro-slice by a certain threshold. If that detection is part of a defined zone, its contact is closed during the detection period to indicate detection. Figure 4.12 shows the RTMS microwave beam footprint, while Figure 4.13 depicts its installation at the Timber Dr. intersection on US 70.

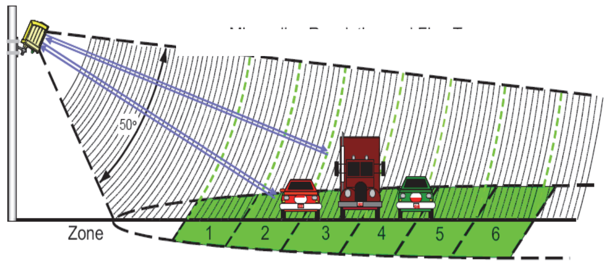
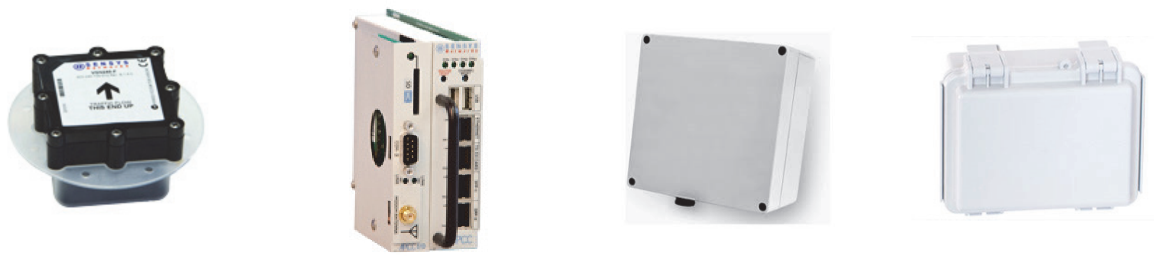


Figure 4-12 RTMS Microwave and Beam Footprint

Figure 4-13 RTMS Field Installation

4.3.3.2 Sensys Wireless Sensor Data (http://www.sensysnetworks.com/)

With the intention to collect high resolution vehicle actuation data, the project team acquired and installed 24 Sensys wireless sensors with two APCC's, two Access Points and 10 repeaters. The Sensys wireless detectors are developed by Sensys Networks[@] and use magneto-resistive sensors embedded in the pavement to detect individual vehicle actuations. The sensors are self-powered and have two-way low-power radio communication capabilities. An access point serves as the wireless bridge between the sensor and a contact closure card, which can be installed in a standard detector rack of a controller cabinet. When the sensor is located out of range of the access point, wireless repeaters can be installed between the access point and the sensors to extend the communication. Figure 4.14 shows the various device components.

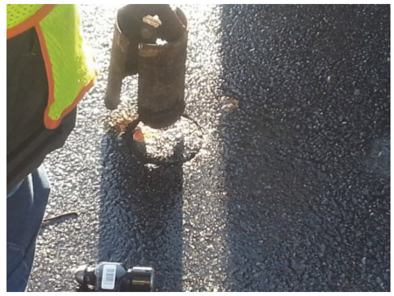


Sensor APCC Access Point Repeater

Figure 4-14 Sensys Devices

The twenty-four Sensys sensors were installed in the pavement by coring 4-inch diameters where the detector is placed, which are then covered with epoxy. NCDOT Division 4 and the Traffic Control team helped coring and installing the sensors. Two APCC's were installed in the traffic controller cabinet at the Shotwell Rd. intersection and S. Robertson St. intersection on the US 70 arterial. Two Access Points and 10 Repeaters were installed 20 feet from a wooden power pole on the street. Figure 4.15 shows the installation process.

1



2

3

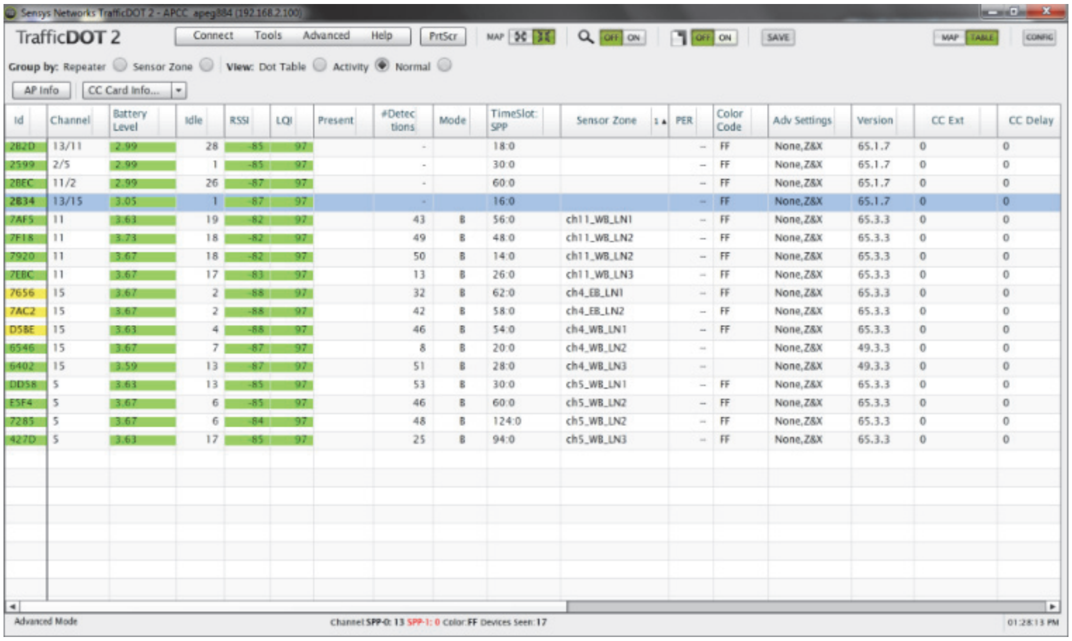
4

5

6

Figure 4-15 Sensys Sensor Installation

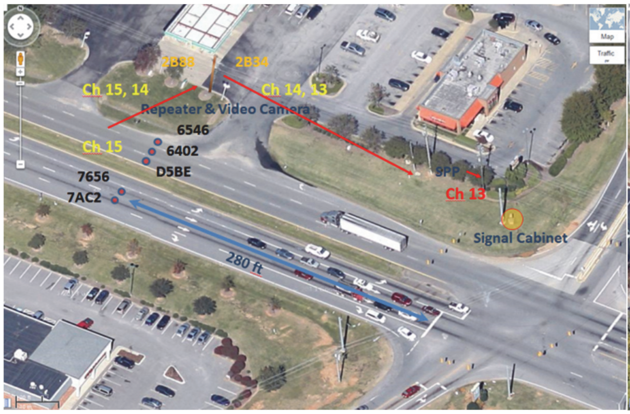
Each approach lane at each intersection was equipped with a sensor. After completing the installation, each repeater and sensor status can be monitored by TrafficDOT2 software produced by Sensys Networks. Figure 4.16 shows Traffic DOT 2 screen. Figure 4.17 depicts all the devices and sensors' locations in the field.



Id	Channel	Battery Level	Idle	RSSI	LQI	Present	#Detections	Mode	TimeSlot: SPP	Sensor Zone	PER	Color Code	Adv Settings	Version	CC Ext	CC Delay
2B2D	13/11	2.99	28	-85	97		-		18:0		–	FF	None,Z&X	65.1.7	0	0
2599	2/5	2.99	1	-85	97		-		30:0		–	FF	None,Z&X	65.1.7	0	0
2BEC	11/2	2.99	26	-87	97		-		60:0		–	FF	None,Z&X	65.1.7	0	0
2B34	13/15	3.05	1	-87	97		-		16:0		–	FF	None,Z&X	65.1.7	0	0
7AF5	11	3.63	19	-82	97		43	B	56:0	ch11_WB_LN1	–	FF	None,Z&X	65.3.3	0	0
7F18	11	3.73	18	-82	97		49	B	48:0	ch11_WB_LN2	–	FF	None,Z&X	65.3.3	0	0
7920	11	3.67	18	-82	97		50	B	14:0	ch11_WB_LN2	–	FF	None,Z&X	65.3.3	0	0
7EBC	11	3.67	17	-83	97		13	B	26:0	ch11_WB_LN3	–	FF	None,Z&X	65.3.3	0	0
7656	15	3.67	2	-88	97		32	B	62:0	ch4_EB_LN1	–	FF	None,Z&X	65.3.3	0	0
7AC2	15	3.67	2	-88	97		42	B	58:0	ch4_EB_LN2	–	FF	None,Z&X	65.3.3	0	0
D5BE	15	3.63	4	-88	97		46	B	54:0	ch4_WB_LN1	–	FF	None,Z&X	65.3.3	0	0
6546	15	3.67	7	-87	97		8	B	20:0	ch4_WB_LN2	–		None,Z&X	49.3.3	0	0
6402	15	3.59	13	-87	97		51	B	28:0	ch4_WB_LN3	–		None,Z&X	49.3.3	0	0
DD58	5	3.63	13	-85	97		53	B	30:0	ch5_WB_LN1	–	FF	None,Z&X	65.3.3	0	0
E5F4	5	3.67	6	-85	97		46	B	60:0	ch5_WB_LN2	–	FF	None,Z&X	65.3.3	0	0
7285	5	3.67	6	-84	97		48	B	124:0	ch5_WB_LN2	–	FF	None,Z&X	65.3.3	0	0
427D	5	3.63	17	-85	97		25	B	94:0	ch5_WB_LN3	–	FF	None,Z&X	65.3.3	0	0

Figure 4-16 TrafficDOT 2 Sensor Status

In addition, three carefully placed speed traps (devices in tandem on the same lane) were deployed on the arterial segment, also shown in Figure 4.17 (parts b, c, and d). The sensors actuation data are transmitted through the repeater and archived in an SD card in the APCC.



(a) Shotwell Rd. West

(b) Shotwell Rd. East

(c) S Moore St. West

(d) S Moore St. East

(e) S. Robertson St. West

(f) S. Robertson St. East

Figure 4-17 Sensys Sensors and Each Device Locations with IDs

4.3.3.3 Video Data

Video cameras and Digital Video Recorders (DVR) were also installed at the Timber Drive intersection on US 70. The purpose of collecting video data was essentially to verify the OASIS loop detector log data as well as the accuracy of the RTMS unit. Cameras were mounted on the power pole located at the Timber Drive and US 70 intersection. Figure 4.18 shows the video camera installation at a vantage point.

(a) Mounting Cameras (b) Mounted Cameras on the Power Pole

Figure 4-18 Video Camera Installation

The cameras and DVRs recorded all approach lanes' vehicle movements and the collected data was processed both by Auto Scope (a video image processing system) as well as manually.

The project team anticipates conducting an initial study at a single site in order to test and refine the data collection procedures. The first selected site is US 70 arterial in Garner, NC. The research team conducted an intensive field study on this site and then refined the data collection method and procedures.

4.4 Study Data Collection Sites:

4.4.1 Pilot Site: US 70 Arterial in Garner

The first field study was conducted on US 70 in Garner, NC during March and April 2012. Figure 4.19 depicts the location of and spacing between the system intersections. Figure 4.20 shows the existing detector configuration at each intersection. Simultaneously during those same two months, data were collected from the OASIS log, Bluetooth devices, floating cars, and RTMS (Figure 4.12). In addition, four video cameras with two DVR's (Figure 4.18) were installed at the Timber Dr. intersection for recording vehicle movements.

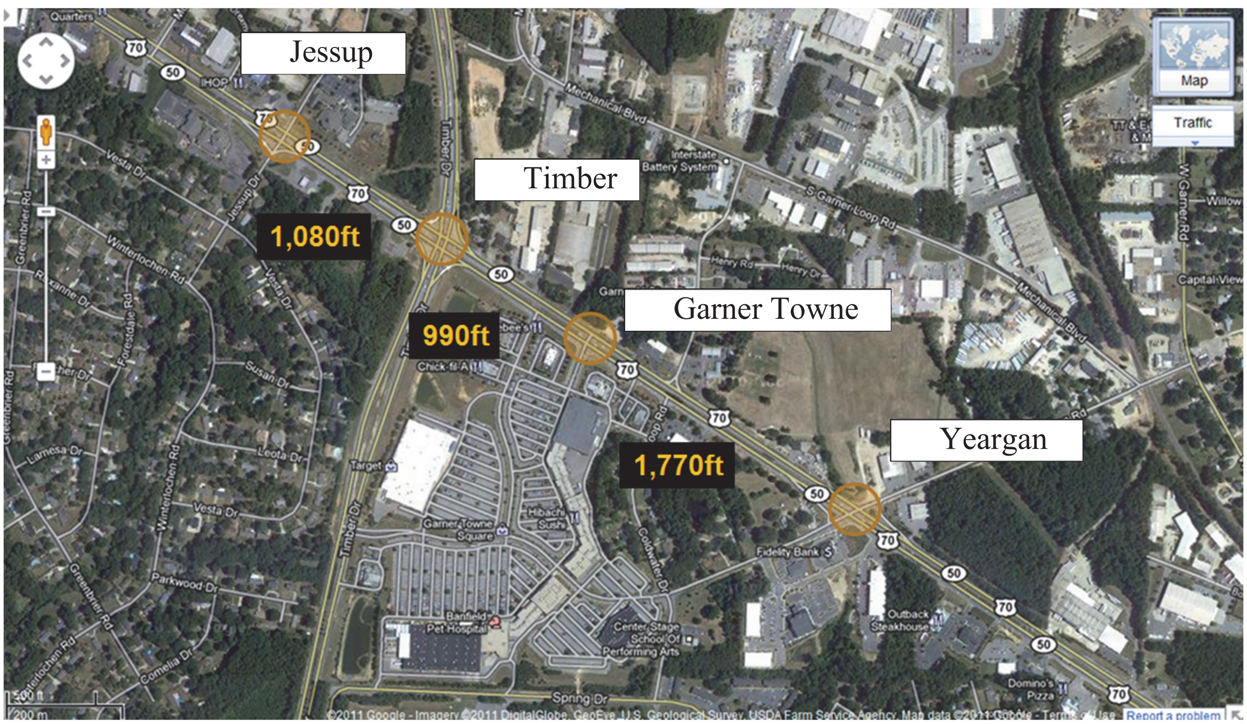
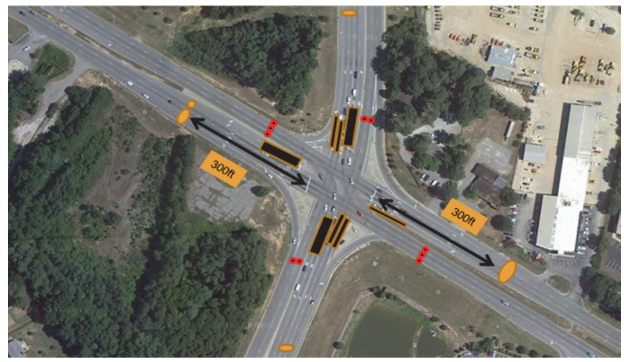


Figure 4-19 Pilot Study Site in Garner, NC

Jessup Dr. & US 70 has 5 system detectors and 3 long loop detectors on the side street. The Timber Dr. intersection has 5 system detectors and 10 downstream detectors on each approach lane. All system detectors except for the first EB lane detector at the Timber Dr. intersection are link detectors covering two or three lanes, thus unable to provide lane volumes or occupancies. There are also 8 long loop detectors for non-coordinated movements. The Garner Towne Square intersection has 5 system detectors and 3 long loop detectors. The Yeargan road intersection has 5 system detectors and 8 long loop detectors. Except for the Timber Dr. system detectors, all system detectors and downstream detectors do provide lane by lane volume and occupancy. All of the system detectors are deployed 300ft upstream of the approach intersection stop line.

Jessup Dr. & US 70



Timber Dr. & US 70

Garner Towne Square & US 70

Yeargan Rd. & US 70

Figure 4-20 US 70 Arterial Intersection Geometry with Detector Location

Five Bluetooth units (see Figure 4.21) were deployed on the US 70 arterial to measure arterial travel time. Each Bluetooth location is depicted in Figure 4.21. Spacing between BT1 and BT2 is 0.23 miles, BT2 and BT3 0.25 miles, BT3 and BT4 0.23miles and BT4 and BT5 0.34Miles. The total distance between BT1 to BT5 is 1.05 miles. The data and findings at this pilot site are described in detail in Chapter 5.

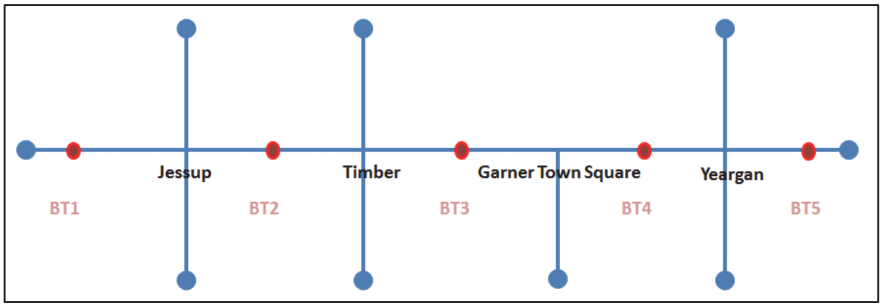


Figure 4-21 Bluetooth Locations on US 70 Arterial in Garner

4.4.2 Other Data Collection Sites

As mentioned in Chapter 3, the project team along with NCDOT selected three study sites. The other two sites are on the NC 55 arterial in Apex, NC and US 70 in Clayton. Detailed site information was provided in Figure 3.2 in Chapter 3.

Similar to the pilot site, the team collected OASIS split monitor log, detector log as well as Bluetooth travel time and INRIX TMC travel time data for the NC 55 arterial in Apex.

The US 70 arterial in Clayton was selected as the test bed for high resolution data collection. Sensys wireless sensors with APCCs, Repeaters and Access Points were installed on the arterial. In addition, both BluFAX and BlueMAC devices are used for monitoring and collecting travel time data.

4.5 Summary

In this chapter, the available data sources from current NCDOT closed loop systems are investigated. Additional data collection systems and plans were also introduced.

The OASIS system temporally archives seven log files in its RAM. Archived log files can be downloaded using the TransLink 32 software. The project team focused on “Split Monitor Logs” and “Detector Logs”. Split monitor logs provide each phase allotted green time and displayed green time in the field. Those data indicate the dynamic green duration and sequence which is directly related with each phase's dynamic g/C and capacity. Detector logs provide a minimum one minute resolution of vehicle counts (or calls) along with occupancy and speed. The quality of detector logs depends on the detector configuration. Lane-based detectors provide more accurate information compared to link-based detectors since they avoid the undercount of link based detectors. In addition, OASIS detector logs for both occupancy and speed round-down their calculations when the system archives its calculations.

GPS enabled floating car and Bluetooth devices are used for arterial travel time and speed monitoring. RTMS and video cameras with DVR were deployed for testing the quality of the OASIS log file. In addition, Sensys wireless sensors are installed on the US 70 arterial in Clayton for contrasting the capability of high resolution data against the coarser aggregated data produced by other devices.

Three arterial field studies were conducted during several months in 2012 and 2013. The project team conducted the first field study on the US 70 arterial in Garner where it collected OASIS log data, RTMS unit data, video data, Bluetooth travel times, GPS enabled floating car travel time and speed data.

The second field study was conducted at the NC 55 arterial in Apex. All signalized intersection OASIS log data, Bluetooth arterial travel time data and INRIX TMC data were collected. The third field study site was on US 70 in Clayton. At that site, the project team collected high resolution detector data using Sensys wireless sensors along with OASIS log data, and Bluetooth travel time data. Findings from all three sites are provided in the next few chapters.

CHAPTER 5. INVESTIGATE RELATIONSHIPS AND DEVELOP CANDIDATE MODELS (TASK 5)

The US 70 arterial in Garner, NC is selected as a pilot study site. The purposes of selecting the pilot study site are:

- Conducting a detail travel speed (time) study,
- Monitoring OASIS Split Monitor Logs,
- Monitoring OASIS Detector Log and data quality test
- Development of a possible monitoring model,

In this Chapter, OASIS log data for the selected pilot sites, current timing plans, and arterial travel times (speed) are presented. Several performance monitoring methods for intersection and arterial streets are introduced in this Chapter.

5.1 Current Signal Timing Plan

The US 70 arterial in Garner, NC consists of four intersections: Jessup Dr, Timber Dr, Garner Towne Square, and Yeargan Rd. intersections. All weekdays (Monday to Friday) have the same time of day plan: AM Peak, Midday, and PM peak plans. The intersection spacing is 1,080ft, 990ft and 1,770ft from Jessup to Yeargan. Table 5.1 shows US 70 Garner arterial time of day plan.

Table 5-1 US 70 Garner Arterial Time of Day Plan

	Plan	Plan Start time	Plan End time	Cycle (s)	Offset (s)				Plan Number
					Jessup	Timber	Garner	Yeargan	
Monday to Friday	1	6:15:00	11:00:00	120	62	55	45	115	10-1
	2	11:00:00	16:00:00	170	0	10	155	70	23-1
	3	16:00:00	19:00:00	170	145	0	10	15	21-1

A common cycle length of 120 seconds is used for the AM peak plan. For the midday and PM peak plans the common cycle length is 170 seconds. The Time Of Day (TOD) plan starts at 6:15 AM and ends at 19:00 PM. Table 5.2 through Table 5.5 show intersection signal timing parameters (cycle, split and phase sequence) for the US 70 arterial. The coordinated phases are phase 2 and 6 which are East and West directions.

Table 5-2 Jessup Dr. Intersection Programmed Signal Timing

Plan Number	Phase Sequence		Split (s)								Cycle (s)
	Phase 1	Phase 5	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8	
10-1	Lag	Lead	20	75	0	25	20	75	0	25	120
23-1	Lead	Lead	25	125	0	20	15	135	0	20	170
21-1	Lead	Lead	30	120	0	20	20	130	0	20	170

Table 5-3 Timber Dr. Intersection Programmed Signal Timing

Plan Number	Phase Sequence		Split (s)								Cycle (s)
	Phase 1	Phase 5	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8	
10-1	Lead	Lead	15	50	35	20	15	50	15	40	120
23-1	Lag	Lead	20	77	25	48	16	81	48	25	170
21-1	Lead	Lag	20	77	25	48	16	81	48	25	170

Table 5-4 Garner Towne Square Intersection Programmed Signal Timing

Plan Number	Phase Sequence		Split (s)								Cycle (s)
	Phase 1	Phase 5	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8	
10-1	Lead		20	75	0	25	0	95	0	0	120
23-1	Lead		25	120	0	25	0	145	0	0	170
21-1	Lead		25	120	0	25	0	145	0	0	170

Table 5-5 Yeargan Rd. Intersection Programmed Signal Timing

Plan Number	Phase Sequence		Split (s)								Cycle (s)
	Phase 1	Phas e 5	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phas e 8	
10-1	Lag	Lead	25	60	0	35	17	68	0	35	120
23-1	Lag	Lead	25	105	0	40	25	105	0	40	170
21-1	Lead	Lag	25	110	0	35	25	110	0	35	170

5.2 Travel Time Studies

Travel time on the study site data was collected during two weeks form March 12, 2012 to March 23, 2012. Travel time data methods are:

1- Floating car

2- Bluetooth

3- INRIX

5.2.1 Floating Car

Floating car travel times are collected along the subject arterial by using Geologger, Tru-Traffic, and TravTime software. Table 5.6 shows the result of 21 travel time runs for the EB and 22 runs for the WB in AM peak (7am to 9am). In order to properly analyze the travel time statistics, two scenario maps were created for segmenting runs. The first map began at an arbitrary distance upstream of Jessup St. and ended at an arbitrary distance downstream of Yeargan for the eastbound direction. This method was reversed for the westbound direction. However, the arbitrary distance contributed to several problems in estimating travel time statistics. First, possible routes that began or ended after or before the beginning and end of the routs may have been skipped. These arbitrary distances would also yield an unrealistic travel time between Jessup and Yeargan as the distance travelled would be longer. In order to determine correct travel time statistics, a second scenario map was created with segments that began and ended at Jessup and Yeargan, respectively. This way, reliable travel time information was provided for the distance between the two intersections. However, this scenario would ignore vehicles that had to stop at Jessup heading Eastbound and Yeargan heading Westbound, which would be detrimental to analyzing the signal coordination. When looking at the first scenario map runs, it could be determined that four runs had to stop at these intersections, three at Jessup, and one at Yeargan. Using the results produced by the first scenario, the stopped times at these intersections for these runs were added to the travel times of the runs produced by the second scenario. This produced accurate travel time runs for the corridor for Jessup through to Yeargan Eastbound on US 70 and Yeargan to Jessup Westbound on US 70. It also took into account the delay at the first intersections and added it to the total travel time, which also allowed for proper evaluation of the signal coordination.

Table 5-6 Floating Car Travel Time for Full Segment

		US 70 EB (sec)	US 70 WB (sec)
Sample Size		21	22
Average Travel Time		76.9	75.5
Number of Stop	**0 stops**	58.66	67.29
	1 stop	102.68	83.9
	2 stops	-	111.6
Average Speed		33.41	34.06
Number of Stop	**0 stops**	43.36	39.13
	1 stop	24.85	30.27
	2 stops	-	23.07
Minimum Travel Time		46.2	49.8
Maximum Travel Time		123	112.2
Average Number of Stop		0.52	0.5
Average Stopped Time		12	4.8
LOS		C	B
TTI Index		1.35	1.32

The total number of stops was increased from eighteen in the first run to 22 in the last run. The average number of stops and average stopped times were adjusted. All other average totals were not adjusted as the effects on them would be negligible. The results exported from TravTime and adjusted for accuracy are displayed Table 5.6. Figure 5.1 and Figure 5.2 shows the travel time distributions associated with the number of stops on the eastbound and westbound approaches, respectively.

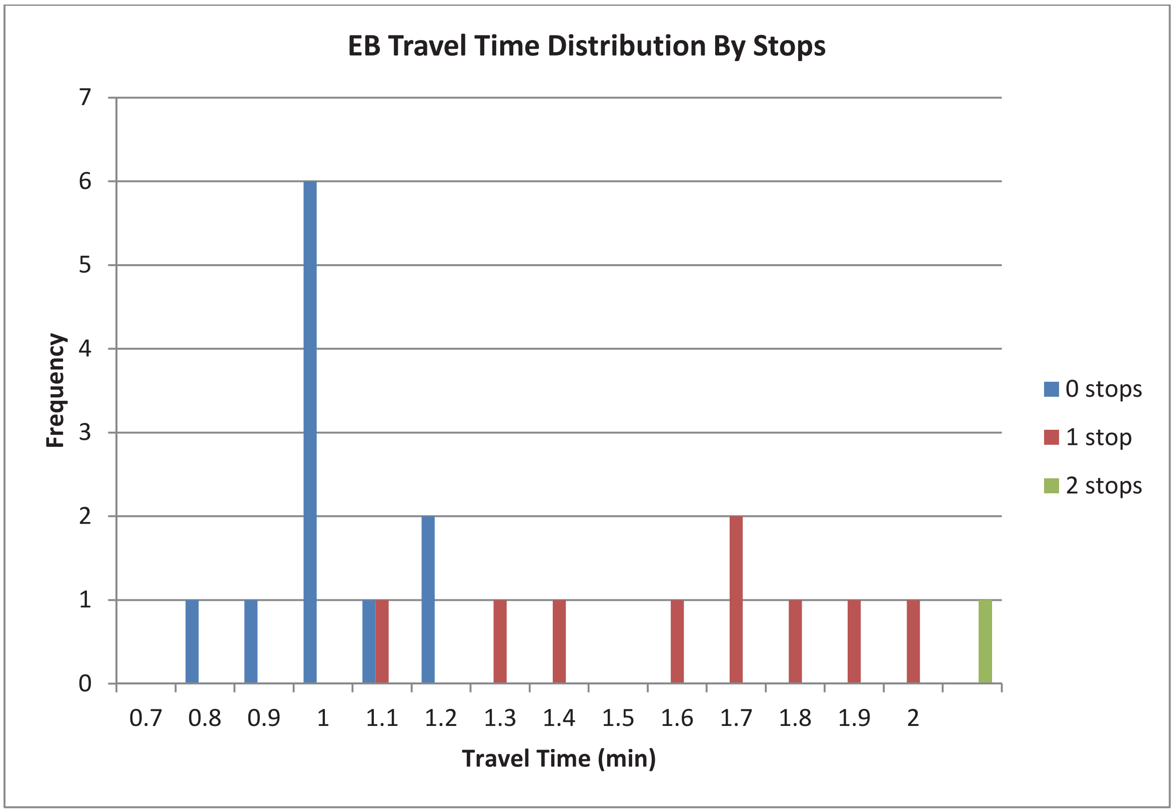


Figure 5-1 Travel Time Distribution by Number of Stops on EB

Figure 5.1 and Figure 5.2 provide clear evidence that arterial travel time has multi-modal distribution. In addition, travel time is highly correlated to the number of stop.

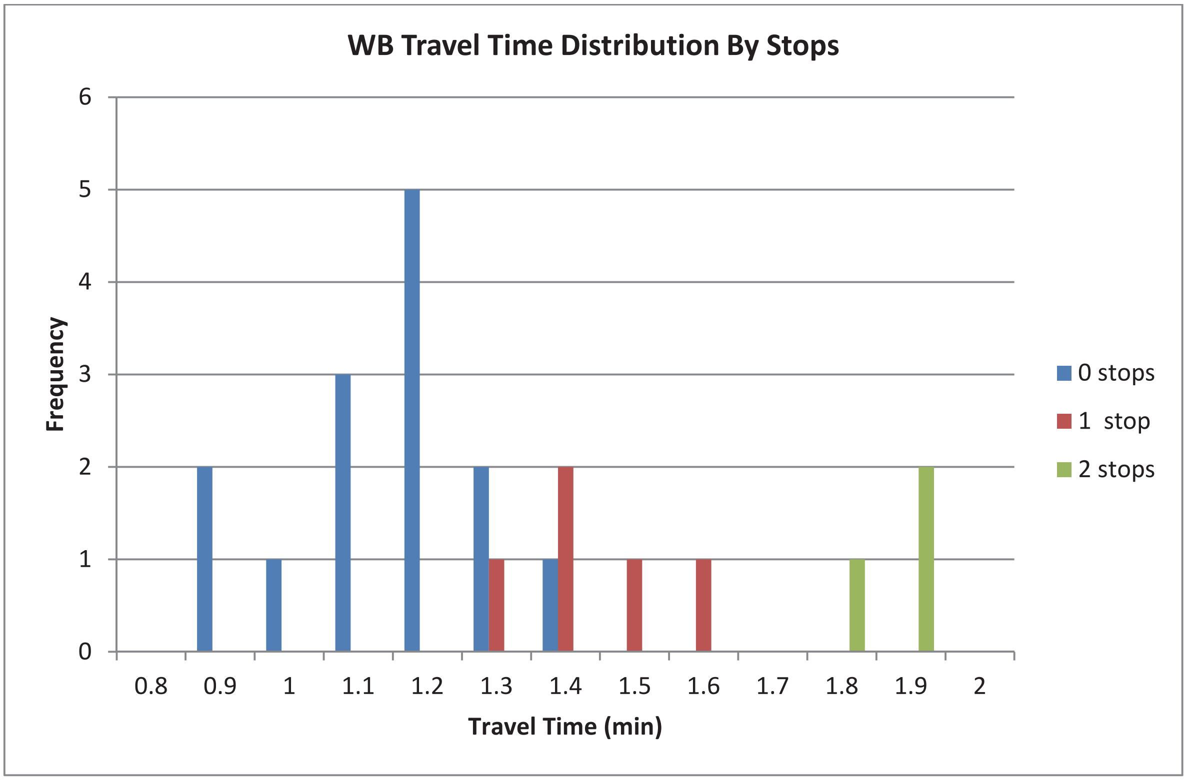


Figure 5-2 Travel Time Distribution by Number of Stops on WB

Table 5.7 shows the average travel speed on the corridor links and the average running speed on the links where vehicles do not have to stop. It can be seen that on links where the running speed is nearly equivalent to the average travel speed, vehicles do not have to stop at the downstream intersection. These links are Timber-GTS, GTS-Yeargan, and Timber-Jessup.

Table 5-7 Floating Car Travel Time for Each Section

Direction	Checkpoint	Average Travel Speed (mph)	Running Speed (mph)
EB	Jessup-Timber	20.93	36.7
	Timber-GTS	42.14	42.68
	GTS-Yeargan	48.62	48.96
WB	Yeargan-GTS	36.05	42.3
	GTS-Timber	27.9	36.22
	Timber-Jessup	44.53	44.84

Figure 5.3 displays the trajectories of vehicle runs completed using Tru-Traffic in the Time-Space Diagram in the AM peak plan. This figure shows the progression of vehicles through the arterial street. Determining whether a vehicle has to stop or not depends on the time it progresses through the upstream intersection. An example of this can be seen as vehicles travel from Jessup St. to Timber St. All of the vehicles that leave Jessup St. during the beginning of the green band are able to travel s through the entire westbound corridor without a stop. However, vehicles that leave Jessup at a later time during the green band are likely stop at Timber St. More than 50% of time the Yeargan intersection released vehicles earlier than its programmed offset (due to early return to green) and most of those early released vehicles had to stop at downstream intersections.

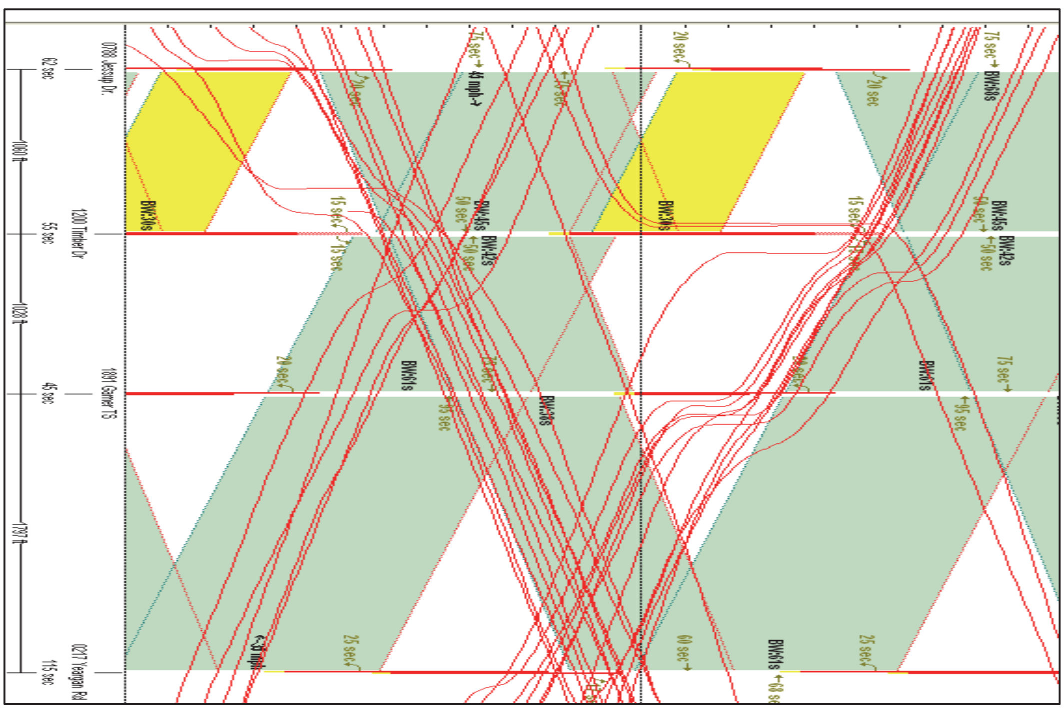


Figure 5-3 Travel Time Collection and Trajectory in Time-Space Diagram

Figure 5.4 and Figure 5.5 show the corridor time-space diagram with floating car trajectories and allows for an easier representation of signal coordination. It can be observed that eastbound vehicles only stopped at either Jessup or Timber and on rare occasions at both. Once any eastbound vehicle cleared the Timber Drive intersection, it was able to progress through the intersection without a stop. Westbound vehicles stopped at Yeargan, Garner Town Square, and Timber. While no vehicles had to stop at all three, several had to stop at either Yeargan or Garner Town Square and Timber. It can be observed that there is a high probability of two stops one at Garner Town Square and another at Timber Drive for vehicles that are not in leading positions in platoons. Figure 5.3 illustrates this and it can be seen in further detail in TravTime (see Figure 5.5 and Figure 5.6), that vehicles near the beginning of a queue or platoon that began at Garner Town Square were always able to clear the Timber Drive intersection without stopping. No vehicles had to stop once they progressed past Timber Dr. as Jessup is well coordinated with Timber.

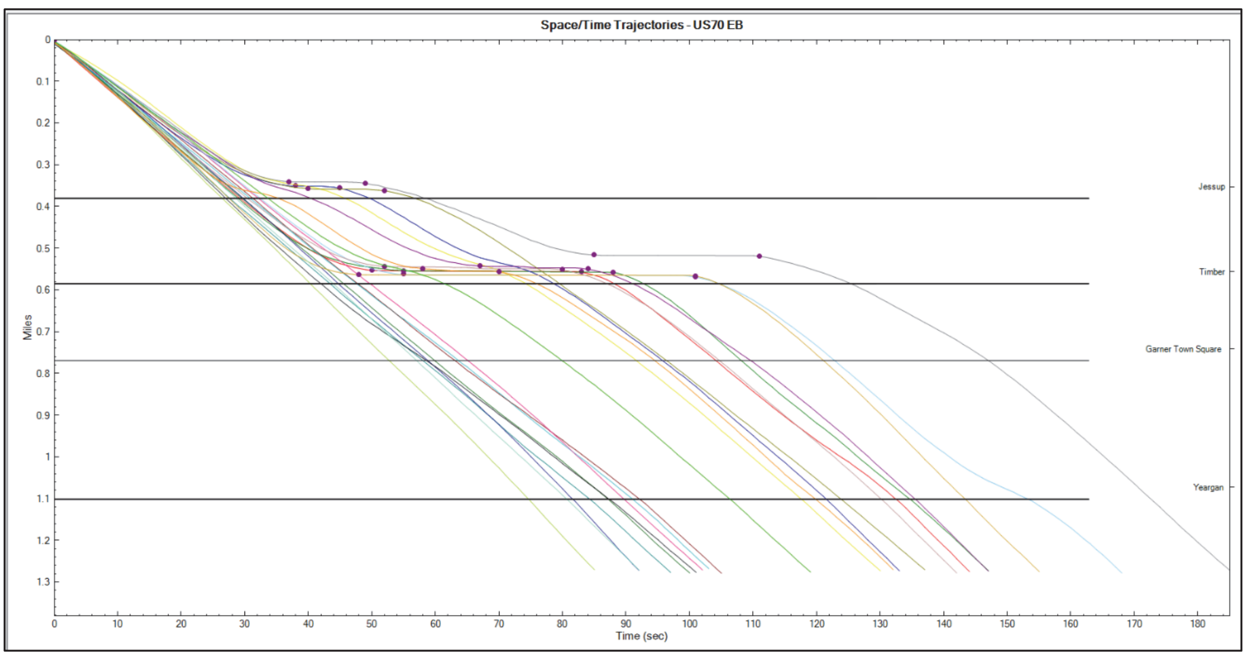


Figure 5-4 U.S. 70 Eastbound Corridor Space/Time Trajectories

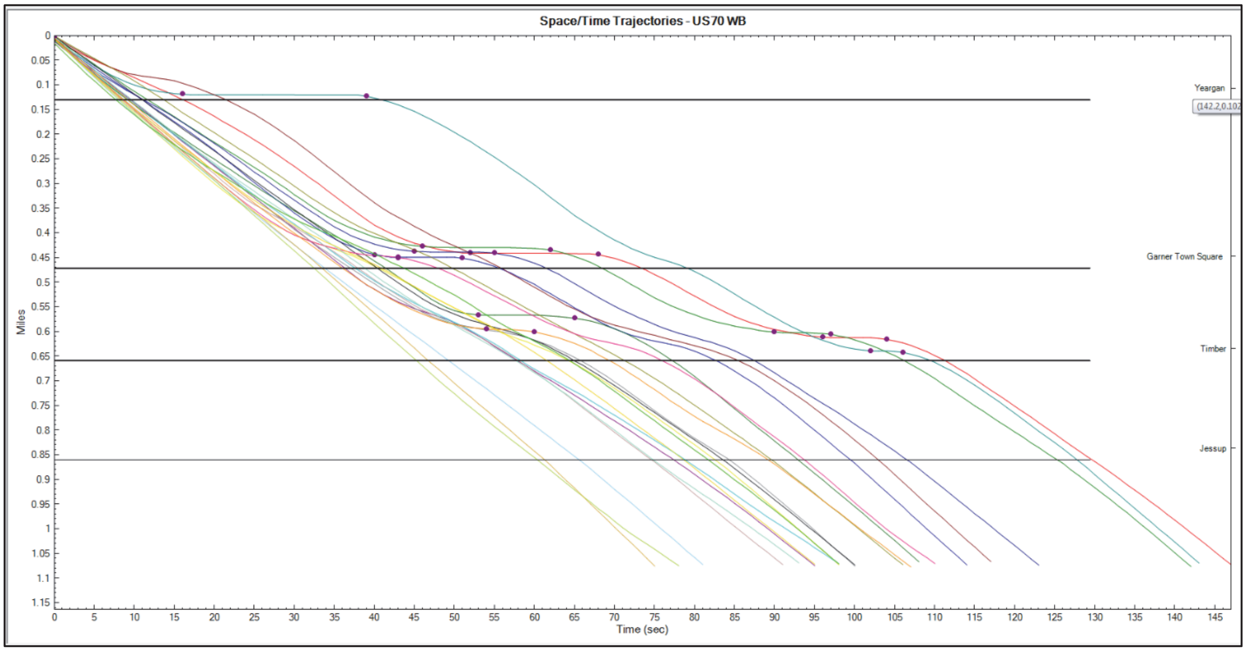


Figure 5-5 U.S. 70 Westbound Corridor Space/Time Trajectories

Table 5.8 displays the number of travel time runs with zero, one, and two stops in each direction. Over half of the runs traveled through the arterial without a stop in each direction. While the westbound direction allowed for a larger number of runs without a stop, it also had more runs with multiple stops. The overall number of stops for both directions was the same at eleven. Eastbound direction had a heavier traffic level in the AM peak (study period) and consequently is associated with a higher number of stops.

The coordination in the corridor does a good job of servicing the vehicles in platoons as they progress through the corridor by limiting the number of vehicles that have to make a stop. Even with the heavier traffic and congestion limitation, the westbound direction is able to allow more vehicles to traverse the intersection unimpeded than the eastbound. The appropriate direction is being given priority.

Table 5-8 Number of Stops and Percentage from Floating Car

WB	0 stops	1 stop	2 stops
Number	14	5	3
Percentage	63.64%	22.73%	13.64%
EB	0 stops	1 stop	2 stops
Number	11	9	1
Percentage	52.38%	42.86%	4.76%

5.2.2 Bluetooth

As mentioned earlier, five Bluetooth units are deployed on the US 70 arterial in Garner, NC. Using two weeks of Bluetooth data collected on Tuesdays, Wednesdays, and Thursdays (from March 12. 2012 to March 23. 2012), the US 70 EB and WB travel time distributions are built.

The total EB sample size is 113 vehicles for 7:00 to 800 AM period and 124 for 8:00 to 9:00 AM period. Average travel time for EB and WB directions were 1.85 and 1.75 minutes and standard deviation was 0.49 and 0.479, respectively. The travel time distribution is multimodal depending on number of stops.

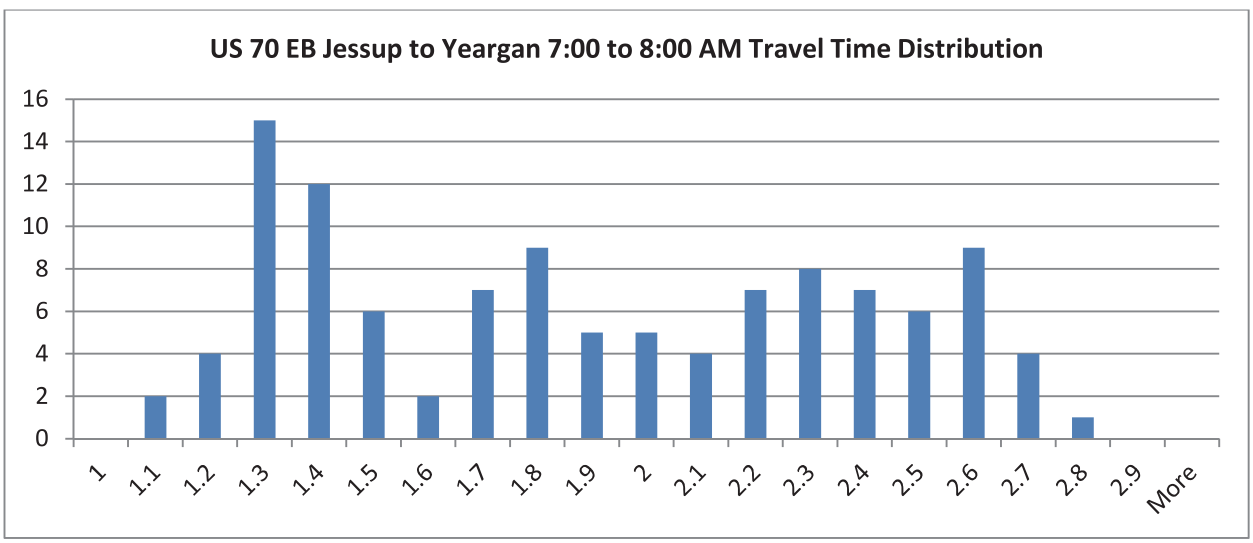


Figure 5-6 U.S. 70 Eastbound Travel Time Distribution (7AM to 8AM)

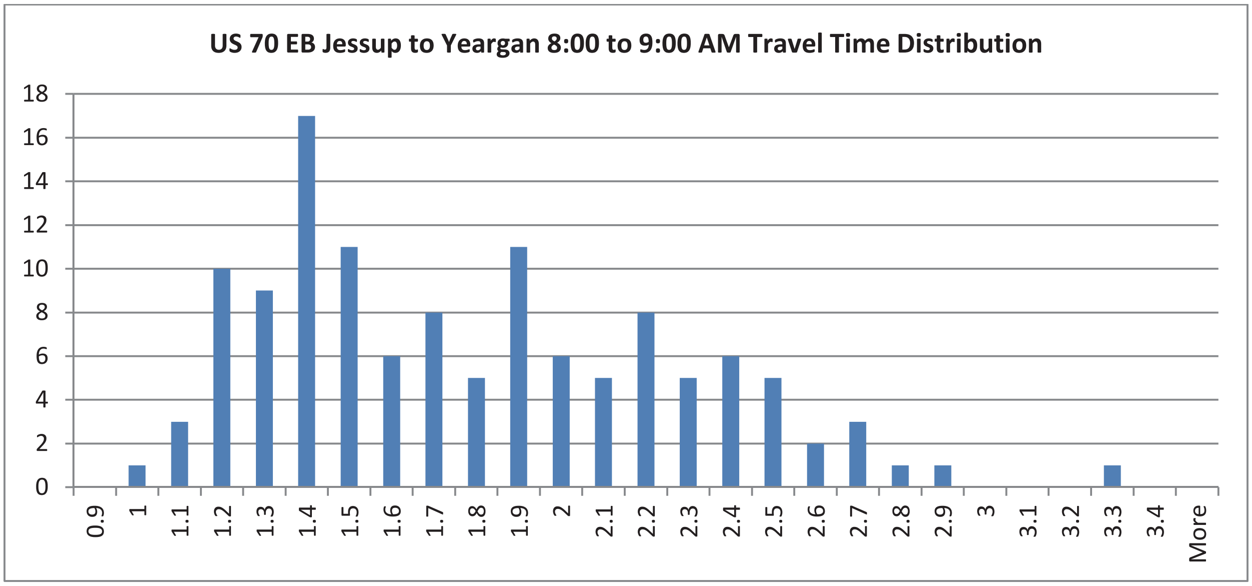


Figure 5-7 U.S. 70 Eastbound Travel Time Distribution (8AM to 9AM)

The two AM study periods on US 70 WB approach have sample sizes of 267 and 180. Westbound average travel time and speed during 7:00 to 8:00 AM are 2.082 minutes and 30.259 miles/hour. From 8:00 to 9:00 AM average travel time was 1.8 minutes and average travel speed was 35miles/hour.

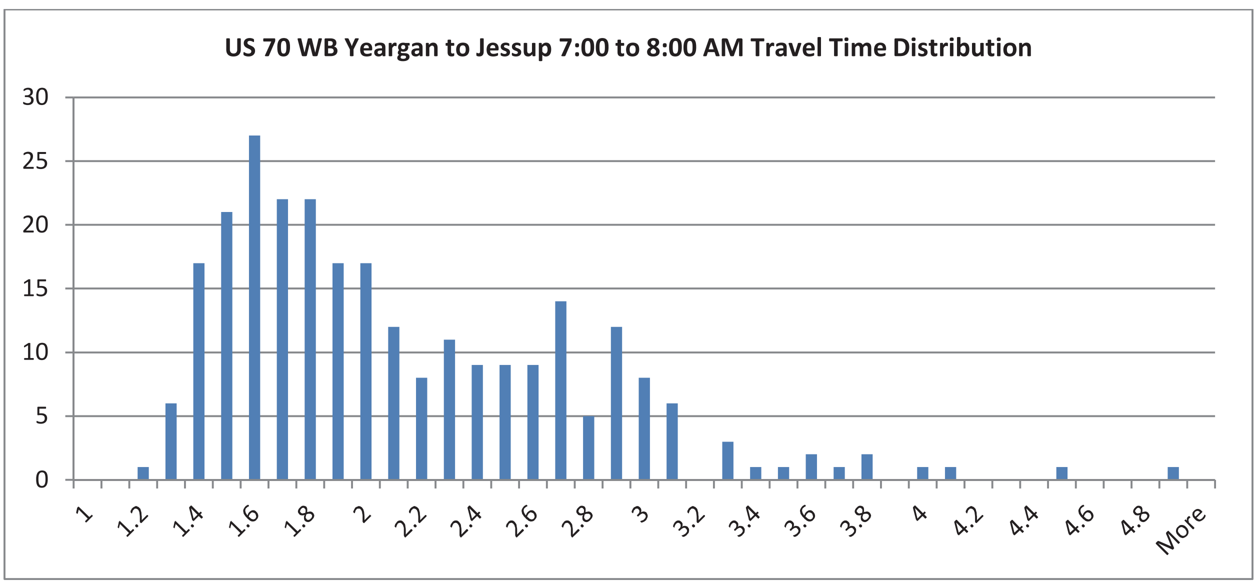


Figure 5-8 U.S. 70 Westbound Travel Time Distribution (7AM to 8AM)

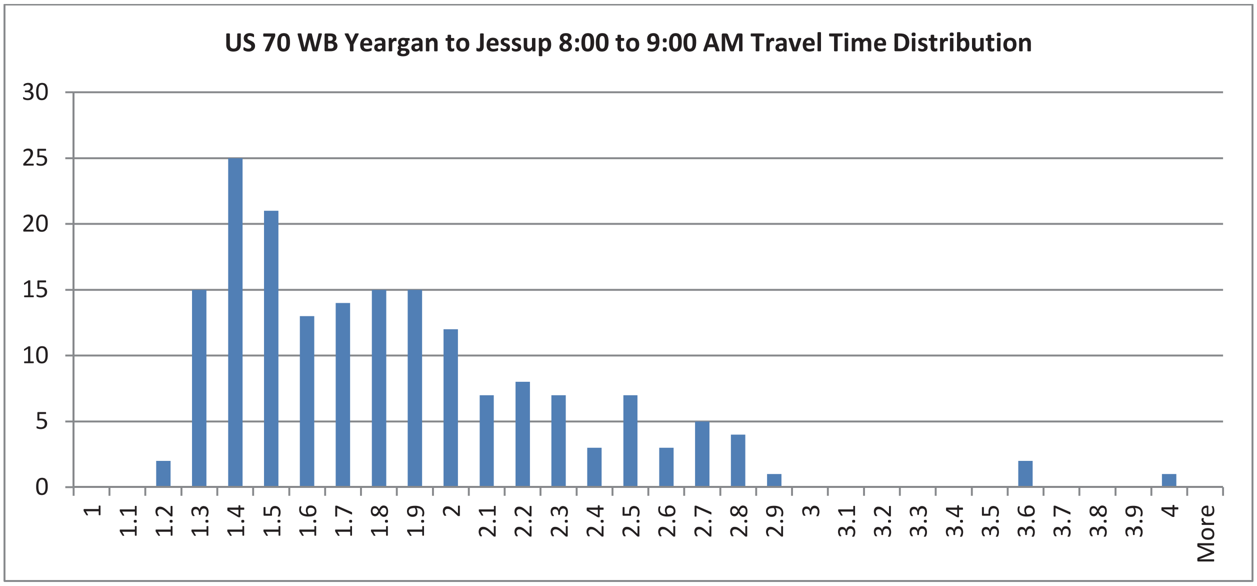


Figure 5-9 U.S. 70 Westbound Travel Time Distribution (8AM to 9AM)

5.2.3 Travel Time Comparison

Table 5.9 displays a comparison the Bluetooth and Geologger travel times. The Bluetooth travel times were produced by BlueSTATS while the Geologger travel times were produced by TravTime.

Table 5-9 Floating Car vs Bluetooth Travel Time

	EB (WB) Sample Size		EB (WB) Average of Travel Time		EB (WB) Standard Deviation of Travel Time	
	Floating Car	Bluetooth	Floating Car	Bluetooth	Floating Car	Bluetooth
70-Jessup	21 (22)	225 (523)	22.2 (16.8)	25.8 (22.2)	10.2 (1.2)	13.2 (10.8)
Jessup-Timber	21 (22)	256 (308)	38.4 (29.4)	41.4 (36)	22.8 (7.2)	26.4 (19.8)
Timber-GTS	21 (22)	265 (659)	18.6 (26.4)	22.8 (28.2)	1.2 (9)	12 (11.4)
GTS-Yeargan	21 (22)	288 (739)	24.6 (29.4)	30 (36.6)	2.4 (9)	12 (16.2)
Route	21 (22)	237 (447)	103.8 (101.4)	108 (118.2)	27 (19.8)	29.4 (35.4)

The values listed are fairly similar for the route and link values. While the Bluetooth data has a larger sample size, it has a higher variability in travel time. This unreliability originates from not knowing the path or route of the vehicles that travelled the arterial. While all floating car runs were straight through, the Bluetooth runs could vary in their path and also have mismatches.

5.3 OASIS Detector Log Monitoring

As mentioned in Chapter 4, OASIS provides vehicle counts, which consists of three attributes: a) vehicle volume, b) percentage of detector occupancy during a fixed time interval, and c) the space mean speed. OASIS system can report data with a minimum of one minute aggregation level.. In addition, non-coordinated movement lanes normally have long loop detectors operating in the “present” mode. Furthermore, no detector is used for right turn lanes typically, making it difficult to determine the exact volume of non-coordinated movements.

5.3.1 Detector Log Data Quality

NCDOT typically uses 6x6 single loop detectors in its closed loop systems as a downstream detector. The research team collected system detector log as well as video data to capture approach volumes. Recorded video files are manually processed to calculate minute-by-minute volumes.

The video was recorded in midday on April 13, 2012 for the Timber Dr. intersection EB direction. Timber Dr. intersection EB approach has two type of system detectors: link-based and lane-based detectors. Figure 5.10 shows detector configurations on Timber Dr. intersection. There are three system detectors 300 feet upstream of the EB stop line but lane 2and lane 3 detectors are tied together so they only provide aggregated counts across both lanes.

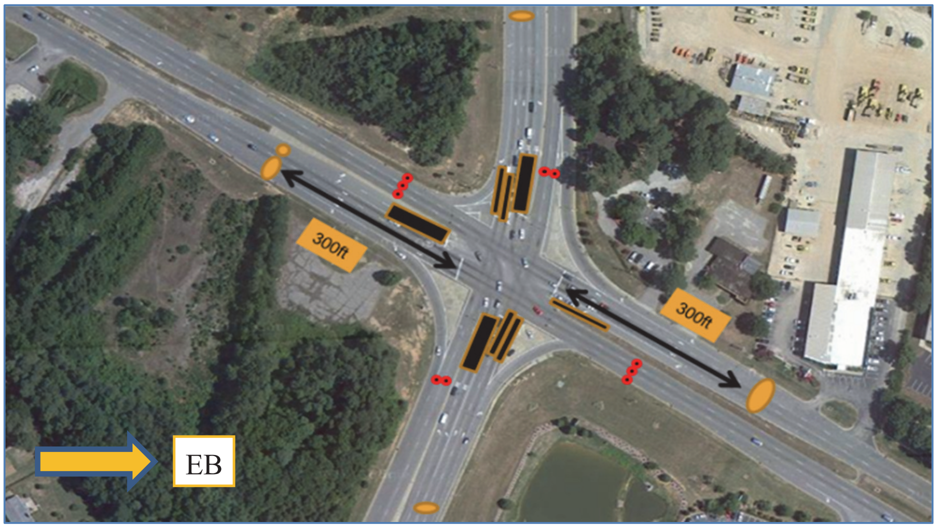


Figure 5-10 Detector configuration of Timber Dr. Intersection on US 70

The Table 5.10 shows video count results compared with the OASIS detector log. The first lane has lane-based detectors and counts the difference between video and detector log, which is 6 vehicles. The detector error rate is 1.17%. Lane-based OASIS detector log provides quite accurate volume information. However, the link-based detector error rate is 7.59% during the midday period (1 hour). The link-based detector volume error rate is increased during peak hour since high demand rates will increase the chance to occupy link-banded detector at the same time.

Table 5-10 OASIS Detector Log with Video Counts Comparison

	Lane 1		Lane 2 & 3	
Time Stamp	OASIS	Video Count	OASIS	Video Count
4/13/2012 11:56	5	6	9	7
4/13/2012 11:57	5	4	11	6
4/13/2012 11:58	14	13	8	18
4/13/2012 11:59	10	7	9	14
4/13/2012 12:00	7	6	10	7
4/13/2012 12:01	10	11	15	17
4/13/2012 12:02	4	8	13	11
4/13/2012 12:03	10	12	16	12
4/13/2012 12:04	7	5	10	11
4/13/2012 12:05	7	6	12	14
4/13/2012 12:06	8	11	19	17
4/13/2012 12:07	7	7	12	13
4/13/2012 12:08	6	5	10	11
4/13/2012 12:09	15	14	10	21
4/13/2012 12:10	9	8	11	13

4/13/2012 12:11	5	6	9	8
4/13/2012 12:12	14	12	12	21
4/13/2012 12:13	4	3	10	10
4/13/2012 12:14	8	7	8	10
4/13/2012 12:15	16	15	10	19
4/13/2012 12:16	5	9	15	12
4/13/2012 12:17	5	4	7	8
4/13/2012 12:18	14	15	17	23
4/13/2012 12:19	4	8	11	12
4/13/2012 12:20	7	6	11	6
4/13/2012 12:21	7	7	11	16
4/13/2012 12:22	6	7	8	8
4/13/2012 12:23	15	16	21	19
4/13/2012 12:24	12	11	9	15
4/13/2012 12:25	3	7	11	9
4/13/2012 12:26	10	11	9	10
4/13/2012 12:27	6	4	9	11
4/13/2012 12:28	4	2	6	9
4/13/2012 12:29	11	14	17	14
4/13/2012 12:30	8	5	10	13
4/13/2012 12:31	6	8	12	11
4/13/2012 12:32	9	10	16	7
4/13/2012 12:33	7	8	11	12
4/13/2012 12:34	7	9	11	8
4/13/2012 12:35	7	7	12	12
4/13/2012 12:36	9	8	14	16
4/13/2012 12:37	6	8	10	13
4/13/2012 12:38	9	9	15	15
4/13/2012 12:39	7	5	9	12
4/13/2012 12:40	13	8	16	15
4/13/2012 12:41	12	13	14	14
4/13/2012 12:42	4	6	6	12
4/13/2012 12:43	16	17	21	17
4/13/2012 12:44	8	2	9	12
4/13/2012 12:45	6	3	4	6
4/13/2012 12:46	12	12	16	14
4/13/2012 12:47	9	7	13	14
4/13/2012 12:48	6	7	9	9
4/13/2012 12:49	12	15	22	14
4/13/2012 12:50	7	9	13	12
4/13/2012 12:51	7	8	9	7
4/13/2012 12:52	15	7	12	23
4/13/2012 12:53	8	11	17	15
4/13/2012 12:54	10	8	10	10
4/13/2012 12:55	15	12	14	20
Sum	515	509	711	765

5.3.2 RTMS Unit

An RTMS unit is installed on the Timber Dr. intersection on the US 70 arterial. Figure 5.11 shows the location of the RTMS unit. The device is calibrated by a manual count and it covers both directional movements.

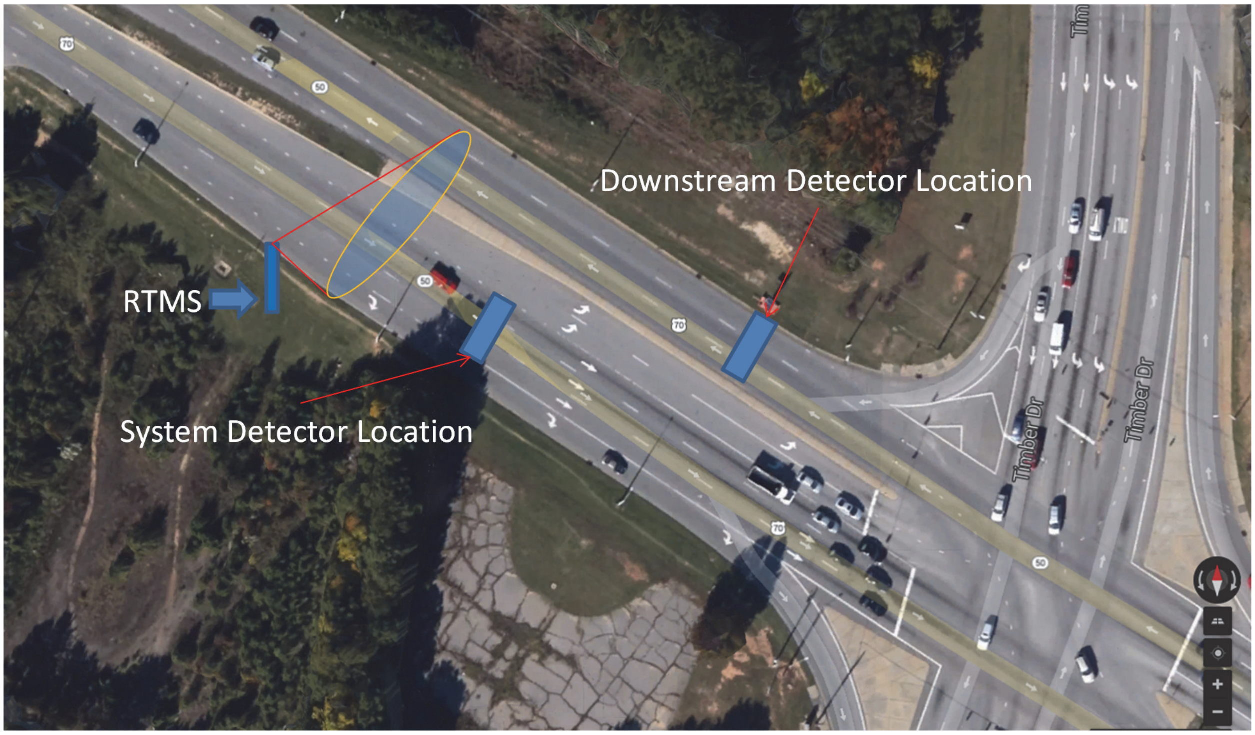


Figure 5-11 RTMS Unit Location Map

RTMS Provides one minute aggregated volume, occupancy, and speed data similar to OASIS detector log. The collected RTMS data is compared with manually counted video data during 1 hour PM Peak (5:30 to 6:30). The video recording shows 569, 576, and 216 vehicles for EB first lane, second lane and third lane, respectively. The RTMS unit provides 534, 561, and 694 vehicles for the same time duration. The EB third lane counts shows a significant difference since RTMS counts include most of the right turn vehicles. After manually filtering out the right turn vehicles, the third lane vehicle counts are 156. RTMS underestimated vehicle counts by 8.08%. The one hour count difference between RTMS and video count is 87 vehicles (video counts: 1,224 and

RTMS counts: 1,096) for WB direction and the error rate is 10.45%. At the same time, OASIS detector log's error rate for WB is 3.92% (total counts: 1,176). The results indicate that inductive loop detectors provide more accurate volume information than RTMS units.

5.3.3 Volume Profile with Time of Day Plan

Lane-based inductive loop detectors provide quite accurate volume information. OASIS detector log file was used for generating volume profile from March 1 to March 14, 2014. Figure 5.12 show the results of two weeks (8 weekdays data: Tuesday to Thursday) average volume profile from OASIS detector log and different time of day plans. The red line indicates WB two weeks average of 15 minutes flow rate per lane and blue line shows EB two weeks average of flow rates. In addition, Orange lines are time-of-day breakpoints. Outside of time-of-day breakpoint, which means before 6:15 AM and after 7:00PM, the signals run in free-run mode.

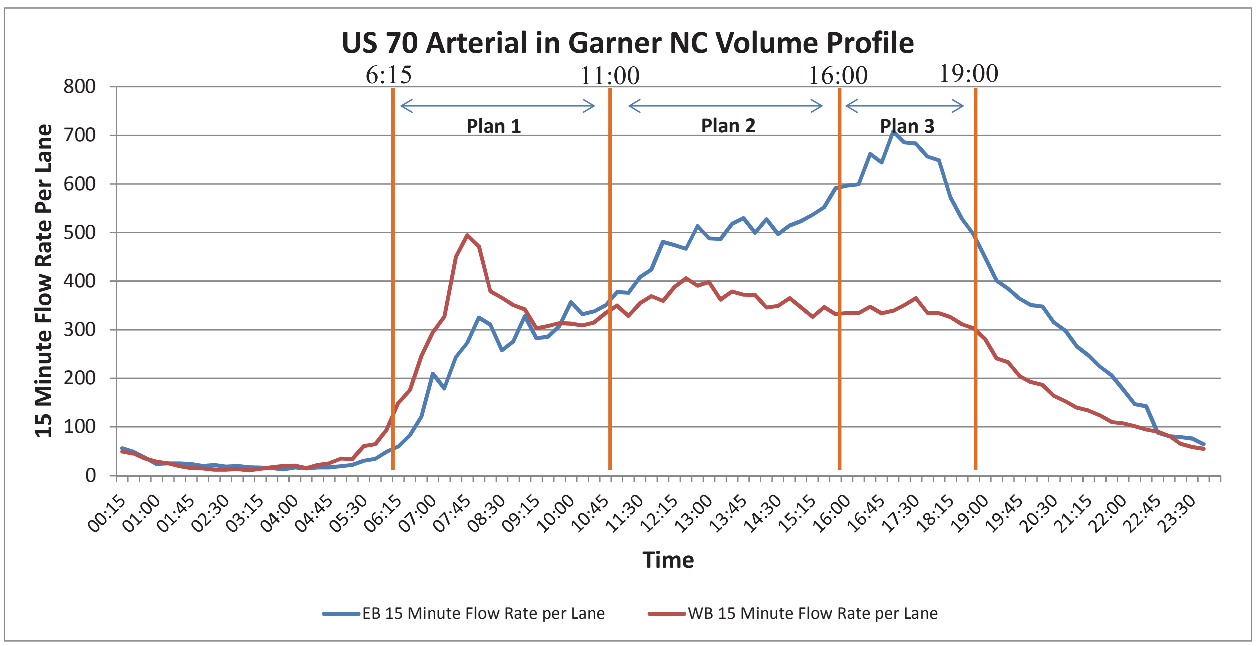


Figure 5-12 US 70 Volume Profile

The plot shows that the AM and PM peak volumes are captured reasonably well by the current TOD plans. However, after the PM peak plan breakpoint, there is still considerable demand, so there may be some benefits in adding another PM plan after 7 PM with a short cycle length.

5.4 OASIS Split Monitor Log Monitoring

In Chapter 4, the OASIS Split Monitor log data format was introduced. OASIS provides detailed information about each phase's displayed green time, which does not include yellow and all red times. This log file allows the operator to monitor current as well as historical used green time distribution, g/C ratio, early return to green distribution and non-coordinated phase gap-out and max-out percentage in each phase.

5.4.1 Coordinated Phase g/C Profile

Intersection capacity is determined by g/C ratio and saturation flow rate (saturation headway). The capacity of a given lane group is:

$$c = N * s * \frac{g}{C}$$

Where, c: lane group capacity,

N: Total number of lanes for the given lane group,

s: Adjusted saturation flow rate for the given lane group,

g: Effective green time for the given movement, and

C: Cycle length of the intersection.

A semi-actuated controller does not allocate a green signal to side streets as long as there is no demand for it. The controller allows skipping or gapping out of that specific phase so that the unused green time is allocated to the major street. This is called an "early return to green." Also, the coordinated phases green can be extended until the yield point or next reference point. Therefore, each cycle green duration is dynamic. Therefore, programmed greens of coordinated phases indicated minimum amount of green duration for coordinated movements. However, programmed effective green value (minimum effective green value) is conventionally used for estimating lane group capacity, intersection signal failure, and level of service due to absence of field observed effective green information.

Table 5.11 shows the programmed g/C ratio and average of split monitor g/C (over six weekdays) for Timber Dr. intersection, which is the critical intersection on US 70 arterial in Garner, NC. In case of US 70 Timber Dr. intersection, 23% more green was provided compare to programmed green during AM plan. 18% and 20% more green was provided during MD plan for phase 2 and phase 6, respectively. During PM plan, 10% more green was provided for phase 2 and 12 % of more green was provide for phase 6.

Table 5-11 Timber Dr. Intersection Programmed g/C and Field g/C Comparison

	AM		MD		PM	
Phase	**Phase 2 (EB)**	**Phase 6 (WB)**	**Phase 2 (EB)**	**Phase 6 (WB)**	**Phase 2 (EB)**	**Phase 6 (WB)**
Cycle	120		170		170	
Sample size	827 cycles		626 cycles		375 cycles	
Average of Used Green	55.08	55.31	84.42	91.5	77.88	83.6
Split Monitor g/C	**0.46**	**0.46**	**0.5**	**0.54**	**0.46**	**0.49**
Programmed Green	42	42	69	73	69	73
Programmed g/C	**0.35**	**0.35**	**0.41**	**0.43**	**0.41**	**0.43**

Figure 5.13 and Figure 5.14 show g/C profile plots over six weekdays. The black lines indicate coordinated phase's average profile. The g/C plot illustrates that even critical intersection has considerably larger field g/C than its programmed g/C.

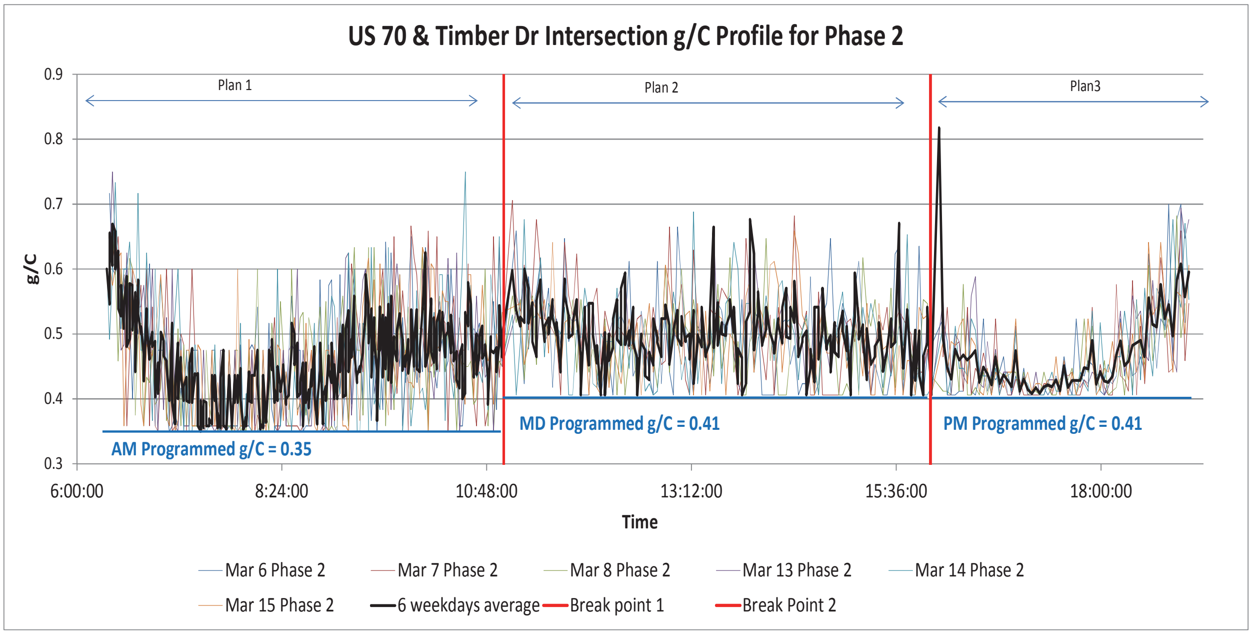


Figure 5-13 Phase 2 g/C Profile for Timber Dr. Intersection

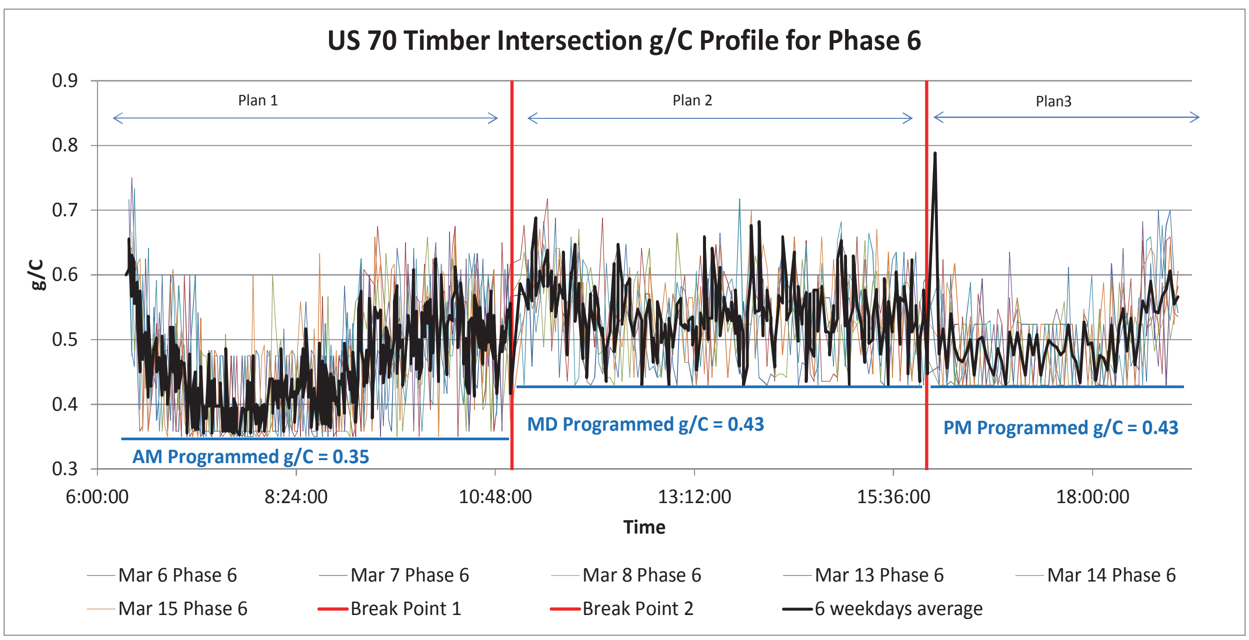


Figure 5-14 Phase 6 g/C Profile for Timber Dr. Intersection

Table 5.12 shows programmed and dynamic g/C ratios on Jessup Dr. intersection (non-critical intersection). Dynamic green was 38% more than programmed green for phase 2 and 53% for phase 6 in the AM plan. 19% and 21% more green was provided during MD plan for phase 2 and phase 6, respectively. During PM plan, 21% more green was provided for phase 2 and 22% more green was provide for phase 6.

Table 5-12 Jessup Dr. Intersection Programmed g/C and Field g/C Comparison

	AM		MD		PM	
Phase	Phase 2 (EB)	Phase 6 (WB)	Phase 2 (EB)	Phase 6 (WB)	Phase 2 (EB)	Phase 6 (WB)
Cycle	120		170		170	
Sample size	815 cycles		622 cycles		361 cycles	
Average of Used Green	96	106	143	153	137.7	151.3
Split Monitor g/C	**0.797**	**0.88**	**0.84**	**0.90**	**0.81**	**0.89**
Programmed Green	69	69	119	129	114	124
Programmed g/C	**0.575**	**0.575**	**0.7**	**0.758**	**0.67**	**0.729**

Figure 5.15 and Figure 5.16 shows g/C profile for coordinated phases in Jessup Dr. intersection. The g/C plot illustrates that the coordinated movements' lane group on non-critical intersection has considerably more g/C ratio compared to the programmed greens. The average g/C line for six weekdays shows that it never reaches its low boundary, which is the programmed g/C. In addition, even single day's plot also hardly hits the programmed g/C line. The g/C plot gives important information about coordinated movement lane group's capacity. According to our findings, coordinated movement lane group's capacity is much higher than its programmed capacity.

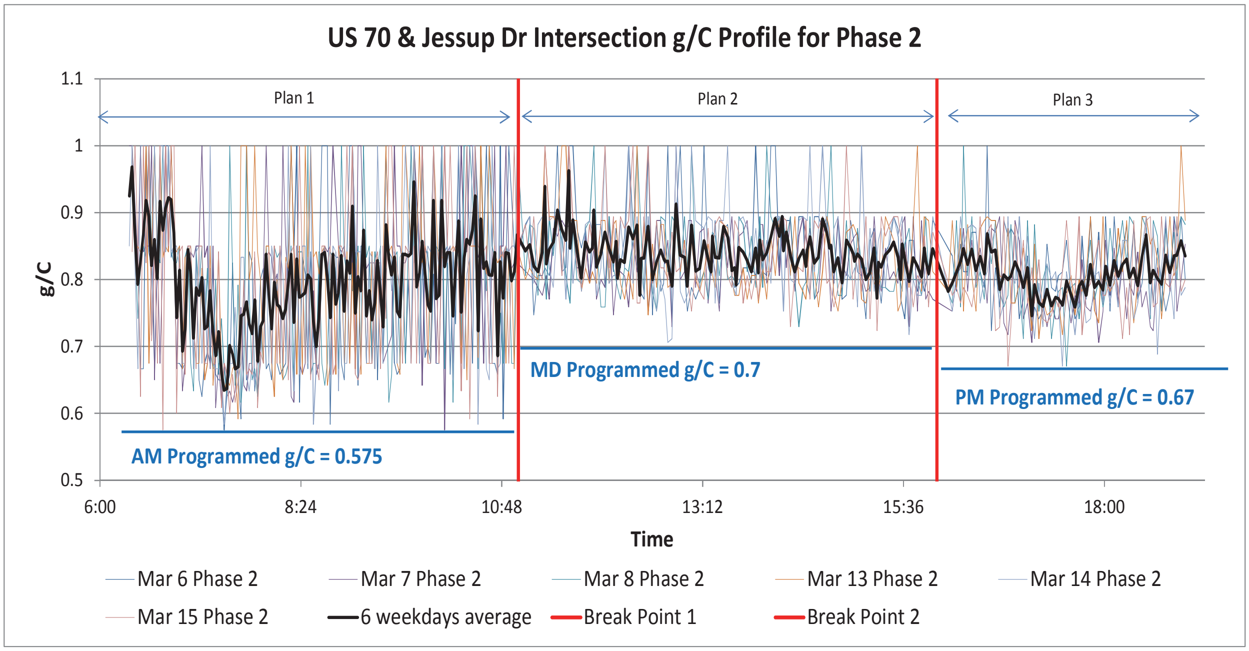


Figure 5-15 Phase 2 g/C Profile for Jessup Dr. Intersection

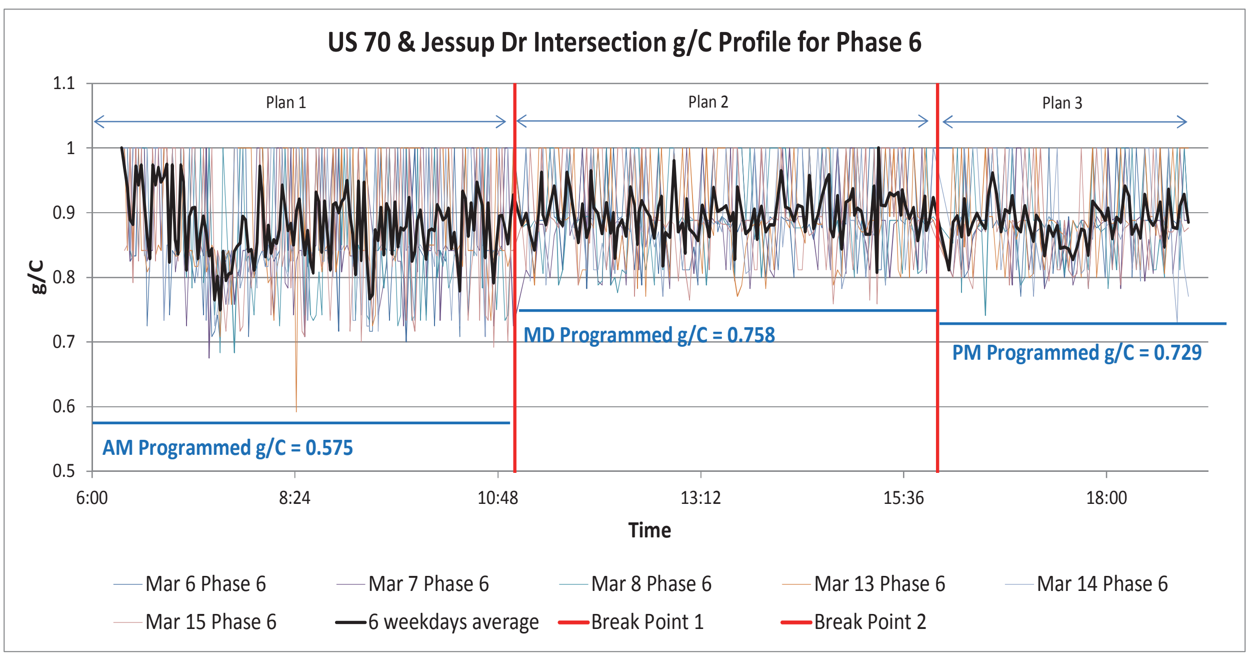


Figure 5-16 Phase 2 g/C Profile for Jessup Dr. Intersection

5.4.2 Flow Rate over Capacity

Average capacity is derived from six weekdays (Tuesday to Thursday) average g/C ratio. Adjusted saturation flow rate is 1,800 veh/h/ln. Figure 5.17 and Figure 5.18 shows the Timber Dr. intersection's estimated capacity and flow plots for coordinated phases. Only EB direction demand level becomes close to its capacity during PM peak hour. There is enough capacity to serve demand for both directions. Therefore, there are no high chances to have signal failure. However, these plots are derived from six weekday average g/C with a15-minute aggregated detector volumes. It is better to use cycle-by-cycle detector volumes and displayed green durations for more accurate analysis. OASIS split monitor log provides cycle-by-cycle used green time duration, but detector log does not provide vehicle actuation log. Since detector log provides minimum 1 minutes aggregated resolution data, it is difficult to directly match green time with its demand.

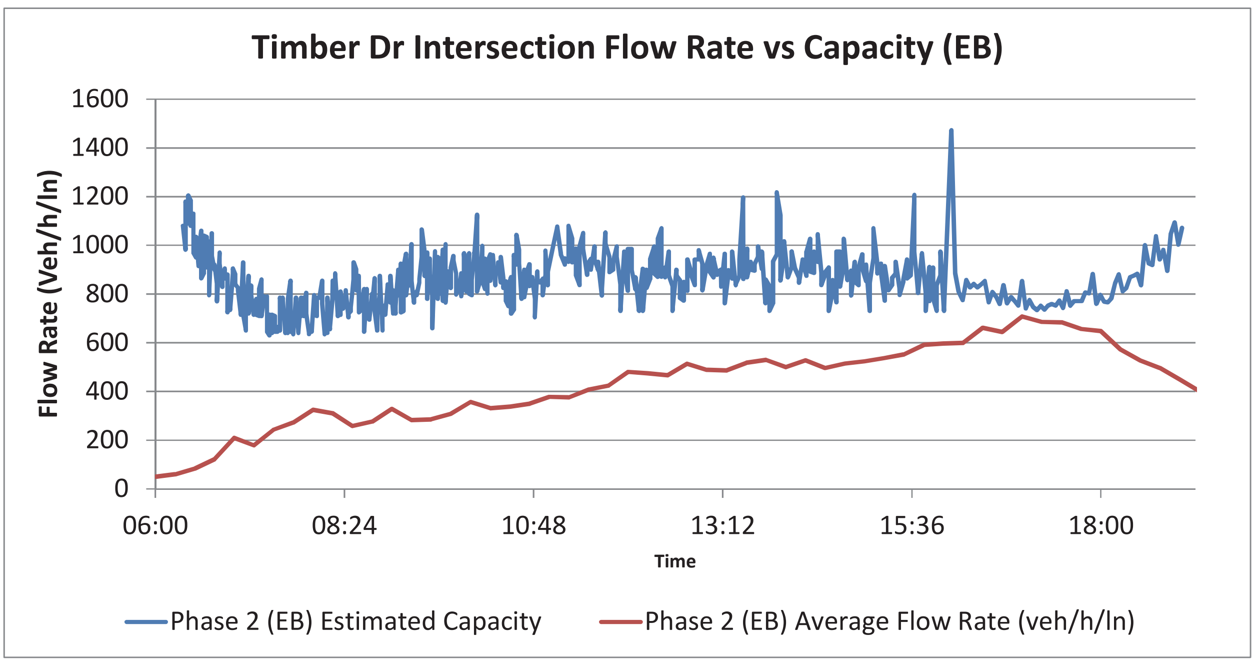


Figure 5-17 Phase 2 g/C Profile for Timber Dr. Intersection

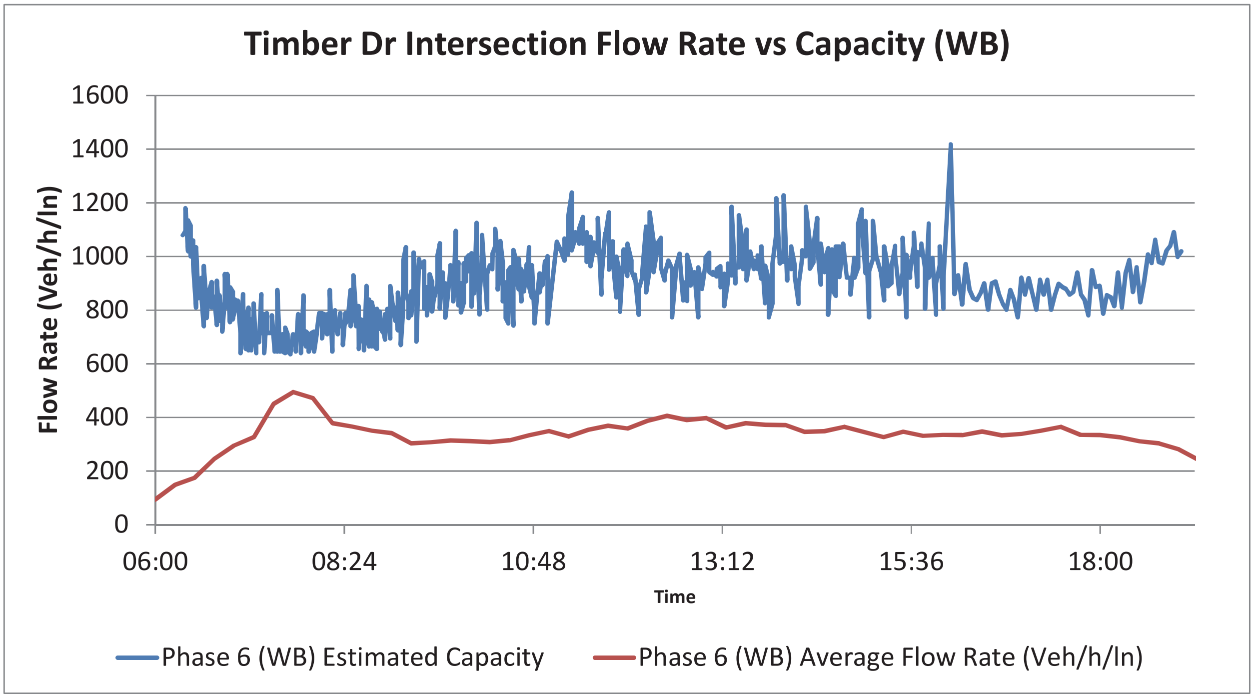


Figure 5-18 Phase 6 g/C Profile for Timber Dr. Intersection

5.4.3 Early Return to Green Distribution

Semi-actuated signal control arterials have the potential for the coordinated phase to begin earlier than expected. This phenomenon happened when non-coordinated phases are gaped out or skipped. This early return to green may increase system delay when this was out of the field engineer's prospect ranges. During offset fine tuning procedure, signal engineers consider how often and how many seconds early return to green occurs in the field.

However, it is hardly possible to select one number, which can nicely represent specific intersection's early return to green. This is because each intersections' early return to green is independent from any upstream or downstream intersection flow, but only depends on its own non-coordinated approach traffic flow and arrival pattern. In other words, each intersection has a different early return to green distribution.

Figure 5.19 and Figure 5.20 shows (critical intersection) the distribution of early return to green for coordinated phases for 824 cycles at Timber Dr. intersection. Although the Timber Dr. intersection is a critical intersection, there are only 9.1% and 7.2% of zero early return to green. For more than 90% of time during the AM period, Timber intersection has early return to green while there is zero green extension.

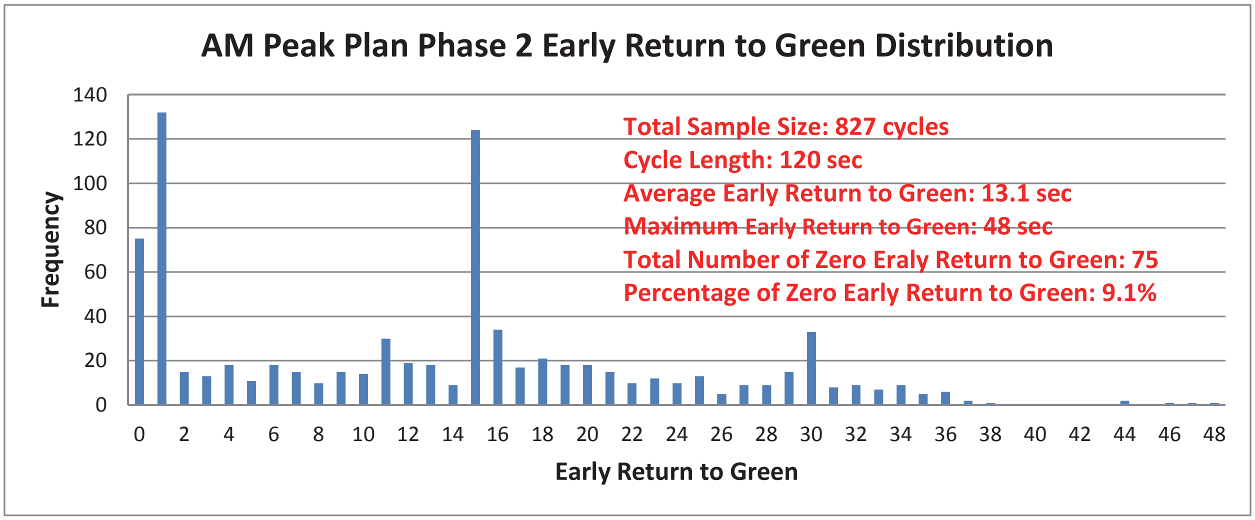


Figure 5-19 Phase 2 Early Return to Green Distribution for Timber Dr. Intersection

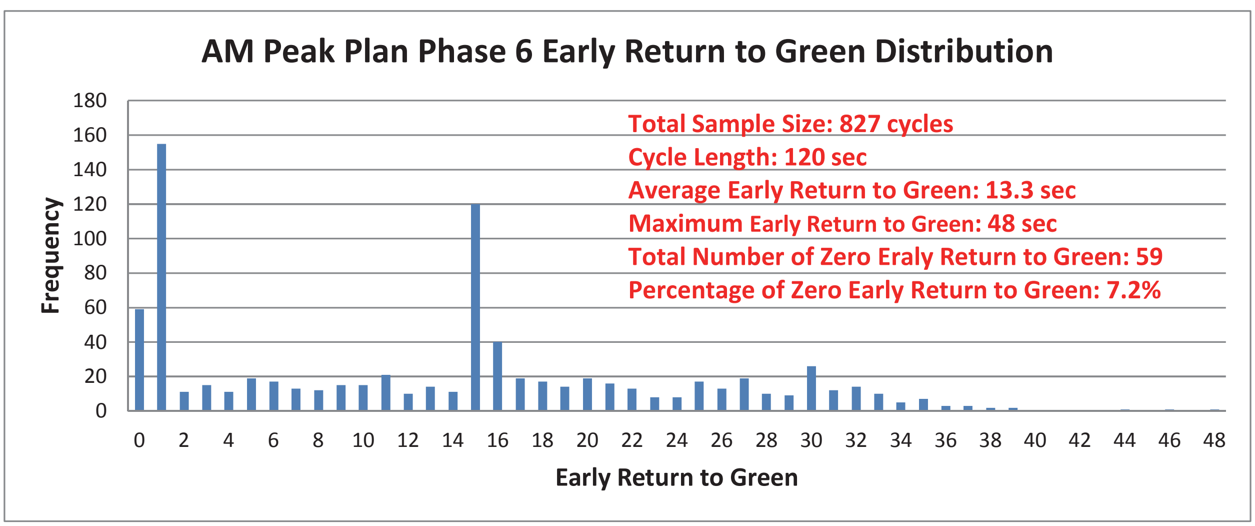


Figure 5-20 Phase 6 Early Return to Green Distribution for Timber Dr. Intersection

Figure 5.21 and Figure 5.22 illustrates Jessup Dr. intersection's (non-critical intersection) early return to green distribution. There are more zero early return greens compare to Timber Dr. intersection. However, in case of phase 2 (EB), there are 200 zero early return to green but only 119 cycles are fully extended. So, there is only 1 zero early return to green and the percentage is 0.12%. In case of phase 6, all of 286 cycles of zero early return to green happened due to full green extension, so there are no pure zero early return to green.

The early return to green distribution is multi-modal and is affected by conflict phase gap out or skips. As mentioned above, it is difficult to pick up one number that can nicely represent a specific intersection's early return to green.

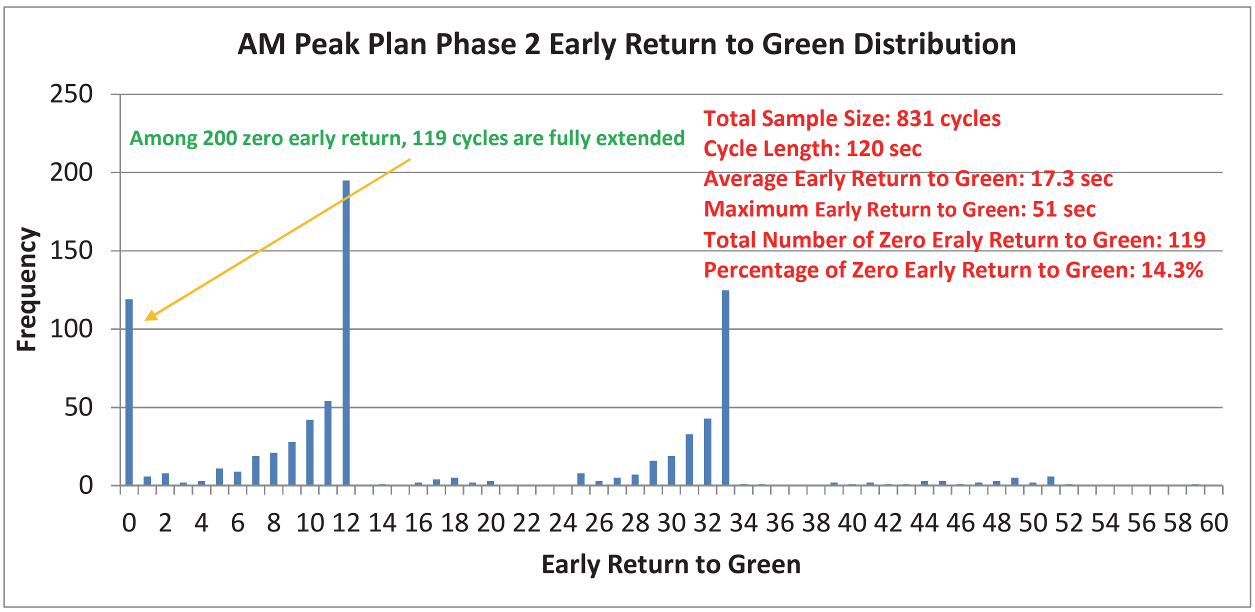


Figure 5-21 Phase 2 g/C Profile for Jessup Dr. Intersection

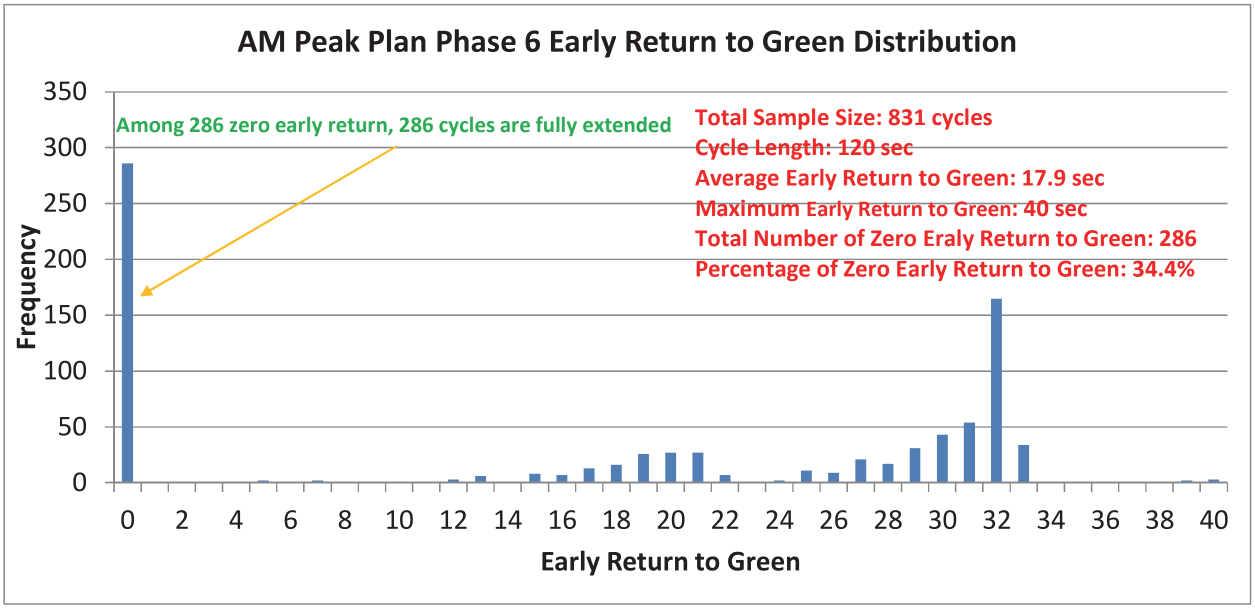


Figure 5-22 Phase 6 g/C Profile for Jessup Dr. Intersection

5.4.4 Non-Coordinated Phase Used Green Distribution

Table 5.13 shows one example of displayed green time of a non-coordinated phase. More than 37 % of time, phase 1 is skipped due to an absence in demand and it used a minimum green time of 7 seconds for 42.8% of time. The Programmed split for phases 1 and 5 are 15 seconds at Timber intersection with 4.0 and 3.0 seconds yellow and all-red indications during the AM peak period, respectively. Therefore, when OASIS displayed green time for more than 9 seconds, phase 1 and phase 5 started earlier than their programmed start time. More than 19 % of time, the Timber Dr. intersection phase 1 is forced off while 3.4% of time, phase 5 is forced-off during the AM peak period.

Figure 5.23 shows the distribution of displayed green time for phase 1 and 5 on the Timber Dr. intersection. Most of the time, both phase 1 and phase 5 are skipped or only used their minimum green time.

Table 5-13 Timber Intersection Phase 1 and 5 Displayed Green Time

	Phase 1		Phase 5	
Displayed Green Time	Frequency	Percentage	Frequency	Percentage
0	312	37.7%	315	38.1%
7 (minimum green)	354	42.8%	404	48.9%
8	95	11.5%	80	9.7%
9	22	2.7%	9	1.1%
10	18	2.2%	10	1.2%
11	11	1.3%	3	0.4%
12	7	0.8%	3	0.4%
13	4	0.5%	1	0.1%
14	1	0.1%	1	0.1%
15	2	0.2%	0	0.0%
16	0	0.0%	1	0.1%
17	1	0.1%	0	0.0%
Total	827	100%	827	100%

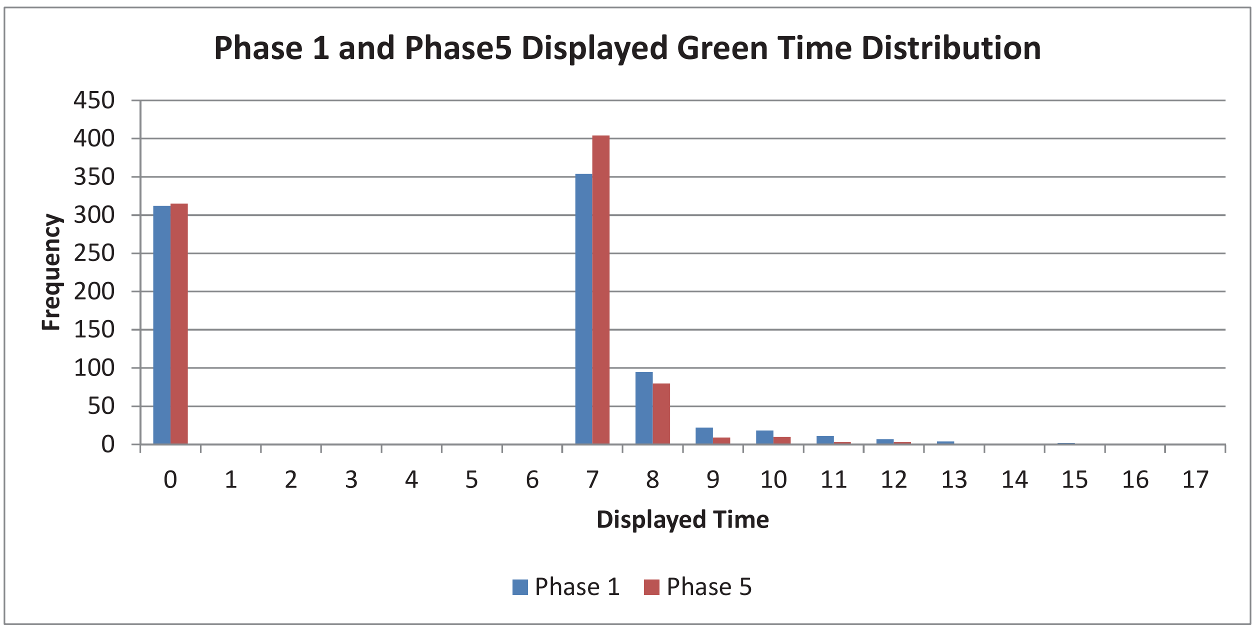


Figure 5-23 Phase 1 and Phase 5 Displayed Green Distribution on Timber Dr. Intersection

Figure 5.24 shows the distribution of displayed green time for both phase 1 and 5 on the Jessup Dr. intersection. The Jessup intersection used “Lag-Lead” phase for left turn phase sequences.

Phase 1 used a lag phase so it cannot gap out until phase 6 is terminated. Therefore, most of time (63.9% of time), phase 1 used all of its programmed time while phase 5 was maxed-out only 0.12%.

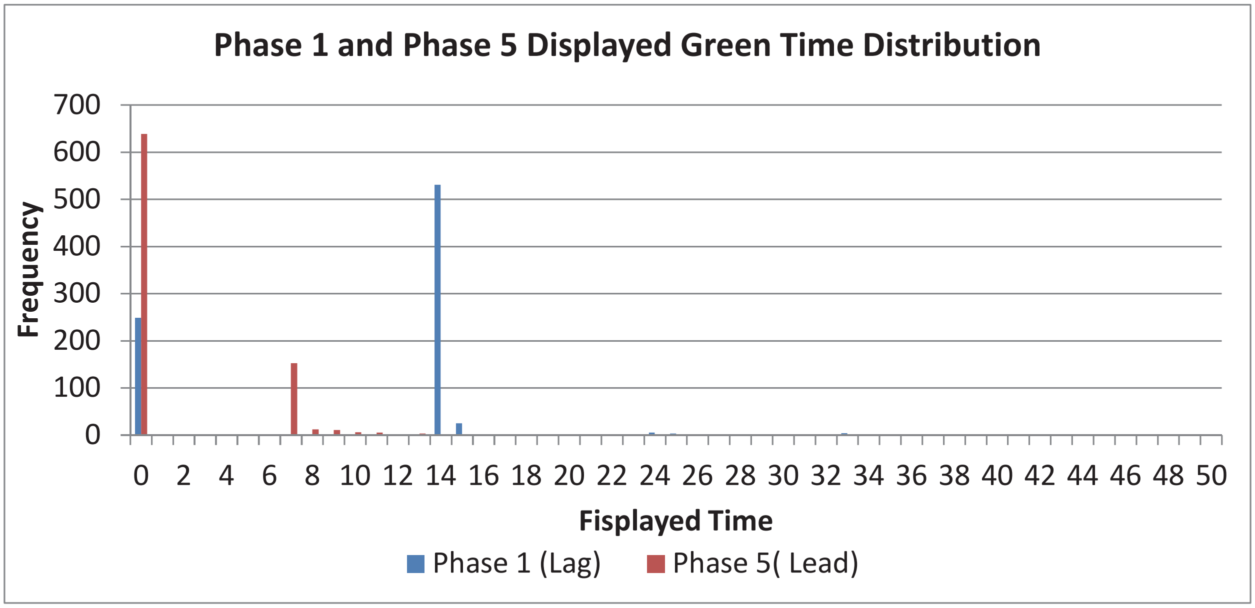


Figure 5-24 Displayed Green Distribution on Jessup Dr. Intersection for AM Plan

This result highlights the disadvantages of a "Lead-Lag" phase sequence. Since lag phase cannot gap out, lag phase will show green ball even without any demand until concurrent phase is terminated.

Jessup Dr. intersection used a "Lead-Lead" phase sequence during the PM peak plan. Phase 1 split is 30 seconds and phase 5 split is 20 seconds. The minimum green is 7 seconds for both phases 1 and 5. Figure 5.25 shows displayed green time in OASIS split monitor for phases 1 and 5 during the PM peak plan (365 cycles). Phase 1 used its minimum green (7 seconds) 16.2 % of the time. It is difficult to estimate the total amount of time that the "lag" phase turned on green without any demand, but the frequency of skipping phase 1 on Figure 5.24 and Figure 5.25 give an idea of inefficiencies of a lag phase. The percentage of skipping during the AM plan for phase 1 is 36.1% while phase 1 only skipped 3.88% of time during the PM plan.

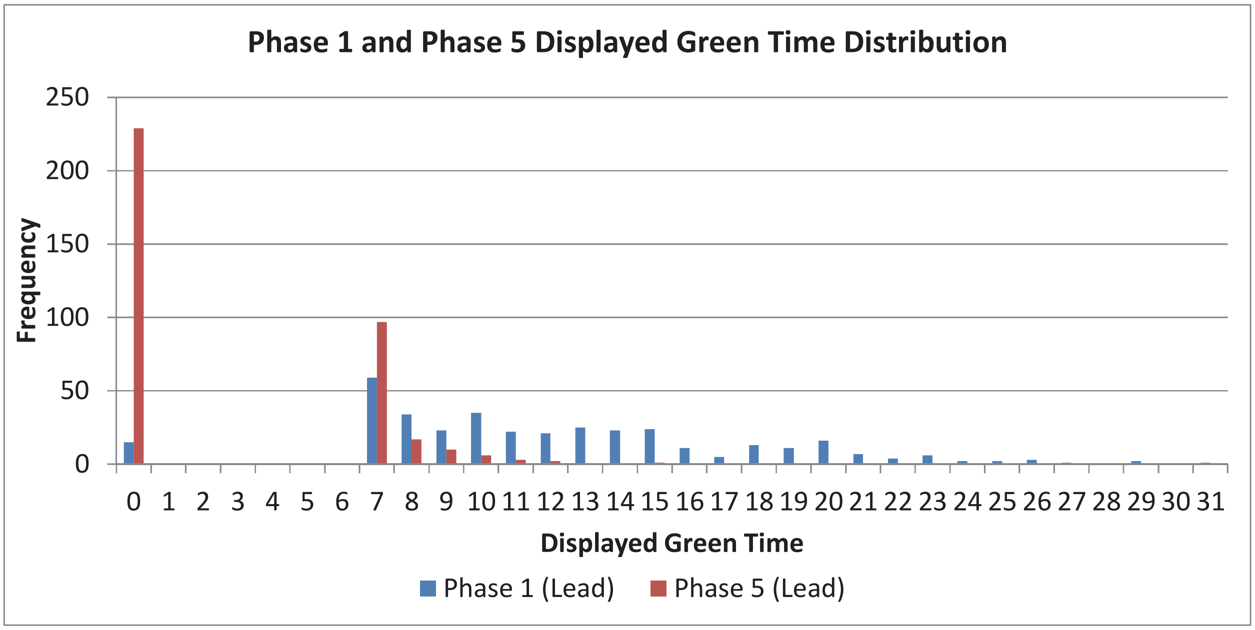


Figure 5-25 Displayed Green Distribution on Jessup Dr. Intersection for PM Plan

5.5 Bandwidth Monitoring

Unlike freeway facilities where delay results primarily from capacity constrained bottlenecks, delay along signalized arterials whose demand does not exceed capacity results from deceleration and stops as vehicles interact with signal control. Therefore mitigating delay along signalized arterials through improved signal timing is a cost effective congestion management approach that can be accomplished without adding physical roadway capacity.

Well-designed signal coordination along arterial streets minimizes the number of stops and consequently travel delay. Synchronizing the onset of green indication for the intersections along an arterial street is one of the key steps in improving coordination and is known as offset optimization. However, most of current offset optimization methods rely on programmed greens and reds, while arterial coordinated phases' green is a dynamic and time dependent variable.

Travel time studies show the relationship between the number of stops and arterial travel time. From OASIS split monitor information, it is possible to monitor real-world bands cycle-by-cycle, hereafter called “Dynamic Bandwidth”.

5.5.1 Conventional Bandwidth

The Federal Highway Administration Traffic Signal Timing Manual defines Bandwidth as "the maximum amount of green time for a designated movement as it passes through a corridor”. Traditional bandwidth is decided by ideal travel time, offset, and constant green and is constant in all cycles.

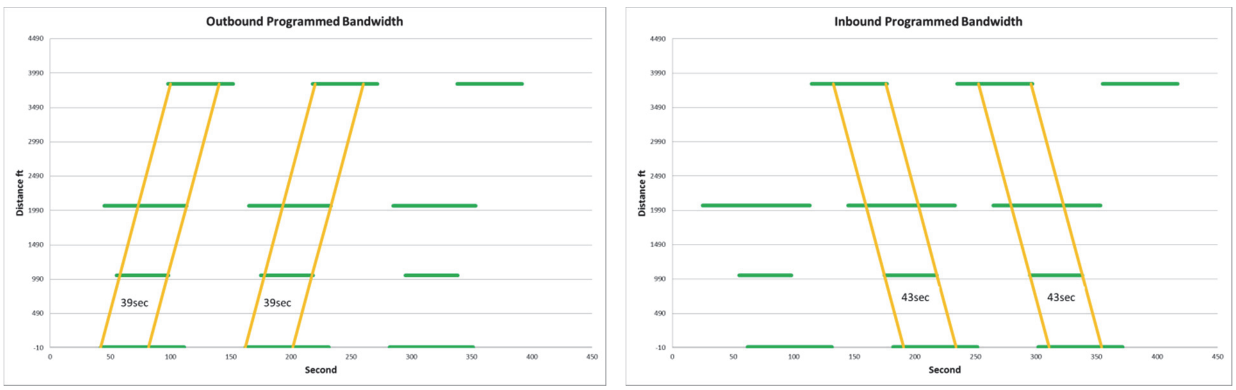


Figure 5-26 AM Plan Programmed Bandwidth for US 70 Arterial in Garner, NC

The Monday to Friday AM plan’s programmed outbound bandwidth is 39 seconds and inbound bandwidth is 43 seconds, which is calculated by programmed green with current offset. Therefore, outbound bandwidth efficiency is 32.5% and inbound bandwidth efficiency is 35.83%. The arterial’s two-way bandwidth efficiency is 34.16%. Figure 5.26 shows outbound and inbound programmed bands.

5.5.2 Dynamic Bandwidth

Dynamic bandwidth should be determined using intersection dynamic green durations. Figure 5.27 shows an example of cycle-by-cycle real-world bandwidth on US 70 arterial in Garner, NC.

Each intersection's cycle-by-cycle green duration is calculated from OASIS's split monitor log and cycle-by-cycle bandwidths are calculated manually.

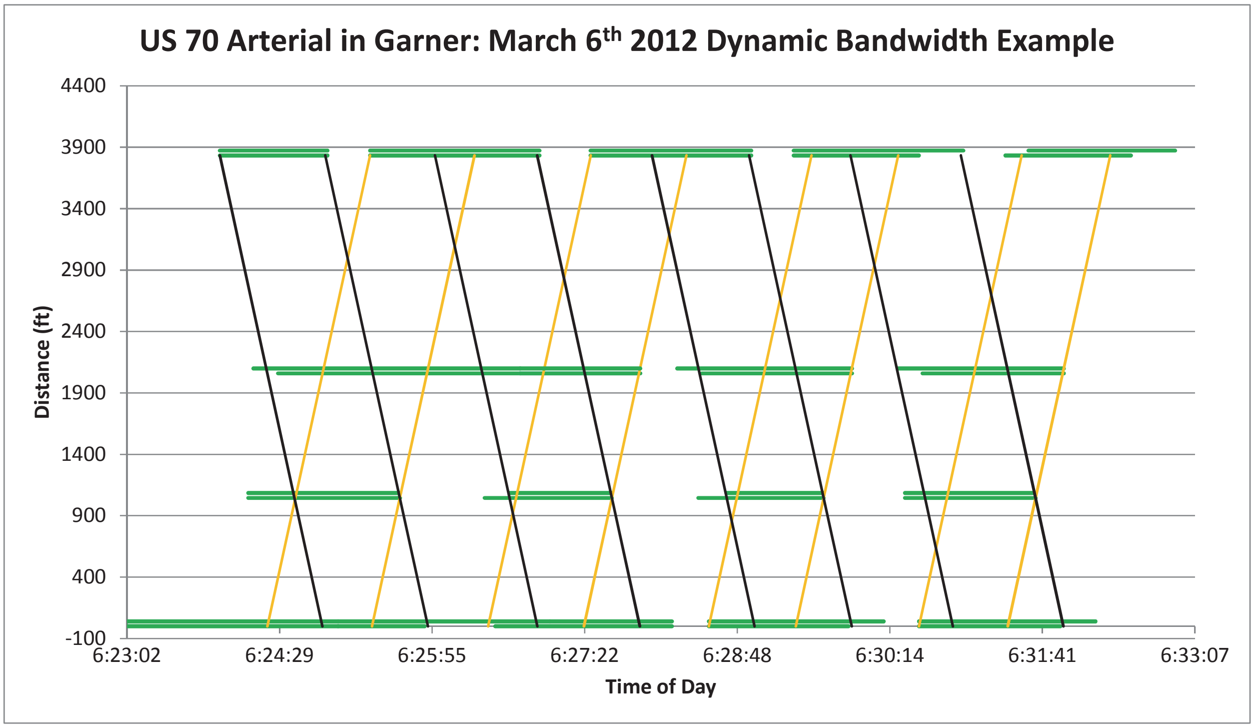


Figure 5-27 Dynamic Bandwidth on US 70 Arterial in Garner, NC

Table 5.14 shows corresponding dynamic band sizes. All the bandwidths are larger than the programmed bandwidth (see Figure 5.26) as expected.

Table 5-14 Cycle-by-Cycle Dynamic Bandwidth on US 70 Arterial in Garner

Outbound Bandwidth	Inbound Bandwidth
59	60
54	58
49	55
50	63

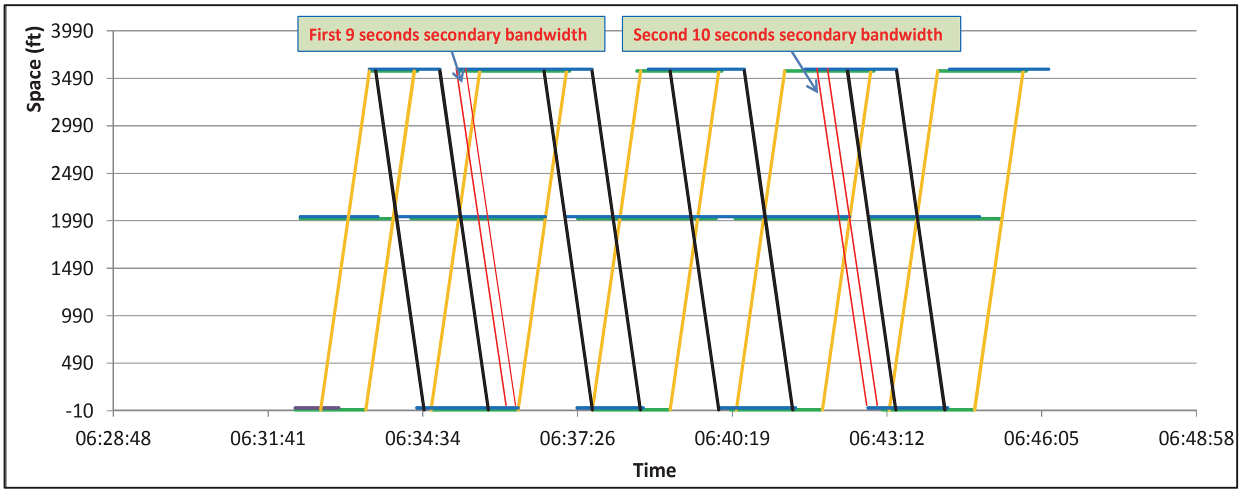


Figure 5-28 Dynamic Bandwidth on US 70 Arterial in Clayton, NC

Figure 5.28 and Table 5.15 show "Secondary Bandwidth" which is smaller than primary bandwidth and is created by early return to green and green extension combinations. The programmed bandwidths are 44 seconds and 39 seconds for outbound and inbound, respectively. All the dynamic bandwidths should be greater than or equal to programmed bandwidth, except secondary bands.

Table 5-15 Cycle-by-Cycle Dynamic Bandwidth on US 70 Arterial in Clayton

Outbound Bandwidth	Inbound Bandwidth
44	77
94	9
87	58
90	85
89	10
	59

Since cycle-by-cycle green durations are dynamic, provided bandwidth should be dynamic and there is possibility to have secondary bandwidth and non-programmed bandwidth (i.e. a band that is not programmed by traffic engineer). However, manually calculating and monitoring all dynamic bandwidth is difficult and time consuming. In addition, there is no tool to allow monitoring cycle-by-cycle bandwidth. This research developed such a tool.

5.6 Dynamic Bandwidth Analysis Tool

Dynamic Bandwidth Analysis Tool (DBAT) was developed for monitoring near real-time arterial dynamic bandwidth. Departing from the traditional method of defining bandwidth size using ideal travel time, offsets, and programmed green, the dynamic bandwidth is determined for each cycle using the actual coordinated green durations.

5.6.1 Processing OASIS Split Monitor Raw Data

An arterial street's bandwidth over the given sample of recorded cycles is the summation of the individual bandwidths in each cycle, as shown in Figure 5.29. Cycle-by-cycle dynamic greens are generated from OASIS split monitor log.

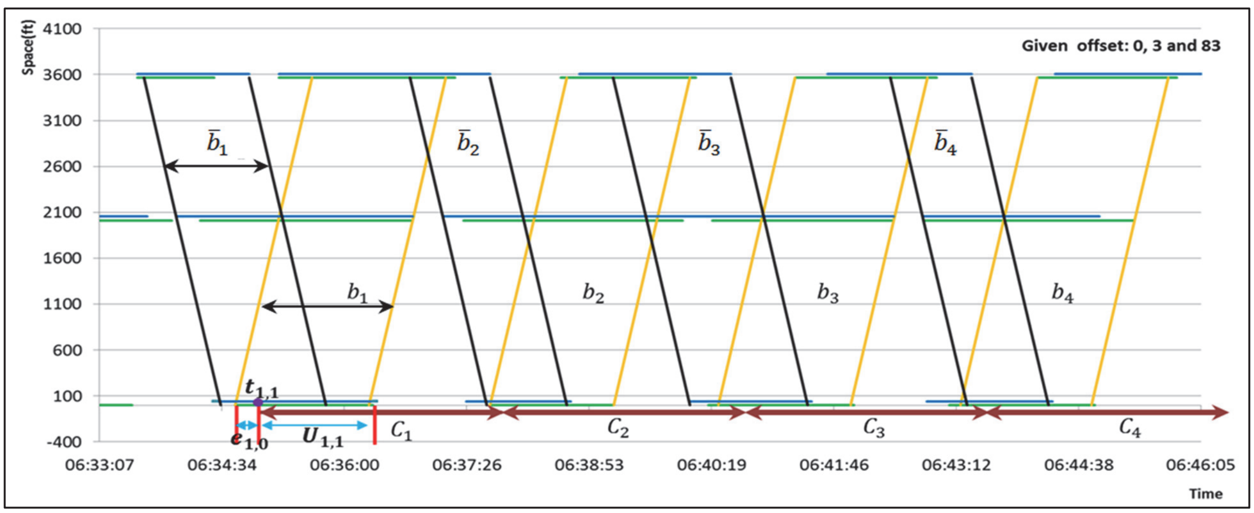


Figure 5-29 Dynamic Bandwidth Example

The following data items from the OASIS split monitor log are key to establishing the actual duration of each cycle's coordinated greens.

- SMTimeStamp: Start time of each cycle (reference point)

- ExtraTimeCP: Remaining time in the current cycle, which is going to be used by the coordinated phase (Used green time before reference point)
- UsedPhase: Actual green time, which is used in the current cycle for each phase (Used green time after reference point)

Figure 5.30 shows split monitor data for two cycles. In this example, the left-turn phase sequence is Lead-Lead so phases 2 and 6 start together. However, the ExtraTime for phase 2 has 13 seconds and ExtraTime for phase 6 has 27 seconds in the first cycle's. Therefore, phase 2 start 13 seconds earlier and phase 6 starts 27 seconds later than reference point (which is 6:34:54) in the second cycle. In addition, phase 2 and phase 6 in the second cycle will be terminated 42 seconds from the reference point.

					Used Phase							
SMTimeStamp	Cycle	Offset	Ex Time Phase 2	Ex Time Phase 6	1	2	3	4	5	6	7	8
10/17/2011 6:32:55	120	55	13	27	7	42	18	11	0	42	8	21
10/17/2011 6:34:54	120	55	16	30	7	42	19	7	0	42	0	34

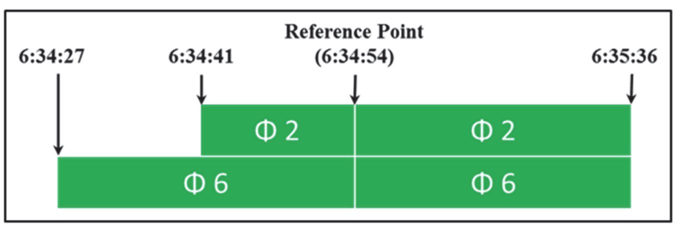


Figure 5-30 Split Monitor Example

Let $I, J,$ and K respectively denote the set of all intersections in the arterial system, all cycles, and both directions of traffic. Let $x_{i,j,k}$ and $y_{i,j,k}$ respectively represent the start and end time of the green phase in cycle $j \in J$ at intersection $i \in I$ in direction $k \in K$; and $t_{i,j}$ be the reference point (SMTimeStamp) in cycle $j \in J$ at intersection $i \in I$. Let $e_{i,j,k}$ (ExtraTimeCP) denote the time difference between the start time of through movement green duration in cycle $j \in J$ for direction

$k \in K$ at intersection $i \in I$ and the reference point. Let $u_{i,j,k}$ (UsedPhase) denote the amount of used green time started from $t_{i,j}$ in cycle $j \in J$ at intersection $i \in I$ for direction $k \in K$. The cycle-by-cycle dynamic green is defined by following equations:

$$x_{i,j,k} = t_{i,j} - e_{i,j-1,k}, \forall i \in I, \forall j \in J, \forall k \in K$$

$$y_{i,j,k} = t_{i,j} + u_{i,j,k}, \quad \forall i \in I, \forall j \in J, \forall k \in K$$

5.6.2 Development of DBAT

DBAT was developed in the C++ programming language. When DBAT reads input data from a dialog box, it generates each intersection's actual coordinated green time as an array. This process requires defining each intersection's green start and end time. The data processing equations shown above are an OASIS^{TM} specific example. For different traffic control software, these equations would need to be modified to match the software's signal data archive convention, to provide accurate green time calculations and time referencing.

DBAT directly reads the split monitor log file and creates dynamic green durations for each intersection and then generates a Boolean array of each intersection dynamic green and red. Afterwards it initializes the clock time to zero and rearranges each intersection's first green start time from the given offset value in the DBAT dialog box. By definition, DBAT sets the first intersection as the reference intersection. Therefore, the bandwidth search always starts at the first intersection and the first green. The minimum search interval is one second, and DBAT calculates bandwidth by counting the number of seconds that would accommodate a through vehicle trajectory traveling at the specified progression speed without encountering a red indication at any of the intersections. The progression speed (an input into the dialog box) is the slope of the vehicle trajectory. The rearranged Boolean array indicates each intersection's signal status.

DBAT creates a virtual vehicle trajectory for each second of the green durations at the first intersection of the system during the entire study period, T (which includes J cycles) and determines whether the trajectory reaches the downstream intersection during the green signal indication for the coordinated phase or during the red signal. If the trajectory intersects the red signal of the coordinated phase, the process is stopped, otherwise it continues to the downstream intersection. If the trajectory makes is through all intersections of the system, it is counted as a success and the same process is performed for the next time step (one second). The total number of successes in a row is the duration of a directional band.

This method can also identify secondary bands that may occur within a cycle as well as non-programmed bands that may arise in cases where the programmed green time provides no static bandwidth in one of the directions. Secondary bands, when they occur, result from a combination of early return green and/or green extension.

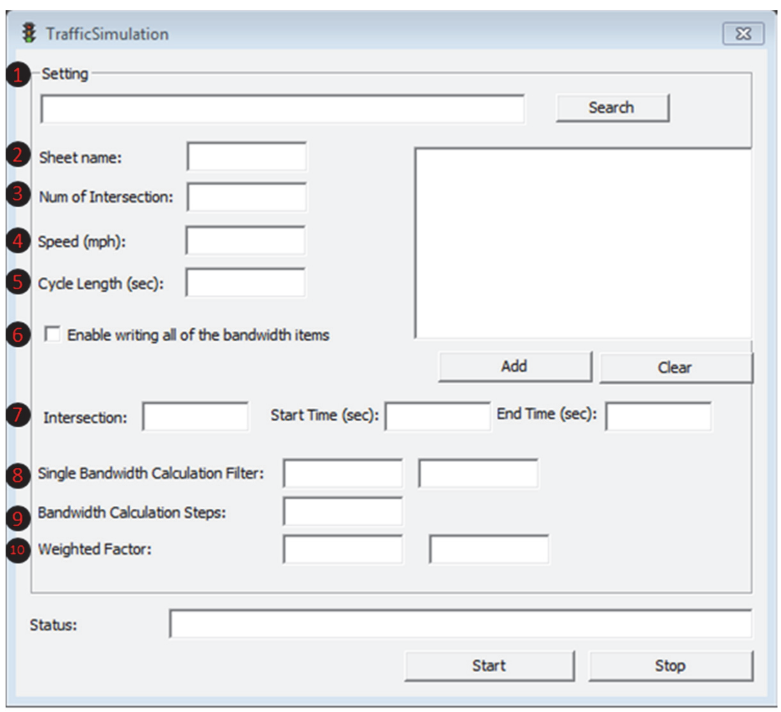


Figure 5-31 DBAT Interface

Figure 5.31 shows the DBAT dialog box. It requests 10 input data in the form. Input data 1 through 5 are essential information to calculate dynamic bandwidth. Input data 7 allows the user to set up the offset searching range and all possible offset combinations can be searched.

DBAT requires the following inputs:

- Free-flow speed or speed limit
- Total number of intersections
- Cycle length for analyzed time of day plan
- Order of intersection within search space
- Bandwidth search interval (minimum is 1 second)
- Directional bandwidth weighting factor

DBAT generates the following outputs:

- Inbound (outbound) sum of bands
- Inbound (outbound) number of bands
- Inbound (outbound) average band size
- Inbound (outbound) standard deviation of band size
- Inbound (outbound) band efficiency
- Bi-directional total sum of bands
- Bi-directional weighted total sum of bands
- Inbound (outbound) individual band per cycle (optional)

In order to calculate bandwidth efficiency, it is necessary to modify the original formulation since the dynamic bandwidth varies each cycle. Furthermore, bandwidth efficiency for dynamic bandwidth analysis has the capability to account for secondary bands and non-programmed bands as described above.

The conventional two-way bandwidth efficiency (CBE) formulation is as follows:

$$\mathrm{CBE} = \frac{BW_1 + BW_2}{2C} * 100$$

Where: C = cycle length in sec,

BW_1 = outbound bandwidth in sec, and

BW_2 = inbound bandwidth in sec.

The proposed two-way dynamic Bandwidth efficiency (DBE) formulation over a time period T is as follows:

$$\mathrm{DBE} = \frac{\sum_{i=1}^{i=m} BW_i + \sum_{k=1}^{k=n} BW_k}{C * (M + N)} * 100$$

Where: BW_i = cycle-by-cycle outbound dynamic bandwidth in sec,

BW_k = cycle-by-cycle inbound dynamic bandwidth in sec,

m = total number of outbound dynamic bands in time period T,

n = total number of inbound dynamic bands in time period T,

M = total number of outbound primary dynamic bands in time period T, and

N = total number of inbound primary dynamic bands in time period T.

5.6.3 DBAT Evaluation

DBAT results were verified using three classic and well-understood two-way progression schemes: alternate progression, double alternate progression, and simultaneous progression. The test results confirmed that DBAT provided the correct bandwidth solution in each case.

5.6.3.1 Alternate Progression

For certain block lengths with 50:50 splits and uniform block lengths, the optimal offset for maximizing bandwidth is a half cycle.

$$\frac{C}{2} = \frac{L}{S} = \text{Ideal offset}$$

Where: $C = \text{Cycle length, sec}$

$L = \text{block length, ft}$

$S = P\text{rogression speed, fps}$

To test DBAT, the following condition is created:

- Three signalized intersections on an arterial,
- Intersection spacing is 2,000 ft,
- Cycle length is 80 seconds,
- Effective green is 40 seconds for all intersections, and
- Progression speed is 50 fps.

Under the above scenario, the offset for the optimum bandwidth solution is 0, 40 and 0 seconds from first to last intersection, respectively. In addition, directional bandwidth is 40 seconds. The DBAT input file is created under the above scenario conditions and Table 5.16 shows the DBAT input file form.

Table 5-16 Input Data for Alternate Progression Scenario Test

Intersession A						Intersession B						Intersession C					
Time	Distance	ExtraTime CP1	ExtraTime CP2	Used Phase2	Used Phase6	Time	Distance	ExtraTime CP1	ExtraTime CP2	Used Phase2	Used Phase6	Time	Distance	ExtraTime CP1	ExtraTime CP2	Used Phase2	Used Phase6
7:00:00	0	0	0	40	40	7:00:00	2000	0	0	40	40	7:00:00	2000	0	0	40	40
7:01:20	2000	0	0	40	40	7:01:20	2000	0	0	40	40	7:01:20	0	0	0	40	40
7:02:40		0	0	40	40	7:02:40		0	0	40	40	7:02:40		0	0	40	40

From the input signal data, DBAT provides the following results with offsets 0, 40 and 0 as shown in Table 5.17. The “Band1” column expresses the total sum of bandwidth for phase 2 and the “Band2” column is the sum of bandwidth for phase 6. The “# of Band” column is the total number of directional bands and the “Ave Band” column is the sum of Bandwidth divided by “# of Band”. The DBAT test result exactly matches the alternate progression case example.

Table 5-17 DBAT test results of alternate progression

Int.1	Int.2	Int.3	Band1	Band2	# of Band1	# of Band2	Ave Band1	Ave Band2	Sum of band1, band2
0	40	0	80	80	2	2	40	40	160

The DBAT result shows that each direction made two bands and the average bandwidth size was 40 seconds. However, Table 5.17 does not provide cycle-by-cycle bandwidth size. When the user clicks the check box (Enable writing all of the bandwidth items), DBAT provides a summary table as well as cycle-by-cycle bandwidth. The result of the cycle-by-cycle bandwidths is 40 seconds for both directions similar to Figure 5.32.

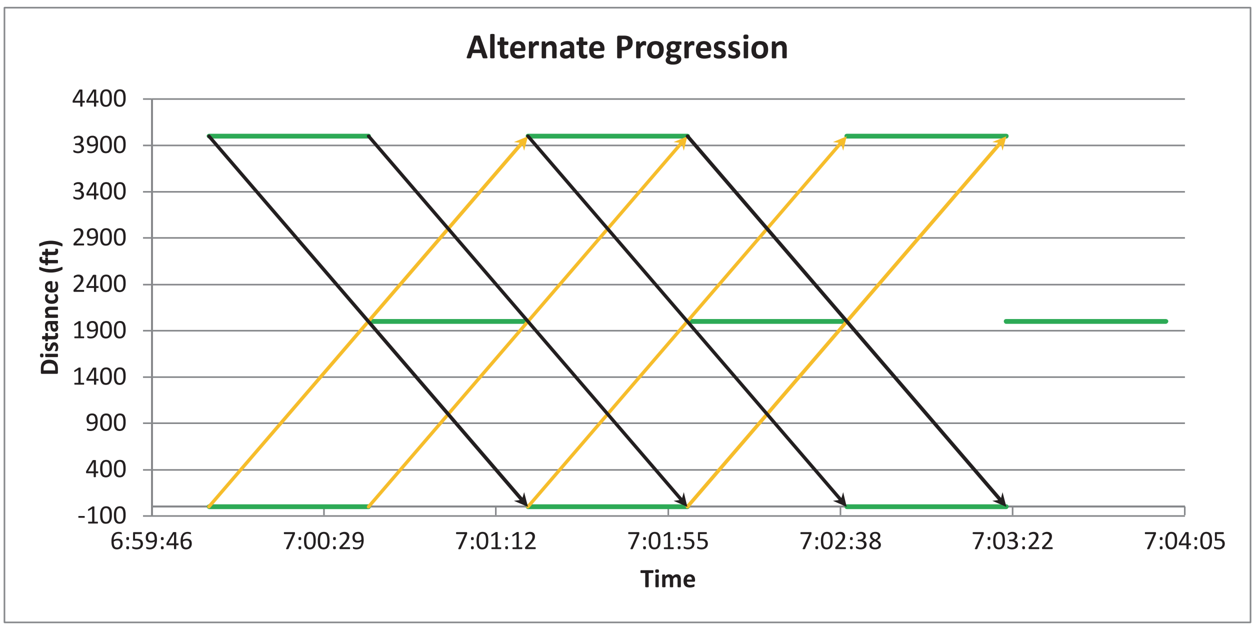


Figure 5-32 DBAT Test Result of Alternate Progression Case

5.6.3.2 Double Alternate System

For certain uniform block lengths with 50:50 splits, the ideal offset in either direction (over two blocks) is 2L/S, so that the sum of the two desired offsets just happens to be:

$$\frac{2L}{S} + \frac{2L}{S} = \frac{4L}{S} = C$$

$$\frac{C}{4} = \frac{L}{S} = \text{over tow block ideal Offset}$$

The travel time of each platoon along two consecutive blocks is exactly one-half of a cycle length, so that two such travel times add up to the cycle length.

For the testing purpose of DBAT, following condition is created:

- Four signalized intersections on arterial,
- Each intersection spacing is 1,000 ft,
- Cycle length is 80 seconds,
- Effective green is 40 seconds for all intersections, and
- Progression speed is 50 fps.

The ideal offset of the double alternate system is 0, 0, 40 and 40 seconds from first to last intersection, respectively. In addition, both directional bandwidth sizes are 20 seconds. Table 5.18 shows DBAT's test result. The DBAT results shows that the outbound direction made 3 bands and the inbound direction made 2 bands. The average bandwidth size is 20 seconds for both directions. DBAT provides correct bandwidths under the same offset.

Table 5-18 DBAT test results of double alternate progression

ID	Int.1	Int.2	Int.3	Int.4	Band1	Band2	# of Band1	# of Band2	Ave Band1	Ave Band2	Sum of band1, band2
1	0	0	40	40	60	40	3	2	20	20	100

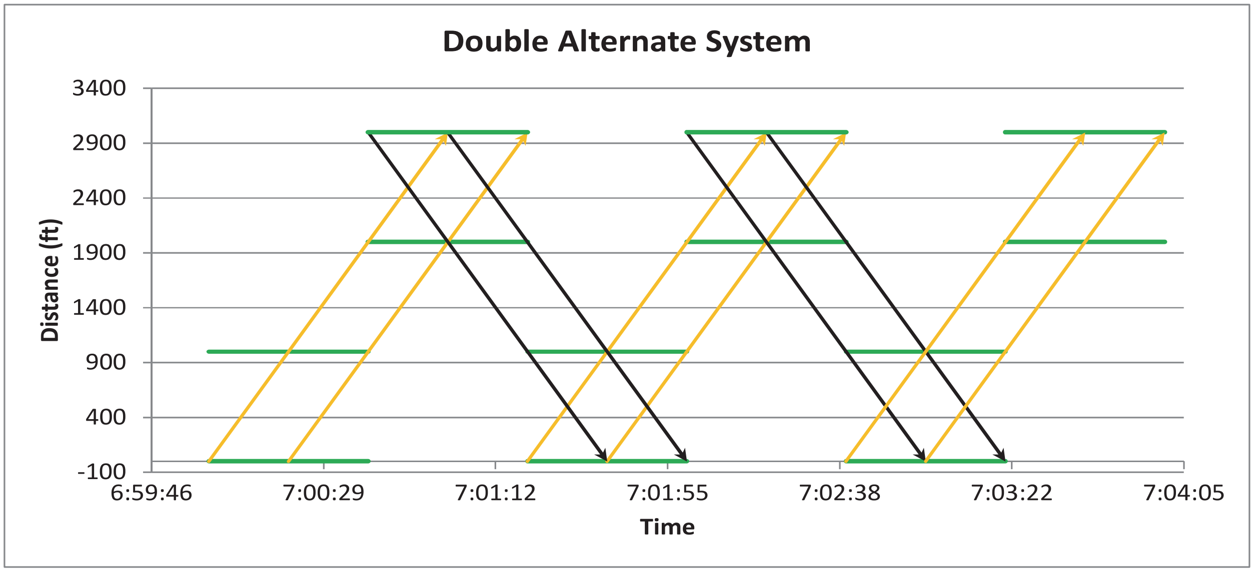


Figure 5-33 DBAT Test Result of Double Alternate System Case

The result of cycle-by-cycle bandwidths is also 20 seconds for both directions like Figure 5.33.

5.6.3.3 Simultaneous System

For very closely space signals, or for rather high vehicle speeds, simultaneous systems might be one of the best operation strategies. For the testing purpose of DBAT, following condition is created:

- Four signalized intersections on arterial,
- Each intersection spacing is 400 ft,
- Cycle length is 80 seconds,
- Effective green is 40 seconds for all intersections,
- Progression speed is 40 fps, and
- Each intersection offset is 0 for all intersection.

Under the given scenario, outbound and inbound bandwidth should be 10seconds, and the DBAT calculation result was confirmed to have the exact same result like Table 5.19 and Figure 5.34.

Table 5-19 DBAT test results of double alternate progression

ID	Int.1	Int.2	Int.3	Int.4	Band1	Band2	# of Band1	# of Band2	Ave Band1	Ave Band2	Sum of band1, band2
1	0	0	0	0	30	30	3	3	10	10	60

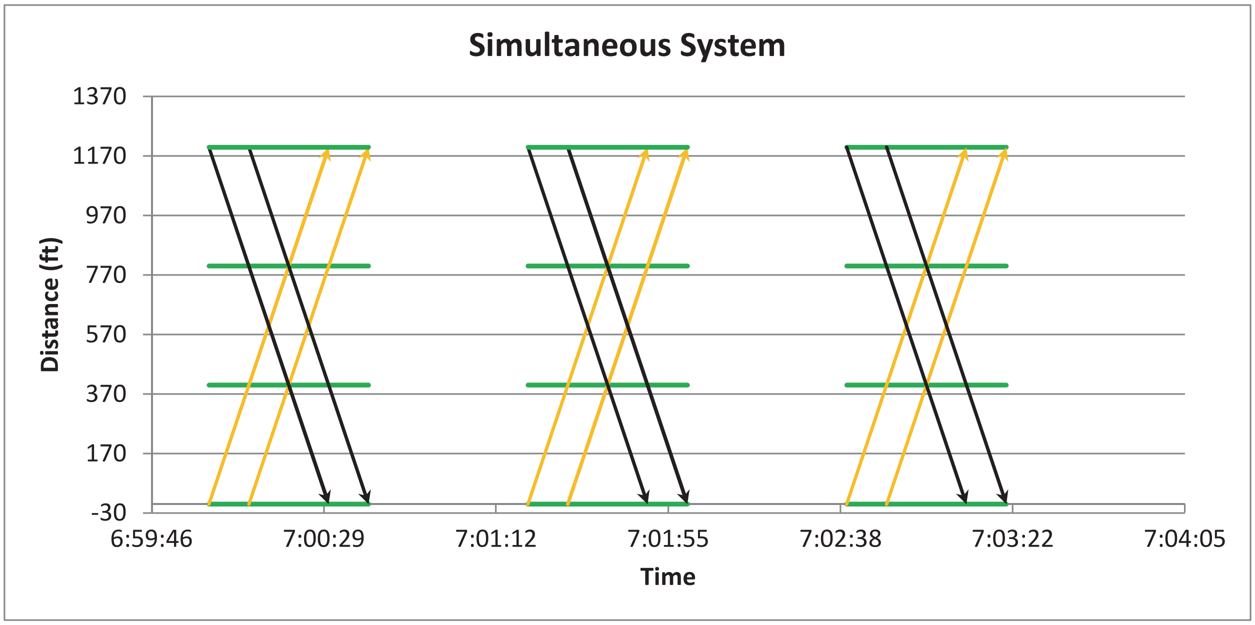


Figure 5-34 DBAT Test Result of Simultaneous System Case

5.7 Summary

Floating car and Bluetooth travel time studies show that arterial travel time is highly correlated to the number of stops. The arterial travel time distribution has a multi-modal distribution (see Figure 5.1 and Figure 5.2) depending on the number of stops. In addition, the number of stops is directly related to with the signal coordination (see Figure 5.3). The Tru-Traffic time-space diagram in Figure 5.3 shows the relationship between through vehicle number of stops and signal coordination. This result supports the importance of signal coordination. In addition, the comparison result of floating car and Bluetooth travel time (see Table 5.9) supports the accuracy of Bluetooth travel time.

Lane-based 6X6 single loop detector detection error rate is 1.17% during non-peak hour and 3.92% during peak hour. From the OASIS detector log, the daily volume profile can be generated and can be used to evaluate the time of day plan (see Figure 5.12).

The split monitor log file is used for creating coordinated phases' dynamic g/C profile, early return to green and green extension distribution, and non-coordinated phases' used green distributions. The combined g/C profile and volume profile show the possibility of signal failure (see Figure 5.17 and Figure 5.18). This monitoring method can be used for evaluating coordinated phases' green duration. Coordinated phases' early return to green and green extension distribution monitoring graph shows its variability. This variability makes signal coordination difficult. As mentioned earlier, early return to green can increase inefficiency of progression quality on the coordinated arterial. Non-coordinated phases' used green distribution monitoring results can be used to evaluate the suitability of phase length, minimum green settings, and maximum green settings.

Since coordinated phases have early return to green and green extensions, conventional bandwidth should not be the same as the field observed bandwidth (which the project team calls dynamic bandwidth). The Project team developed the Dynamic Bandwidth Analysis Tool (DBAT) for monitoring cycle-by-cycle real-world bands using the OASIS split monitor log. DBAT results were verified using three classic and well-understood two-way progression problems: alternate progression, double alternate progression, and simultaneous progression. The test results confirmed that DBAT provided solutions identical to the optimal solution of each case.

CHAPTER 6. ESTIMATE MODEL PARAMETERS, TEST MODEL ACCURACY, AND INVESTIGATE ADAPTIVE IMPLEMENTATIONS (TASK 6)

In this chapter, dynamic bandwidth analysis results will be introduced for the three study sites introduced in Chapter 3. The collected OASIS split monitor logs for dynamic bandwidth analysis are:

- Site A (US 70 arterial in Clayton): April 22 to April 25, 2013 (four weekdays)
- Site B (US 70 arterial in Garner): February 20 to 24, 2012 (five weekdays)
- Site C (NC 55 arterial in Apex) : Aug 27 to 31, 2012 (five weekdays)

6.1 Dynamic Bandwidth Analysis

The peak directions for site A, B and C are outbound, inbound and outbound, respectively. Table 6.1 summarizes the total duration, number, average duration, and overall efficiency of dynamic green bands for outbound and inbound directions for all three sites. These results are provided for different weekdays, and the total sum of bandwidth in both directions is also provided. The number of bands includes secondary and non-programmed bands. All bands shorter than the programmed band are defined as secondary bands as mentioned earlier in Chapter 5. For Site C, NC 55, the programmed signal timings provide no bandwidth in the inbound direction, and therefore the green bands provided are shaded to indicate they are non-programmed bands. The result shows daily fluctuation of bandwidth as well as the number of bands, which are created.

Table 6-1 DBAT Arterial Dynamic Bandwidth Analysis Results

Site	Day	Out-bound Total Duration (Sec)	In-bound Total Duration (sec)	# of Out-bound Bands	# of In-bound Bands	Out-bound Average Duration (sec)	In-bound Average Duration (sec)	Out-bound Efficiency (%)	In-bound Efficiency (%)	Out- & In-bound Total Duration (sec)
A	1	1,661	1,315	21	23	79.1	57.17	46.5	38.7	2,976
	2	1,655	1,360	21	22	78.8	61.82	46.4	40.0	3,015
	3	1,673	1,157	21	22	79.7	52.59	46.9	34.0	2,830
	4	1664	1,302	21	20	79.2	65.1	46.6	38.3	2,966
B	1	6,698	7,220	139	138	48.2	52.3	40.5	43.7	13,918
	2	6,846	7,493	139	137	49.3	54.7	41.6	45.6	14,415
	3	6,634	7,186	137	138	48.4	52.1	40.4	43.7	13,896
	4	6,497	7,109	137	137	47.4	51.9	39.5	43.2	13,682
	5	6,532	7,016	137	137	47.7	51.2	39.7	42.7	13,624
C	1	3,152	196	40	36	78.8	5.4	49.1	3.1	3,419
	2	3,159	186	40	30	79.0	6.2	49.2	2.9	3,413
	3	3,207	172	40	33	80.2	5.2	49.9	2.7	3,447
	4	3,204	193	40	37	80.1	5.2	49.9	3.0	3,465
	5	3,135	126	40	28	78.4	4.5	48.8	2.0	3,329

Non-programmed bandwidths are shaded

Table 6.2 shows Site A's cycle-by-cycle bandwidth results for each analysis day. Outbound bandwidth does not have secondary bands but inbounds has several secondary bands. The gray color represents secondary bands. A total of seven secondary bands are generated naturally four of which are less than 4 seconds and 3 band are longer than 9 seconds.

Figure 6.1 shows the dynamic bandwidth plot of first five cycles on the first day on site A. It shows two secondary bands. The first secondary band is 9 seconds and the second one is 10 seconds.

Table 6-2 Dynamic Bandwidth Computation Result of Site A from DBAT

Outbound				Inbound			
Day 1	Day 2	Day 3	Day 4	Day 1	Day 2	Day 3	Day 4
44	44	44	44	77	76	59	57
94	85	97	92	9	73	82	84
87	89	82	83	58	81	11	77
90	87	79	88	85	59	40	59
89	93	86	92	10	3	58	58
92	86	84	89	59	57	59	87
90	91	100	87	84	93	82	59
69	69	74	86	60	76	73	58
72	80	71	69	73	56	60	54
59	69	79	72	56	73	59	77
69	72	72	68	78	56	59	56
79	72	69	74	70	79	74	77
72	82	86	72	1	74	58	82
81	70	75	69	59	76	1	59
78	73	81	82	59	67	60	56
81	82	86	80	80	3	73	57
78	83	80	85	59	49	39	56
74	83	75	78	74	51	54	58
91	77	85	86	56	73	39	73
85	84	83	81	56	58	39	58
87	84	85	87	78	73	39	
				39	54	39	
				39			

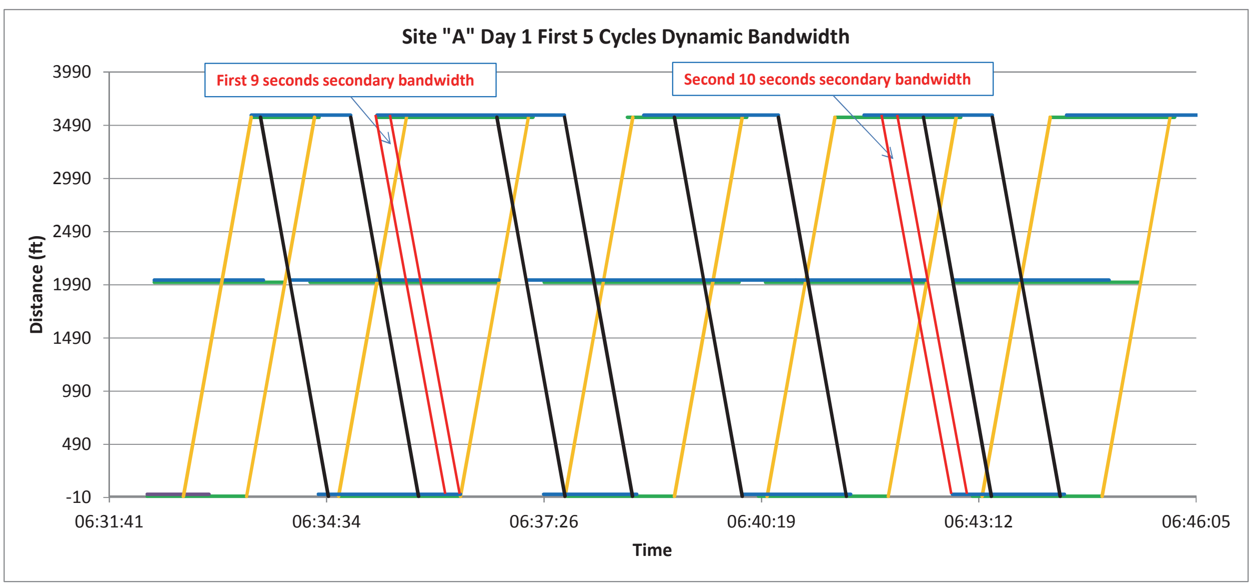


Figure 6-1 US 70 Arterial in Clayton Primary Band with Secondary Band

6.2 Comparison of Dynamic and Programmed Bandwidth Efficiency

Table 6.3 shows the programmed bandwidth and daily dynamic bandwidth efficiency for all three study-sites. As expected, the programmed bandwidth underestimated the actual bandwidth that was provided in all cases. For Site A, programmed bandwidth underestimates the actual directional bandwidths by 48% to 81%. For Sites B and C, programmed bandwidth underestimated the actual bandwidths by 19% to 28% and 13% to 23%, respectively. Furthermore, for the NC 55 arterial in Apex, non-programmed bands were detected. Therefore, the non-coordinated direction has measurable bandwidth efficiency although the average band duration was at most six seconds at this site.

Table 6-3 Bandwidth Efficiency Comparisons

Site	Type		Outbound Bandwidth Efficiency	% Difference	Inbound Bandwidth Efficiency	% Difference	Two way Bandwidth Efficiency	% Difference
A	**Programmed Bandwidth**		**25.88**	**-**	**22.94**	**-**	**24.41**	**-**
	Observed Dynamic Bandwidth	Day 1	46.53	79.79	38.68	68.61	42.70	74.93
		Day 2	46.36	79.13	40.00	74.37	43.26	77.22
		Day 3	46.86	81.07	34.03	48.34	40.60	66.33
		Day 4	46.61	80.10	38.29	66.91	42.55	74.31
B	**Programmed Bandwidth**		**32.5**	**-**	**35.83**	**-**	**34.16**	**-**
	Observed Dynamic Bandwidth	Day 1	40.49	24.58	43.73	22.05	42.11	23.27
		Day 2	41.64	28.12	45.58	27.21	43.61	27.66
		Day 3	40.35	24.15	43.71	21.99	42.03	23.04
		Day 4	39.52	21.60	43.24	20.68	41.38	21.14
		Day 5	39.73	22.25	42.68	19.12	41.20	20.61
C	**Programmed Bandwidth**		**43.12**	**-**	**0**	**-**	**21.56**	**-**
	Observed Dynamic Bandwidth	Day 1	49.13	13.94	3.06	-	26.10	21.06
		Day 2	49.19	14.08	2.91	-	26.05	20.83
		Day 3	49.92	15.77	2.69	-	26.31	22.03
		Day 4	49.88	15.68	3.02	-	26.45	22.68
		Day 5	48.83	13.24	1.97	-	25.4	17.81

The FHWA *Traffic Signal Timing Manual* defines *Great progression*, *Good progression*, and *Fair progression* based on bandwidth efficiency ranges of 37% to 100%, 25% to 36%, and 13% to 24%, respectively. For the programmed bandwidth, FHWA arterial coordination guidelines would label the existing two-way bandwidth efficiency for US 70 in Clayton (Site A) as *Fair progression* during the AM peak plan period. In contrast, dynamic bandwidth for the four days analyzed was considerably larger than the programmed bandwidth. Calculating the efficiency of this arterial based on the dynamic bandwidth results in *Great progression* in the AM peak period.

6.3 Field Dynamic Bandwidth Distribution

Figure 6.2 and Figure 6.3 illustrate the outbound and inbound dynamic bandwidth distribution for the US 70 arterial in Clayton (Site A). The programmed outbound and inbound bandwidths are 44 seconds and 39 seconds, respectively. However, most of the observed dynamic bands are considerably larger than these values. In fact, the mode of both outbound and inbound bandwidth is larger than the programmed band.

Table 6.4 provides a summary of dynamic green band frequency for the three study sites. In case of US 70 in Garner (Site B), the inbound programmed bandwidth was also the most frequently observed dynamic green band. Nonetheless, both outbound and inbound dynamic bandwidths cover a wide variety of durations at this site. Even in this case where the programmed bandwidth is the most frequently observed dynamic green band, programmed bandwidth alone do not accurately represent the reality of the coordination. For NC 55 in Apex (Site C), the inbound direction was not coordinated. However, a total of 164 dynamic bands were created over the 200 cycles. Although it should be noted that most of the non-programmed bands were shorter than 6 seconds, several green bands with durations of longer than 10 seconds were observed.

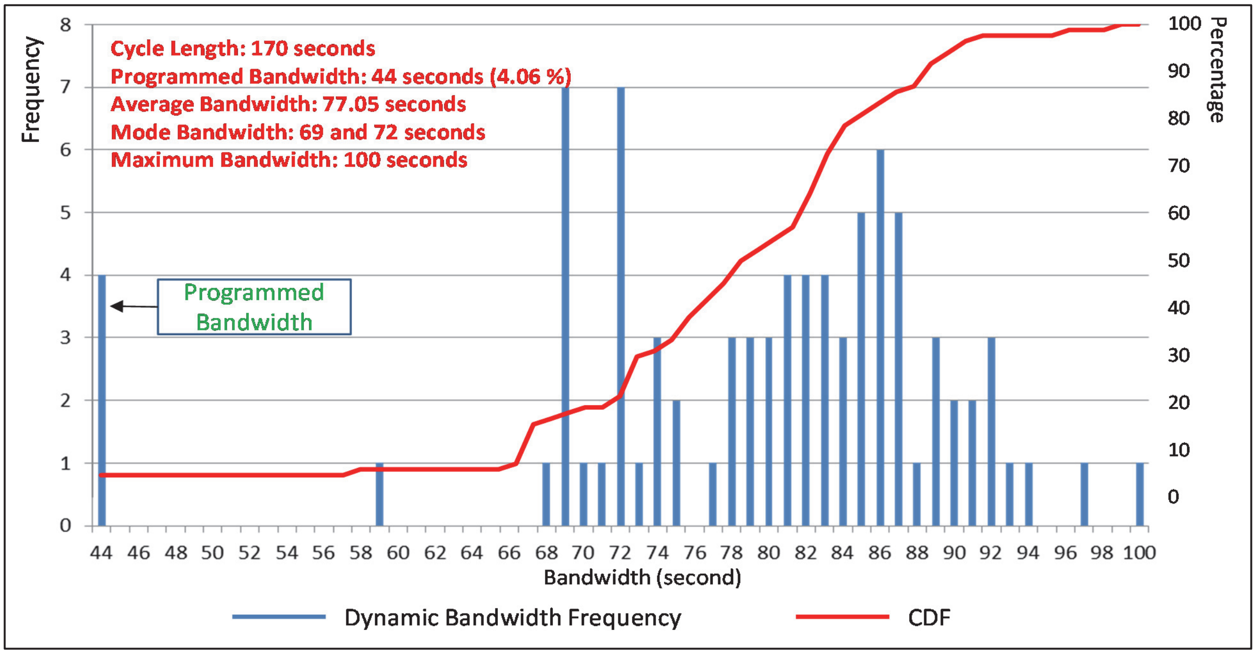


Figure 6-2 Site “A” Outbound Bandwidth PDF and CDF

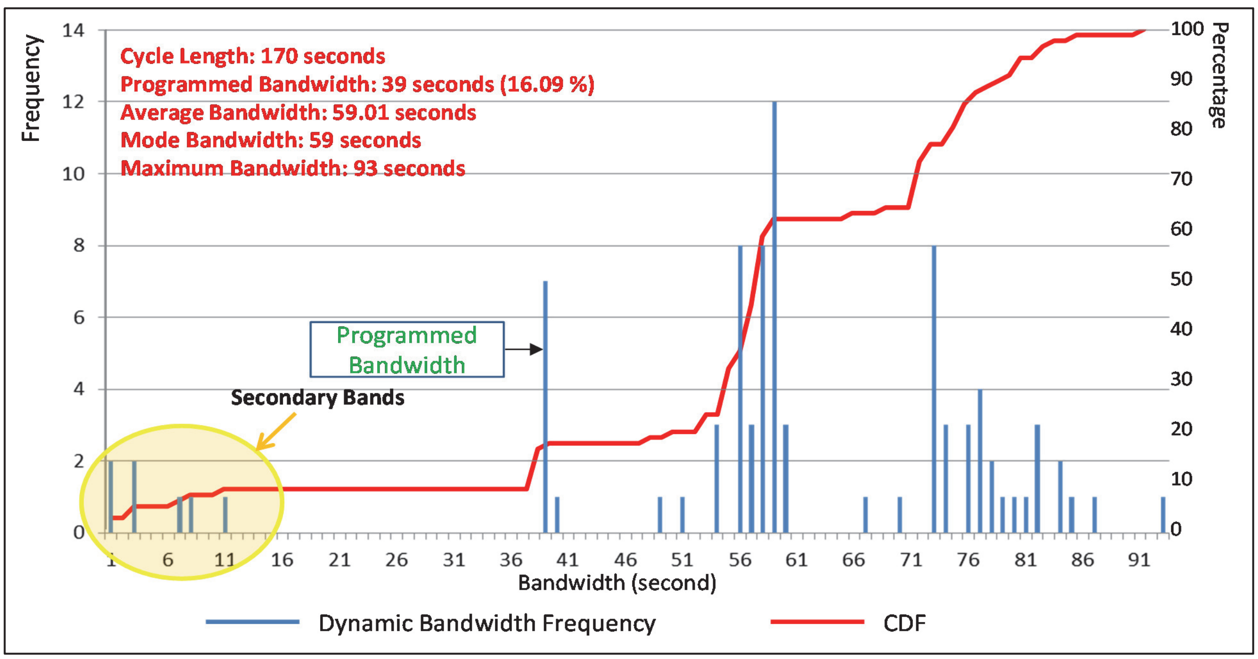


Figure 6-3 Site “A” Inbound Bandwidth PDF and CDF

Table 6-4 Bandwidth Comparisons

Site	Direction	Programmed Bandwidth (Relative Frequency of Programmed Bandwidth)	Actual Dynamic Bandwidth			
			Number of Bands	Average (sec)	Mode (sec)	Max (sec)
A	Outbound	44 sec (4.06%)	84	77.05	69 and 72	100
	Inbound	39 sec (16.09%)	87	59.01	59	93
B	Outbound	39 sec (0.29%)	689	48.87	51	77
	Inbound	43 sec (22.27%)	687	53.19	43	77
C	Outbound	69 sec (5.50%)	200	79.28	75	93
	Inbound	0 sec (18.00%)	164	5.32	6	33

Programmed bandwidth's proportional frequency ranged from 0.29% to 22.27% across the three study sites. In addition, each arterial has a unique dynamic bandwidth distribution making it difficult to draw general conclusions and to represent dynamic bandwidth distributions with a single statistic.

6.4 Summary

In this chapter, dynamic bandwidth analysis results are presented. For all three study sites, one week OASIS split monitor data were processed by the developed Dynamic Bandwidth Analysis Tool. DBAT uses dynamic bandwidth analysis methodology that provides actual dynamic bandwidth durations using closed loop signal data to assess the performance of semi-actuated coordinated arterial streets. This methodology was designed to calculate dynamic bandwidth from signal ATMS data. Traditionally, calculated programmed green bandwidth provides limited insight for evaluating the quality of arterial coordination because it cannot assess the impact of early return to green and green extension for the coordinated phases. Detailed analysis' at three study sites confirmed that dynamic coordinated green times along signalized arterials have complex multimodal distributions. This finding underscores the value of cycle-by-cycle dynamic bandwidth analysis for both the evaluation offsets. The developed methodology represents an important first step in formalizing and enabling dynamic bandwidth using cycle-by-cycle phase duration data from OASIS Split monitor log.

Key findings from the field data analysis include:

- Total dynamic bandwidth and bandwidth efficiency are considerably larger than the corresponding programmed bandwidth and bandwidth efficiency
- Dynamic bandwidths can include secondary and non-programmed bands
- Dynamic bandwidths distributions are complex and characteristically multi-modal

CHAPTER 7. PERFORM RIGOROUS COMPARATIVE ASSESSMENT OF MODEL PERFORMANCE VERSUS CONVENTIONAL PLAN EVALUATION METHODS (TASK 7)

The early return to green and green extension bring up the concept of using dynamic greens extracted from OASIS split monitor log for offset optimization. In particular, optimizing the offsets based on the dynamic green times to maximize dynamic bandwidth has great potential to yield even more efficient arterial performance.

The primary objective of this investigation is to test the feasibility of improving dynamic bandwidth efficiency by optimizing offsets, for a set of observed cycles. For this purpose, an exhaustive search method is used that enumerates all possible offset combinations and determines bandwidth efficiency for each combination using DBAT. Next, the solutions are sorted based on a performance measure of interest (e.g., bandwidth efficiency or weighted sum of bi-directional bandwidth). From this sorted list, the best or a certain target value of top solutions can be selected. The proposed exhaustive search approach can be used to evaluate the possibility of further improvements in dynamic bandwidth efficiency and to observe the search space of the problem.

7.1 DBAT Exhaustive Search Method

In order to perform an exhaustive analysis with dynamic bandwidth, the DBAT algorithm described above was embedded in a routine that enumerates all possible offset combinations. As with the DBAT bandwidth analysis algorithm, the minimum offset search interval was set to 1 second. The search space of the offset optimization problem is a function of the cycle length and the number of intersections as follows:

$$\text{Number of feasible solutions} = C^{I-1}$$

Where, C = cycle length

I = number of intersections

The DBAT exhaustive search routine considers directional demand using a target value "k" as defined in the MAXBAND formulation. "k" represents the desired ratio of the sum of inbound to the sum of outbound dynamic bandwidth and should be set based on the ratio of the corresponding directional traffic demand.

$$k = \frac{\sum \overline{b_i}}{\sum b_i}$$

Where, $k = target\ value$

$\bar{b} = inbound\ dynamic\ bandwidth$

$b = outbound\ dynamic\ bandwidth$

The exhaustive search method ensures that the solution is globally optimum and provides important information on the shape of the solution space and the robustness of different solutions. However, calculation time increases to the point of becoming impractical as the number of intersections increases. If spanning of the search space is desired for arterials with higher numbers of intersections, it is possible to reduce the number of solutions by increasing the search time increment.

7.2 DBAT Exhaustive Search Results

All three sites were analyzed with DBAT's exhaustive search routine. For each site, the current programmed offsets were highlighted in the solution set along with two globally optimal solutions.

One of the highlighted optimal solutions duplicates the directional weighting implied by the programmed bandwidth, and the other one is based on equal directional weighting.

The results in Figure 7.1 were generated using this function applied to the AM peak plan for the US 70 arterial in Clayton (site A), which has three intersections. The cycle length in the AM peak plan is 170 seconds. Therefore, the total number of unique offset combinations with a 1 second search interval is $170^2 = 28{,}900$. This is a manageable search space for the exhaustive search method. Figure 7.1 shows the total bandwidth (on the y-axis) for all the combinations of offset (on the x-axis) at this site. The x-axis is plotted using a unique ID assigned to each offset combination. The maximum sum of bandwidth with directional weighting equal to the programmed ratio for this site is 3,002 seconds (see Table 7.1 for more information). In the case of site A, the programmed offset dynamic bandwidth is quite close to the best solution found by the exhaustive search approach. The difference between them is a mere 26 seconds over a one-hour time period. In addition, there are 12 Pareto optimal solutions under equal directional bandwidth weighting, each yielding an optimal equally weighted sum of dynamic bandwidth of 2,860 seconds.

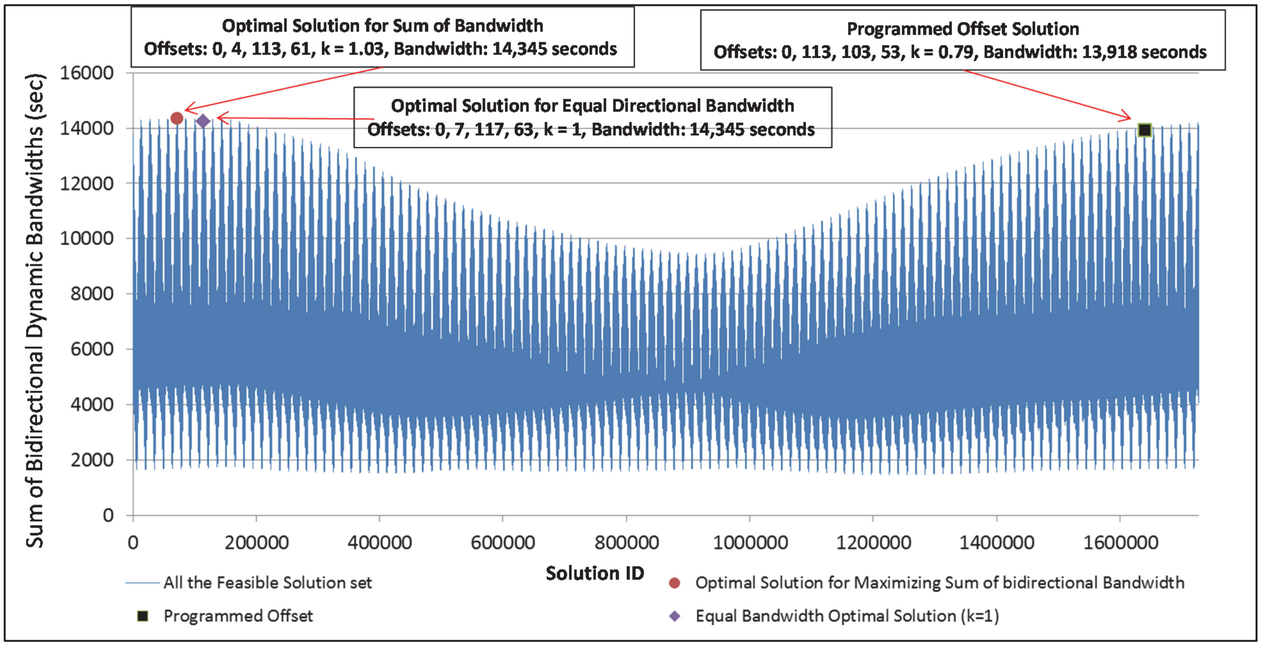


Figure 7-1 Site "B" Day 1 AM Peak Plan Exhaustive Search Result

Figure 7.2 shows the US 70 in Garner's (site B) exhaustive search results. Site B has four signalized intersections with a 120 second cycle length. Therefore, there are $120^3 = 1{,}728{,}000$ feasible offset solutions. Among those feasible solutions, Figure 7.2 highlights the two optimal solutions described above. The optimal solution with weighting based on the programmed offsets yields a dynamic bandwidth total of 14,345 seconds and the equal weighting optimal solutions yields 14,224 seconds. Compared with programmed offsets, the global optimal offset provides 427 seconds more total bandwidth over the four hours of the AM peak plan.

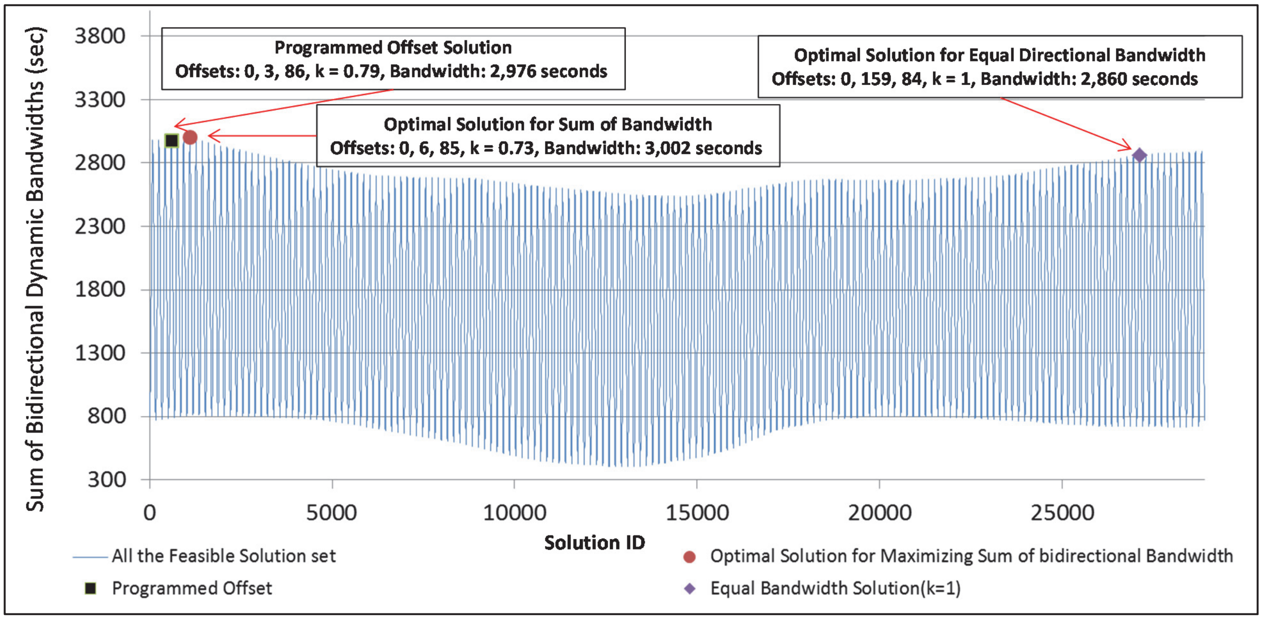


Figure 7-2 Site “A” Day 1 AM Peak Plan Exhaustive Search Result

Figure 7.3 shows the results of the exhaustive search method for the NC 55 arterial in Apex (site C). This arterial has seven intersections. Analyzing all seven intersections with the exhaustive search is impractical, and therefore the study case uses the last three intersections only (intersection 5, 6, and seven in Figure 3.2).

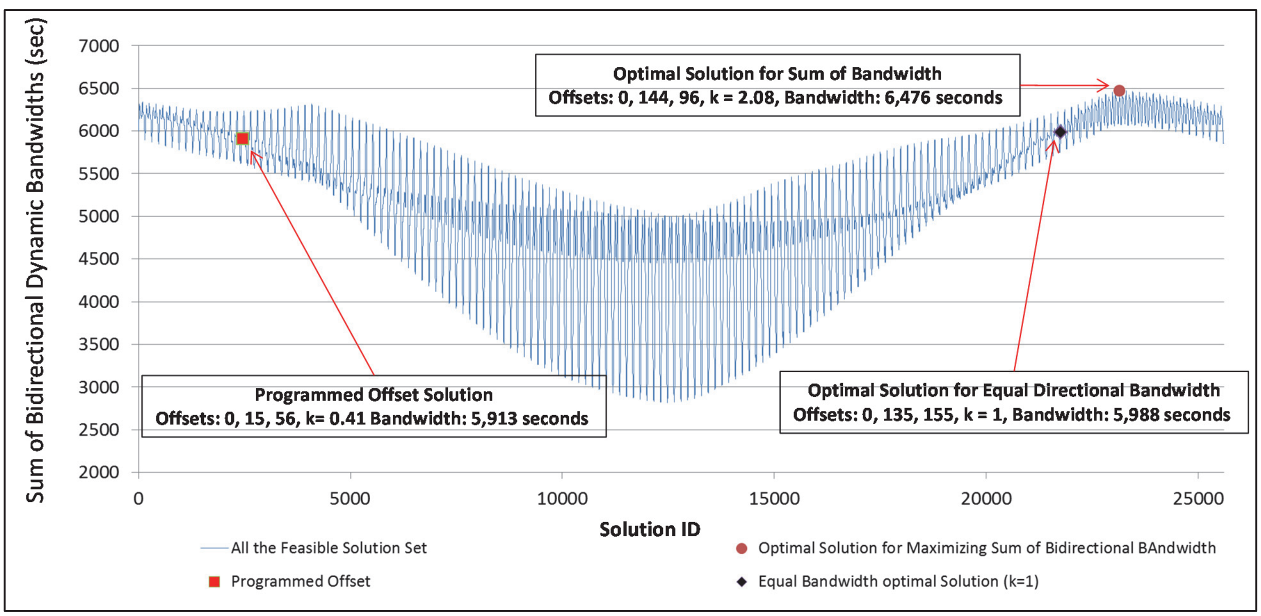


Figure 7-3 Site “C” Day 1 AM Peak Plan Last 3 Intersection Exhaustive Search Result

The selected time of day plan is AM peak from 7:00 to 9:10 and the cycle length is 160 seconds. From the exhaustive search, the global directionally weighted solution has a total sum of bandwidth of 6,476 seconds. In addition, it can be seen in Figure 7.3 that this global solution iS. Robust against offset changes as minor changes in offset reduce the total sum of bandwidth to around 6,000 in the worst case. The low sensitivity to offset permutation of this solution indicates that the corresponding offsets can provide good progression even if expected early return to green and green extension values vary from those represented in the analyzed cycles. Compared to the programmed offset sum of dynamic bandwidth (5,913 seconds), the optimal offset solution provides 563 seconds more total bandwidth. This means that the optimal solution would have provided an average of 12.5 seconds more bandwidth per cycle across 45 cycles analyzed in site C.

Table 7-1 Programmed Dynamic Bandwidth with Un-weighted Optimal Solutions

Site	Solution type	k	Offset (sec)	Outbound Sum of Bandwidth (sec)	Inbound Sum of Bandwidth (sec)	Overall % Change
A	Observed Dynamic Bandwidth	0.79	0,3,86	1,661	1,315	-
	Optimal Solution for Sum of Bandwidth	0.73	0,6,85	1,729	1,273	0.87%
	Optimal Solution for Equal Bandwidth	1	0,159,84	1,430	1,430	-3.9%
B	Observed Dynamic Bandwidth	1.08	0, 113, 103, 53	6,698	7,220	-
	Optimal Solution for Sum of Bandwidth	1.03	0, 4, 113, 61	7,079	7,266	3.07%
	Optimal Solution for Equal Bandwidth	1	0, 7, 117, 63	7112	7112	2.20%
C	Observed Dynamic Bandwidth	0.41	0, 15, 56	4,176	1,737	-
	Optimal Solution for Sum of Bandwidth	2.08	0, 144, 96	2,099	4,377	9.52%
	Optimal Solution for Equal Bandwidth	1	0, 135, 155	2,994	2,994	1.27%

7.3 Evaluation of DBAT Solution

Using an exhaustive search, the set of offsets that are expected to provide the highest value of weighted sum of bandwidths on the US 70 arterial in Clayton, NC (Site A) were found for each time of day plan. To find these offsets, two weeks data from split monitor logs were collected in April 2013 and then analyzed. An exhaustive search method was used to cover all possible combinations of offsets. For each set, total band size and weighted sum of band size were determined. Note that these values are “expected” to be observed in the field if the offsets were to be implemented; however, due to variations in demand level, we may not see identical values. The set of offsets that provided the highest expected weighted sum of bandwidth was selected and implemented in the field. For a period of two weeks in August 2013, green durations and start times were recorded to determine the band size in each direction, total band size, and weighted sum of band size. In addition, arterial through travel times were collected before and after offsets were changed using Bluetooth units.

Table 7-2 Site "A" April Two weeks Split Monitor data DBAT Process Result

Cycle Length (sec)	Plan Start (Plan End)	Plan	Offset (sec) *		EB Band Size (sec)	WB Band Size (sec)	Total Band Size (sec)	Weighted Sum of Band (% Change) **	EB (WB) Weight ***
			Int. 2	Int. 3					
170	6:20 (7:30)	April field Plan	3	86	1,547	1,202	2,749	1,325	0.36 (0.64)
		Estimated Optimal Plan****	0	85	1,524	1,229	2,753	1,334 (0.68%)	
140	7:30 (9:30)	April field Plan	135	73	2,445	2,478	4,923	2,463	0.46 (0.54)
		Estimated Optimal Plan****	136	79	2,206	2,723	4,929	2,485 (0.89%)	
120	9:30 (11:30)	April field Plan	108	62	2,458	2,237	4,695	2,365	0.58 (0.42)
		Estimated Optimal Plan****	107	60	2,519	2,241	4,759	2,402 (1.56%)	
130	11:30 (13:30)	April field Plan	118	53	2,411	1,709	4,121	2,140	0.61 (0.39)
		Estimated Optimal Plan****	117	57	2,327	1,908	4,235	2,165 (1.17%)	
150	13:30 (16:30)	April field Plan	4	47	4,948	1,848	6,797	3,913	0.67 (0.33)
		Estimated Optimal Plan****	137	54	4,783	2,774	7,557	4,112 (5.09%)	
220	16:30 (18:30)	April field Plan	211	3	3,080	1,895	4,975	2,735	0.71 (0.29)
		Estimated Optimal Plan****	172	5	3,156	1,853	5,009	2,777 (1.54%)	

* Intersection 1 is the reference intersection. Therefore, its offset is always set at zero
** Objective Function: Two week average directional band size multiplied by directional weight
*** Two week average directional volume ratio
**** Maximized weighted sum of band using two week split monitor

Table 7.2 summarizes the actual and estimated optimal EB, WB, Total, and Weighted sum of bandwidths at Site A. In all time of day plans except for one (shaded), total band size and weighted sum of bandwidths are almost equal. This means that the implemented offsets for these time-of-day plans were quite close to the estimated optimal ones. On the other hand, for the first pm plan, the result of the exhaustive search estimated a significant 760 seconds improvement in total band size, if the offsets were changed from 0, 4, and 47 to 0, 137, and 54 at intersections 1, 2, and 3 respectively. Therefore, after field implementation, we expect to observe a significant increase in total band size as well as considerable decrease in travel time in plan 5. Figure 7.4 shows DBAT exhaustive search results for eight weekdays at Site A.

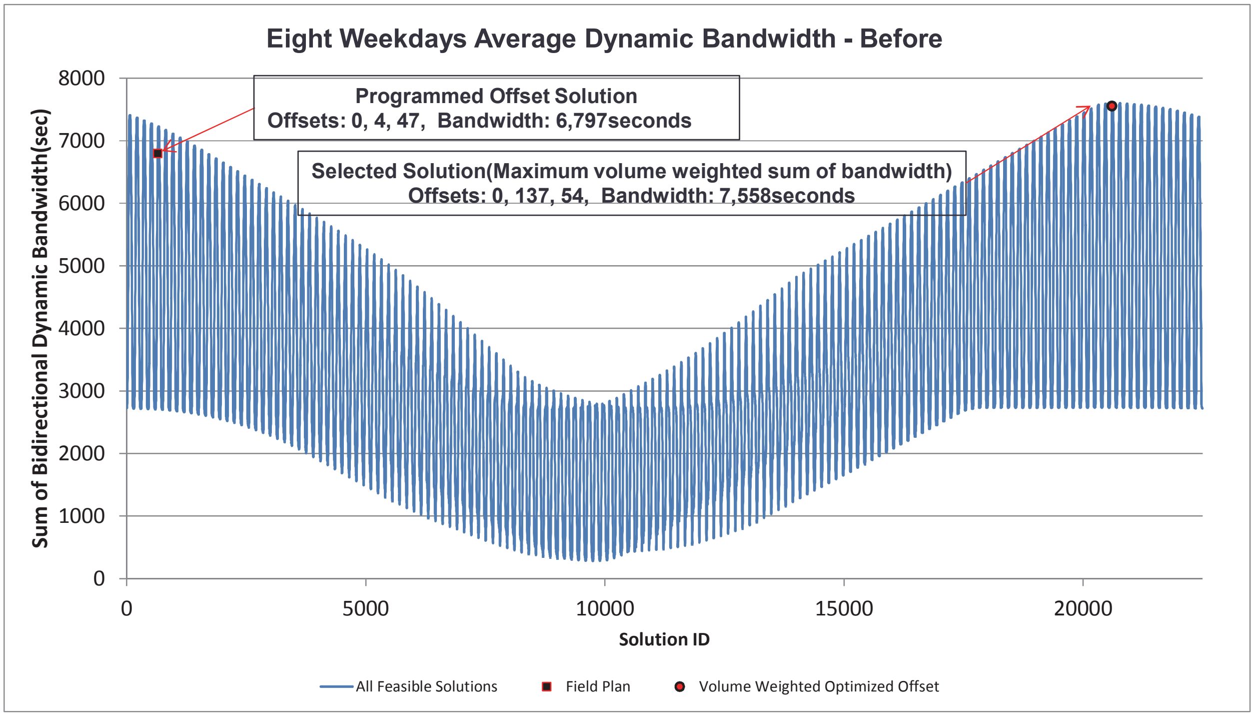


Figure 7-4 DBAT Exhaustive Search Result in Plan 5

In order to ascertain that any potential change in arterial travel time is solely due primarily to changes in the offsets and not to different signal operations or demand (volume), before and after detector counts for through movements and coordinated phase’s used green times were studied.

A t-test was used for testing demand difference in before and after conditions. The test was conducted under a significant level of 0.05 and mean difference of 0.4vehicles/minute. Among the six different time of day plans, only the fifth plan shows meaningful bandwidth improvement so 0.4vehicles/minute was decided to detect one vehicle difference in the fifth plan cycle (150 seconds). Table 7.3 summarizes important before and after data. The results show that the average number of detector calls per lane per minute did not significantly change before and after field implementation of the optimal offsets at any intersection. This indicates that through volume level did not appreciably change in the two data collection periods.

Table 7-3 Test Results for Demand Variation Before and After Offset Change

Intersection	Direction	Number of lane	Data Collection Duration (min)		Average number of Detector Calls (veh/min/ln)		Standard Deviation of Detector Calls (veh/min/ln)		Power	p-value
			Before	After	Before	After	Before	After		
Shotwell & US 70	EB	2	2,887	2,890	8.179	8.143	4.449	4.5	0.960	0.379
	WB	3	4,329	4,336	3.206	3.181	4.008	4.018	0.999	0.384
S Moore & US 70	EB	2	2,896	2,883	10.37	10.28	6.07	6.081	0.804	0.285
	WB	3	4,341	4,318	5.459	5.385	3.477	3.757	1	0.169
Robertson & US 70	EB	2	2,895	2,573	9.77	9.663	6.052	5.446	0.821	0.247
	WB	3	4,341	3,856	4.921	4.88	2.73	2.77	1	0.251

In addition, the Kolmogorov-Smirnov test shows no significant change in the used green time distribution before and after field implementation of the optimal offsets, see Table 7.4. Since the distribution of used greens of the coordinated phases is highly correlated with the early return to greens and green extensions, and they both are highly correlated with the demand pattern on non-coordinated phases, the analysis suggests that the demand level at non-coordinated phases did not significantly change either. As a result, there is no strong evidence to conclude that the demand level in the before and after data collection phases are different.

Table 7-4 K-S Test Result for Before and After green used time of Coordinated Phase

Intersection	Phase	D	P-Value
Shotwell & US 70	EB (Phase2)	0.041	0.407
	WB (Phase 6)	0.013	0.913
S Moore & US 70	EB (Phase2)	0.026	0.695
	WB (Phase 6)	0.007	0.971
S. Robertson & US 70	EB (Phase2)	0.024	0.998
	WB (Phase 6)	0.034	0.920

Figure 7.5 shows Shotwell Dr. intersection's before and after coordinated phase used green time distribution.

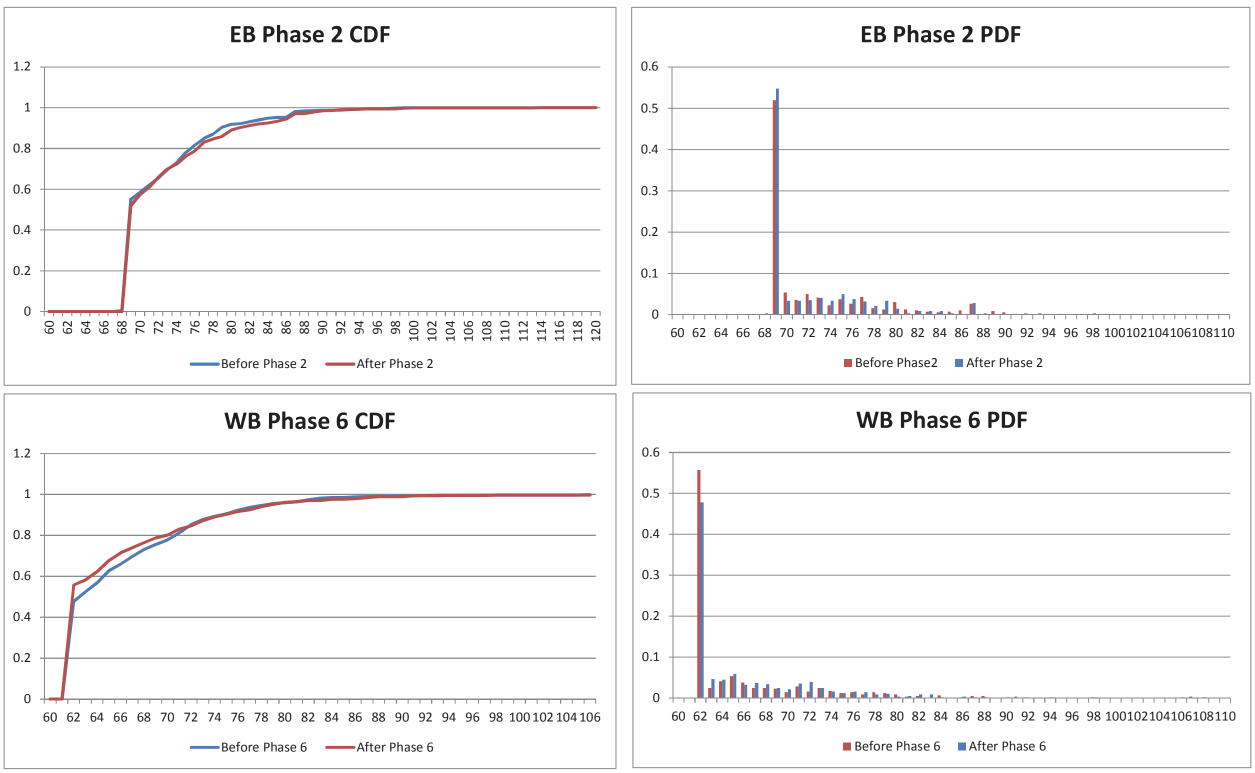
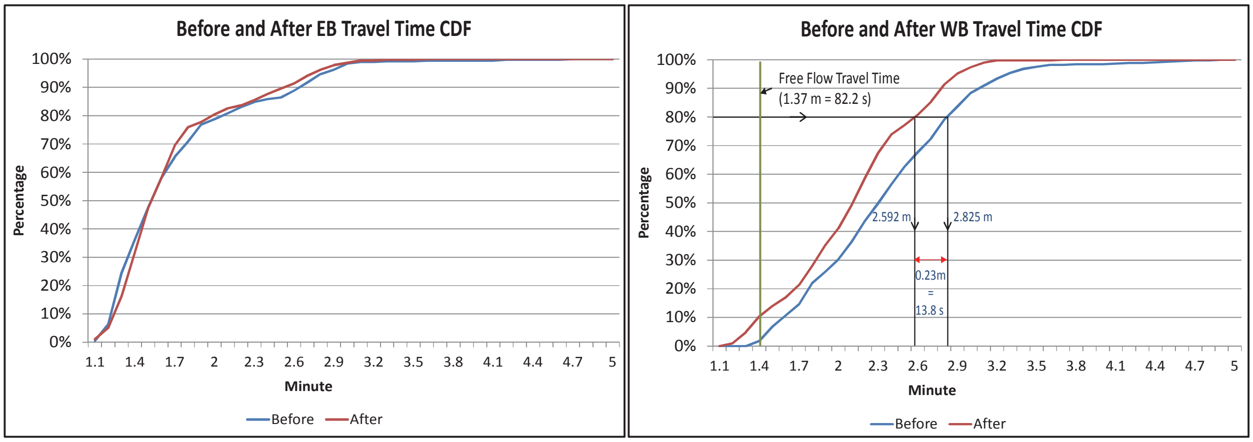


Figure 7-5 Coordinated Phase Used Green Time Comparison (Shotwell Dr. Intersection)

Figure 7.6 presents cumulative travel time distributions for the study site before and after field implementation of the optimal offsets for the 1:30 pm Plan. The cumulative travel time distributions do not show a considerable change in EB travel time, the peak direction (see Figure

7.6-a); however, they show a significant shift in the distribution towards lower travel times in the westbound direction as a result of using the optimal offsets (see Figure 7.6-b). This shows that while optimizing the offsets did not significantly change travel time and bandwidth on the major direction, it significantly reduced travel time in the minor direction.



a) Eastbound Direction b) westbound Direction

Figure 7-6 Directional Before and After Travel Time CDF's for 1:30 pm Plan at Site A

Table 7.5 summarizes the before and after directional band size, total band size, weighted sum of band size, average travel time, and standard deviation of travel time in the study site for all time of day plans. As expected, there is not a considerable change in average travel time in most time of day plans. The main reason is that using the optimal set of offsets did not significantly change the "expected" band size for those plans. On the other hand, for the 1:30 pm plan, the optimal offsets reduced average travel time in the westbound direction by 10.5% without creating a significant change in the eastbound average travel time.

Table 7-5 Before and After Travel Time Observation at Site A

Cycle Length (sec)	Plan Start (Plan End)	Plan	Offset *		EB Band Size (sec)	WB Band Size (sec)	Total Band Size (sec)	Weighted Sum of Band (Objective Function)	EB Weight Value (WB Weight Value)	Bluetooth Sample Size		Average Travel Time (sec)		STDEV Travel Time (sec)	
			Int. 2	Int. 3						EB	WB	EB	WB	EB	WB
170	6:20 (7:30)	Before	3	86	1,547	1,202	2,749	1,325	0.36 (0.64)	94	297	103	123	33.3	38.9
		After	0	85	1,543	1,235	2,778	1,346		97	263	104	122	32.5	39.1
140	7:30 (9:30)	Before	135	73	2,445	2,478	4,923	2,463	0.46 (0.54)	254	417	104	109	29.4	31.6
		After	136	79	2,223	2,749	4,973	2,507		232	420	107	107	28.8	33.5
120	9:30 (11:30)	Before	108	62	2,458	2,237	4,695	2,365	0.58 (0.42)	223	284	102	110	37.1	35.2
		After	107	60	2,589	2,283	4,873	2,461		222	295	98	107	32.2	34.6
130	11:30 (13:30)	Before	118	53	2,411	1,709	4,121	2,140	0.61 (0.39)	264	315	115	104	37.1	33.5
		After	117	57	2,378	1,890	4,267	2,187		273	348	116	103	35.2	31.9
150	13:30 (16:30)	Before	4	47	4,948	1,848	6,785	3,913	0.67 (0.33)	379	441	103	141	34.6	40.0
		After	137	54	4,802	2,751	7,553	4,117		375	494	102	126	33.8	29.1
220	16:30 (18:30)	Before	211	3	3,080	1,895	4,975	2,735	0.71 (0.29)	398	308	125	125	44.3	41.8
		After	172	5	3,198	1,860	5,058	2,810		326	316	122	125	44.8	42.3

* Intersection 1 is the reference intersection. Therefore, its offset is always set at zero.

It is very important to note that the optimal offsets were not fine-tuned in the field, yet they still yielded a significant reduction in travel time in the westbound direction without creating any significant change in other travel times. This is a very important finding and shows that the proposed approach has great potential for eliminating the fine-tuning process, which is time consuming and costly and cannot guarantee an optimal performance especially when the number of intersections increases. Our proposed approach, by analyzing two weeks of used green times could accurately account for patterns that are expected to be observed in early returns to green and green extensions.

7.4 ACTRA

ACTRA is a PC-based transportation management system that monitors and controls traffic from a central control center developed by Siemens ITS for implementation in system using EPAC™, 170, 2070ATC, or EPIC™ controllers. ACTRA allows integration of convenient traffic analysis optimization tools including AAP™ (Passer™ and Transyt-7F™) and Synchro™. Also, ACTRA includes integrated GIS-based area maps. The ACTRA system is currently used by the City of Raleigh.

7.4.1 ACTRA Reports

The ACTRA system provides *System* reports and *Intersection* reports. System reports are system wide reports that are available to be run through Time of Day. Intersection reports include ten different reports as listed below:

- Intersection Communications Faults Report
- Intersection Cycle Measures of Effectiveness Report
- Intersection Detector Faults Report
- Intersection Detector Volume Report
    - Date and Time - current system date and time (when report was generated)
    - Local Name - the name of the selected Intersection
    - Report Start/Report End - the time range this report covers
    - Date/Time - start time for that line of data
    - Detector Number - identification of the detector
    - Volume - counts
- Intersection EDI Monitor Fault Report
- Intersection Local Alarm Report
- Intersection Measures of Effectiveness Report

- Local Name - the name of the selected Intersection
- Date and Time - current system date and time (when report was generated)
- Report Start/Report End - the time range this report covers
- Start Time
- Dial/Split/Offset
- Phase/Volume/Stops
- Delay x 10
- Utilization - in seconds

- Intersection MMU Monitor Fault Report
- Intersection Speed Data Report
  - Date and Time - current system date and time (when report was generated)
  - Local Name - the name of the selected Intersection
  - Report Start/Report End - the time range this report covers
  - Date/Time - beginning time for the pattern monitored
  - Dial/Split/Offset of the pattern monitored
  - Percent Lower, Percent Within, and Percent Higher - the percentage of the vehicles that were lower, within, or higher than the set speed range for the specified pattern
- Intersection System Detector Report
  - Date and Time - current system date and time (when report was generated)
  - Local Name - the name of the selected Intersection
  - Report Start and Report End dates and times - the interval covered by this report
  - Sample Time - date and starting time
  - Interval - the sample interval in minutes
  - Detector number
  - Raw Volume - counts
  - Raw Occupancy - counts (number of seconds)
  - Average Volume
  - Average Occupancy

EPAC controllers can store up to 72 of the most recent volume log records for the current report interval with up to 24 detectors to collect volume data for the selected intersection. For example, if you use a report interval of 10 minutes, the 72 logged records would cover a 12-hour period. As the number of records exceeds 72, the oldest record is deleted to make room for the new record. The minimum time interval for volume log data storage is 1 minute.

7.4.2 ACTRA Split Monitoring

ACTRA system also provides "Split Monitor" result report. The Split Monitoring feature allows the user to monitor intersection split data over specified time intervals. The split data can vary from cycle to cycle based upon different traffic demands while cycle length and offset stay the same as the programmed input value. This feature enables DBAT to conduct cycle-by-cycle bandwidth monitoring and offset optimization process. Figure 7-7 shows ACTRA system's Split Monitoring Data Display window. The monitoring result can be also downloaded as a report file.

Figure 7-7 ACTRA Split Monitoring Data Display Window

However, ACTRA system uses a different offset reference point than does the OASIS™ system. There are three well-known offset reference points which are NEMA TS1, NEMA TS2, and 170. NEMA TS1 references the offset point to the start of the green indication of the second coordinated phase, and NEMA TS2 references the offset point to the start of the green indication of the first coordinated phase. 170 typically references the offset point from the start of the coordinated phase yellow indication. Figure 7-8 show these offset reference points diagrammatically. NCDOT OASIS™ system uses the NEMA TS2 reference point scheme while ACTRA uses the NEMA TS1 reference point scheme.

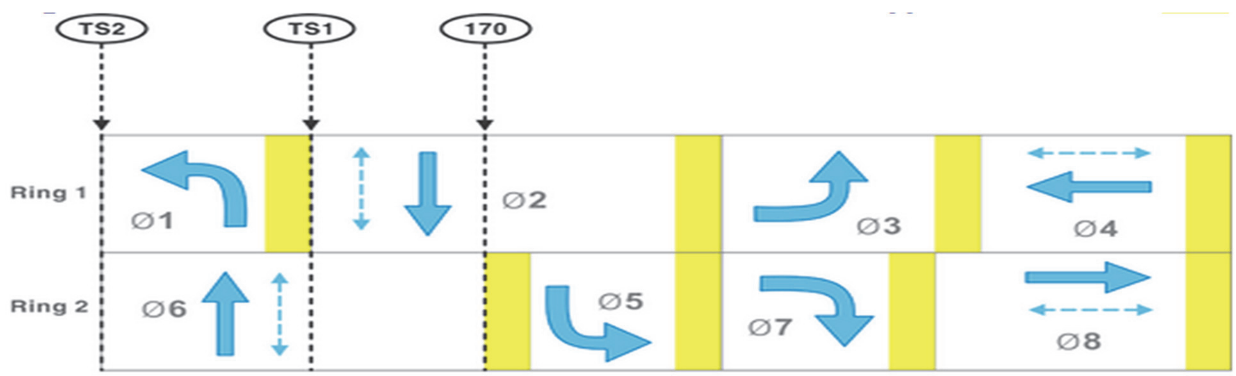


Figure 7-8 Offset Reference Point

As mention early, DBAT is designed specifically for OASIS™ split monitor data. Therefore, the ACTRA split monitor data must be modified in order to use DBAT. Specifically, the cycle-by-cycle offset referencing must be converted from NEMA TS1 to NEMA TS2 offset referencing.

7.4.3 DBAT Process Result

As mentioned above, the City of Raleigh uses the ACTRA system to operate and monitor the city's traffic signals. The Western Blvd arterial was selected as a test site, and 6 weekdays PM peak plan (4:30 to 6:30) ACTRA split monitor reports were collected and provided to the project team by the City of Raleigh Transportation Operations Division of the Public Works Department. Figure 7-9 show the selected study site which consists of six signalized intersection.

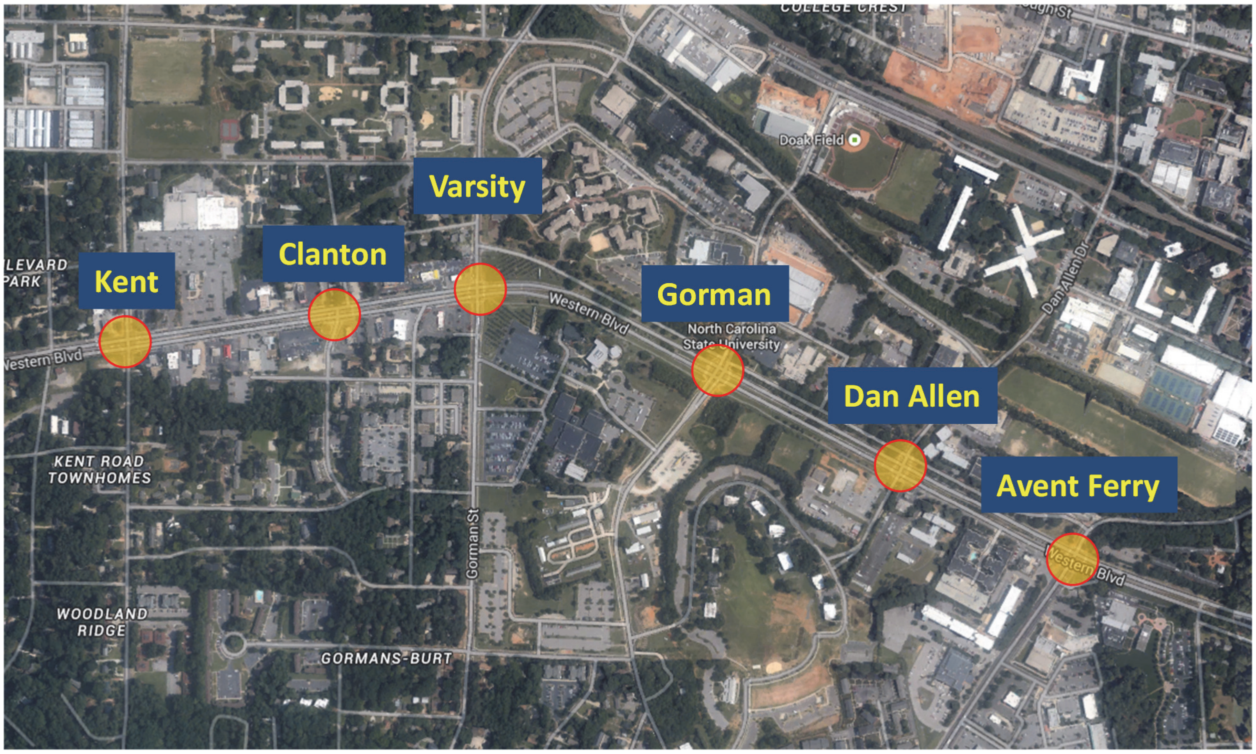


Figure 7-9 Western Blvd Study Sites

The selected PM peak plan uses a 200-second common cycle length. The current offsets are 0, 179, 179, 10, 191 and 18 from Avent Ferry to Kent Dr. intersection, respectively. The project team calculated weighting factors based on directional volume per lane. The calculated weighting factors are 0.627 (outbound) and 0.307 (inbound). Table 7-6 shows current field offset DBAT processing result.

Table 7-6 Current Offset DBAT Analysis Result

Time Period	Average Bandwidth		Bandwidth Efficiency		Total Sum of Bandwidth
	Outbound	Inbound	Outbound	Inbound	
PM	64.95	6.60	31.51	1.54	2,231

Since selected arterial consists of 6 intersections with 200 seconds of cycle length, the total number of possible solutions would be 200^5 (320,000,000,000). This solution space is too large to practically allow for a full exhaustive search. Therefore, the coordinated intersections were divided into two groups. The first group consists of the Avent Ferry Rd., Dan Allen Dr., Varsity Rd., and Gorman St. intersections. The optimal offset solution set for weighted bandwidth is 0, 184, 184, and 42 from Avent Ferry Rd. to Gorman St. Figure 7-10 shows the first intersection group's exhaustive search results with optimal solution and current field solution.

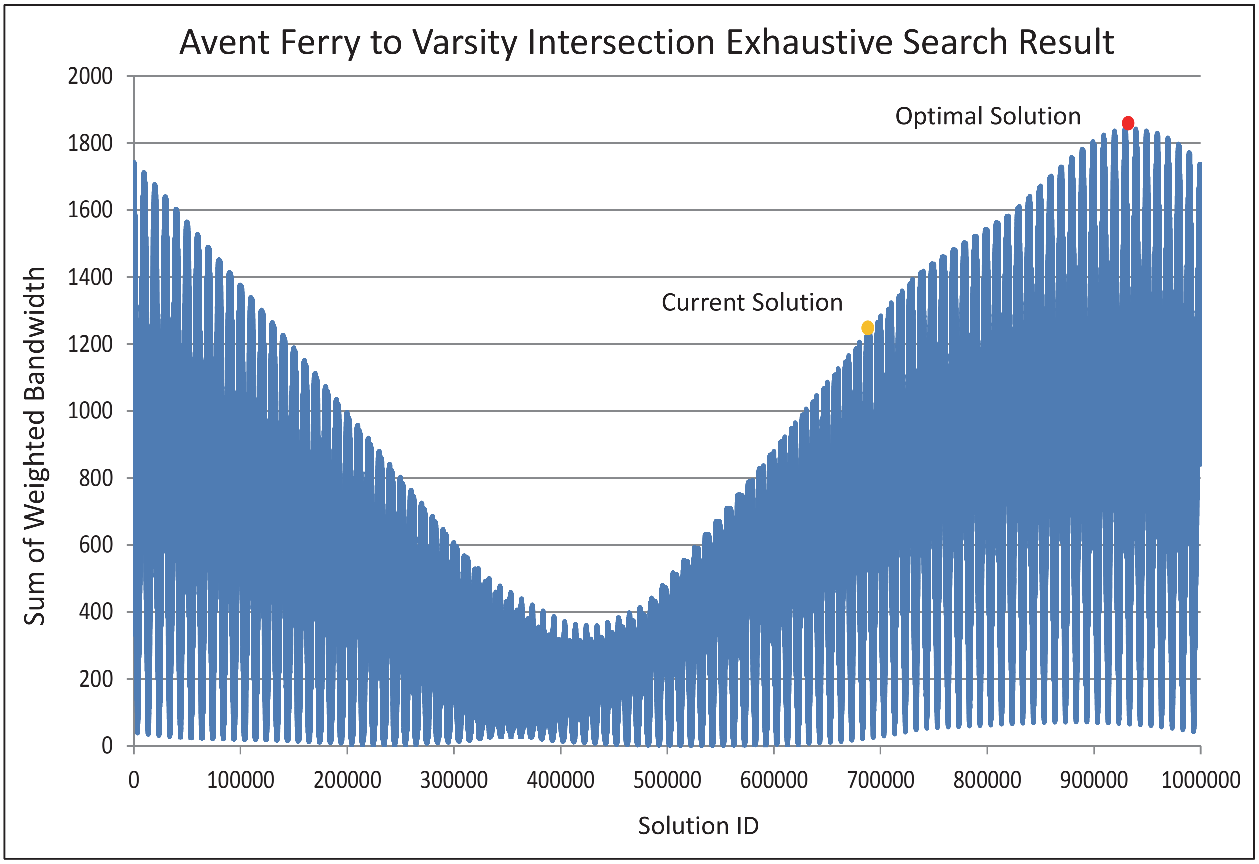


Figure 7-10 Exhaustive Search Result by DBAT

With the optimal offsets for the first four intersections fixed the next step was to add the remaining two intersections and repeat the exhaustive search to find the final two offset values. Figure 7-11 shows the result of this second step of the exhaustive search optimization.

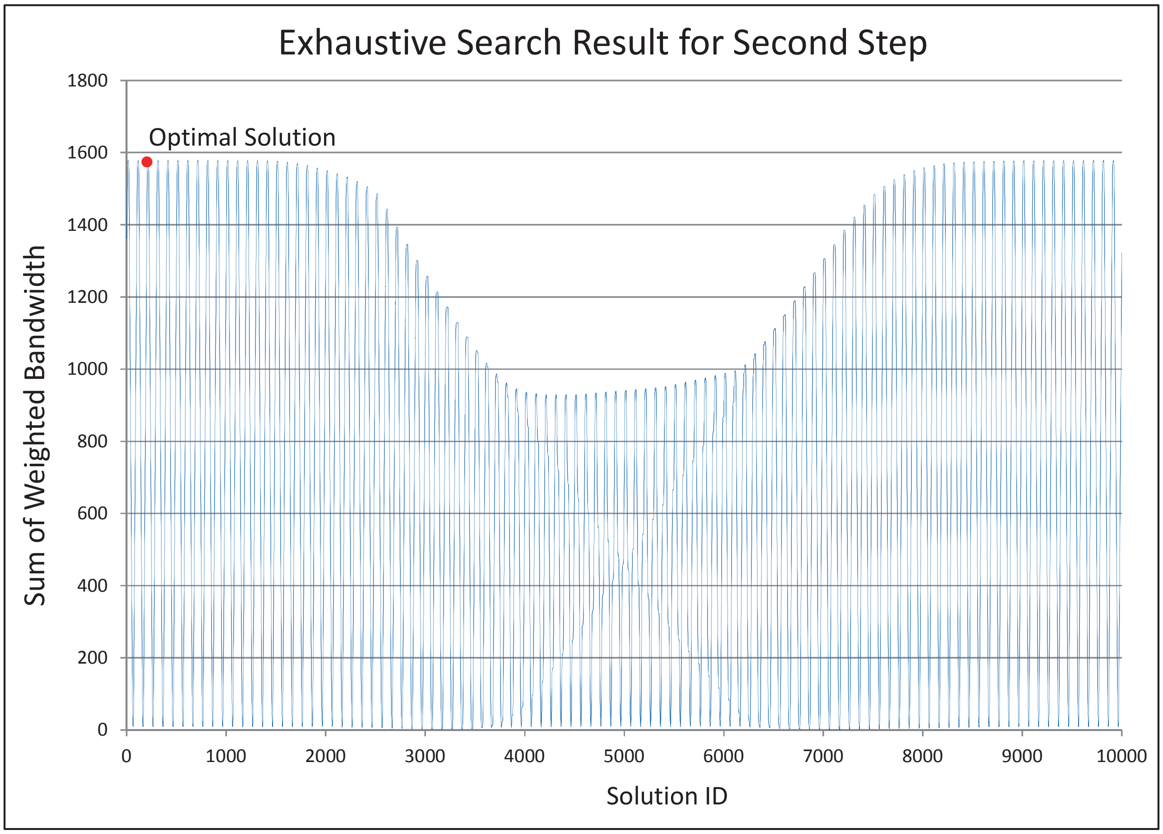


Figure 7-11 Final Exhaustive Search Result

The optimal offsets are 0, 184, 184, 46, 6 and 30 from Avent Ferry Rd. to Kent Rd., respectively. Table 7-7 the compares the current field offset bandwidth efficiency to the expected bandwidth efficiency for the optimal offsets. This small study illustrates that the DBAT analysis and optimization tool can be effectively used for signal control systems other than OASIS^{TM}.

Table 7-7 Comparison of Current field Offset to Optimal Offset

	Average Bandwidth		Bandwidth Efficiency		Total Sum of Bandwidth
	Outbound	Inbound	Outbound	Inbound	
Current Offset	64.95	6.60	31.51	1.54	2,231
Optimal Offset	83.35	5.21	40.42	1.22	2,755

7.5 Summary

In this chapter, the dynamic bandwidth optimization method is introduced. In terms of dynamic bandwidth optimization, the presented exhaustive search method can provide all feasible, including globally optimum solutions. From this information, optimum offset can be selected.

The field evaluation result provides strong evidence of the robustness of the proposed exhaustive search method. DBAT processed six time of day plans using two weeks worth of data. The selected global solutions from DBAT have very similar offset values to current field offsets for 5 time of day plan, but it provides a much improved bandwidth result for one time of day plan (see Table 7.2). Field implementation results indicated a 10.5% travel time decrease for the non-peak direction without any significant change on the other direction for the 1:30 PM plan. Implementing the optimal offset combination has shown to improve efficiency and reduce travel time without the need for field fine-tuning.

The exhaustive search method is practical only for coordinated systems with a small number of intersections due to the exponential growth of the search space as the number of intersections increases. Although it is possible to mitigate excessive calculation time by increasing the search interval or decreasing search space, it is not likely that an exhaustive search will provide a real global solution.

CHAPTER 8. DEVELOP RECOMMENDATIONS FOR IMPLEMENTATION OF BEST PERFORMING MODELS (TASK 8)

There are limitations to estimate arterial travel time using OASIS log data since OASIS detectors provide one minutes aggregated data for system detectors. Most of the time, coordinated common cycle length is greater than 60 seconds, so it is hard to estimate how many vehicles are arriving during green from the OASIS detector log. For this reason, the project team proposes several monitoring methods using OASIS log data and developed a near real time dynamic bandwidth monitoring program, which is called the dynamic bandwidth analysis tool (DBAT). In addition, the project team not only provides real time dynamic bandwidths monitoring program, but also introduces dynamic bandwidth optimization method for signal offsets fine-tuning. The introduced exhaustive search method with DBAT gives information on all possible offset combination's dynamic bandwidth solutions under a given time of day plan. This result allows evaluating current signal coordination quality as well as easy offset fine-tuning.

In Chapter 5, a few monitoring methods were introduced. The methods are:

1. Monitoring and creating coordinated movements flow (see Figure 5.12),
2. Monitoring Time of Day Plan suitability (see Figure 5.12),
3. Monitoring cycle-by-cycle coordinated movements capacity (see Figure 5.13 to Figure 5.16),
4. Monitoring flow to capacity ratio (see Figure 5.17 and Figure 5.18)
5. Monitoring Early return to green and green extension for coordinated phases (Figure 5.19 to Figure 5.22)
6. Monitoring non-coordinated phase displayed green distribution (Figure 5.23 to Figure 5.25)

7. Dynamic bandwidth analysis tool (see Chapter 5.6)

As mentioned earlier, bandwidth is highly correlated with arterial progression quality, and intersection offsets have a direct impact on the bandwidth magnitude. Coordinated intersection offsets are the decision variables that are typically optimized to maximize bandwidth. The majority of bandwidth optimization studies published in the signal coordination literature were conducted using programed (fixed) green times. However, most coordinated arterial signal systems operate in semi-actuated mode as explained above. One approach to dealing with varying green time would be to use the observed average coordinated green durations. However, field studies show that the distribution of coordinated green times are typically asymmetrical (see Chapter 5). Therefore, using average green durations for developing a semi-actuated control strategy will not guarantee finding a near optimal set of offsets. In practice, signal engineers traditionally conduct field visits to observe the early return to green and the initial queue to fine tune the offsets and improve arterial performance based on engineering judgment.

Chapter 7 and Chapter 8 of this report introduced dynamic bandwidth monitoring and dynamic bandwidth optimization method for coordinated arterials. DBAT allows monitoring near real-time arterial dynamic bandwidth. It can help offset optimization for dynamic bandwidth maximization. However, the exhaustive search method requires long computation time when numbers of coordinated intersections are increased.

In this Chapter, new dynamic bandwidth optimization methods are introduced. In addition, even if dynamic bandwidth is a useful arterial performance indicator, travel time (speed) is also a very important monitoring item. As a side evaluation and recommendation, the team will evaluate the accuracy of the Bluetooth traffic surveillance technology, and develop initial long-term recommendations concerning the possible incorporation of permanent Bluetooth surveillance to allow more direct measurement of arterial system performance.

8.1 Linear Programming for Dynamic Bandwidth Optimization

The concept of the proposed bandwidth optimization LP is that green band is continuously created per each cycles, so it is possible to directly constrain each directional band start and end points inside of green durations. After defining green start and end time at each intersection, LP can search for optimal offsets, which maximize the sum of directional bandwidth. The main constraints are each green start and end time at each cycle. The numbers of input cycles are decided by progression speed and arterial length. Figure 8.1 shows basic geometry of the new bandwidth optimization model.

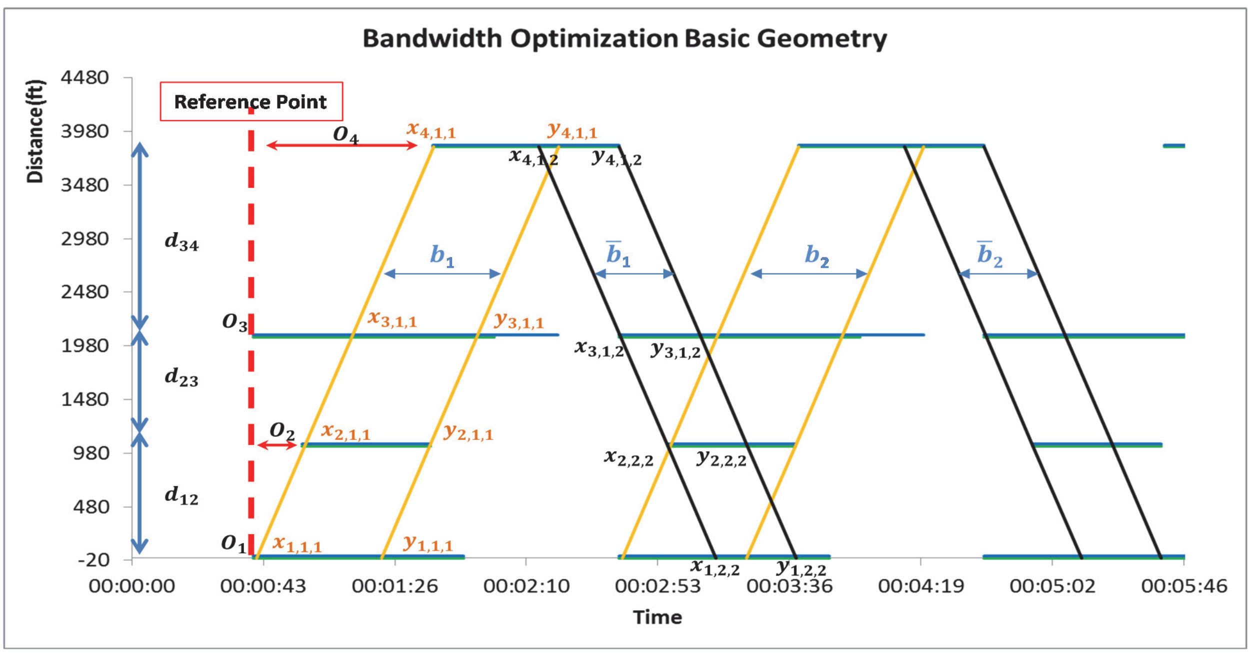


Figure 8-1 Bandwidth Optimization Basic Geometry

Variables are defined as follows:

$b_i = outbound\ bandwidth;$

$\bar{b}_i = inbound\ bandwidth;$

$g_{i,j,k} = Green\ phase\ duration\ for\ intersection\ i, in\ cycle\ j, direction\ k;$

$i = 1, 2, 3, \cdots, l \qquad l = number\ of\ intersection$

$j = 1, 2, 3, \cdots, m \qquad m = number\ of\ cycles$

$k = 1, 2 \qquad 1 = outbound\ direction, 2 = inbound\ direction$

$gs_{i,j,k} =\ green\ start\ time\ for\ intersection\ i, in\ cycle\ j, direction\ k;$

$ge_{i,j,k} =\ green\ end\ time\ for\ intersection\ i, in\ cycle\ j, direction\ k;$

$d_{i,i+1} = distance\ between\ \ intersection\ i\ and\ i + 1\ (ft);$

$v_{i,i+1} =\ progression\ speed\ between\ intersection\ i\ and\ i + 1\ (ft/sec);$

$L_{i,i+1} =\ Lower\ bound\ of\ speed\ between\ intersection\ i\ and\ i + 1\ (ft/sec);$

$H_{i,i+1} =\ upper\ bound\ of\ speed\ between\ intersection\ i\ and\ i + 1\ (ft/sec);$

$t_{i,i+1} = travel\ time\ between\ intersection\ i\ and\ i + 1\ (\frac{d_{i,i+1}}{v_{i,i+1}}\ sec);$

$x_{1,j,k} =\ outbound\ band\ start\ time\ for\ intersection\ 1, in\ cycle\ j, direction\ k;$

$y_{1,j,k} =\ outbound\ band\ end\ time\ for\ intersection\ 1, in\ cycle\ j, direction\ k;$

$O_i =\ offset\ for\ intersection\ i\ (sec);$

$D =\ outbound\ average\ demand(vehicle)\ per\ cycl(sec);$

$\bar{D}\ =\ inbound\ average\ demand(vehicle)\ per\ cycl(\mathrm{sec});$

$k =\ inbound\ demand\ over\ outbound\ demand\ (\frac{\bar{D}}{D});$

$\tau_i(\bar{\tau}_i) =\ oubound\ (inbound) queue\ clearance\ time\ for\ intersection\ i;$

$\alpha =\ ratio\ of\ critical\ direction\ bandwidth\ to\ demand;$

$M =\ very\ large\ number;$

For the bandwidth optimization, the objective function is:

$$Maximize \sum_{i=1}^{r}(b_i + k\bar{b}_i) + \alpha \cdot M \tag{1}$$

Where, $r = the\ number\ of\ outbound(inbound)\ \ bands$

$b_i = y_{1,j,1} - x_{1,j,1}$

$\bar{b}_i = y_{l,j,2} - x_{l,j,2}$

The objective function (equation 1) considers a target value to maximize the sum of two-way bandwidths. In addition, the given formula overcomes MAXBAND over constraint problem using the decision variable α and the big number M. The objective function tries to maximize outbound bandwidth, inbound bandwidth and the α value. α is controlled by equation (13) and (14). Mathematically, the maximum α is the the ratio of bandwidth over the seconds required to process the demand (assuming h_s seconds per vehicle where h_s is saturation headway) for the critical direction.

For the bandwidth optimization, the decision variables are:

$x_{1,j,1} =\ outbound\ band\ start\ time\ for\ intersection\ 1, in\ cycle\ j, direction\ 1\ ;$

$y_{1,j,1} =\ outbound\ band\ end\ time\ for\ intersection\ 1, in\ cycle\ j, direction\ 1\ ;$

$x_{l,j,2} =\ inbound\ band\ start\ time\ for\ intersection\ l, in\ cycle\ j, direction\ 2\ ;$

$y_{l,j,2} =\ inbound\ band\ end\ time\ for\ intersection\ l, in\ cycle\ j, direction\ 2\ ;$

$v_{i,i+1} =\ progression\ speed\ between\ i\ and\ i+1\ (ft/sec)$;

$\alpha =\ ratio\ of\ critical\ direction\ bandwidth\ to\ demand;$

$O_i =\ offset\ for\ intersection\ i$ (sec).

Model constraints include:

Constrain (2) and (3) ensure that the bandwidth end point (time) is after bandwidth start point.

$$y_{1,j,1} \geq x_{1,j,1} \qquad for\ j = 1 \cdots r \tag{2}$$

$$y_{l,j,2} \geq x_{l,j,2} \qquad for\ j = 1 \cdots r \tag{3}$$

Constraint (4) to (7) ensure that bandwidth start and end point strictly stay inside of green duration.

$$gs_{i,j,1} \leq x_{1,j,1} + \sum_{i=2}^{l} t_{i-1}, t_i \leq ge_{i,j,1} \quad for\ i = 1, \cdots, l, \quad j = 1, \cdots, r \tag{4}$$

$$gs_{i,j,1} \leq y_{1,j,1} + \sum_{i=2}^{l} t_{i-1}, t_i \leq ge_{i,j,1} \quad for\ i = 1, \cdots, l, \quad j = 1, \cdots, r \tag{5}$$

$$gs_{i,j,2} \leq x_{l,j,2} + \sum_{i=2}^{l} t_{i-1}, t_i \leq ge_{l,r,2} \quad for\ i = 1, \cdots, l, \quad j = 1, \cdots, r \tag{6}$$

$$gs_{i,j,2} \leq y_{l,j,2} + \sum_{i=2}^{l} t_{i-1}, t_i \leq ge_{i,j,2} \quad for\ i = 1, \cdots, l, \quad j = 1, \cdots, r \tag{7}$$

Constraint (8) expresses the start time of the first greens at non-reference intersection.

$$gs_{i+1,1,k} = gs_{1,1,k} + O_i \quad for\ i = 1, \cdots, l-1 \quad k = 1, 2 \tag{8}$$

Constraint (9) introduces a lower and an upper bound for the speed on each link.

$$L_{i,i+1} \leq v_{i,i+1} \leq H_{i,i+1} \quad for\ i = 1, \cdots\ l \tag{9}$$

Constraints (10) and (11) account for queue clearance time at each intersection.

$$gs_{i,j,1} \leq x_{1,j,1} - \tau_i \quad for\ i = 1, \cdots, l, \quad j = 1, \cdots, r \tag{10}$$

$$gs_{i,j,2} \leq x_{l,j,2} - \bar{\tau}_i \quad for\ i = 1, \cdots, l, \quad j = 1, \cdots, r \tag{11}$$

$$O_1 = 0 \tag{12}$$

$$-C < O_i < C \tag{13}$$

$$0 \leq \alpha < 1 \tag{14}$$

Constraints (15) and (16) are designed to consider directional demand with optimal α. When both directional demands are smaller than bandwidth α will be one but when critical direction's demand exceeds band capacity the optimum α will be calculated and it represents the ratio of the bandwidth capacity to seconds required to process the demand on the critical direction.

$$b_i \geq \alpha \cdot D \tag{15}$$

$$\bar{b}_i \geq \alpha \cdot \bar{D} \tag{16}$$

8.2 LP Model Test

When the arterial has a semi-actuated signal controller, each intersection possibly has an early return to green and green extension. Therefore, green durations vary over cycles. The proposed linear program was applied to Site A (US 70 arterial in Clayton, NC). Table 8.1 shows dynamic green time information, which is downloaded from OASIS split monitor.

Table 8-1 Site A OASIS Split Monitor Log

US 70 & Shotwell Rd				
Time	**ExtraTimeCP1**	**ExtraTimeCP2**	**Used Phase 2**	**Used Phase 6**
06:32:11	18	34	79	49
06:35:01	8	25	79	79
06:37:51	24	37	79	49
06:40:41	24	40	79	49
06:43:31	16	37	79	49
US 70 & S Moor St				
Time	**ExtraTimeCP1**	**ExtraTimeCP2**	**Used Phase 2**	**Used Phase 6**
06:31:41	46	62	104	87
06:34:31	30	44	104	104
06:37:21	24	0	125	170
06:40:11	45	45	104	104
06:43:01	45	70	104	79
US 70 & R Robertson St				
Time	**ExtraTimeCP1**	**ExtraTimeCP2**	**Used Phase 2**	**Used Phase 6**
06:33:06	70	70	54	79
06:35:56	41	28	54	79
06:38:46	46	23	54	79
06:41:36	45	32	54	79
06:44:26	58	44	54	79

Both the DBAT exhaustive search method and the proposed LP method were applied to site A. LP provides optimal offset as of 0, 5.49, and 85.96 seconds. The calculated objective function, which is sum of both directional dynamic bandwidths, is 692.18 seconds.

Figure 8.2 and Table 8.2 shows proposed LP calculation results.

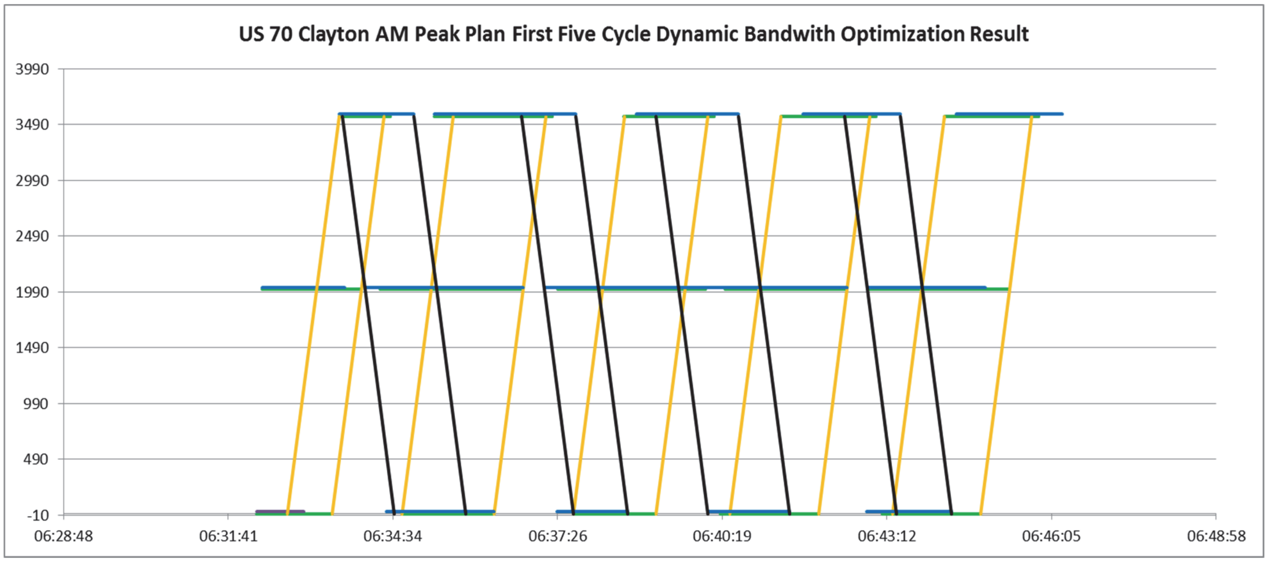


Figure 8-2 Dynamic Bandwidth LP Optimization Result

Table 8-2 LP Optimization Result

	Band 1	Band 2	Band 3	Band 4	Band 5
Outbound Bandwidth	47.06	97.00	87.00	93.06	92.06
Inbound Bandwidth	-	75.00	57.00	86.00	58.00

The exhaustive search method provides 5 different optimal offsets, and the sum of both directions dynamic bandwidths is 707 seconds. Figure 8.3 shows the DBAT exhaustive search result. Table 8.3 shows the summary of optimal offsets. The "Number of Band 2" column indicates there are two secondary bands for inbound since linear programming only provided four inbound bands. In addition, the exhaustive search method provides a bigger objective function value result than linear programming, since linear programming cannot consider the secondary bands.

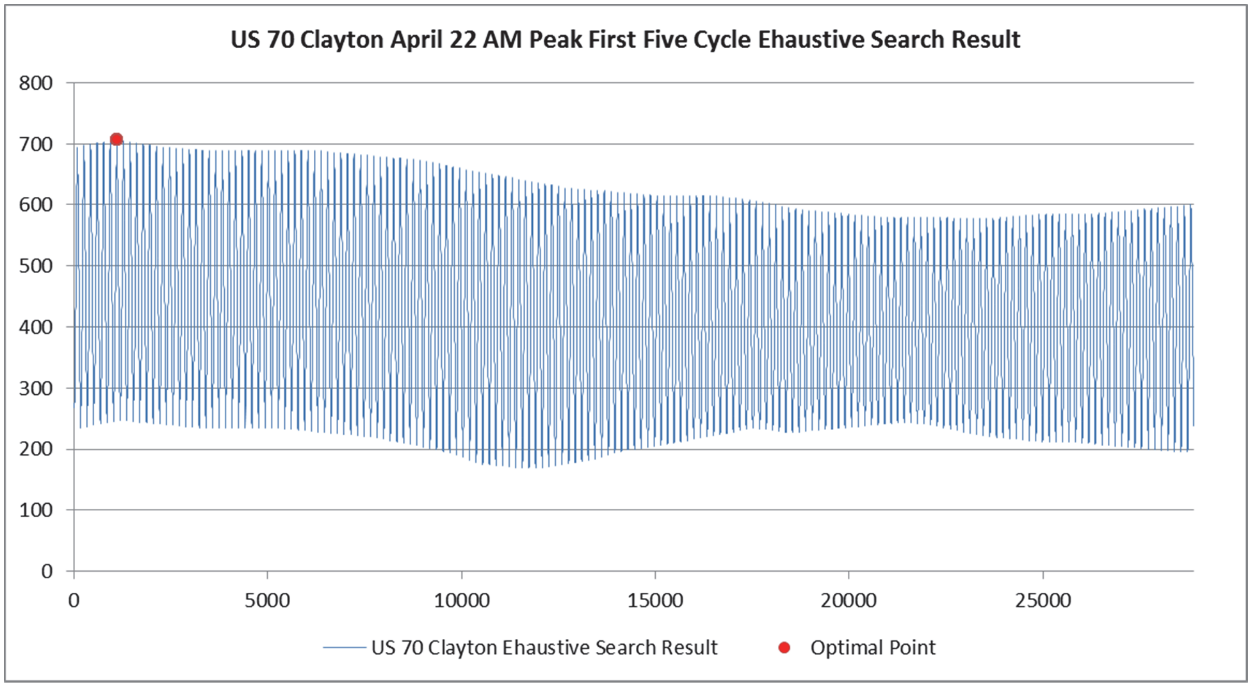


Figure 8-3 Dynamic Bandwidth DBAT Exhaustive Search Result

The secondary bands were eliminated using a secondary band filter in DBAT and then the exhaustive search result provided Table 8.4 results.

Table 8-3 DBAT Exhaustive Search Result Including Secondary Bands

ID	Int. 1	Int. 2	Int. 3	Outbound Bandwidth	Inbound Bandwidth	Number of Outbound Bands	Number of Inbound Bands	Sum of Directional Bandwidth
1101	0	6	80	434	273	5	6	707
1102	0	6	81	431	276	5	6	707
1103	0	6	82	428	279	5	6	707
1104	0	6	83	425	282	5	6	707
1105	0	6	84	422	285	5	6	707
1106	0	6	85	419	288	5	6	707

After eliminating secondary band, exhaustive search only provides one optimal solution. The calculated number of bands is identical to LP, but sum of bands is smaller. The LP objective function is larger than the exhaustive search since exhaustive search takes only integer value for its offsets and minimum search interval is 1 second.

Table 8-4 DBAT Exhaustive Search Result Excluding Secondary Bands

ID	Int. 1	Int. 2	Int. 3	Outbound Bandwidth	Inbound Bandwidth	Number of Outbound Bands	Number of Inbound Bands	Sum of Directional Bandwidth
1106	0	6	85	419	269	5	4	688
1105	0	6	84	422	265	5	4	687
1104	0	6	83	425	261	5	4	686
1103	0	6	82	428	257	5	4	685
1102	0	6	81	431	253	5	4	684
1101	0	6	80	434	249	5	4	683

After adding one constraint (enforcing offsets to take only integer values), the LP and exhaustive search methods give identical solutions. Therefore, the global solution for maximum sum of two way dynamic bandwidth is when each intersection offset is 0, 6, and 85 seconds. The total amount of the sum of bandwidths is 688 seconds.

The proposed LP model cannot consider secondary bands, but its computation time is significantly less than the exhaustive search method. In addition, both methods provide an identical optimal solution. Therefore, project team suggests using DBAT for monitoring near real-time dynamic bandwidth and using both DBAT and the proposed LP for fine-tuning offsets.

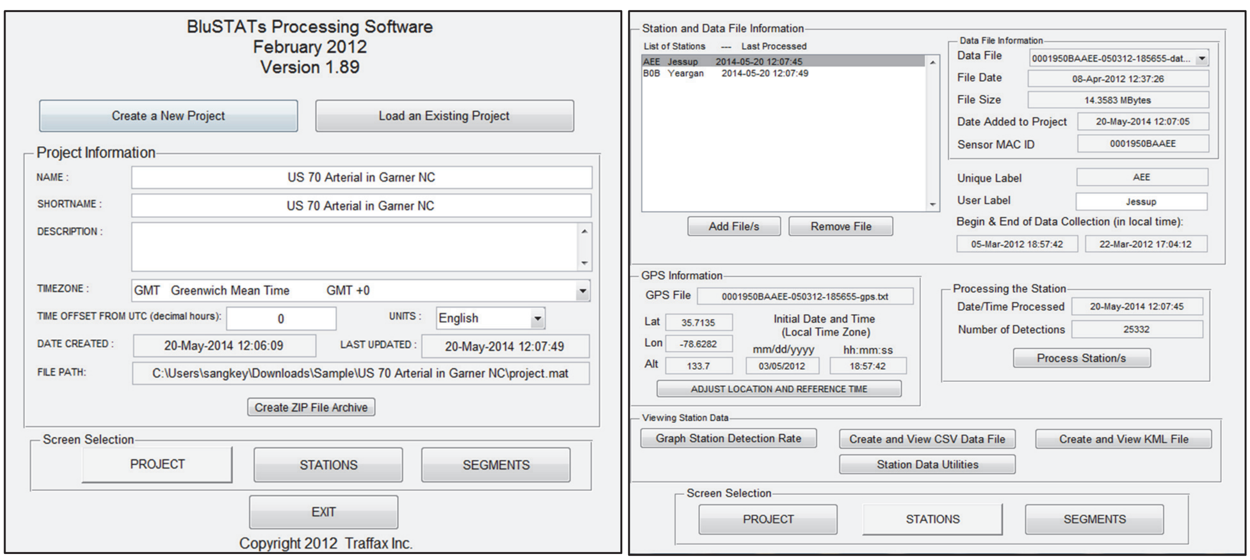
8.3 Comparison of BluFax and BlueMAC Travel Times

In Chapter 4, two Bluetooth devices were briefly introduced. Bluetooth technology is being increasingly utilized by government agencies, consulting firms, and researchers as an inexpensive and practical method of measuring travel times. There are many studies to show Bluetooth technology's effectiveness for measuring travel time. For example, Schneider IV et al. completed a study comparing Bluetooth data to floating car data (48). This study showed that Bluetooth Travel times matched floating car travel times. In addition to advantages in accuracy, one more major advantage of using Bluetooth units to collect travel times is its relatively low cost. A few studies estimated that Bluetooth unit may be 500 to 2,500 times more cost-effective than floating car data collection approach. For these reason, using Bluetooth units have become an attractive alternative approach for travel time data collection.

8.3.1 Data Process

8.3.1.1 BluFax (http://www.traffaxinc.com/)

In other to facilitate data processing procedure, BluFax requires to use BluSTATs software. BluSTATs is set up to eliminate redundant detections, match MAC addresses between stations, and filter data to produce O-D and travel time matrices for defined two-station segments. Figure 8.4 shows BluSTATs software interface.



(a) User Intersection for Project (b) User Intersection for Station Data

Figure 8-4 User Interface of BluSTATs Software

Data processing with BluSTATs software is a multi-step process that is described in the following steps:

1. Data is downloaded onto a computer from the SD card
2. The data set for each BluFax device is imported into BluSTATs, forming a “station.” Each station is defined with a GPS file, its own MAC ID and Traffax device label, and user-selected label.
3. Station data is processed to remove redundant detections that were recorded consecutively by the BluFax device. This filtering process removes multiple detections of a single MAC address as it makes a single pass through a station’s detection range. Detections that occur at different times of day due to multiple passes by the same Bluetooth enabled device are not considered redundant and are therefore maintained in station processing.
4. The user then pairs station to create segments that align with travel time objectives of the study. BluSTATs compares MAC address detections between the two stations and preserves only those addresses that were detected at both locations within the minimum and maximum time periods identified by the user.

8.3.1.2 BlueMAC (http://www.digiwest.com/)

BlueMAC devices carry a Global Positioning System (GPS) and Global System of Mobile (GSM) antenna to report to the BlueMAC website at intervals configured by the user. GPS allows the user to map every device’s location in a project to watch how traffic flows throughout the monitored corridors. It does not require the user to process data. The provided BlueMAC website allows the user to create a Project similar to Figure 8.5. When a new project is created, the collected data is automatically transmitted to BlueMAC server and then a travel time report and origin-destination report is created.

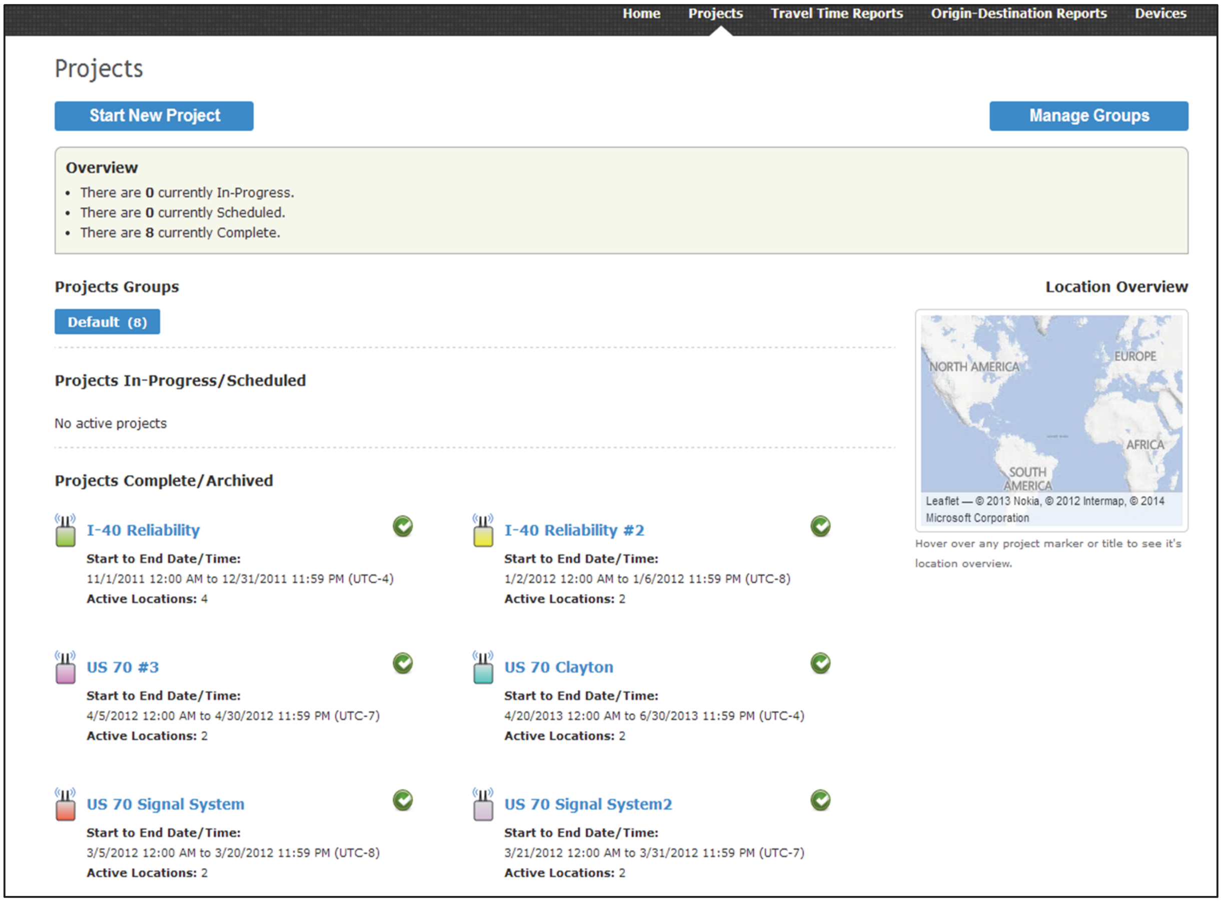


Figure 8-5 BlueMAC Project Website

The website also allows the user to do real-time monitoring on selected study routes as shown in Figure 8.6. It provides a threshold for the selection of ranges for travel time and speed. The website provides the trip distance, expected travel time, total number of trips, average speed, average travel time, etc. The user can select daily, weekly and bi-weekly travel time monitoring results, and travel time can be aggregated in 5 minutes, 15 minutes, hourly and daily increments. All the results will be displayed on the website by selected user values.

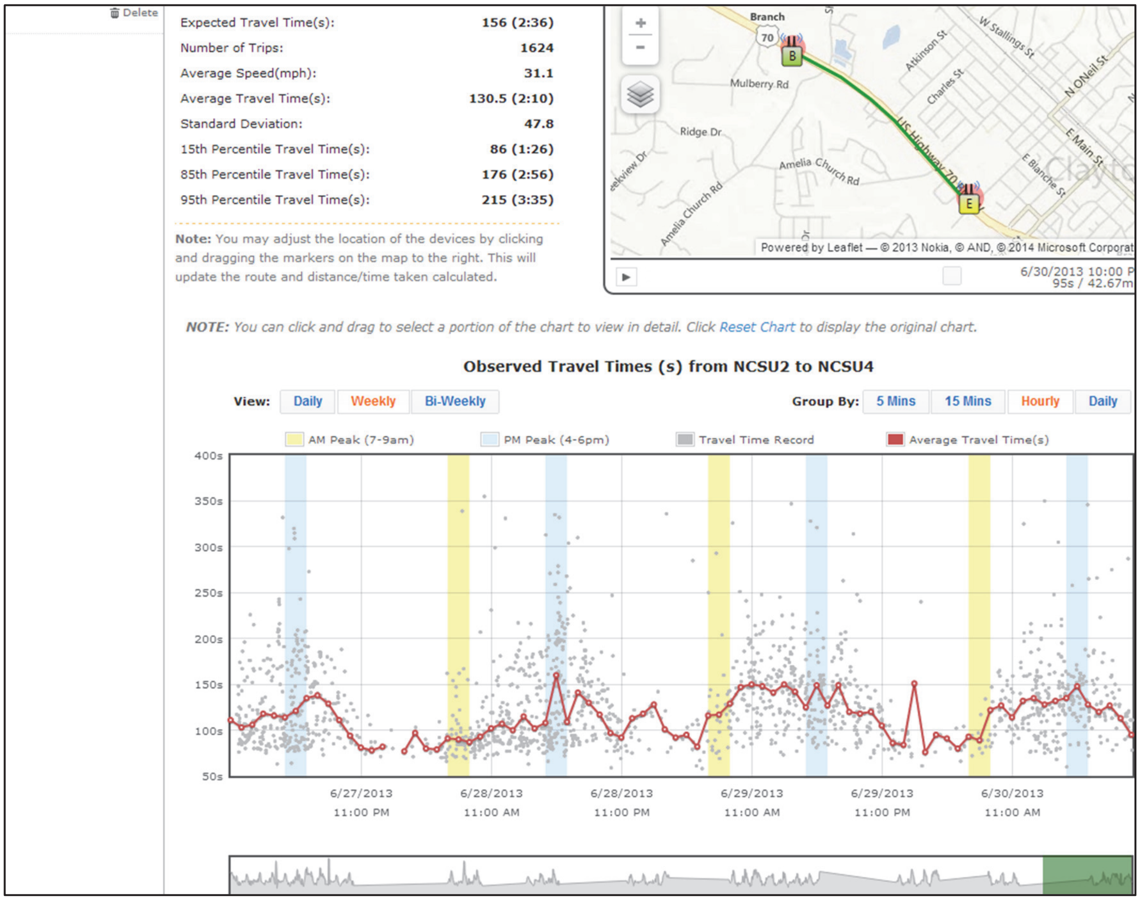


Figure 8-6 BlueMAC Travel Time Monitoring Example

8.3.2 MAC Address Archive

BluFax and BlueMAC devices collect media access control (MAC) addresses from Bluetooth-enabled devices, and calculate travel time by detecting the signals of same MAC address at two locations. The difference between the two devices is that BluFax archives all 12 characters of MAC addresses while BlueMAC truncates the first 5 digits and last one digit of the MAC address. The MAC address is a unique ID, when it has all 12 digits. However, after truncating several digits of MAC address for privacy issues, it will not be a unique ID anymore.

For testing the effect of truncating 5 digits of the MAC address, the project team designed an experiment. Since BluFax units archive all 12 digits of MAC address, the project team used BluFax station data as experimental raw data. From March 5, 2012 to March 22, 2012 (total of 18 days) BluFax units Station data was collected. After making station data using BluSTATS software, the first five digits and last one digit of MAC addresses were truncated. Table 8.5 shows the station data form of both data types. Both stations data are processed to matching MAC addresses and the results are presented in the Table 8.6. For the matching MAC addresses, the experiment used no time boundary.

Table 8-5 DBAT Exhaustive Search Result Excluding Secondary Bands

BluFax Station MAC address	Truncated BlueMAC type MAC Address
10:C6:FC:F3:8F:54	00:00:0C:F3:8F:50
00:23:06:D7:63:07	00:00:06:D7:63:00
D8:B3:77:E4:C6:41	00:00:07:E4:C6:40
00:A0:96:33:1C:11	00:00:06:33:1C:10
D8:B3:77:E4:C6:41	00:00:07:E4:C6:40

The complete MAC address (full of 12 digits) provides 5,735 trips and 6,466 trips for EB and WB, respectively. However, when the first 5 digits and last one digit of the MAC address was truncated, the total numbers of trips were increased by 14 and 12 units on EB and WB. The result shows that even if it is unlikely (less than 0.24%), there is a possibility to create wrong trips.

Table 8-6 Matching Number Comparison

Site (Direction)	Total number of Matching (Trips)		
	BluFax	BlueMAC	Difference
Jessup to Yeargan (EB)	5,735	5,749	14 (0.24%)
Yeargan to Jessup (WB)	6,446	6,458	12 (0.19%)

8.4 Summary

This chapter introduces a linear programming (LP) formulation to enable dynamic bandwidth maximization on semi-actuated arterial streets. The methodology relies on the use of archived signal log data on phase start and end times in each cycle in both directions. That data is used to optimize the offsets that maximize the variable system bandwidth across multiple cycles constituting a coordination plan period. The formulation is also flexible to optimize signal offsets using programmed (fixed) green durations at each intersection. The proposed formulation offers four significant enhancements compared to traditional methods.

1. The formulation is strictly linear (complexity of class P) as opposed to the traditional mixed integer programming formulations (complexity of class NP-hard).
2. It can work with either static or dynamic (cycle varying) green durations.
3. Traditional bandwidth optimization methods have explicit constraints to enforce bandwidth allocation to be proportional to directional demand. However, as longs as the minimum required bandwidth is allocated to each direction, a proportional allocation of bandwidth is not necessary and in fact, can result in reporting sub-optimal solutions. The proposed formulation overcomes this shortcoming.
4. The proposed formulation predicts the maximum proportion of traffic demand that can be served in each band, a unique attribute absent from other formulations found in the literature.

In addition, two Bluetooth devices are introduced for monitoring arterial travel time. Arterial travel time is an important performance measure but it is difficult to monitor by the current OASIS log. Bluetooth units can be a good method for monitoring arterial travel times.

CHAPTER 9. DEVELOP RECOMMENDATIONS FOR IMPLEMENTATION OF OTHER USES OF MODEL OUTPUTS (TASK 9)

In this chapter, INRIX arterial travel time (speed) data is compared with Bluetooth data. Two sites are selected and multiple days' results are compared. In addition, project team collected and analyzed high resolution detector data using Sensys wireless sensors for recommendation and analysis. The research reported in this chapter represents a broadening from the original task description. The motivation behind the research was to both test the accuracy of INRIX arterial times in turn testing the usefulness of the INRIX data in assessing coordination plan quality and to evaluate the accuracy of the OASIS detector log data. These evaluations are foundational to support sound decisions on how the INRIX and OASIS data should be used for all purposes, including coordination plan assessment.

9.1 INRIX and Bluetooth Travel Time Comparison

INRIX provides link based travel time (speed) data. INRIX uses mainly GPS-enabled probe vehicles to collect speed information on over 1 million miles of roads across the United States. INRIX uses the industry standard Traffic Message Channel (TMC) coding system to represent directional roadway segments. The TMS segments are defined with a geo-located beginning and ending point. INRIX records and reports Travel Time, Average Speed, Reference Speed, C-Score and C-Value at a one-minute temporal resolution for all TMC segments. Average speed is a time of day and day of week average that is periodically updated, while Reference speed is the 85th percentile measured speed capped at 65 mph. The C-Score indicates if the speed is historical (reference speed), real time or a blend of real time and historical data, while the C-Value is a measure of confidence for the real-time data.

Archived one-minute INRIX travel time data were collected from the Regional Integrated Transportation Information System (RITIS), and the INRIX travel times were compared with Bluetooth travel time. When comparing the travel time, care must be taken to properly consider the sampling differences between the Bluetooth and INRIX data. The unique potential for bias must be considered for each data source, and with increased usage of network probe data it is important to understand the factors that contribute to differences in path travel time estimates between the two methods. The Bluetooth observations were analyzed using the BlueStats software provided by TRAFFAX. BlueStats includes an outlier detection method based on the non-parametric IQ4 method. However, none of the BlueTooth travel time observations were identified as outliers by BlueStats.

9.1.1 NC 55 Arterial Comparison Result

The first selected site is on the NC 55 arterial in Apex. The arterial segment (TMC 125+06184) starts from Hunter St. intersection and ends at the US 64 EB off-ramp. The segment length is 1.02 miles. Both INRIX and Bluetooth travel time data are collected for 8 weekdays from August 27, 2012 to September 6, 2012.

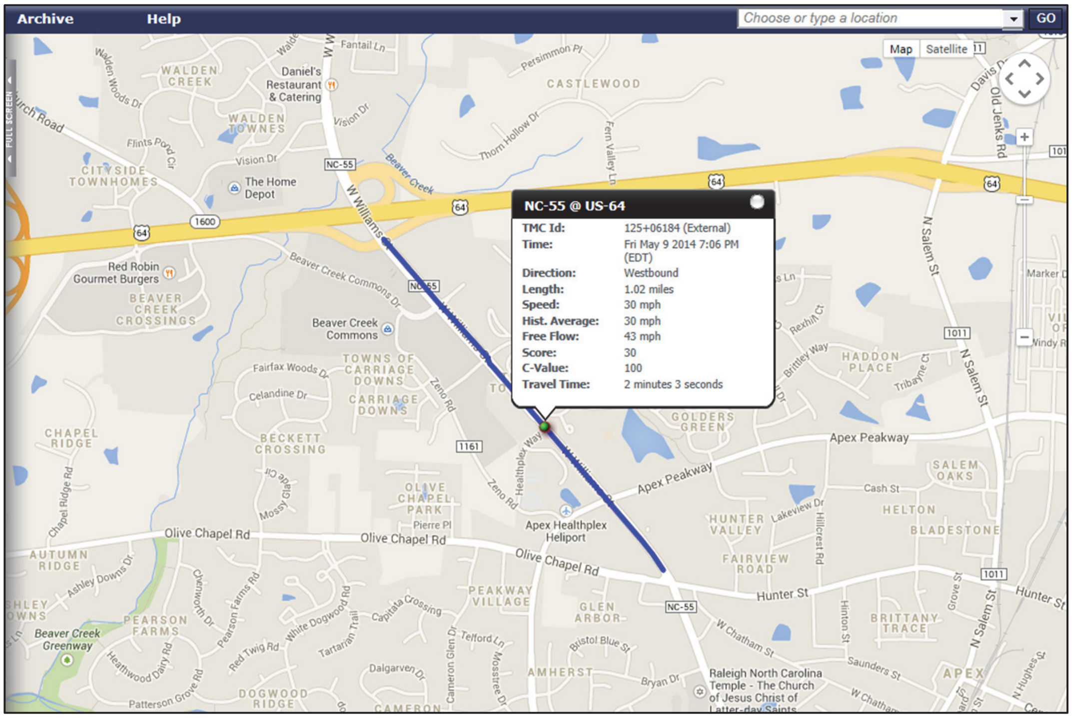


Figure 9-1 INRIX TMC Segment 125+06184

Table 9.1 provides the TMC 125+06184 segment start and end coordinates. BluFAX unit were installed at both ends of the segment as close as practicable to these two positions.

Table 9-1 TMC 125+06184 Segment Information

TMC	Road	Direction	County	Start Latitude	Start Longitude	End latitude	End longitude	miles
125+06184	NC-55	WESTBOUND	WAKE	35.736157	-78.862898	35.747442	-78.874657	1.022389

Figure 9.2 shows the six weekdays INRIX and Bluetooth travel times and provides a visual comparison.

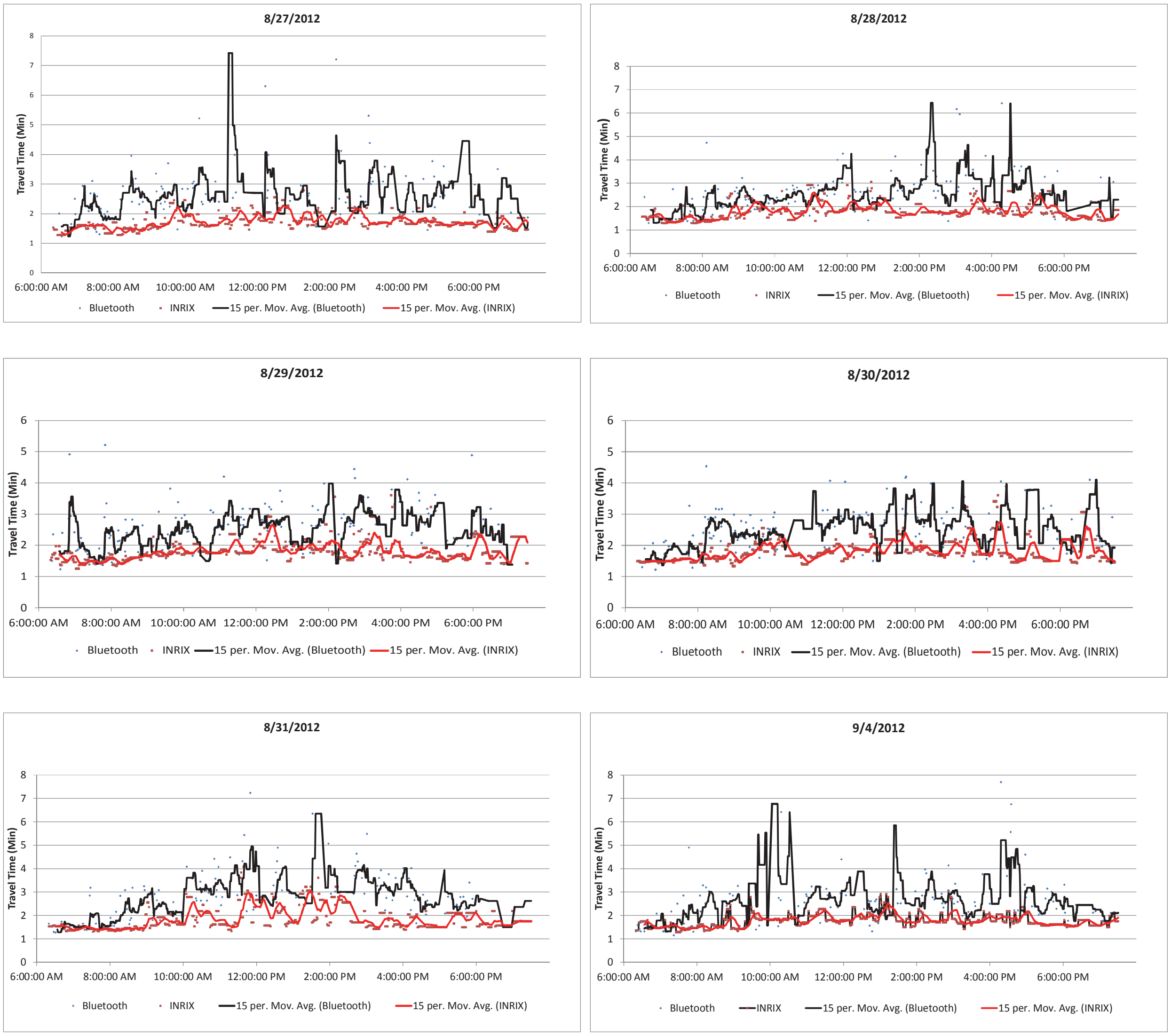


Figure 9-2 Six Weekdays INRIX and Bluetooth Travel Time Comparison

As demonstrated in Chapter 5, Bluetooth travel times and floating car travel times display very similar magnitudes and distributions. However, Figure 9.2 shows two systems (INRIX and Bluetooth) provide quite different results in terms of average travel time and that INRIX and Bluetooth travel times are especially different when the Bluetooth data are indicating travel times much longer than the free flow travel time. Table 9.2 highlights the discrepancies in average travel time. The INRIX reported TMC travel time is approximately 31.3% less than Bluetooth travel time (BluFAX observations processed using the BluSTATS software).

Table 9-2 Average Travel Time Comparisons (NC 55)

	8/27/2012	8/28/2012	8/29/2012	8/30/2012	8/31/2012	9/4/2012	9/5/2012	9/6/2012
INRIX	1.722	1.798	1.802	1.807	1.856	1.773	1.843	1.765
Bluetooth	2.601	2.552	2.520	2.494	2.807	2.645	2.699	2.623
% Difference	33.8%	29.5%	28.5%	27.5%	33.9%	33.0%	31.7%	32.7%

9.1.2 Western Blvd Arterial Comparison Result

The project team also conducted arterial travel time comparisons between INRIX and Bluetooth. This comparison was conducted after INRIX announced improvement of their system using sub-TMC analysis. Simultaneous travel time data were collected during March 25 to April 2, 2014. As above, INRIX one-minute TMC reports were downloaded from the RITIS server, and BluFAX units were again used for collecting the Bluetooth travel time data. The selected TMC segments selected were 125+14768 and 125-14767 (Western Blvd arterial between Avent Ferry Rd. intersection and Gorman St. intersection in both the eastbound and westbound directions). Table 9.3 shows the selected TMC segment's start and end coordinates. Two BluFAX units were installed as close to the TMC star/end positions as practicable. Figure 9.3 illustrates the location of the analyzed TMC.

Table 9-3 TMC 125+06184 (125-14767) Segment Information

TMC	Road	Direction	County	Start latitude	Start longitude	End latitude	End longitude	Miles
125-14767	Western Blvd	EASTBOUND	WAKE	35.784907	-78.687049	35.780727	-78.675529	0.72
125+14768	Western Blvd	WESTBOUND	WAKE	35.780868	-78.67543	35.785053	-78.687049	0.72

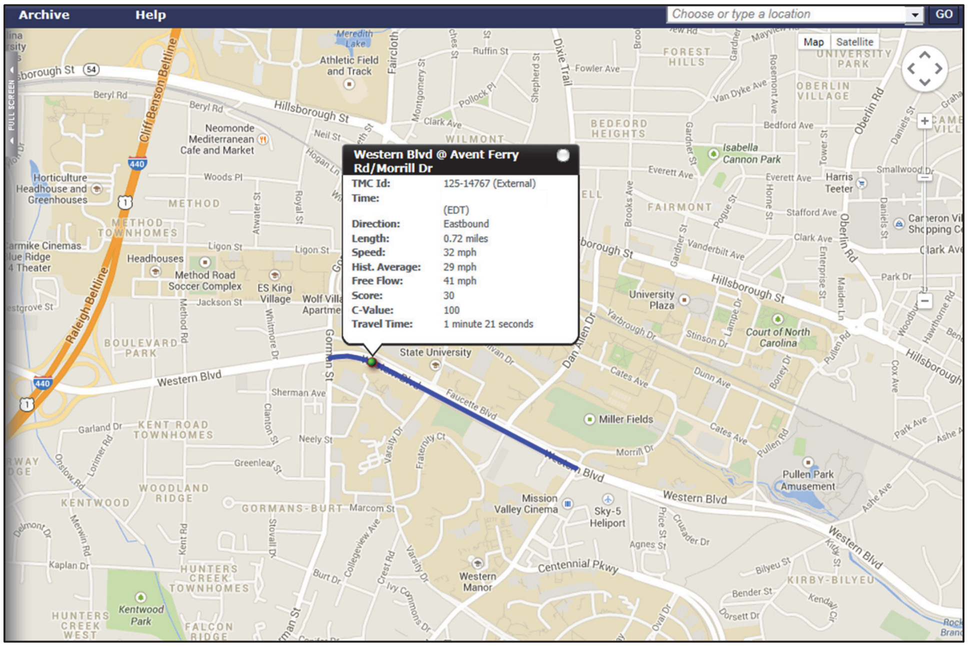


Figure 9-3 INRIX TMC Segment 125-14767

The Western Blvd TMC 125-14767 and 125+14768 travel time comparison results are not significantly differ from the previous (NC 55) results. Table 9.4 shows the daily average travel time for both INRIX and Bluetooth. The total average percentage difference between the two travel times is 37.8% with INRIX once again reporting shorter travel time than Bluetooth.

Table 9-4 Average Travel Time Comparisons (Western Blvd)

Eastbound									
Date	3/25/2014	3/26/2014	3/27/2014	3/28/2014	3/29/2014	3/30/2014	3/31/2014	4/1/2014	4/2/2014
INRIX	1.462	1.305	1.305	1.371	1.169	1.114	1.296	1.329	1.295
Bluetooth	3.610	1.998	2.039	2.384	1.672	1.574	2.191	2.074	2.064
% Difference	59.5%	34.7%	36.0%	42.5%	30.1%	29.2%	40.8%	35.9%	37.3%
Westbound									
INRIX	1.293	1.350	1.330	1.407	1.221	1.145	1.362	1.306	1.323
Bluetooth	2.263	2.013	2.112	2.270	2.105	1.905	1.957	2.015	2.051
% Difference	42.9%	33.0%	37.0%	38.0%	42.0%	39.9%	30.4%	35.2%	35.5%

Figure 9.4 and Figure 9.5 provide the travel time plots for March 26, 2014.

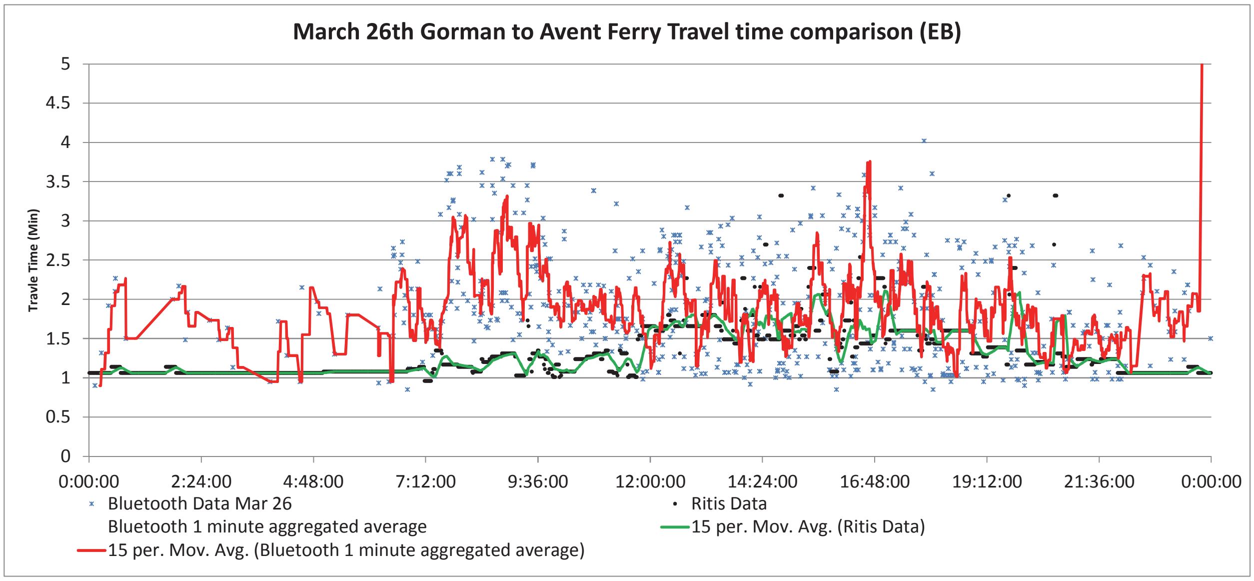


Figure 9-4 EB Travel Time Comparison (TMC 125-14767)

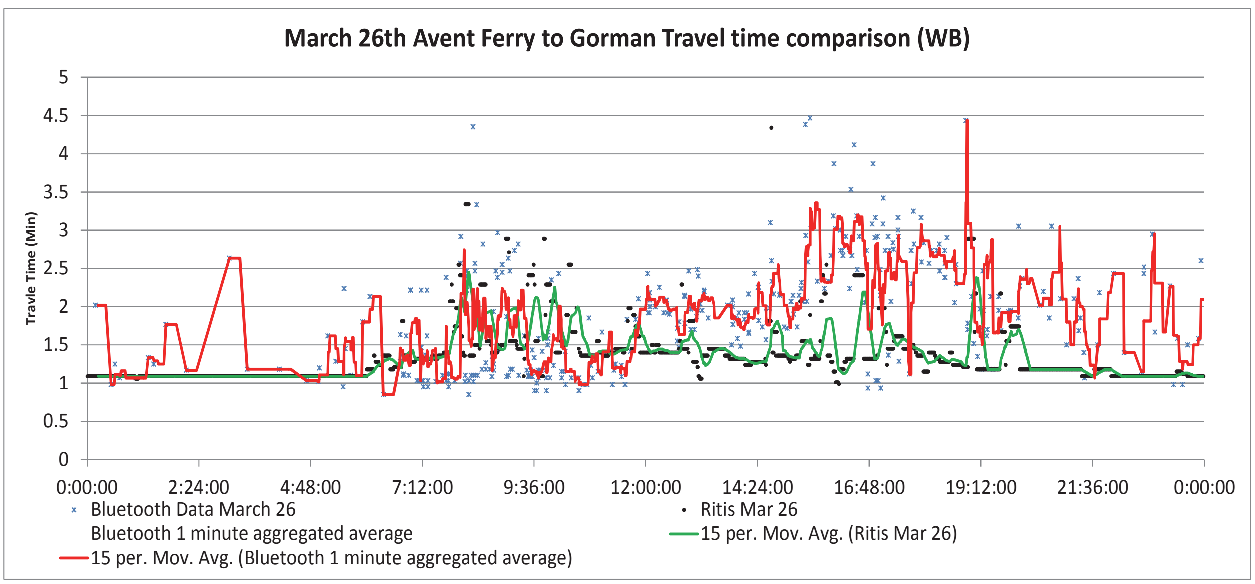


Figure 9-5 WB Travel Time Comparison (TMC 125+14768)

9.2 High Resolution Detector Data

9.2.1 Data Quality Test

As mentioned earlier in Chapter 4 and Chapter 5, the OASIS detector log provides a minimum one-minute aggregated resolution, and the detector log data were compared with manually counted video data. The results indicate the OASIS lane-based detector has less than a 2% error. The S. Robertson WB direction has three system detectors, and project team installed three sensors at the same locations (see Figure 4.17-f). Sensys sensors provide a one hundredth of a second resolution for detector actuation results such as illustrated in Table 9.5. The column of "Detection" indicates sensor status (a value of 1 means the detector is "on" (occupied) and 0 means the detector is "off"(empty)). For testing detection accuracy, raw data was aggregated to a one-minute interval to match the OASIS detector log. The detectors in each of the three lanes were compared to this one-minute resolution for data collected on April 22, 2013.

Table 9-5 Sensys High Resolution Data Example

Sensor ID	Time	Detection
7656	4/22/2013 0:06:48.29	1
7656	4/23/2013 0:06:49.37	0
7656	4/22/2013 0:14:23.81	1

Table 9.6 shows the difference between OASIS and Sensys detector calls. There are less than 3 % difference between OASIS loop detectors and Sensys wireless sensors. Figure 9.6 provides a graphical comparative display of the detector calls per minute.

Table 9-6 Detector Detection Differences

	Lane 1	Lane 2	Lane 3
OASIS	4,950	6,055	3,693
Sensys	5,030	5,992	3610
% Differences	1.61%	1.04%	2.25%

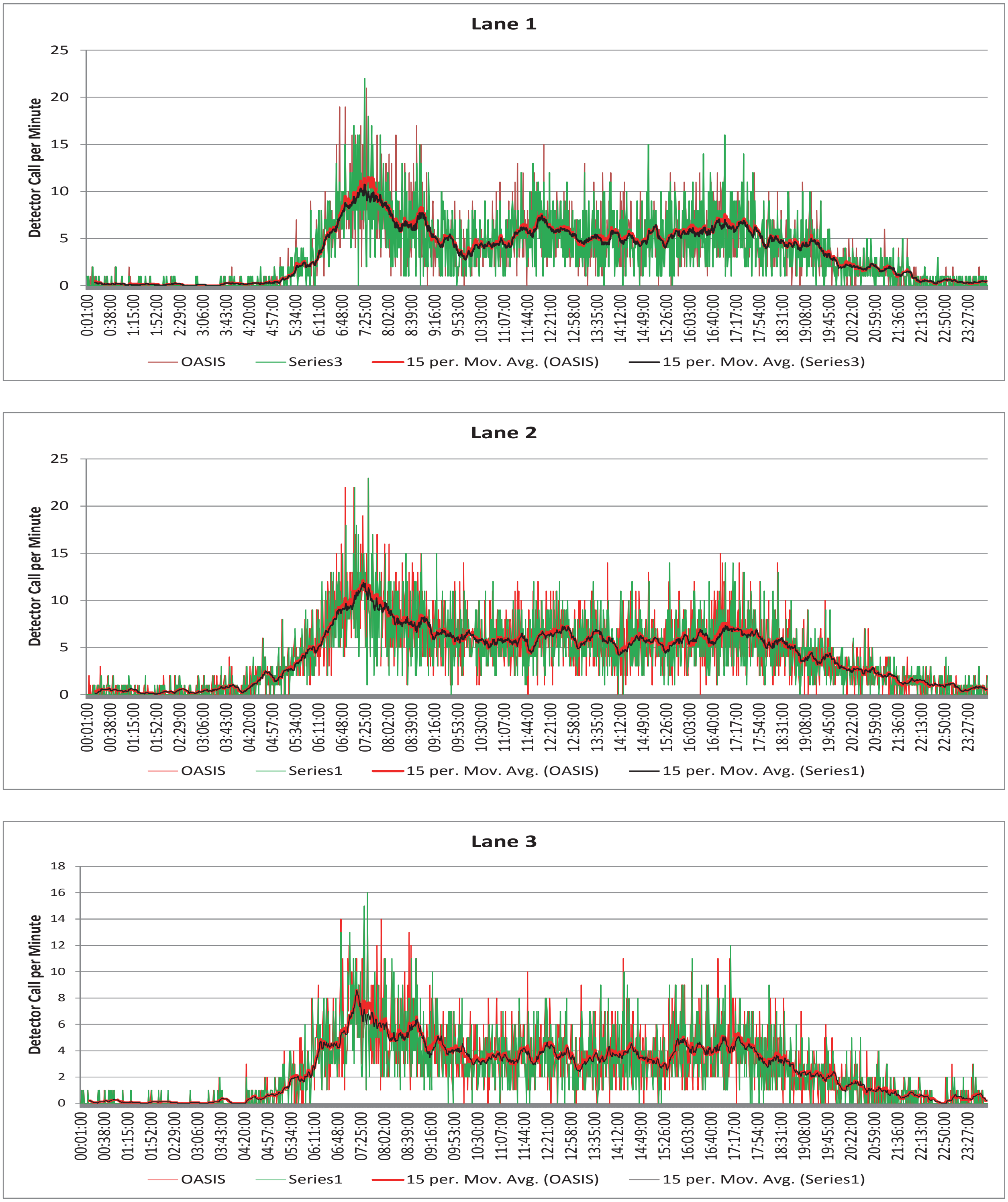


Figure 9-6 OASIS and Sensys Detector Call Comparison

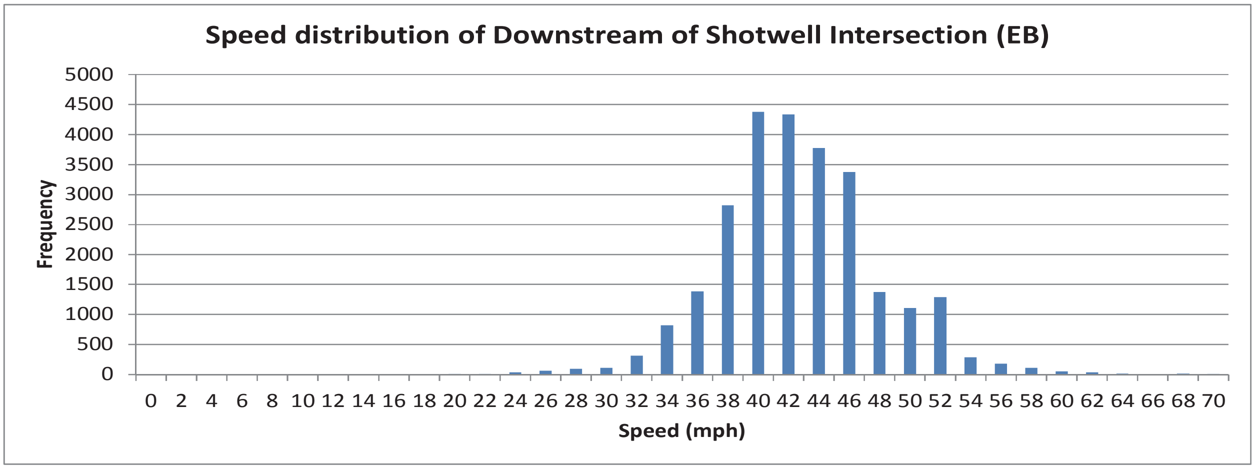
9.2.2 Segment Travel Speed Distribution

The OASIS detector log also provides speed information. The speed calculation is a single loop estimate that relies on a default or user specified value for average vehicle length plus detector detection zone length. Therefore, the average vehicle length plus detection zone length should be calibrated if reasonably accurate speed values are desired. OASIS uses 20 feet as the default value of average vehicle length plus detection zone length.

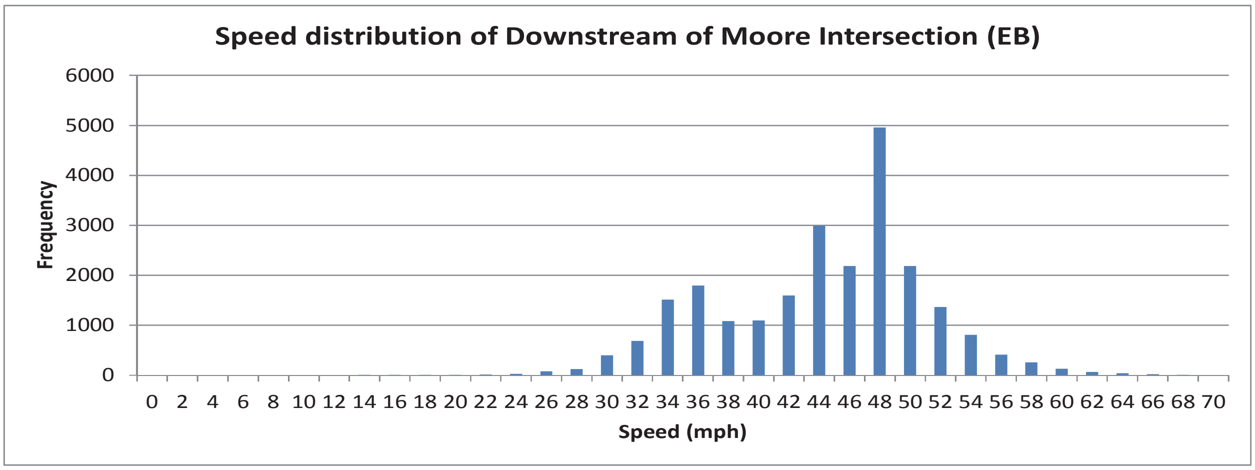
In order to collect more accurate segment travel speeds, the project team created a speed trap with dual sensors. Figure 4.17 (b), (c) and (d) show three speed traps that lie between coordinated intersections. Two sensors are used for creating each speed trap, and the distance between the paired sensors was set at 20 ft. The speed trap locations were located approximately 250 ft. downstream from the intersections. This placement was selected to provide a balance between avoiding queue spill back the downstream intersection while providing enough distance from the upstream intersection for vehicle acceleration. Table 9.7 shows each speed trap's average, mode and 95th percentile speed. The three locations' average speeds are 43.2, 44.8 and 44.2 (the speed limit is 45 mph). The 95th percentile speeds are 52, 54 and 54 mph. Figure 9.7 graphically illustrates each speed trap's speed distributions.

Table 9-7 Speed Trap Speed Summary

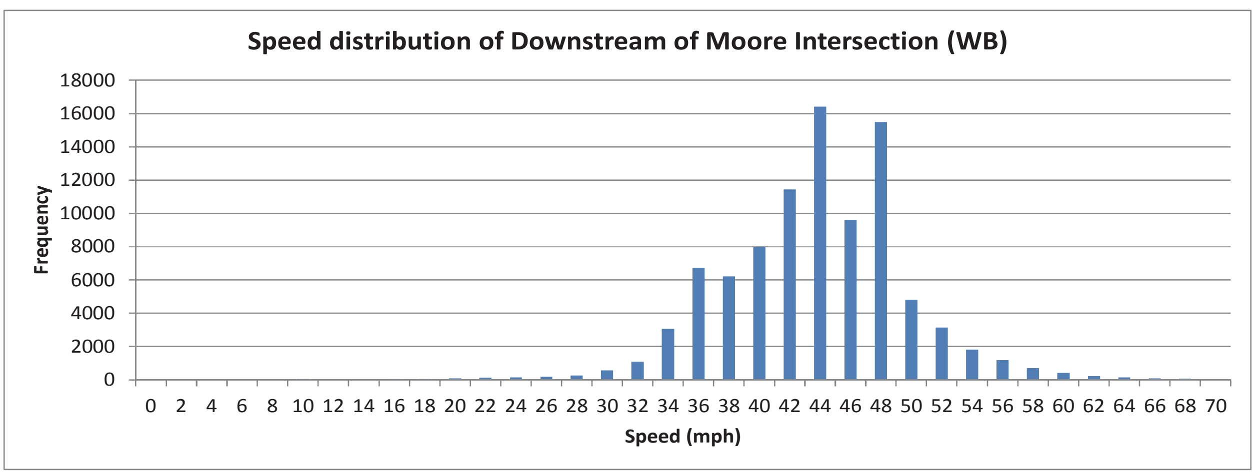
	Downstream of Shotwell (EB)	Downstream of Moore (EB)	Downstream of Moore (WB)
Sample Size	25,975	23,903	92,374
Average of Speed	43.2	44.8	44.2
Mode of Speed	41	48	47
95th of Speed	52	54	54



(a) Speed Distribution of Shotwell Rd. Downstream (Sea Figure 4.17-b) EB



(b) Speed Distribution of Moore St. Downstream (Sea Figure 4.17-d) EB



(c) Speed Distribution of Moore St. Downstream (Sea Figure 4.17-c) WB

Figure 9-7 Segment Travel Speed Distributions

9.2.3 Combine Dynamic Bands and High Resolution Detector Data

The OASIS split monitor log provides quite accurate signal status information in that relatively precise times can be derived for the beginning and ending of all signal indications. This contrasts with the detector log's one-minute aggregation. If high resolution vehicle detector data, such as was recorded from the temporary Sensys detector installations, were available, this vehicle detection level data could be merged with the cycle-by-cycle dynamic green bands and be displayed graphically or otherwise analyzed.

Figure 9.8 (EB) and Figure 9.9 (WB) show the visualized result of dynamic bandwidth and the high resolution detector data. Each dot indicates a vehicle with each dot's color representing the respective travel lane (from left to right – blue is lane one, red is lane two and black is lane three). The eastbound direction has only two lanes except for a lane between Shotwell Rd. and Moore St. that ends as an exclusive right turn. Therefore, the eastbound direction only has black dots between Shotwell and Moore.

Information that can be gleaned from this diagram includes –

1. Entering intersection vehicle arriving type
2. Average platoon headways
3. Platoon size
4. Number of vehicles arriving on green (or percent arriving on green)
5. Number of vehicles inside of dynamic bands (or percentage of vehicles inside of green bands)

The EB traffic entering intersection the three intersection subsystem at Shotwell Rd. does not appear to be characterized by random arrivals. The upstream intersection, Town Center Blvd., is 1.2 miles from the Shotwell Rd. intersection, and although not included in the field study the Town

Center Blvd. intersection is coordinated with the three analyzed intersections. Figure 9.8 appears to show that this case of coordination is effective even with the relatively long distance of over a mile.

Table 9.8 and Table 9.9 provide the percentage of arrivals on green and the percentage of vehicles in the green bands for each direction, and this data can be used to assess the quality of progression. For example, the EB direction movement from Shotwell to Moore has a percentage of arrivals on green and percentage of vehicles in green bands of 90.08% and 78.50%, respectively. However, the WB direction from Robertson to Moore has a percentage of arrivals on green and vehicle in green bands of only 37.04% and 15.59%, respectively. The result illustrates that most of EB direction vehicles can pass the Moore intersection without stopping, while most of the WB direction vehicles must stop at the Moore intersection.

Table 9-8 US 70 Arterial in Clayton EB during Midday 1:30 PM Plan

Downstream of the Shotwell Intersection Approaching Moore	**Total Number of vehicles**	3,014
	Arriving on Green	2,715
	% of Arriving on Green	90.08
	Number of vehicles in the Bands	2,366
	% of vehicles in the Bands	78.50
Downstream of the Moore Intersection Approaching Robertson	**Total Number of vehicles**	3,017
	Arriving on Green	2,468
	% of Arriving on Green	81.80
	Number of vehicles in the Bands	2,284
	% of vehicles in the Bands	75.70

Table 9-9 US 70 Arterial in Clayton WB during Midday 1:30 PM Plan

Downstream of the Robertson Intersection Approaching Moore	Total Number of vehicles	2,605
	Arriving on Green	965
	% of Arriving on Green	37.04
	Number of vehicles in the Bands	406
	% of vehicles in the Bands	15.59
Downstream of the Moore Intersection Approaching Shotwell	Total Number of vehicles	2,376
	Arriving on Green	1,850
	% of Arriving on Green	77.86
	Number of vehicles in the Bands	1362
	% of vehicles in the Bands	57.32

The EB direction is very well coordinated since most of the platoons are inside of dynamic bands in contrast to the WB direction, which exhibits poor coordination. Analysis of the high definition data plotted with the dynamic green bands reveals that the Robertson St. intersection has a green start that is too early and the Moore St. intersection green start is too late. This creates a situation where the platoons released from the Robertson St. intersection must stop at the Moore St. intersection. Therefore, the high definition data plots indicate that WB progression would be improved by starting the coordinated green at the Moore St. intersection earlier and starting the coordinated green at the Robertson St. intersection later than was the case in the April 2013 plan. The joint high resolutions detector data and dynamic bandwidth diagram gives an intuitive sense of offset fine-tuning opportunities. Table 9.10 shows the Chapter 7 dynamic bandwidth optimization results compared with the current offsets. The results shows the Moore St. intersection offset should start 17 seconds earlier and the Robertson St. intersection should start 7 seconds later compared to the April 2013 offsets in order to maximize the total sum of weighted bandwidth.

Table 9-10 Dynamic Bandwidth Optimization Results

Cycle Length (sec)	Plan Start (Plan End)	Plan	Offset (sec) *			EB Band Size (sec)	WB Band Size (sec)	Total Band Size (sec)	EB (WB) Weight ***
			Int. 1	Int. 2	Int. 3				
150	13:30 (16:30)	April field Plan	0	4	47	4,948	1,848	6,797	0.67 (0.33)
		Estimated Optimal Plan****	0	137	54	4,783	2,774	7,557	

The results reveal that a modification to the OASIS software that would allow temporary or ongoing logging of detector actuations would support detailed analysis of coordination quality for overall plan modification or offset fine tuning. In addition to modification of the OASIS software, the only hardware that would be needed would be for the purpose of storing the high resolution detector data. An external hard drive or field robust solid state memory could easily serve this purpose. The combination of the ability to store high resolution detector data and the graphical and statistical analyses described above would provide a cost effective alternative to labor intensive field studies for plan assessment and fine tuning.

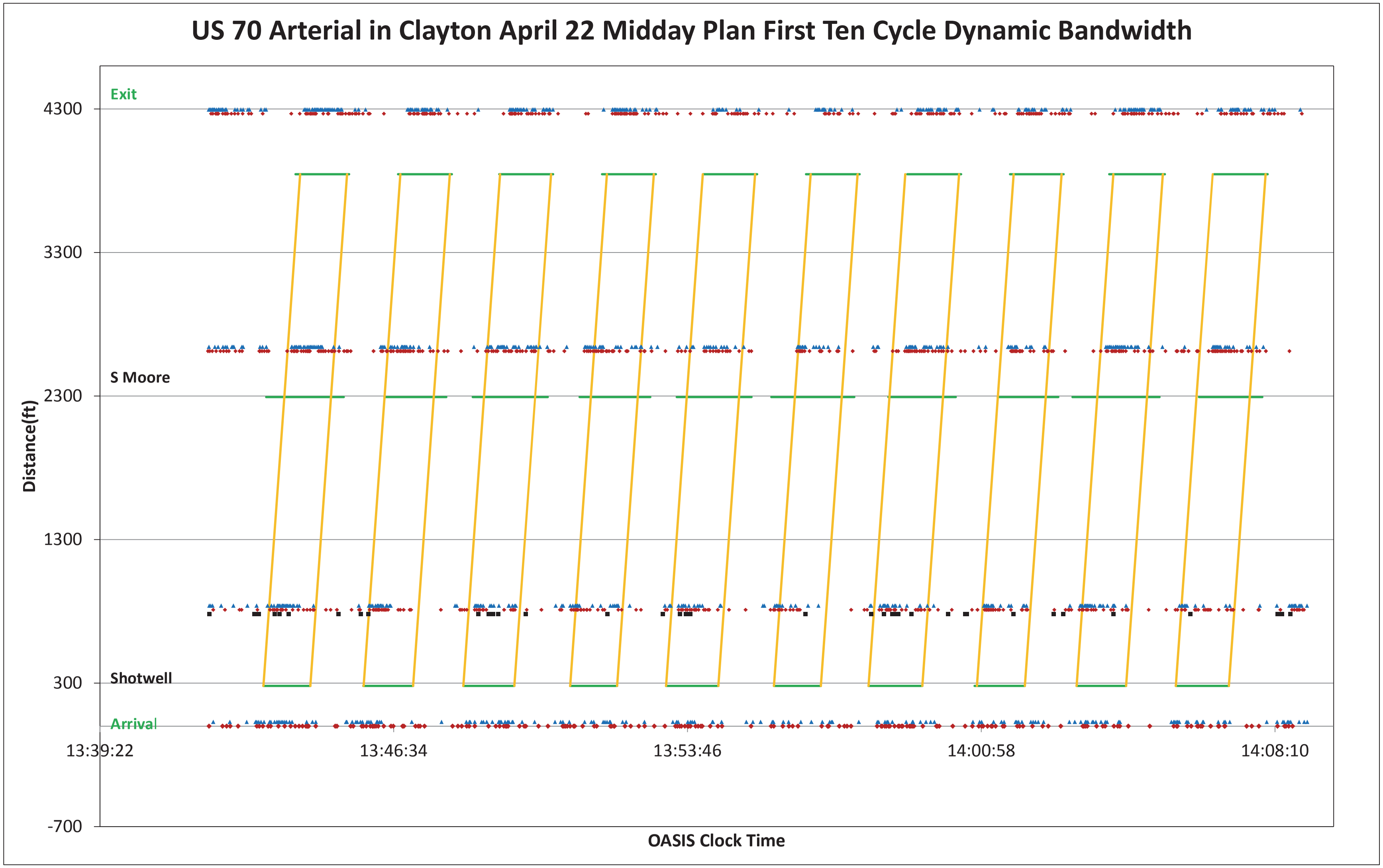


Figure 9-8 Dynamic Bandwidth and High resolution Detector Data (EB)

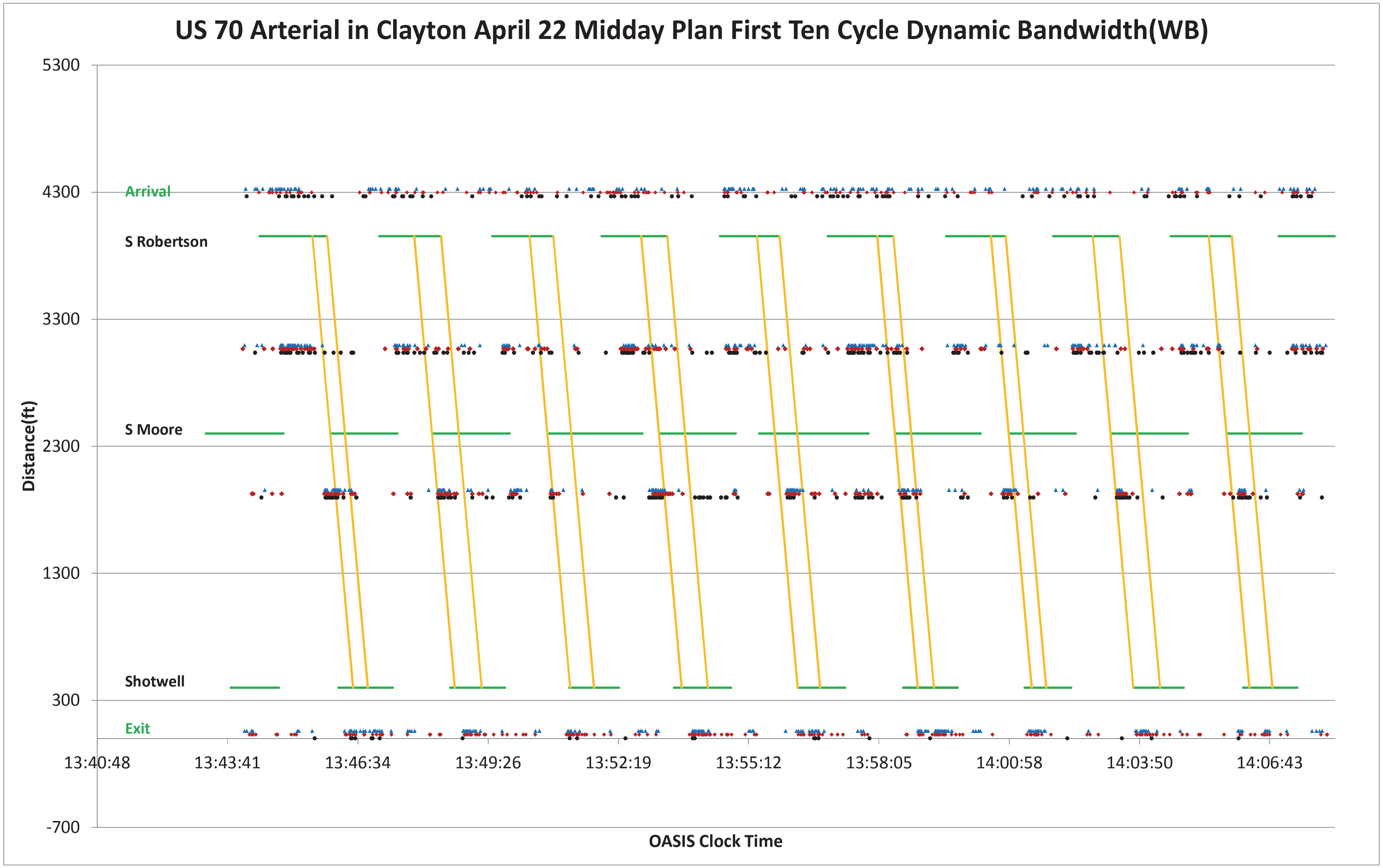


Figure 9-9 Dynamic Bandwidth and High resolution Detector Data (WB)

9.3 Summary

In this chapter, INRIX arterial TMC reported travel time data and Bluetooth travel time data were compared, and the results of a study that involved the collection and analysis of high resolution detector data was presented. The motivation for the comparison study between INRIX and Bluetooth was to answer the question of whether or not arterial travel times derived from INRIX TMC segment travel times could serve as an effective alternative or supplement to field travel time runs. Given that the INRIX TMC segment data are readily available at a one-minute resolution for many of the closed-loop systems under the COST section's responsibility, the primary question is whether or not INRIX segment travel times are sufficiently accurate. If the INRIX arterial segment travel times are accurate relative to directly measure travel times, then the INRIX could allow the COST section to easily and remotely monitor closed-loop system travel times. This would not only minimize the cost of conducting field travel time runs but would allow analysis of a much broader operational time window than is possible through field study. The motivation for the high resolution detector data study was to assess the timing plan assessment and fine tuning potential of joint analysis of system detector actuations and dynamic green band analysis based on the OASIS split monitor log.

The comparison between INRIX and Bluetooth travel times shows that INRIX TMC reported travel time is approximately 31.3% less than Bluetooth travel time for NC 55 Apex (TMX 125+06184) and 37.8% less than Bluetooth travel time for Western Blvd (TMC 125-147 67 and TMC 125+14768). It appears that this discrepancy in average travel time primarily results from the fact that the INRIX arterial peak travel times are much lower than the Bluetooth peak travels (see Figure 9.2, Figure 10.4 and Figure 10.5).

High resolution detector data was collected by 24 Sensys wireless sensors. In addition to basic information on platooning and vehicle arrival characteristics, the high resolutions detector data can help the signal offset fine-tuning process when combined with split monitor data that has been further processed to show the dynamic, cycle-by-cycle green bands (see Figure 9.8 and Figure 9.9). The high resolution data plotted along with dynamic bandwidth give a clear sense of the quality of signal coordination and visual clues for offset fine tuning. Statistics derived from the joint detector actuation/dynamic bandwidth data further provide direct estimation of important progression quality MOEs, such as percentage of vehicles traveling within the green bands and percentage of vehicles arriving under green. Cost effective implementation of this type of high resolution detector data analysis could be accomplished through a modification to the OASIS software that would allow temporary or ongoing logging of system detector calls coupled with an external memory storage device.

CHAPTER 10. SUMMARY OF FINDINGS, CONCLUSIONS, AND RECOMMENDATIONS

The chapter provides the project's key findings and conclusions summarized by task. The project team's recommendations based on these findings and conclusions are summarized in the following chapter. Findings and conclusions are provided for each project task, and recommendations are provided when appropriate for certain tasks.

10.1 Task 1 - Literature Review – Findings and Conclusions

- Several adaptive control systems are used for advanced traffic management in United States.
    - These systems in general required installing more detectors than required for the current standard NCDOT specifications and also involve more complex operational parameters. Therefore, these systems would involve increased installation and maintenance costs as well as increased cost and time for signal engineer retraining.
    - ACS-Lite is a reduced-scale version of the FHWA adaptive control software. It offers small and medium-size communities a lower-cost traffic control system that provides some real-time adaptive control features.
- Advances have been made in arterial and signal monitoring systems, and these systems are in the early stages of availability and deployment.
    - SMART-SIGNAL uses queue estimation modeling and vehicle acceleration and deceleration modeling in order to estimate arterial travel time using a virtual probe vehicle generation method.

  - Researchers at Purdue University have created several useful monitoring methodologies to monitor both intersection and arterial level of signal operating performance.
  - These arterial monitoring systems are theoretically robust and show strong promise based on the monitoring results in published literature.
  - Both systems require additional detectors as well as the capability to collect high resolution detector data.
- The current NCDOT closed-loop system specifications do not directly support SMART-SIGNAL or the Purdue Arterial Monitoring Method for the following reasons.
  - The OASIS software does not provide high resolution detector data. This data is needed for the cycle-by-cycle queue estimation in SMART-SIGNAL and to generate the vehicles arrival during green and red for the Purdue Coordination Diagram.
  - NCDOT's signal system detector specifications do not generally call for a stop bar detector and an upstream detector for non-coordinated movements. This detector configuration is needed for SMART-SIGNAL's turning movement estimation routine and for the Purdue intersection level non-coordinated movement monitoring routines.
- There are three generally used arterial performance measures, namely number of stops, travel speed, and bandwidth. Neither stops nor travel speed can be measured directly or estimated using for the current general NCDOT signal system design and the current

OASIS software implementation. Bandwidth can be analyzed from the OASIS split monitor log using the analysis procedures developed under Task 6.

10.2 Task 2 - Select Study Location – Findings and Conclusions

Project team selects three study sites for field data collection and developed model test. The selected study sites are:

- US 70 arterial in Clayton (including 3 signalized intersections)
- US 70 arterial in Garner (including 4 signalized intersections)
- NC 55 arterial in Apex (including 7 signalized intersections)

10.3 Task 3 - Design Data Collection Plan – Findings and Conclusions

In this task, available data sources from current NCDOT closed loop systems were investigated and additional data collection devices and plans were introduced.

- OASIS system temporally archives seven log files in its local memory, and these log files can be downloaded over a dialup modem connection using the TransLink32 software.
- The split monitor log provides each phase’s allotted green time and displayed green time.
- The detector log provides minimum one-minute resolution of vehicle volume (call), occupancy, and speed.
- The quality of detector log data depends on detector configuration.
    - Lane-based detectors provide more accurate information compare to link-based detector.
    - Second, 6’ by 6’ system detectors provides more detailed information than long loop detectors.

- The OASIS detector log for both occupancy and speed uses a strictly round-down approach before the system archives the calculated results.

10.4 Task 4 - Conduct Field Studies – Findings and Conclusions

The field collection devices all provided useful information. However, the high mount video cameras installed at the prototype field site on US 70 in Garner involved troublesome installation and less than desirable results in terms of the quality of results from digital image processing. This led to the need to do manual counts from the video. Also, the portable mast-mounted side fire radar detector used on the prototype field site did not provide reliable results due to the difficulty in maintaining the device's aim and lane calibration. Finally, although the Sensys installation used for the US 70 Clayton site provide much useful information, the complexity and labor investment in installing and removing the in pavement detectors and mounting and configuring the communications devices may be too extensive for use as temporary data collection system.

10.5 Task 5 - Investigate Relationships and Develop Candidate Models

10.5.1 Findings and Conclusions

This task involved field study analysis, intersection level performance analysis, and dynamic bandwidth investigation. The key findings and conclusions are –

- Signalized arterial travel times have a multi-modal distribution (see Figure 5.1 and 6.2) that is governed primarily by the number of stops.
- The number of stops is directly related to signal coordination (see Figure 6.3) and the presence of early return to green.
- The comparison of floating car and Bluetooth travel time (see Table 5.9) supports the accuracy of Bluetooth travel time.

- Lane-based 6’ by 6’ single loop detectors exhibited a detection error rate of 1.17% during the non-peak hour and 3.92% during the peak hour.
- The OASIS detector log can be used to generate a daily volume profile useful for evaluating time of day timing plans (see Figure 5.12).
- A plot that combines a g/C profile calculated using the split monitor log with the volume profile created from the detector log can be used to highlight the possibility of signal failure (see Figure 5.17 and Figure 5.18).
- The split monitor log clearly reveals the prevalence and importance of early return to green and green extension for the coordinated phases and further highlights the resulting cycle by cycle variability of coordinated phase green time.
- The non-coordinated phases used green distributions from the split monitor log can be used to evaluate the suitability of phase length and minimum and maximum green settings for these actuated phases.
- The project team developed the Dynamic Bandwidth Analysis Tool (DBAT) for monitoring cycle-by-cycle experienced bandwidth using the OASIS split monitor log.
- DBAT results were verified using three classic and well-understood two-way progression schemes: alternate progression, double alternate progression, and simultaneous progression.

10.5.2 Recommendations

The project team recommends that NCDOT’s ongoing interaction with its signal system software vendor include the requests for and cooperation in developing the following additions and improvements –

- Improvements in system data archival and access.

- Data availability at a finer temporal resolution than the current one-minute minimum.
- Computational rigor in terms of accuracy and precision of derived system measures including detector speed, volume, and occupancy (for example using customary rounding at the chosen precision rather than rounding down.)
- Ubiquitous use of GPS clocks at non-networked controllers to minimize the effect of local intersection clock drift.

In addition, the project team also makes the following detector configuration and data collection equipment recommendations –

- Using lane-based versus link-based detection in all cases to improve count accuracy and detail.
- If data from other devices (e.g. RTMS or Bluetooth) is to be synced with OASIS log data, it is important to check the clock time difference between the OASIS master clock and data collection devices before commencing data collection.

10.6 Task 6 - Estimate Model Parameters, Test Model Accuracy, and Investigate Adaptive Implementations

10.6.1 Findings and Conclusions

In this task, one week of OASIS split monitor log data for three sites were processed using the Developed Dynamic Bandwidth Analysis Tool (DBAT). Key findings and conclusions are –

- The methodology accurately determines cycle-by-cycle dynamic bandwidth from the detailed signal indication data recorded in the OASIS split monitor log.
- Traditionally calculated programmed green bandwidth provides limited insight for evaluating the quality of arterial coordination because it does not consider the impact of early return to green and green extension for the coordinated phases.

- Dynamic bandwidth totals and bandwidth efficiencies are considerably larger than the corresponding programmed bandwidth and bandwidth efficiency for the plans analyzed.
- Dynamic bandwidths can include both secondary (addition transient bands within a cycle) and non-programmed bands (bandwidth in the direction for which the programmed coordinated green provides no bandwidth).
- Dynamic bandwidths distributions are complex and characteristically multi-modal.
- The developed methodology represents an important first step in formalizing and enabling dynamic bandwidth using cycle-by-cycle phase duration data from OASIS Split monitor log.

10.6.2 Recommendations

The recommendation for task 6 is that NCDOT should use developed DBAT program to monitor dynamic bandwidth. Signal engineers already know that field bandwidths are different from programmed bandwidth, but it is difficult to monitor cycle-by-cycle. The developed DBAT software tool reads the OASIS split monitor log and provides summary dynamic bandwidth statistics as well as the ability to record the cycle-by-cycle dynamic bandwidths including secondary bands and non-programmed bands. In addition to providing a powerful tool for analyzing current dynamic bandwidth, DBAT will also be useful for before and after signal retiming studies.

10.7 Task 7 - Perform Rigorous Comparative Assessment of Model Performance versus Conventional Plan Evaluation Methods

10.7.1 Findings and Conclusions

In this task, the dynamic bandwidth optimization method is introduced. The presented exhaustive search method can provide all feasible, including globally optimum, solutions. From this information, optimum offsets that consider cycle-by-cycle coordinate green time variability can be selected. In addition, the field evaluation results provide strong evidence of the effectiveness and robustness of proposed optimization method. The field evaluation was conducted on the US 70 Garner site. Key findings and conclusions were –

- The identified global optimal solutions from DBAT provided very similar offset values compared to the current field offsets for five of the six time of day plans analyzed, but the solutions indicated that much improved bandwidth results could be achieved by modifying the offsets for one of the time of day plans (see Table 7.2).
- The identified global solutions were implemented in the field to provide a controlled before and after arterial travel time study.
- The before and after field study result shows a 10.5% travel time decrease for non-peak direction without any significant changing on the other direction for 1:30 PM plan.
- The optimal offset combinations were implemented without field fine-tuning and achieved improved efficiency and reduced travel time.
- The exhaustive search optimization method is practical only for coordinated systems with a small number of intersections due to the exponential growth of the search space as the number of intersections increases.

- It is possible to mitigate excessive calculation time by increasing the search interval or decreasing search space.

10.7.2 Recommendations

The project team recommends that NCDOT use the DBAT tool for signal offset optimization when closed loop systems include a small number of intersections. The DBAT tool takes archived OASIS split monitor log data and provides all feasible solution sets as well as identifying the global solution or solutions.

10.8 Task 8 - Develop Recommendations for Implementation of Best Performing Model

10.8.1 Findings and Conclusions

This task introduces a linear programming (LP) formulation to enable dynamic bandwidth maximization on semi-actuated arterial streets with a larger number of signalized intersection than is feasible with the DBAT exhaustive search tool. As with the DBAT tool, the methodology relies on the use of archived signal log data regarding phase start and end times in each cycle in both directions. This data is used to optimize the offsets that maximize the variable system bandwidth across multiple cycles constituting a coordination plan period. The formulation also is flexible and can be used to optimize signal offsets using programmed (fixed) green durations at each intersection. The proposed formulation offers four significant enhancements compared to traditional methods. Key findings and conclusions were –

- The formulation is strictly linear (complexity of class P) as opposed to the traditional mixed integer programming formulations (complexity of class NP-hard).
- It can work with either static or dynamic (cycle varying) green durations.

- Traditional bandwidth optimization methods have explicit constraints to enforce bandwidth allocation to be proportional to directional demand. However, as longs as the minimum required bandwidth is allocated to each direction, a proportional allocation of bandwidth is not necessary and in fact, can result in reporting sub-optimal solutions. The proposed formulation overcomes this shortcoming.
- The proposed formulation predicts the maximum proportion of traffic demand that can be served in the bandwidth, a unique attribute absent from other formulations found in the literature.

10.8.2 Recommendations

The project team recommends that the NCDOT COST Section implement and ongoing closed-loop signal system monitoring system that include the following elements –

Elements based on the OASIS Detector Log

1. Creating and analyzing the coordinated movements flow plot (see Figure 5.12)
2. Analyzing the coordinated movements flow plot for assessing time of day plan suitability (see Figure 5.12)

Elements based on the OASIS Split Monitor Log

3. Monitoring cycle-by-cycle coordinated movements capacity (see Figure 5.13 to Figure 5.16)
4. Monitoring early return to green and green extension for coordinated phases (Figure 5.19 to Figure 5.22)
5. Monitoring non-coordinated phase displayed green distribution (Figure 5.23 to Figure 5.25)

6. Monitoring Dynamic bandwidth using Dynamic bandwidth analysis tool

Element based on the OASIS Detector Log and the Split Monitor Log

7. Monitoring flow to capacity (see Figure 5.17 and Figure 5.18)

In addition, the project team proposes that NCDOT use Bluetooth units for monitoring arterial travel time. Bluetooth units for travel time data collection are significantly more cost-effective than floating car data collection in terms of the number of data points produced.

The last recommendation is that NCDOT use the developed dynamic bandwidth optimization linear program (LP) be used for dynamic bandwidth maximization. The proposed LP was compared with the DBAT exhaustive search tool, and the test results confirmed that the LP provided the correct optimum solution. Therefore, when the closed loop system has a large number of signalized intersections, provides a feasible method for offset optimization with respect to dynamic bandwidth.

10.9 Task 9 - Develop Recommendations for Implementation of Other Uses of Model Outputs

10.9.1 Findings and Conclusions

In this task, INRIX arterial TMC and Bluetooth travel time data were compared and high resolution detector data were collected and analyzed. Key findings and conclusions were –

- The comparison result shows that INRIX TMC reported travel time is approximately 31.3% less than Bluetooth travel time for NC 55 Apex (TMX 125+06184).
- Western Blvd (TMC 125-147 67 and TMC 125+14768) comparison result shows INRIX reported travel time is 37.8% less than Bluetooth travel.

- Peak period INRIX arterial travel times are much lower than peak period Bluetooth travel times (see Figure 9.2, Figure 9.4 and Figure 9.5).

High resolution detector data were collected by 24 Sensys wireless sensors. The high resolution data can directly provide the following information which is not currently readily available to signal timing engineers –

- Entering intersection vehicle arriving type
- Average platoon headways
- Platoon size
- Number of vehicles arriving on green (or percent arriving on green)
- Number of vehicles inside of dynamic bands (or percentage of vehicles inside of green bands)

Additional, the high resolution detector data when combined with a plot of dynamic bandwidth could be very helpful to the signal offset fine-tuning process (see Figure 9.8 and Figure 9.9).

10.9.2 Recommendations

The project team does not recommend that NCDOT use INRIX arterial TMC data for arterial travel time monitoring based on the significant differences between INRIX and Bluetooth travel times. However, this recommendation should be periodically re-evaluated as INRIX continues to develop its arterial travel time product.

In case of high resolution data collection, the project team strongly recommends that the COST Section move toward collecting high resolution detector data. As mention earlier, high resolution data directly provides much useful information that is not currently available to signal timing engineers, and when high resolution data is combined with dynamic bandwidth information, the resulting diagram could prove very helpful to the signal offset fine-tuning process (see Figure 9.8 and Figure 9.9). This recommended high resolution data collection capability can be met through what should be a minor OASIS software modification coupled with additional field data storage either through increased controller memory or an external storage device.

CHAPTER 11. IMPLEMENTATION AND TECHNOLOGY TRANSFER PLAN

11.1 Introduction and Deliverables

The project team developed a series of implementable recommendations for ongoing closed-loop system monitoring that can be accomplished using the current detector and signal data archiving features of the OASIS system software. These recommendations summarized in Chapter 10, section 10.8.2. These graphical and statistical analyses included in these recommendations are described and illustrated in Chapter 5 and all of the recommended procedures can be conducted through straightforward application of Excel using the OASIS detector and split monitor logs with the exception of dynamic bandwidth analysis.

The recommended dynamic bandwidth analysis is enable by the project's key deliverable – A validated dynamic bandwidth analysis tool (DBAT) that reads OASIS split monitor log data and determines all cycle-by-cycle green bands, produces summary statistics, and if desired will determine the offsets that maximize dynamic bandwidth for the observed cycle-by-cycle signal indications (Tasks 5, 6, and 7 covered in Chapter 5 through Chapter 7).

In developing the DBAT tool, the project team extended the tool's capabilities beyond merely assessing historical cycle-by-cycle dynamic bandwidth by adding the capability to perform an exhaustive search of possible offset combinations to determine the set of offsets that would have maximized dynamic bandwidth for the archived cycle-by-cycle phase indications. This exhaustive search routine can be feasibly applied only to systems with no more than four or five intersections depending on the cycle length and offset search interval. This is because if all possible offset combinations in increments of one second are evaluated at each intersection the number of offset combinations is equation to the cycle length in seconds raised to the power of the number of intersections minus 1 (see Chapter 7, section 7.1).

Therefore, the project team also developed and validated a linear programming (LP) implementation of dynamic bandwidth optimization. The LP version, which overcomes the exhaustive search computational inefficiency for larger arterial systems, was prototyped in Excel. However, although the team recommends that NCDOT ultimately apply the LP formulation to larger arterial systems, creation of a user-friendly, ready to implement LP tool was beyond the scope of this project.

Finally, the project team recommends that NCDOT work toward the capability of collecting and analyzing high resolution (actuation level) detector data. The time stamped vehicle actuation data is valuable on its own and has significant potential for supporting signal coordination fine-tuning when analyzed jointly with dynamic bandwidth.

11.2 OASIS Log-Based Analysis and Monitoring

As mentioned above, the recommended graphical and statistical analyses can be conducted by importing and analyzing the OASIS Detector and Split Monitor log data in Excel. The discussion and examples in this report should be sufficient to guide COST Section personnel in performing the analyses. Nonetheless, project team members will provide additional assistance as needed and can provide the Excel workbooks created for the research.

11.3 Dynamic Bandwidth Analysis Tool (DBAT)

The DBAT tool was created as a standalone program with a simple user interface. The program is simple yet powerful and is easy to use. Chapter 5 and Chapter 6 of this report should provide sufficient instruction on preparing the OASIS split monitor log data and executing the program for dynamic bandwidth analysis. Chapter 7 provides details on how to use the DBAT exhaustive search method for optimizing offsets to maximize dynamic bandwidth. The project team will provide further assistance to COST Section personnel as needed for implementing the DBAT tool.

11.4 Dynamic Bandwidth Linear Program

The DBAT tool's exhaustive search method for determining optimal offsets works quite well for systems with only a few intersections. However, as mentioned above, the search space grows exponentially as the number of intersections increases. Therefore, the project team developed a linear program formulation that can provide optimal offsets using efficient solution algorithms. The LP is currently implemented in Excel and is not as user friendly as the DBAT tool. Development of a standalone, user friendly version of the LP was beyond the scope of this project. However, the project team will share the Excel implementations with COST Section personnel and assist with the assessment of whether NCDOT should pursue an implementable version of the bandwidth optimization LP.

11.5 High Resolution Detector Data Collection and Analysis

Chapter 9 provides description and examples of using high resolution vehicle detector data alone and in conjunction with dynamic bandwidth information to enhance the closed-loop signal system performance assessment process. In order to fully implement this recommendation, NCDOT will need to work with the system software vendor to request a software enhancement that would allow temporary and/or ongoing archival of the detector actuation time stamps. Additionally, implementation of this recommendation would require providing additional memory storage at each controller for which the detailed data is to be collected. This could be achieved by a memory upgrade to the controller or through an external storage device.

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