Report No. UT-06.11

EVALUATION OF ADVANCE WARNING SIGNAL INSTALLATION

Phase I Final Report

Prepared For:

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June 2006

UDOT RESEARCH & DEVELOPMENT REPORT ABSTRACT

1. Report No. UT-06.11	**2. Government Accession No.**	**3. Recipients Catalog No.**
4. Title and Subtitle Evaluation of Advance Warning Signal Installation Phase I Final Report	**5. Report Date** June 2006 **6. Performing Organization Code:**	
7. Author(s) Grant G. Schultz, Ph.D., P.E., PTOE Ryan Peterson, EIT Bradley C. Giles Dennis L. Eggett, Ph.D.	**8. Performing Organization Report No.**	
9. Performing Organization Name and Address: Brigham Young University Department of Civil & Environmental Eng. Provo, UT 84602	**10. Work Unit No.** **11. Contract No.**	
12. Sponsoring Agency Name and Address: Utah Department of Transportation Research and Development Division 4501 South 2700 West Salt Lake City, UT 84114-8410	**13. Type of Report and Period Covered** **14. Sponsoring Agency Code**	
15. Supplementary Notes:		

16. Abstract

Advance warning signals installed upstream of a high-speed signalized intersection (HSSI) warn motorists of impending signal changes in an effort to reduce the frequency of red-light running (RLR) and crashes. A new advance warning signal design was tested on an approach to a HSSI in Salt Lake County to study the effects of the design on motorist behavior. The new design utilized an overhead dynamic blank-out sign and flashers. A state-of-the-art Digital Wave Radar™ evaluation system was installed at the study site to collect continuous data of vehicle speeds and RLR events by a non-intrusive method. Crash data were collected from the jurisdiction responsible for the study site. Data were collected prior to, immediately after, and eight months after installation of the system.

Initial results to the installation showed that motorists were adhering to the advance warning as evidenced by the speed data and a reduction in RLR events. Eight months after installation, however, vehicle speeds and RLR had returned to their original conditions. As a result, recommendations were made to begin to improve performance of the system, while additional monitoring was recommended to evaluate the limited recommendations made.

17. Key Words Advance warning signal, intersection safety, red-light running, speed, dilemma zone, decision zone		**18. Distribution Statement** Available from: Utah Department of Transportation Research and Development Division Box 148410 Salt Lake City, Utah 84114-8410 Brigham Young University Department of Civil & Environmental Eng. 368CB Provo, Utah 84602	
19. Security Classification (Of this report) None	**20. Security Classification (Of this page)** None	**21. No. of Pages** 246	**22. Price**

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ACKNOWLEDGEMENTS

This research was made possible with funding from the Utah Department of Transportation (UDOT) and Brigham Young University (BYU).

Special thanks to the following individuals who served as members of the technical advisory committee (TAC). Additional thanks to Guy Buckner, Troy Noall, and the signal technicians at UDOT who helped the researchers to complete this study. Special thanks also to Mike Miller, Logan Harris, and others at Wavetronix LLC who aided in setting up the data collection equipment and aiding in the overall data collection.

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Executive Summary

Motorists approaching signalized intersections are routinely required to make split-second decisions when traffic signals turn yellow. Motorists must decide whether they have sufficient time or distance to either safely stop or to safely proceed through the intersection before the conflicting traffic is granted the right-of-way. Motorists without a safe option or who do not make a safe decision to stop or proceed through the intersection increase the risk of red-light running and/or rear-end and right-angle crashes, both of which are becoming more and more common across the United States.

In order to reduce the number of red-light running violations and crashes at signalized intersections, the intersections and signals need to be designed and timed in such a way that they make the decision making process easier for motorists, allow motorists a safe and legal maneuver, and reduce the number of conflicts and decisions a motorist must make. Intersections and signals that are not designed and timed appropriately induce conflict zones in which motorists are unable to make safe and/or legal maneuvers, and may become confused as to what they should do. Conflicts increase the risk of red-light running and crashes. For all of these reasons, there is a need to determine ways to increase the safety and efficiency at high-speed signalized intersections (HSSIs) to increase safety and ultimately, to save lives.

The purpose of this report is to present the findings of a study conducted to evaluate the effectiveness of a new technology designed to increase safety at HSSIs in Salt Lake County. The study was part of a research project funded by the Utah Department of Transportation (UDOT) and conducted by researchers at Brigham Young University (BYU) in conjunction with participation of professionals from the private industry and UDOT.

Project Background

Several years ago, maintenance crews from UDOT began to express concerns about the high concentration of skidding that was occurring at signalized intersections on S.R. 154 (Bangerter Highway). The maintenance crews were concerned that repeated exposure to high deceleration rates and subsequent skidding would prematurely damage the pavement on the approaches. At the same time, UDOT traffic and safety officials were concerned with the potential for red-light running and related rear-end and right-angle crashes as a result of these conditions.

In response to the concerns of both maintenance and safety engineers, UDOT hired a consultant to evaluate and design an effective new system to mitigate the problems identified by their engineers. The consultant recommended a new system designed in accordance with research findings published by McCoy and Pesti at the University of Nebraska-Lincoln for the Nebraska Department of Roads (NDOR) that included both advance detection (AD) and advance warning signal (AWS) components.

Although the system was based on the NDOR research, the new AD/AWS design installed by UDOT was unique. The AD/AWS design incorporated a new AD/AWS layout and signal timing plan along with blank-out advance warning signs that are linked to the signal controller in order to provide dynamic information to motorists on the condition of the approaching signal and to provide advance warning of impending signal changes. The blank-out signs are mounted over the through lanes instead of on the side of the road as is the case with more traditional AWS designs. The new AD/AWS design has been termed the blank-out overhead dynamic advance warning signal (BODAWS) system.

The BODAWS system is comprised of three distinct components: 1) an AD component, 2) an AWS component, and 3) a signal timing component. The purpose of the BODAWS system was to increase motorist awareness of impending signal changes and to provide decision zone protection. It was hypothesized that motorist awareness could be increased by mounting the BODAWS signs and flashers over the roadway. The BODAWS signs were dynamic and blanked-out when not in use, providing only real-

time pertinent information to motorists so that they would be less inclined to lose respect for the warnings over time.

UDOT determined to install the new BODAWS systems at four locations in Salt Lake County, including three on Bangerter Highway and one on S.R. 201 (2100 South). The Bangerter Highway locations included the intersections of Bangerter Highway with S.R. 68 (Redwood Road), 2700 West, and 13400 South. The installation of Bangerter Highway and 13400 South was evaluated for this research.

Project Objectives

UDOT contracted with researchers at BYU to conduct an analysis of the BODAWS system to determine if the new design was effective at reducing both dilemma zone and decision zone conflicts. Measures of effectiveness specified for the research included: 1) adverse risk parameters such as the frequency of red-light running events; 2) the frequency and severity of right-angle and rear-end crashes; and 3) the speed distributions of approaching vehicles. The size and location of the decision zone, or greatest variation in motorist behavior, was also measured and analyzed. The outcomes of the study, as well as guidelines and recommendations for system improvements, are presented in this executive summary.

System Design and Configuration

The BODAWS system is comprised of: 1) the BODAWS signs and flashers, 2) the BODAWS detector, and 3) the BODAWS signal timing plan. The BODAWS sign and flashers consisted of a blank-out sign with two flashing beacons mounted over the travel lanes on the approach to the intersection. The location of the BODAWS signs and flashers corresponded to the recommendations of an NDOR study conducted by McCoy and Pesti (*4*). The NDOR design for an 85th percentile speed of 65 mph recommends the utilization of a design speed 10 mph less than the 85th percentile speed (i.e., 55 mph) with the signs and flashers located 445 feet in advance of the intersection. The NDOR design

was established using a rigorous cost-benefit analysis that compared crash and delay costs of a “new” configuration with a “conventional” design based on the then current stopping-sight distance definition of the dilemma zone. This design was recommended by the consultant for use in the Utah installations.

The detection component of the BODAWS system was installed to reduce the percentage of vehicles in the decision zone at the onset of the yellow change interval. The design of the BODAWS detectors was again based on recommendations of the NDOR design. From the research performed for NDOR, the AD component consisted of a single optical detection zone created using a video camera mounted on a standard lamp post installed upstream of the study site intersection. The BODAWS detector was located 755 feet from the stop bar. The lamp post and detector camera were located 705 feet from the stop bar.

The BODAWS signal timing plan consisted of four distinct events occurring in sequence. The first event occurred at the beginning of the extendable portion of the green interval when the stop bar detectors are not active and the BODAWS detectors are active. When the BODAWS detectors are active, the signal controller is programmed to identify an appropriate gap in traffic in order to end the major street through phase when as few vehicles are in the decision zone as possible. The maximum unit extension time recommended for the study was 3 seconds, which is just long enough to allow a motorist traveling at or greater than the design speed of 55 mph to travel from the BODAWS detector to a point 70 feet from the BODAWS signs and flashers. If a motorist was within 70 feet of the BODAWS sign and flashers it was anticipated that they would not see the variable message or notice the beacons flashing. The second event of the BODAWS signal timing plan began when the signs and flashers were activated due to the passage time being met. When the BODAWS signs were active the message “PREPARE TO STOP” appeared on the blank-out sign and flashing beacons on each side of the sign began to flash in alternating half-second bursts. According to the NDOR study, a lead flash time of 6.5 seconds was recommended, where the lead flash time is the time between the activation of the BODAWS sign and the onset of the yellow change interval. Although the NDOR study recommended a 6.5 second lead flash, the consultant for UDOT recommended a 6.0 second lead flash for the installation. The third and fourth

events of the BODAWS system signal timing plan included the 6 second yellow change interval and the 2 second all-red clearance interval. The BODAWS signs and flashers remained active throughout the red interval.

Evaluation Metrics

After the initial design had been completed and the installation locations, determined, UDOT retained researchers at BYU to evaluate the effectiveness of the BODAWS system. Possible positive impacts of BODAWS installations that were determined worthy of evaluation at these locations included: 1) crash rate reductions, 2) improved motorist reaction time, and 3) a reduction in red-light running violations. Potential negative impacts included increased speeds on intersection approaches following the activation of the BODAWS signs and flashers, as well as the potential for increased collisions as vehicles increase their speed to beat the light.

The metrics proposed for evaluation by the research team included an evaluation of safety impacts of the BODAWS design installation at the study site through crash and red-light running differentials, as well as an evaluation of the impact on speed trends immediately following activation of the BODAWS signs and flashers, and before and after the onset of the yellow change interval. In addition, a decision zone study was also conducted for the analysis.

Data were gathered before, immediately after, and eight months after activation of the BODAWS system installation. The data collection equipment at the study site incorporated non-intrusive Digital Wave Radar™ sensor technology developed by Wavetronix LLC, of Lindon, Utah. The radar sensors utilized for the data collection detected vehicle passage and speed to determine speed trends and trigger the capture of red-light running data.

The BODAWS system installation was activated on June 8, 2005. Data were collected before BODAWS installation from April 27, 2005 to May 2, 2005, and from May 25, 2005 to June 8, 2005. Data were collected immediately after BODAWS installation from June 8, 2005 to June 23, 2005, and from July 8, 2005 to July 22, 2005.

Data were collected approximately eight months after installation from February 8, 2006 to February 15, 2006, and from March 17, 2006 to March 24, 2006.

In order to effectively reference each data collection period with respect to the installation of the BODAWS system, the period before BODAWS installation will be referred to as period 1 (P1), the period immediately after BODAWS installation will be referred to as period 2 (P2), and the period eight months after BODAWS installation will be referred to as period 3 (P3).

Research Results

Speed, red-light running, and crash data were collected during P1, P2, and P3 while decision zone boundary data were collected during P1 and P2. The speed, red-light running, crash, and decision zone study data were statistically analyzed to determine if the BODAWS system was effective in reducing red-light running violations and crashes and to determine if the speeds of vehicles and the boundaries of the decision zone were affected by the BODAWS system. A summary of the speed, red-light running, and decision zone results are provided in the following sections. The crash analysis results were somewhat inconclusive and are not discussed in this executive summary, but are included in the full report.

Speed Data Results

Speed data were collected by radar sensors in seven different zones on each approach to the study site. Cumulative distribution plots, box plots, and probability graphs were created for the speed data. Cumulative distribution plots were created to visually compare the speed trends of vehicles approaching the intersection. A visual comparison of the speed distributions allowed trends to be identified that illustrated motorist driving behavioral patterns before and after installation of the BODAWS system.

The cumulative speed data plots from P1, P2, and P3 indicated that the 85th percentile speed of approaching vehicles varied from study period to study period and by the number of seconds before red (SBR). For example, during period P1 motorists tended to travel at lower and more uniform speeds regardless of the number of SBR. During P2 motorists traveled at higher speeds before activation of the BODAWS system (16 to 12 SBR), while traveling at lower speeds after the onset of the yellow change interval (6 to 0 SBR). During P3 motorists increased their speeds throughout all of the detection zones during each SBR indicating that although the data tend to suggest that the BODAWS system was effective in increasing motorist awareness and caution for a limited time after installation (P2), it appeared that motorists may have lost respect for the advance warning as the study went on (P3). It is even possible that motorists may have started to take advantage of the system as they became more familiar with it.

Red-Light Running Data Results

Red-light running violations were recorded by a stop bar radar sensor detection zone on each approach to the study site. In order to determine if a red-light running event occurred, timestamps from the speed data at the stop bar sensor zone of each approach were compared to the timestamps for the onset of the red signal in each direction. Due to the high number of right turning vehicles on the southbound approach that were recorded by the radar sensors, a filter was applied to the data using a 4 second time-after-red limit and a 20 mph minimum speed. UDOT employees also conducted on-site red-light running observations in an attempt to verify the sensor data results. Red-light running violation results were calculated by the number of red-light running violations per 1,000 entering vehicles.

The red-light running trends indicated that fewer motorists committed red-light running violations during P2 than during P1. A reduction in red-light running may be attributed to the fact that motorists were exercising more caution immediately after BODAWS installation as they became accustomed to the system. The visibility of the BODAWS system may have also contributed to increased caution and respect for the yellow change interval immediately after installation. Multiple hours of on-site

observations during P2 tended to support this theory. However, the number of red-light running violations increased during P3. The speed data collected during this time period also supported the increase in red-light running violations as the average speeds of vehicles recorded 50 feet from the intersection at 0 SBR were high enough that motorists would not have been able to brake in time to avoid running the red-light.

The red-light running data tends to confirm the possibility of motorist abuse of the advance warning due to the increase in red-light running events after BODAWS installation.

Decision Zone Data Results

A decision zone study was conducted to determine if installation of the BODAWS system affected the size and location of the decision zone boundaries. The decision zone study occurred during P1 and P2 and was conducted on-site and by observation of the braking habits of vehicles approach the intersection. The decision zone boundaries (i.e., 90 percent and 10 percent probability of stopping) results were provided in terms of both distance from the intersection and the number of seconds of travel time from the intersection. The probability of stopping distances was calculated using a logit model of statistically significant parameters. Independent relationships between study parameters were also evaluated using a Chi-Square test. The parameter estimates were obtained using maximum likelihood estimates in SAS Proc Logistic. The results of the analysis indicated that although there was a statistically significant difference in the decision zone boundaries between time period P1 and P2, the difference was not practically significant. The results of the decision zone study indicated that the decision zone occurred between approximately 218 feet (2.5 seconds) and 554 feet (6.3 seconds) depending on speed and vehicle type.

The decision zone data results tended to indicate that the BODAWS system did not affect the locations that motorists deemed as safe to proceed from or stop at after the onset of the yellow change interval.

Conclusions and Recommendations

Based on the data collected and the analysis performed, speed and red-light running data tend to suggest that motorists decreased their speeds by 5 to 10 mph during the yellow change interval immediately after BODAWS installation while they were adapting to the new system. At the same time, speeds during the green interval increased to levels closer to the posted speed limit while the BODAWS signs and flashers were inactive suggesting that motorists understood they would receive warning before the onset of the yellow change interval and felt comfortable traveling through the intersection closer to the speed limit. Eight months after BODAWS installation, however, motorists increased their speeds on average by 5 to 10 mph greater than before installation. The greatest speed increase eight months after installation occurred during the yellow change interval, suggesting that motorists may have become accustomed to the sign and knew that they could use the advance warning of impending signal changes to try and clear the intersection before the signal turned red.

The red-light running data tended to support the speed data assumptions. The red-light running data showed that motorists were not running the red-light immediately after BODAWS installation as much as they were before installation suggesting that the BODAWS system contributed to a decrease of as much as 4 violations per 1,000 entering vehicles as motorists were adapting to the new system. However, eight months after installation the number of red-light running violations increased to frequencies in some instances higher than before BODAWS installation by as many as 3 violations per 1,000 entering vehicles. The red-light running data verified the increased speeds recorded during the yellow change interval indicating that motorists may have been trying to beat the red light.

The decision zone study found that the size of and location of the decision zone changed slightly after installation of the BODAWS system. The location of the 10 percent probability stopping distance for a passenger car moved from 237 feet before BODAWS installation to approximately 259 feet after BODAWS installation while the 90 percent probability stopping location for a passenger car moved from 440 feet to 481 feet during the same time period. Although the change in the size and location of the

decision zone boundaries was determined to be statistically significant, it did not appear to be practically significant. The decision zone study results suggest that motorists feel comfortable proceeding through an intersection or stopping after the onset of the yellow change interval based on distance and travel time from the intersection regardless of the advance warning or information they are provided by the BODAWS system.

The speed, red-light running, and decision zone data results suggest that motorists might have been more cautious immediately after BODAWS installation but by eight months after installation they may have adapted to the system and used the advance warning to try and proceed through the intersection when it would have been safer for them to stop.

The BODAWS system has proven that it affects motorist behavior and changes driving patterns. However, not all of the changes in motorist behavior were positive. It is possible that the warning that motorists received came too far in advance of the onset of the yellow change interval allowing motorists to abuse the system and persuading motorists that the warning did not have to be obeyed to safely proceed through the intersection. Therefore, UDOT and BYU should continue to monitor driver behavior patterns at the study site and collect data for at least another 10 months which would provide them with 18 months of data after BODAWS installation. In the meantime, UDOT should consider reducing the BODAWS lead flash time as a function of the distance to the BODAWS sign, the minimum sign recognition distance, and the probability of stopping time. On-site observations would indicate that the current lead flash timing provides too much advance warning, thus leading motorists to abuse the system and to possibly lose respect for the information provided.

1 Introduction

The purpose of this report is to present the findings of a study conducted to evaluate the effectiveness of a new technology designed to increase safety at a high-speed signalized intersection (HSSI) in Salt Lake County. The study was part of a research project funded by the Utah Department of Transportation (UDOT) and conducted by researchers at Brigham Young University (BYU) that began in July 2004. The study also required the participation of professionals from the private industry. The findings of the study will be presented to UDOT. This chapter is divided into four sections including a problem statement section, a project background section, a project objectives section, and a report organization section.

1.1 Problem Statement

Motorists approaching signalized intersections are routinely required to make split-second decisions when traffic signals turn yellow. Motorists must decide whether they have sufficient time or distance to either safely stop or to safely proceed through the intersection before the conflicting traffic is granted the right-of-way. Motorists without a safe option or who do not make a safe decision to stop or proceed through the intersection increase the risk of red-light running (RLR) and/or rear-end and right-angle crashes.

RLR violations and crashes are a major problem in the United States. A joint study conducted by the Institute of Transportation Engineers (ITE) and the Federal Highway Administration (FHWA) estimates that in 2001 there were approximately 218,000 RLR related crashes that resulted in approximately 181,000 injuries, 880 fatalities, and nearly $14 billion dollars in damages (*1*). National Highway Traffic Safety

Administration (NHTSA) statistics cited by Singh estimate that rear-end crashes accounted for approximately 30 percent of all crashes, injuries, and property damage in the year 2000 (*2*). Reducing the frequency of RLR violations and crashes that occur at signalized intersections should be a leading safety priority for transportation agencies across the nation.

In order to reduce the number of RLR violations and crashes at signalized intersections, the intersections and signals need to be designed and timed in such a way that they make the decision making process easier for motorists, allow motorists a safe and legal maneuver, and reduce the number of conflicts and decisions a motorist must make. The following subsections discuss the conflicts that motorists face on the approach to signalized intersections and the mitigation measures available to reduce those conflicts.

1.1.1 Conflicts at Signalized Intersections

Intersections and signals that are not designed and timed appropriately induce conflict zones in which motorists are unable to make safe and/or legal maneuvers, and may become confused as to what they should do. Conflicts increase the risk of RLR and crashes. Two types of conflict zones are common on approaches to signalized intersections including (*3*):

- The dilemma zone conflict, and
- The decision zone conflict.

Although the Literature Review in Chapter 2 contains a more detailed description of the decision and dilemma zone conflicts, they will be briefly discussed in the following paragraphs to introduce the reader to the purpose of the study.

If traffic signals are improperly timed, motorists may become trapped in a dilemma zone (DMZ) when the traffic signal turns yellow. The DMZ occurs when motorists do not have sufficient distance to safely stop or sufficient time to clear the stop bar or intersection before the traffic signal turns red (*3*). Motorists caught in the DMZ

have to choose between accelerating in an effort to clear the intersection and risk causing a crash with conflicting traffic or rapidly decelerating and risk causing a rear-end collision. Even when DMZs are mitigated or eliminated, motorists may still become confused and make an erratic decision which leads to decision zone (DCZ) conflicts.

The area on the approach to an intersection where the greatest variation in motorist behavior is manifest when traffic signals turn yellow is called the DCZ (*3*). Although motorist reaction varies during onset of the yellow signal, a wide variation in behavioral patterns might signify that the assumptions used to design the intersection and program the signal timing are not being met. Greater variations might also signify that motorists are responding to unexpected maneuvers by other motorists as well. The goal of traffic engineers is to properly time and design intersections to account for DCZ and DMZ conflicts. Traffic engineers can mitigate DCZ and DMZ conflicts using modern technology and specially designed traffic control devices.

1.1.2 Mitigation Measures for Signalized Intersection Conflicts

The most effective way to mitigate the hazards of DMZs is to properly time traffic signals to account for the behavioral characteristics of motorists using the intersection. DCZs can be mitigated by installing advance detection (AD) systems that detect vehicles as they approach the intersection through the use of one or more advance detectors. The purpose of an AD system is to monitor traffic and choose an appropriate gap in traffic to end the green phase when as few vehicles as possible are in the DCZ. Another method of reducing the size and location of DCZs involves providing more information to motorists in the form of an advance warning of impending signal changes and/or the current condition of the traffic signal. Advance warning can be provided through the use of an advance warning signal (AWS) (*4*).

Transportation agencies often combine the use of AD and AWS technologies (AD/AWS) in order to provide advance warning to motorists while also attempting to reduce the number of motorists that will need to make a decision (*4*).

1.2 Project Background

Several years ago, maintenance crews from UDOT expressed concerns about the high concentration of skidding that was occurring at signalized intersections on S.R. 154 (Bangerter Highway). The maintenance crews were concerned that repeated exposure to high deceleration rates and subsequent skidding would prematurely damage the pavement on the approaches. At the same time, UDOT traffic and safety officials were concerned with the potential for RLR and related rear-end and right-angle crashes as a result of these conditions.

In response to the concerns of both maintenance and safety engineers, UDOT hired a consultant to evaluate and design an effective new system to mitigate the problems identified by their engineers. The consultant recommended a new system designed in accordance with research findings published by McCoy and Pesti at the University of Nebraska-Lincoln that included both AD and AWS components (*4*).

The new AD/AWS design installed by UDOT is unique. The AD/AWS design incorporates a new AD/AWS layout and signal timing plan and blank-out advance warning signs that are linked to the signal controller in order to provide dynamic information to motorists on the condition of the approaching signal and to provide advance warning of impending signal changes. The blank-out signs are mounted over the through lanes instead of on the side of the road as is the case with more traditional AWS designs. The new AD/AWS design will hereafter be referred to as the blank-out overhead dynamic advance warning signal (BODAWS) system.

1.3 Project Objectives

UDOT contracted with researchers at BYU to conduct an analysis of the BODAWS system to determine if the new design was effective at reducing DMZ and DCZ conflicts. Measures of effectiveness specified for the research included: 1) adverse risk parameters such as the frequency of RLR events; 2) the frequency and severity of right-angle and rear-end crashes; and 3) the speed distributions of approaching vehicles. The size and location of the DCZ, or greatest variation in motorist behavior, was also

measured and analyzed. The outcomes of the study, as well as guidelines and recommendations for system improvements, are presented in this report.

1.4 Report Organization

This report is organized into the following seven chapters: 1) Introduction; 2) Literature Review; 3) Background; 4) Implementation; 5) Results; 6) Discussion of Results; and 7) Conclusions and Recommendations. A reference section and multiple appendices also accompany this report.

Chapter 2 is a literature review outlining traffic signal timing concepts, traffic engineering design principles, and traffic engineering technologies that are used in the industry to increase safety at high-speed signalized intersections. The literature review includes such topics as: 1) traffic signal timing; 2) yellow change interval conflicts; 3) AWS technologies and configurations; 4) AD technologies and configurations; 5) positive and negative consequences of AWS and AD installations; 6) AWS and AD installation guidelines; and 7) the methods employed to locate the DCZ. The literature review provides a technical background to the research project and serves as a reference source to compare the research methods, results, recommendations, and conclusions of this report with other professional studies.

Chapter 3 provides background information regarding: 1) the need for BODAWS on Bangerter Highway; 2) the 13400 South study site; 3) the BODAWS system design and configuration; and 4) BODAWS evaluation metrics. The chapter also details the design parameters recommended by the private consultant who designed the BODAWS system based on the results of a research project conducted by the Nebraska Department of Roads.

Chapter 4 details the steps that were involved in implementing the research project including: 1) the data collection equipment technology and configuration; 2) the crash data analysis process; and 3) the study methods employed to locate the boundaries and determine the size of the DCZ. The chapter also includes explanations of the BODAWS evaluation metric data gathering procedures and the statistical analysis techniques that were used to analyze the data.

Chapter 5 presents the statistical and empirical results of the research project in quantitative and qualitative forms using graphs, tables, and figures. The chapter includes: 1) speed data results; 2) RLR data results; 3) crash data results; and 4) DCZ study results.

Chapter 6 theorizes the meaning and practical significance of the data results presented in Chapter 5. The chapter discussions analyze: 1) speed trends; 2) RLR trends; 3) crash data comparisons; and 4) DCZ study results. The discussions relate to the impacts of the BODAWS system on driving behaviors and to whether or not the BODAWS system increases or decreases safety at the study site as currently designed.

Chapter 7 provides conclusions summarizing the findings of the report as well as recommendations for design changes that might increase the effectiveness of the BODAWS system at reducing RLR and crashes. The chapter also recommends future research possibilities and areas of interest that need to be explored further.

2 Literature Review

The literature review contains a brief introduction to, and discussion of, academic and professional literature relating to safety at high-speed signalized intersections during the transition between the green and red intervals. The literature review provides information relating to intersection safety and design, details of previous studies that have been conducted to define the conflicts that motorists face on the approach to signalized intersections, and techniques employed by transportation agencies across North America to reduce or eliminate those conflicts. The literature review also contains information relating to other studies and methods that have been conducted to evaluate conflict mitigation technologies and systems similar to the BODAWS system installed by UDOT.

The literature review is divided into the following sections:

2.1 Traffic Signal Timing – introduces the terminology and describes the events that occur during a normal traffic signal cycle at a signalized intersection.

2.2 Yellow Change Interval Conflicts – describes the conflicts and dangers that motorists face during the transition between green and red intervals.

2.3 AD Technologies and Configurations – describes the purposes, configurations, and functions of AD systems.

2.4 AWS Technologies and Configurations – describes the AWS technologies and configurations available to reduce conflicts and increase safety at signalized intersections.

2.5 Positive and Negative Consequences of AWS and AD Installations – describes the positive and negative consequences of installing AWS and AD technology.

2.6 AWS and AD Installation Guidelines – describes the guidelines that other transportation agencies have developed to aid them in deciding where to install AWS and AD systems.

2.7 DCZ Study Methods – describes the techniques used by other professionals to determine the size and location of the DCZ.

2.8 Literature Review Chapter Summary – provides a summary of the key points outlined in the chapter.

2.1 Traffic Signal Timing

The basic terminology and design of traffic signal timing will be discussed in this section including an introduction to: 1) the signal timing cycle and right-of-way; 2) the green, yellow, and red signals; 3) fixed time and variable time signal designs; 4) the events that happen during the transition between the green and red intervals; 5) the equations used to calculate and design the transition between the green and the red intervals; and 6) the assumptions used to time traffic signals.

2.1.1 The Traffic Signal Cycle

Signalized intersections are timed according to one complete cycle. A cycle consists of distinct phases, all of which occur in sequence. Each phase grants the right-of-way to one or more non-conflicting movements through the signalized intersection. For example, a signal timing plan at a signalized intersection might include phases to allow the through movements of vehicles heading in opposite but parallel directions on each approach and another phase granting left turning vehicles the right-of-way while conflicting movements are required to wait (*5*, *6*, *7*).

Each phase of a traffic signal cycle consists of three distinct intervals. The intervals include the green interval, the yellow change interval, and the red interval. During the green interval certain vehicles are granted the right-of-way. During the yellow change interval vehicles that are assigned the right-of-way are warned that the

right-of-way is about to be terminated and given to another phase. During the red interval the right-of-way has been granted to another phase (*5*, *6*, *7*).

The duration of each cycle at a signalized intersection can be fixed or can vary based on vehicle approach demands and volumes. When signal cycles are fixed they are said to be pre-timed and each cycle length is equal to the previous cycle length. When cycle lengths vary, based on demand and approach volumes, the signalized intersection is said to be actuated (*5*).

2.1.2 Actuated Traffic Signals

Actuated traffic signal controllers obtain information from detectors placed in or near the roadway on one or more of the approaches to the intersection. The detectors detect vehicles in the traffic stream and vary the amount of time assigned to each phase based on the demand of the vehicles whose movements are assigned to the phase. Actuated controllers can be programmed to allow more green time to be assigned to the approaches with higher demand volumes (*5*). Detectors can also be used at actuated signalized intersections to detect vehicles on the approaches to the intersection that do not currently have the right-of-way. Vehicles waiting for the right-of-way at pre-timed intersections may have to wait even when there are no vehicles using the green time on the approach where the right-of-way is assigned. When a detection of a waiting vehicle occurs at an actuated intersection a “call” is placed at the traffic controller. The green time of the approach with the right-of-way can then be adjusted to end when an appropriate gap in the through traffic is found. Detector design will be discussed in more detail in Section 2.3.

2.1.3 The Green Interval

The green interval begins when one or more of the approaches receive the right-of-way. The total green time at pre-timed signalized intersections is fixed. The total

green time for actuated signals varies based on demand and consists of a minimum green time, an extendable green time, and a maximum green time.

The minimum green time is required to allow vehicles queued at the intersection to clear before the controller is allowed to end the interval due to calls on other approaches. Once the minimum green time has been met, the extendable portion of the green begins. The extendable portion of the green is the green time that is allocated to facilitate a safe transfer of the right-of-way from one approach to another after a call has been received. The extendable portion of the green will be discussed in more detail in Section 2.3. The maximum green time is the maximum amount of green time that is allotted to a phase. When the maximum green time is met, the signal will begin the process of transitioning the right-of-way from one movement to another (*5*).

Most safety problems occur during the transition from the green interval to the red interval (*1*). The transition from the green interval to the red interval is called the yellow change interval because the traffic signal indication is amber or "yellow" color (*6*, *7*, *8*).

2.1.4 The Yellow Change Interval

The yellow change interval exists to transition motorists from the green interval to the red interval during normal traffic signal operation. The ITE *Traffic Engineering Handbook* and the *Manual on Uniform Traffic Control Devices* (MUTCD) Section 4D.10 define the yellow change interval as the "first interval following every circular green or green arrow indication" and state that the purpose of the yellow change interval is "to warn approaching traffic of the imminent change in the assignment of right-of-way" (*6*, *8*). Traffic codes and laws regulate the appropriate maneuvers that motorists are allowed to make after the onset of the yellow change interval. However, regulations used to define the yellow change interval vary from jurisdiction to jurisdiction and are not consistent (*9*).

Once motorists have been warned of the imminent change in the assignment of right-of-way they must decide to proceed through the intersection or stop at the intersection. Sufficient time is required for a motorist to proceed through the intersection and sufficient distance is required for a motorist to stop. Some motorists will be close

enough to the intersection that they can safely proceed through the intersection, given the amount of time allotted to them during the yellow change interval, before the conflicting traffic receives the right-of-way. Other motorists will not have enough yellow time to proceed through the intersection and will need to stop to avoid a collision with conflicting traffic.

Guidelines are provided to properly time the yellow change interval. ITE recommends that the yellow change interval be timed using Equation 2-1 (*6*, *10*).

$$YCI = t_r + \frac{v}{2a + 2Gg} \tag{2-1}$$

where: YCI = yellow change interval (sec),
t_r = perception-reaction time (sec),
v = approach speed (ft/sec),
a = deceleration rate (ft/sec^2),
G = acceleration of gravity (32.2 ft/ sec^2), and
g = percent grade (positive for upgrade, negative for downgrade).

Equation 2-1 contains two main components. The first component accounts for the time that motorists need to perceive a change in the signal indication, decide to react, and begin to respond. The variable t_r represents the allotted amount of time to perceive and react to the signal change and is called the perception-reaction time. The second component of Equation 2-1 accounts for the time that motorists need to decelerate to a stop if they decide to do so. According to Equation 2-1, if the design speed of an intersection is 60 mph, the intersection approach is level (no grade), the assumed perception-reaction time of approaching motorists is 1 second, and the design deceleration rate is 11.2 ft/sec^2, the yellow change interval would need to be 5 seconds long. The MUTCD recommends a yellow change interval of 3 to 6 seconds with the longer intervals reserved for signalized intersections with high approach speeds (*7*).

Equation 2-1 is recommended in jurisdictions where motorists are not allowed to be in the intersection on red. Some jurisdictions allow motorists to enter the intersection

on yellow and be in the intersection on red (a permissive yellow), which could occur if a vehicle barely passes the stop bar before the signal turns red (*11*). If a permissive yellow is designed, then an all-red clearance interval needs to be added. The MUTCD states, "the yellow change interval may be followed by a red clearance interval, of sufficient duration to permit traffic to clear the intersection before conflicting traffic movements. . . are released" (*7*). During the all-red clearance interval the traffic signals of all of the approaches are simultaneously red. The all-red clearance interval timing can be added onto the yellow change interval equation as illustrated in Equation 2-2, and is called the change period (*6*, *7*).

$$CP = t_r + \frac{v}{2a + 2Gg} + \frac{W + L}{v} \tag{2-2}$$

where: CP = change period (sec),
t_r = perception-reaction time (sec),
v = approach speed (ft/sec),
a = deceleration rate (ft/sec^2),
G = acceleration of gravity (32.2 ft/ sec^2),
g = percent grade (positive for upgrade, negative for downgrade),
W = width of intersection, curb to curb (ft), and
L = length of vehicle (typically 20 ft).

Once a motorist perceives the change in the signal, he/she must choose between three responses which include: 1) proceeding through the intersection while maintaining a constant speed, 2) accelerating to proceed through the intersection, or 3) decelerating to a stop at the intersection. The time that a motorists needs to come to a complete stop accounts for the speed of the vehicle, the approach grade, the deceleration rate of the vehicle, and the effects of gravity.

The maximum distance a vehicle can travel during the yellow change interval, at the design speed, and make it safely to the stop bar before the signal turns red is called the yellow change interval protection zone. The yellow change interval protection zone

is calculated by multiplying the duration of the yellow change interval by the design speed of the approach. Although Equation 2-1 accounts for the time necessary for a vehicle to clear the intersection, it does not directly provide the distance that a vehicle might need to safely stop. The following subsection explains the equation used to calculate the safe stopping distance of vehicles and is directly related to the yellow change interval equation.

2.1.5 Stopping Sight Distance

Motorists who decide to bring their vehicles to a stop during the yellow change interval must have sufficient distance to do so. A conservative physical estimate of the maximum distance a vehicle can be from the intersection and come to a safe stop at the stop bar is based on a widely used American Association of State Highway and Transportation Officials (AASHTO) equation called the stopping-sight distance equation (*12*). The stopping-sight distance equation recommended by AASHTO (*12*) is defined in Equation 2-3.

$$SSD = 1.47Vt + 1.075\frac{V^2}{a} \tag{2-3}$$

where: SSD = stopping sight distance (ft),
V = design speed (mph),
t = brake reaction time (2.5 sec), and
a = deceleration rate (11.2 ft/sec^2).

The stopping-sight distance accounts for the distance a vehicle travels while the motorist recognizes the need to stop, applies the brakes, and stops. The time a motorist takes to perceive a need to brake and begins to apply the brakes is called the brake reaction time (t) and is similar to the perception-reaction time used to calculate the yellow change interval. The distance a vehicle will travel as it is stopping is based on pavement conditions, slope, and vehicle deceleration rates. According to Equation 2-3, a vehicle

traveling at 60 mph, with a motorist who has a brake reaction time of 1 second and decelerates at 11.2 ft/sec^2, would need approximately 434 feet to safely stop. Equations 2-1, 2-2, and 2-3 contain variables that are based on assumptions of the behavioral characteristics and capabilities of the motorists who will be using the intersection. It should be noted that Equation 2-1 and Equation 2-3 are interchangeable if the grade is ignored. The only significant difference between the two equations is the unit conversion and the value used for the reaction time. In Equation 2-1 a value of 1.0 seconds is generally assumed as this is representative of the reaction time in a signalized system where drivers are expecting a signal and anticipating the need to react. Equation 2-3, however, represents a more unexpected condition; hence a value of 2.5 seconds is generally assumed for the reaction time. The following subsection describes the basic assumptions used to time the yellow change interval and account for the stopping-sight distance.

2.1.6 Standard Design Assumptions Used to Calculate the Yellow Change Interval and Stopping-Sight Distance

The perception-reaction time, brake reaction time, deceleration rate, and length of vehicle variables used in Equations 2-1, 2-2, and 2-3 are based on assumptions about the driving behaviors and capabilities of motorists as well as the physical capabilities and characteristics of their vehicles. For example, the standard vehicle length (*L)* specified by ITE for use as a standard estimate in Equation 2-2 is 20 feet (*6*). In most cases 20 feet would be a conservative estimate of the length of the majority of passenger vehicles that pass through the intersection.

It is important for engineers using the yellow change interval and stopping-sight distance equations to understand the basic design assumptions used to create a safe environment for motorists approaching a signalized intersection. The following subsections describe some of the most common assumptions, their origins, and the importance of verifying that the assumptions chosen for the yellow change interval and stopping-sight distance equations fit the scenario that they are being design for.

2.1.6.1 Perception-Reaction Time Assumptions A study of brake reaction times conducted by Johannson and Rumar (*13*) and cited in the latest version of the AASHTO design manual () found that brake reaction times vary based on whether or not the event that causes a motorist to brake is expected or unexpected.

Johannson and Rumar found that events that are unexpected lead to perception-reaction times as high as 2.7 seconds (*13*). AASHTO recommends using a 2.5 second perception-reaction time for unexpected events, based on the Johannson and Rumar study, because it "exceeds the 90th percentile of reaction time for all drivers" (, *13*).

Because motorists approaching signalized intersections expect the yellow change interval to begin at any time, the perception-reaction time chosen for signal timing calculations falls within the expected event category. The standard professional assumption for perception-reaction time used to calculate the yellow change interval is 1 second. The 1 second perception-reaction time used in the design of signalized intersection is based on the 1940 edition of the AASHTO design manual (*14*) as noted by Fambro et al. in the National Cooperative Highway Research Program (NCHRP) Synthesis Report 400 (*15*).

The Johannson and Rumar study confirms the 1940 AASHTO recommendations. The Johannson and Rumar study found that the 85th percentile perception-reaction time for motorists who respond to a routine signal change is 1 second (*13*). ITE has adopted the expected event (i.e., signal change) perception-reaction time of 1 second for signal timing calculations consistent with the NCHRP recommendations (*16*).

2.1.6.2 Deceleration Rate Assumptions Assumptions used for the deceleration rate are as varied as the assumptions for perception-reaction time and brake reaction times. As recently as 1998, the ITE *Traffic Engineering Handbook* and the ITE *Manual of Traffic Signal Design* handbook recommended using a deceleration rate of approximately 10 ft/sec^2 (, *7*). Research conducted by Fambro et al., found that 90 percent of motorists will decelerate at 11.2 ft/sec^2 or faster. As a result, 11.2 ft/sec^2 has become the design deceleration rate design standard recommended in the most recent AASHTO design manuals. However, motorists will generally decelerate at 14.8 ft/sec^2 or faster if confronted by unexpected objects (*15*).

The assumptions used to calculate perception-reaction time, brake reaction times, deceleration rates, and vehicle lengths vary based on the circumstances. Engineers must understand the assumptions and know how to appropriately account for them in their designs. A properly timed yellow change interval provides sufficient yellow time for a motorist who is beyond their safe stopping-sight distance to make it to the stop bar before the signal turns red. The following section describes the conflicts that arise on approaches to signalized intersections if the design assumptions used to calculate the yellow change interval and stopping-sight distance are incorrect, while the remainder of the literature review will focus on the events that occur during the yellow change interval and the red interval as well as the technologies that exist to increase safety during the yellow change interval.

2.2 Yellow Change Interval Conflicts

There are a number of conflicts that may occur on approaches to signalized intersections. Some of these conflicts may occur as a result of engineering or signal design, some may occur as a result of motorist decision (or indecision), some may occur as a result of driving conditions, and others may occur for a variety of other reasons. The two general types of conflicts that occur on approaches to signalized intersections that are evaluated in this research include: 1) conflicts that occur due to poor signal design and 2) conflicts that occur due to motorist indecision. Both types of conflicts are manifest on approaches to signalized intersections in distinct areas with measurable boundaries or zones. These two conflict zones are called (*3*):

- The “dilemma” zone (DMZ), and
- The “decision” zone (DCZ).

A DMZ is created at the onset of the yellow change interval when a motorist does not have sufficient distance to safely stop or sufficient time to safely proceed through the intersection (*3*, *17*, *18*, *19*). The DCZ is so named because it is the zone where motorists exhibit the greatest variation in their behavior (*3*, *20*). Some confusion exists in the

literature relating to the naming conventions used to describe these two conflicts. Some researchers use the term DMZ to describe the events that would be best described as occurring in the DCZ and do not recognize the conflicts that occur due to poor signal timing (*4*, *6*, *19*, *21*, *22*). Other researchers use both the terms DMZ and DCZ to describe the DCZ (*23*, *24*, *25*). It is important to distinguish between the two concepts so that proper mitigating technologies can be designed into signalized intersections for each of the conflicts. A distinction between the DMZ and the DCZ is detailed in the following subsections.

2.2.1 The DMZ

The DMZ is so named because motorists face a dilemma caused by no fault of their own. Poorly timed signals trap motorists in the DMZ. Poorly timed signals are a result of improper design assumptions used to calculate the yellow change interval signal timing and the safe stopping-sight distance. Poorly timed signals may result from oversight, inexperience, or failure of traffic engineers to understand the driving patterns of motorists approaching the intersection they are designing. The DMZ illustrated in Figure 2-1 illustrates a vehicle that does not have enough stopping-sight distance and that is outside the protection provided by the yellow change interval. At established signalized intersections; changes in the land use patterns, driving conditions, and demographics may require adjustments to the yellow change interval.

Motorists caught in the DMZ must choose between decelerating at an unsafe rate and risk causing a rear-end collision or speeding up and risk running a red-light and causing a crash with conflicting traffic (*21*). Signals must be timed appropriately to allow motorists who obey the traffic laws and pay attention to the approaching signal with sufficient time to make a safe and legal maneuver. However, even when signals are timed according to correct physical and mathematical principles, conflicts may still arise.

Milazzo et al. state that “dilemma zones can be eliminated for drivers who meet all assumptions, but even for these drivers, they must still choose correctly (stop or go). Slight misjudgments, incorrect decisions, or insufficient reaction time or deceleration rates can lead to small, often inadvertent, RLR violations” (*26*). The following

subsection describes the conflicts that arise due to variations in motorist behavioral characteristics.

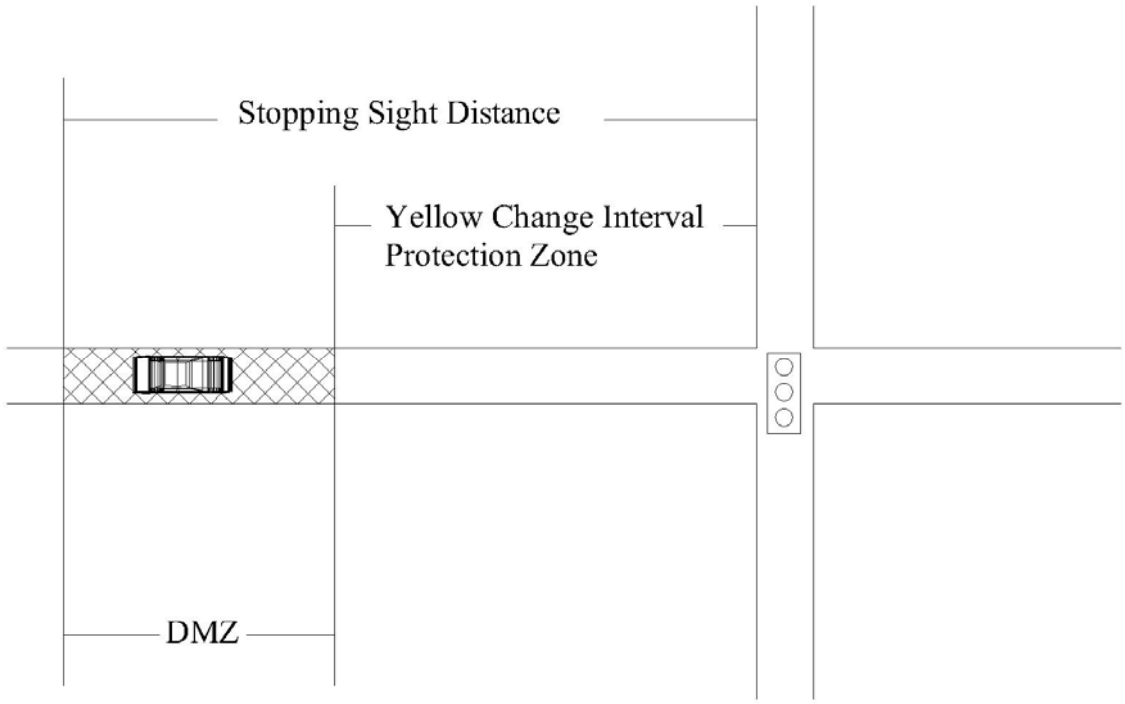

Figure 2-1 Dilemma zone on the approach to a signalized intersection (adapted from).

2.2.2 The DCZ

The DCZ is the area on the approach to an intersection where motorists must decide to proceed through the intersection or stop at the onset of the yellow signal (). As discussed previously, each motorist perceives and reacts differently to unexpected changes to the signal indication. The ability to judge relative speeds and distances also varies from motorist to motorist and may become more difficult on the approach to an HSSI than on approaches to intersections with lower approach speeds. An HSSI is considered any intersection with approach speeds over 35 mph (*6*).

Milazzo et al. state that "the minimum stopping distance depends on the assumptions of deceleration rate and reaction time . . . but not all drivers can or will

achieve these standard assumptions every time. In other words, the minimum stopping distance, which is the basis of the yellow time calculation, is different for each driver" (*26*). Milazzo et al. further explain that "it turns out that it only takes a slight increase in reaction time or a slight decrease in deceleration rate from the 'standard' assumptions for a driver to be susceptible to a 'dilemma zone' situation in which the driver does not have a legal maneuver" (*26*).

Unlike the DMZ, the DCZ exists at every intersection and cannot be mitigated with proper signal timing alone (). Smith et al. explain that the DCZ is the "zone of proximity to a yellow signal which may include the dilemma zone, and within which the driver faces uncertainty as to whether to stop or to proceed, and therefore confronts alternate decision-making choices. Variable driver behavior under these circumstances may disrupt the smoothness of traffic flow and . . . may also pose a traffic safety hazard" (*3*).

Smith et al. further explain that "there is a decision zone for every intersection that subjects interact with. Unlike dilemma zones, whose boundaries rest upon calculations of vehicle travel and stopping distance at different speed limits, determination of decision zones is based upon empirical observations of driver stopping behavior. A decision zone exists for all signalized intersections (unlike the case for a dilemma zone)" (*3*).

The "zone of proximity" phrase used by Smith et al. can be defined in terms of space or time. The zone of proximity relating to space is usually defined as the distance between the point of the approach at which 90 percent of motorists will stop at the onset of yellow and the point of the approach at which 10 percent of motorists will stop at the onset of yellow as illustrated in Figure 2-2 (*3*, *19*, *25*, *27*, *28*).

The DCZ for a specific intersection can be calculated based on observations of traffic at a signalized intersection during the yellow change interval. Data collected during the observation would include measurements of the locations of vehicles that stop or proceed through the intersection to determine characteristic stopping probabilities (*19*, *27*). Some researchers use stopping probability data to define the boundaries of the DCZ using cumulative frequency curves or logit models to characterize motorists tendencies (*3*, *28*, *29*, *30*).

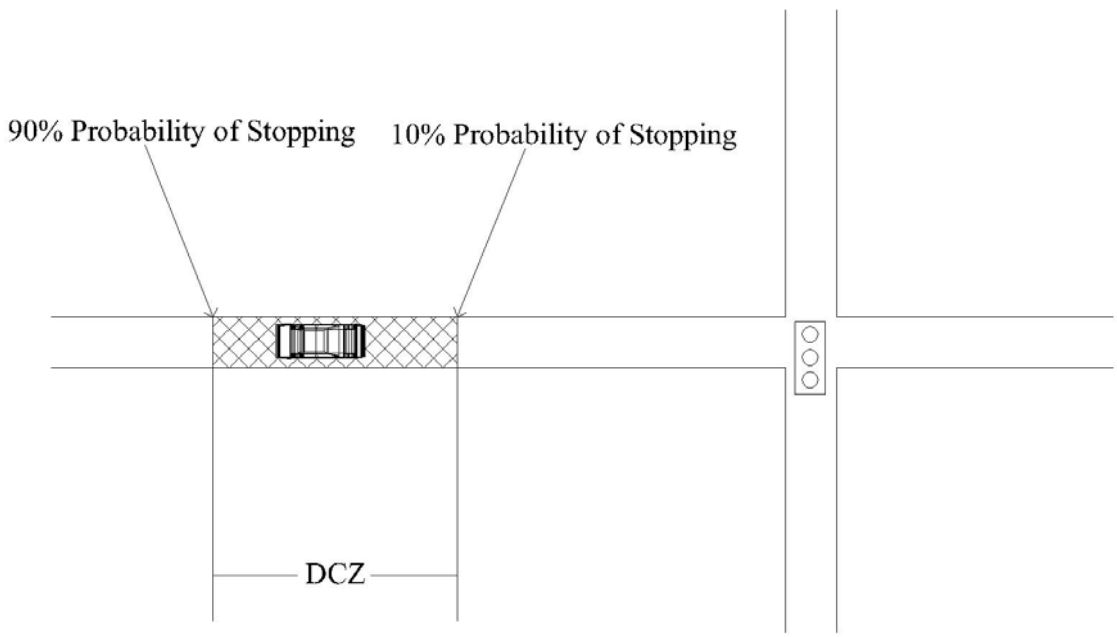

Figure 2-2 DCZ on an approach to a signalized intersection (adapted from *3*).

The zone of proximity of the DCZ relating to time is usually defined by the number of seconds of travel time that an approaching vehicle is from the stop bar, which usually ranges between 2 and 5 seconds (*3*, *25*, *27*, *31*). A Minnesota Department of Transportation (MnDOT) driver simulation study found that a majority of erratic driving behavior occurred when vehicles were between 2 seconds (10 percent probability of stopping) and 4.5 seconds (90 percent probability of stopping) from the intersection when the signal turned yellow with the most dangerous behavior occurring when motorists were between 2 and 3.5 seconds from the intersection (*3*).

If motorists are unable to meet the standard design assumptions used to create solutions to DMZ conflicts, such as the values used to calculate perception-reaction time and deceleration/acceleration rates, then motorists will continue to face dangerous conflicts on approaches to HSSIs. Furthermore, motorists traveling toward an HSSI may not be experienced at making space-time decisions at high speed causing them to be more confused and less decisive in choosing to proceed through the intersection or stop at the onset of the yellow change interval and will be faced with a DCZ conflict.

Motorists who face a DCZ or DMZ conflict may run a red-light and be involved in a crash causing property damage, serious injury, or even death. To mitigate the DMZ, signals need to be timed using acceptable mathematical and physics concepts as well as correct assumptions regarding motorist behavior. The DCZ, however, will always exist even when signals are timed properly. The negative effects of the DCZ can be mitigated by utilizing safety systems and technologies. The following sections describe the technologies, techniques, and applications used to mitigate the DCZ.

2.3 AD Technologies and Configurations

AD systems are installed at HSSIs to allow traffic signals to adapt to varying traffic flow conditions and to mitigate DCZ conflicts by reducing or eliminating the number of vehicle that might be caught in the DCZ at the onset of the yellow change interval (*7*, *21*). For the purposes of this report, only the use of AD technology for DCZ conflict mitigation will be discussed. Various technologies are used to create DCZ protection at signalized intersections. This section is separated into two subsections. The first subsection deals with detector types and functions. The second subsection deals with design principles such as detector placement and signal timing options.

2.3.1 Advance Detector Types and Functions

AD detectors are designed for many purposes. The type of detector chosen for a signalized intersection depends on the purpose and needs at the intersection. Passage or point detectors detect discrete events such as whether or not vehicles are passing a certain point on the approach to the intersection and how much time occurred between each event. Therefore, detection is said to have occurred at a point. Presence or area detectors are designed to monitor events occurring within a zone or specified area. Presence or area detectors also monitor events over periods of time and not just single events (*5*).

Five main types of detectors described in the following paragraphs are passage/point detectors or presence/area detectors that are commonly in use today. The five main types of detectors include (*6*, *7*):

- Inductive loop detectors,
- Magnetic detectors,
- Radar detectors,
- Pressure pad detectors, and
- Video imaging detectors.

2.3.1.1 Inductive Loop Detectors Inductive loop detectors, the most common type of detector, consist of wires placed into the pavement that complete an inductive circuit. An amplifier in the traffic signal control cabinet senses changes in the current of the loop when a vehicle passes over the loop. Inductive loops are used for passage/point and presence/area detection (*6*, *7*).

2.3.1.2 Magnetic Detectors Magnetic detectors are similar to inductive loop detectors except that instead of sensing changes in electrical current they sense changes in the earth's magnetic field. Magnetic detectors are used for passage/point detection but are unable to detect vehicles traveling less than 5 mph (,).

2.3.1.3 Radar Detectors Frequency modulated continuous wave radar sensors actively measure the presence, range, and speed of multiple vehicles using frequency modulated electromagnetic pulses. Continuous wave radar detectors measure only the presence and speed of a single vehicle using the Doppler principle. The radar waves, or pulses, transmitted by a radar sensor will bounce off of both stopped and moving vehicles. Radar sensors can be used for passage, pulse, or presence detection depending on the capabilities and configuration of the particular sensor model. Radar detectors are installed roadside or over the roadway. Radar detection is non-intrusive which means that the physical roadway does not need to be disturbed for installation or maintenance.

2.3.1.4 Pressure Pad Detectors Pressure pad detectors are installed in the roadway pavement like inductive loop and magnetic detectors. Pressure pads are usually rubber and contain two metallic contact closure plates embedded in them. When a vehicle passes over a pressure pad the weight of the vehicle causes the contact closure plates to

meet and an electrical current is generated that sends a signal to the controller. Pressure pad detectors can be used for both passage/point and presence/area detection ().

2.3.1.5 Video Imaging Detectors Video imaging detectors are installed over the roadway. Video imaging detectors operate by optically sensing contrasts of light. When a vehicle passes through the video camera image a change in contrast of light occurs and a signal is sent to the traffic signal control cabinet. Video imaging detectors are used for both passage/point and presence/area detection ().

Once an appropriate detector type has been chosen, the detector design and placement must be determined. The location and design of traffic detectors will be discussed in the following subsection.

2.3.2 AD Detector Design Principles

The type, location, and size of detectors depend on the design of the signalized intersection, the design speed, and the location of the DCZ. Detectors installed to mitigate DCZ conflicts need to be placed upstream of or over the DCZ. DCZ mitigating detectors are most commonly used to report passage/point detection to traffic signal controllers. Together, the traffic signal controller and the detectors provide an AD system as outlined in the following paragraphs.

The purpose of AD is to provide DCZ protection by "prevention of phase termination" while a vehicle is in the DCZ. "This protection can be achieved by strategically locating detectors on the intersection approach and adjusting their detector unit settings such that a vehicle can hold the green while it travels through the [DCZ]" (*32*). When a vehicle passes over an advance detector an actuation is said to have occurred. The amount of time necessary for a vehicle to hold the green is called the unit extension time. The unit extension time is generally selected to allow a vehicle traveling at the design speed sufficient time to travel from one detector to the next, from the detector to the stop bar, or from the detector to the end of the DMZ. The unit extension time (U) is also called the gap time or passage time because it is also the maximum amount of time between successive vehicles that pass over the advance detector before

the onset of the yellow change interval will occur (*5*). Figure 2-3 illustrates the events that occur during the green interval of an actuated controller.

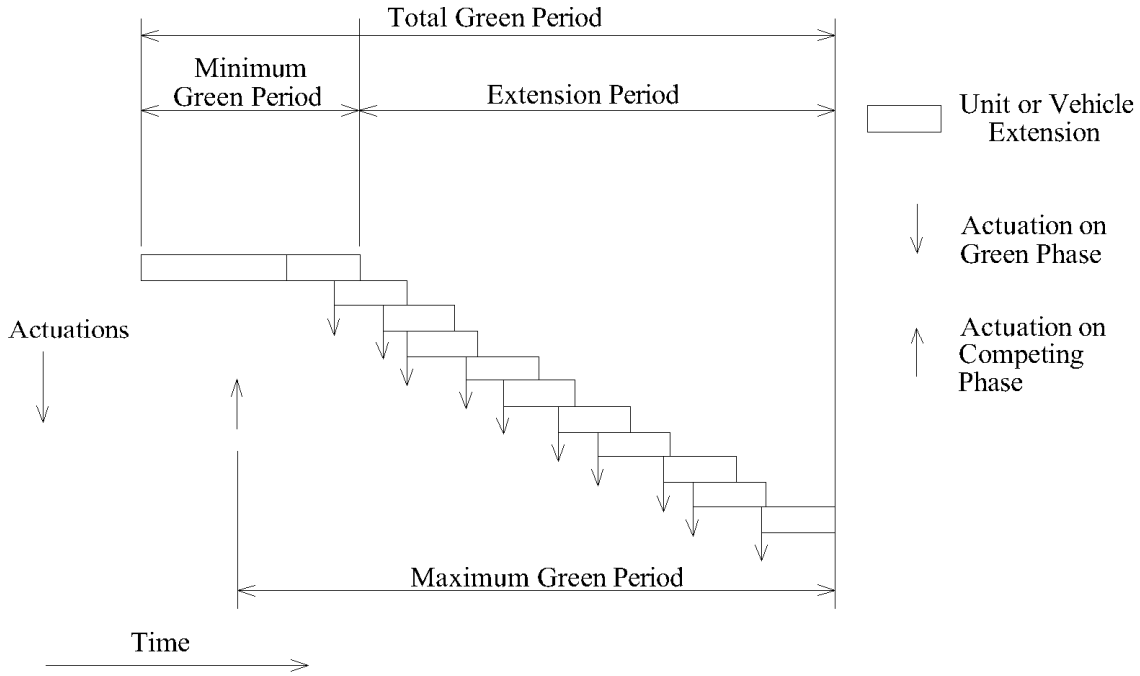

Figure 2-3 Actuated controller green time (*33*).

When the green interval begins the right-of-way is assigned to an approach. The green will continue to be displayed at least until the minimum green time has been met. If a vehicle passes over the AD during the last portion of the minimum green (a portion equal to or less than one unit extension) a unit extension is added to the minimum green. Every additional actuation will also add one unit extension to the green. The unit extension time is only added from the time of the detection and not to the end of the previous unit extension (*5*).

If the passage time passes before an additional actuation occurs on the approach during the extendable portion of the green interval then an appropriate “gap” has been found in traffic and “gap-out” occurs triggering the onset of the yellow change interval.

However, if actuations continue during the extendable portion of the green, the green may continue to be extended until the maximum green time is met.

Once the maximum green time is met the signal must max-out. When max-out occurs DCZ protection is lost because the yellow change interval will begin regardless of how many vehicles are trapped in the DCZ (*4*).

Volume-density controllers are controllers that are able to vary the green time based on traffic demand and can be programmed to reduce the number of max-outs that occur due to continuous actuations of the advance detectors (*5*). Reduction in max-outs can occur by a systematic reduction in the gap or passage time between successive vehicles that makes it more difficult to extend the green as time progresses.

Figure 2-4 illustrates the gap reduction process. At the start of the extendable portion of the green, the maximum allowable gap (U_1), or unit extension time/passage time, is used. Once an actuation has occurred on a competing phase, a preset time-before-reduction (t_1) period is allowed to pass. After t_1 passes a preset time-to-reduce (t_2) begins during which time the gap time needed to extend the green is reduced linearly from U_1 until the minimum allowable gap time (U_2). The green will continue to be extended by a unit extension equal to U_2 until gap-out or max-out occurs. Linearly reducing the gap time required to extend the green makes it more difficult for vehicles to extend the green the longer the green continues. Greater difficulty means that less vehicles will be able to extend the green and less max-outs should occur (*5*).

Advance detectors need to be placed upstream of the DCZ to be effective. The ITE *Traffic Engineering Handbook* contains a table of values defining the boundaries of the DCZ for various speeds (*6*). The ITE table has been reproduced in Table 2-1 correlating approach speeds with the 10 percent and 90 percent DCZ boundary locations.

Based on the ITE manual, if the design speed for a signalized intersection is 55 mph, an advance detector would need to be placed farther upstream from the signalized intersection than the 90 percent boundary of the DCZ listed in Table 2-1 as 386 feet. Based on the placement of the advance detector, the passage time would need to be programmed so that the controller will extend the green long enough for the last vehicle that passed the detector to be clear of the DCZ boundaries before the onset of the yellow change interval.

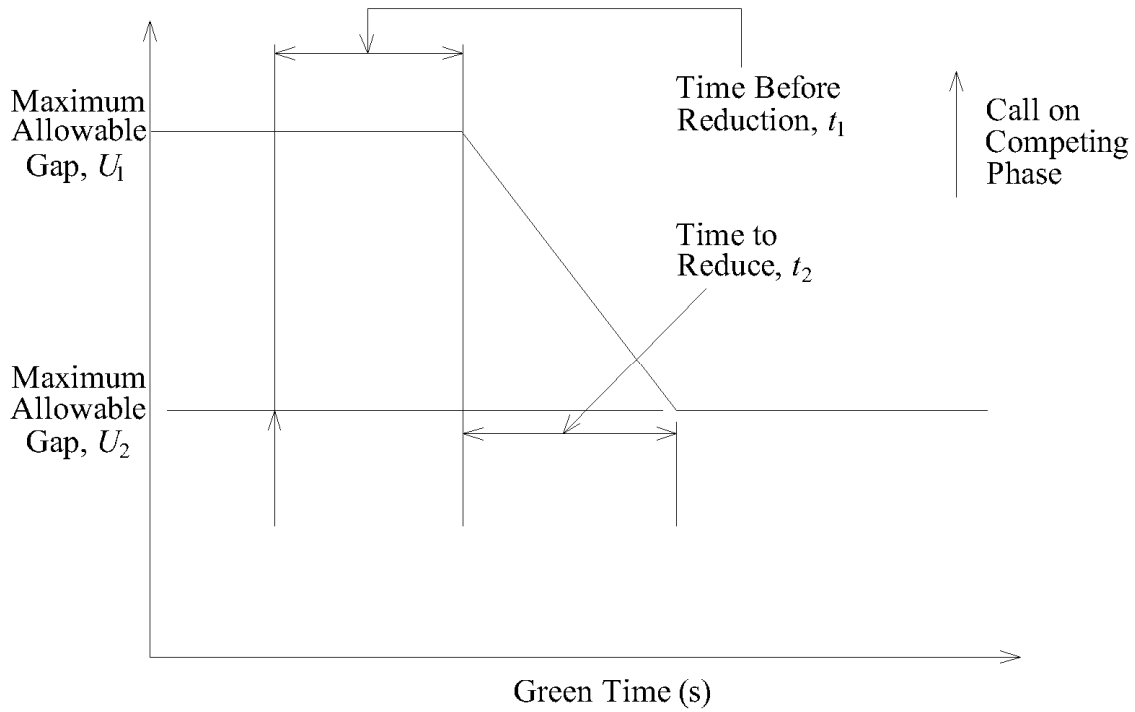

Figure 2-4 Gap reduction process of a volume density controller (*33*).

Table 2-1 Decision Zone Boundaries ()

Approach Speed (mph)	DCZ Boundaries (feet)	
	10%	90%
35	102	254
40	122	284
45	152	327
50	172	353
55	234	386

Sometimes, AWS systems have been installed in conjunction with AD systems in order to provide better DCZ protection to motorists. The following section discusses AWS technologies and configurations as well as combinations of AWS and AD systems.

2.4 AWS Technologies and Configurations

AWS systems also can be utilized to mitigate DCZ conflicts. AWS systems are usually installed on approaches to HSSIs to provide advance warning of impending signal changes in an attempt to reduce driver indecision and behavioral variability (*21*). An AWS system usually consists of a warning sign, with or without flashing beacons, placed upstream of a signalized intersection (*34*).

AWSs can be either static (sometimes called passive) or dynamic (sometimes called active). Static AWSs warn motorists of approaching signalized intersections but do not provide real-time information on the status of the signal and are not interconnected with the signal controller (*34*). Dynamic AWS signs are interconnected with the signal controller so that flashing beacons can be activated at a predetermined time before the onset of the yellow change interval.

The MUTCD standards state that warning signs, like AWS signs, "shall be installed on an approach to a primary traffic control device that is not visible for a sufficient distance to permit the road user to respond to the device" (*8*). The MUTCD also lists the following recommendations for the use of warning signs (*8*):

- All warning signs shall be diamond-shaped (square with one diagonal vertical) with a black legend and border on a yellow background unless specifically designated otherwise.
- Warning signs shall be designed in accordance with the sizes, shapes, colors, and legends contained in the "Standard Highway Signs" book.
- When a BE PREPARED TO STOP sign is used in advance of a traffic control signal, it shall be used in addition to a Signal Ahead sign.
- The BE PREPARED TO STOP sign may be supplemented with a warning beacon.
- When the warning beacon is interconnected with a traffic control signal or queue detection system, the BE PREPARED TO STOP sign should be supplemented with a WHEN FLASHING plaque.

The following subsections describe guidelines for the use and design of warning signs and AWS signs in general including descriptions of AWS sign types, AWS setback distances, and AWS lead flasher timing.

2.4.1 AWS Sign Types

AWS signs vary by shape, color, size, and the text message displayed on the sign. A study conducted by Bowman (*35*) details 10 different sign types that have been used across North America in a total of 18 different configurations. Bowman found that the four most common sign types used in the United States and Canada included (*35*):

- Passive symbolic signal ahead signs,
- Continuously flashing symbolic signal ahead signs,
- Flashing symbolic symbol ahead signs, and
- Prepare to stop when flashing signs.

Passive symbolic signal ahead signs are static signs are used to warn motorists that they are approaching a signalized intersection but do not provide information on the status of the signal. A typical passive symbolic signal ahead sign, defined by the MUTCD as a W3-3 Signal Ahead sign, is illustrated in Figure 2-5. The MUTCD specifies that passive symbolic signal ahead signs "shall be installed on an approach to a primary traffic control device that is not visible for a sufficient distance to permit the road user to respond to the device" (*8*).

Continuously flashing symbolic signal ahead signs are passive symbolic signal ahead signs with flashing beacons mounted next to the sign. The term "continuously flashing" means that the beacons are flashing regardless of the signal status. Continuously flashing symbolic signal ahead signs do not provide motorists with advance warning of impending signal changes. The flashing beacons are only used to draw attention to the sign.

Figure 2-5 Passive symbolic signal ahead sign or MUTCD W3-3 signal ahead sign (*8*).

Flashing symbolic signal ahead signs are like continuously flashing symbolic signal ahead signs except that they are dynamic in nature meaning that the flashers are only activated a preset time before the onset of the yellow change interval in an attempt to warn approaching motorists of the impending signal change. Flasher timing will be discussed later on in this section.

The prepare to stop when flashing sign configuration, similar to the sign illustrated in Figure 2-6, is the most common configuration used in North America (*27*, *35*). Prepare to stop when flashing signs utilize a sign with the text message "PREPARE TO STOP" listed in the MUTCD as a W3-4 sign. Because the W3-4 sign is accompanied by one or two flashing beacons, the MUTCD specifies that the accompanying text "WHEN FLASHING" should supplement the sign (*8*).

A study conducted by Sabra (*20*) measured motorist responses to various AWS signs, text messages, and configurations, in driver simulator situations to better determine which sign type was the most identifiable and understandable. Sabra found that flashing symbolic signal ahead signs were identifiable at the greatest distance by the most motorists, and that prepare to stop when flashing signs were the least understood sign. Sabra also found that dynamic signs were more identifiable than static signs. Another study, by Gibby et al., concluded that AWS signs with flashing beacons are the most effective type of advance warning device (*34*).

Figure 2-6 Prepare to stop when flashing sign.

Once the appropriate sign type has been chosen, the setback distance of the AWS sign from the stop bar should be considered. The following subsection describes the techniques used to design the setback distance.

2.4.2 AWS Setback Distances

The MUTCD specifies that warning signs should be located so that they provide adequate time for motorists to "perceive and complete a reaction to the sign" (*8*). The MUTCD states that the time needed to perceive and complete a reaction warning signs "is the sum of the times necessary for Perception, Identification (understanding), Emotion (decision making), and Volition (execution of decision), and is called the PIEV time" (*8*). The distance a motorist travels during PIEV time equates to the stopping-sight distance equation discussed previously (*21*). Once distance traveled during PIEV is calculated, the MUTCD recommends subtracting a sign legibility or sign recognition distance of 175 feet (*8*). Therefore, according to the MUTCD manual, a signalized intersection with an approach speed of 60 mph would require a warning sign to be placed 400 feet from the stop bar (). The MUTCD adds one further cautionary note stating that "warning signs

should not be placed too far in advance of the (intersection), such that drivers might tend to forget the warning because of other driving distractions, especially in urban areas" (*8*).

Other studies also support the MUTCD recommendations. A Nebraska Department of Roads (NDOR) report recommends a setback distance equal to the stopping-sight distance minus a legibility or sign recognition distance of 125 feet (*4*). MnDOT and the British Columbia Ministry of Transportation and Highways recommend placing AWS signs in accordance with the location of the upstream boundary of the stopping-sight distance for the 85th percentile speed or the posted speed limit of the approach, but do not mention a sign legibility distance requirement (*3*, *22*).

Agent and Pigman completed a literature review that found that most agencies place their AWSs somewhere between 600 and 800 feet from the intersection (*36*). Variation in setback distances among agencies is due to the varying philosophies and design assumptions used by the agencies. Once the setback distance has been determined, the lead flash time can be calculated as discussed in the following subsection.

2.4.3 AWS Lead Flash Timing

The lead flash time of a dynamic AWS system is the amount of time in advance of the yellow change interval that the flashing beacons begin to flash. The lead flash time recommendations for dynamic AWS systems vary widely from agency to agency. Variation in lead flash time is usually a function of the setback distance of the AWS sign. For example, one MnDOT AWS configuration, called the Golden Valley design, had a lead flash time that was calculated by "dividing the distance from the AWS to the stop bar by the approach speed" of the intersection (*27*), while another design, called the Oakdale design, used a lead flash time calculated by dividing "the distance from the AWS to a point in front of the decision zone by the approach speed" (*27*).

McCoy and Pesti recommend that lead flash time should adjusted to account for the time it takes a vehicle to travel from the point where a motorist can perceive the flashing beacons to the stop bar at the design speed of the approach (*4*). McCoy and Pesti suggested a flashing beacon recognition distance of 70 feet (*4*).

Agent and Pigman conducted a literature review that found that most agencies used a lead flash time between 4 and 13 seconds (*36*). Again, the variations in lead flash time were based on the design assumptions of the agency in question.

A MnDOT report concluded that the lead flash time of an AWS, if not timed properly, can delay overall operation of signal systems and that the "punch" of AWSs tends to diminish over time. The MnDOT report also found that the need for an AWS is not perceived as a necessity by motorists at most signal systems (*37*).

2.5 Positive and Negative Consequences of AWS systems

Installation of AWSs leads to both positive and negative consequences in both safety and operations. The effectiveness of AWSs is usually evaluated in before and after studies using the following measures of effectiveness:

- Approach speeds,
- RLR,
- Crash rates, and
- Number of motorists caught in their DCZ and/or DMZ at the onset of the yellow change interval.

Both the positive and negative safety implications of AWS installations are discussed in the following subsections.

2.5.1 Impacts of AWS on Approach Speeds

The literature indicates that various factors influence the impacts of AWSs on approach speeds including: 1) AWS sign type; 2) intersection geometry; and 3) the condition of the traffic signal. The following paragraphs provide the details relating to these conclusions.

Pant and Huang looked at vehicle approach speeds during various signal and AWS conditions including: 1) when the signal was green and the AWS flashers were not

active, 2) when the signal was green and the AWS flashers were active, and 3) when the signal was red and AWS flashers were active (*23*).

Pant and Huang found that when the signal was green and flashers were not active, passenger car and truck speeds on curved approaches slowly decreased as the vehicles got closer to the intersection regardless of the type of AWS that was used. However, Pant and Huang found that on tangent approaches to signalized intersections vehicles speeds tended to increase with proximity to the intersection.

Pant and Huang found that when the signal was green and flashers were active, passenger cars speeds increased on tangent approaches and curved approaches when Prepare to Stop When Flashing signs were used. Therefore, Pant and Huang highly discouraged the use of Prepare to Stop When Flashing signs on tangent approaches to HSSIs because motorists tended to speed up as they approached the intersection before the signal turned yellow (*23*).

Pant and Huang found that when the signal was red and flashers were active, passenger vehicles and trucks decreased their speeds more when Prepare to Stop When Flashing signs were located on curved approaches (*23*).

Klugman et al. found that vehicle approach speeds remained the same during the main street green phase but an increase in speed was observed at AWS equipped locations during the main street yellow and all-red clearance intervals (*27*). Farraher et al. explain this phenomenon by stating that "drivers use the flashers to 'over-drive' the signal timing and 'race' the signal system – thereby becoming a hazard" ().

A MnDOT driver simulator study found that, on average, motorists tended to slow down and brake more often when AWSs were present. A modified NDOR AWS design, that included a Prepare to Stop When Flashing sign and one advance detector, was found to have influenced a greater number of motorists to stop at the onset of yellow change interval (*4*, *38*).

2.5.2 Effectiveness of AWS at Reducing RLR

Results of the effectiveness of AWSs in reducing RLR rates, as reported in the literature, are mixed. The literature indicates that factors relating to RLR violations are

more complex than whether or not an AWS sign is present on the approach. Other factors that influence RLR may include: 1) design speed; 2) AWS sign type; 3) combination of AWS systems with other technologies; and 4) motorist familiarity with the AWS signs. The following paragraphs provide the details relating to these conclusions.

Respondents to a MnDOT survey indicated that AWSs helped motorists to slow down and prepare for upcoming signal changes suggesting that fewer motorists would run the red-light if AWSs were installed (*3*). However, data from other studies illustrated that even though there were decreases in RLR when AWSs were installed the results were usually not statistically significant (*21, 27*).

Pant and Huang made a correlation between AWS sign type and RLR statistics indicating that some sign types are more effective at reducing RLR than other sign types. Pant and Huang found that RLR decreased with Prepare to Stop When Flashing signs and Flashing Symbolic Signal Ahead signs but increased with Continuously Flashing Symbolic Signal Ahead signs (*23*).

A MnDOT driver simulator study found that fewer red signals were run during low speed limit trials, but more were run during high speed limit trials suggesting that speed has a greater affect on RLR than the presence of AWSs (*3*).

McCoy and Pesti found that when comparing intersections equipped with AD and AWS to intersections only equipped with AD, the results were mixed as to which design was better at reducing RLR. In fact, a binomial proportions test indicated that there was no statistical difference between the designs (*4*). The McCoy and Pesti results indicate that AWSs can be used with or without AD technology.

Farraher et al. conduct a study of RLR in Bloomington, Minnesota, using motion imaging sensing equipment. Approximately 1,285 hours of data were collected before AWSs were installed at the study intersection in which 546 cars and 203 trucks ran the red-light with 13 vehicles running the red-light 3.6 seconds or more after the signal turned red. During the “after” period, 1,285 hours of data were also recorded in which 436 cars and 76 trucks ran the red-light with 16 vehicles running the red-light 3.6 seconds or more after the signal turned red. After the AWSs were installed, the intersection experienced a 29 percent reduction in overall RLR and a 63 percent reduction in truck

RLR. Farraher et al. further reported that one year after the AWSs were installed, the number of cars running the red-light and the number of total vehicles running the red-light 3.6 seconds or more after the signal changed had almost returned to rates seen before the AWSs were installed ().

2.5.3 Effectiveness of AWS at Reducing Crashes

A review of literature relating to the effectiveness of AWSs at reducing crashes indicates that even though crash rates are reduced at most sites after AWS installation, the results are usually not statistically significant. The literature also indicates that crash rates are not always tied to the presence of AWS signs alone. The following paragraphs provide the details relating to these conclusions.

The reports of Klugman et al., Sayed et al., and Gibby et al. illustrated a decrease in crash rates when AWS signs were present, but most of the reductions were not statistically significant (*21, 23, 27*). For instance, Sayed et al. developed a crash prediction model that found that there was a 12 percent decrease in total crashes, a 14 percent decrease in severe crashes, and a 2.6 percent decrease in rear-end crashes at sites equipped with AWSs, but the findings were not statistically significant at a 95 percent confidence level (*21*).

Klugman et al. found that right-angle and rear-end crash rates for MnDOT's Golden Valley design were reduced from 0.74 crashes per million entering vehicles to 0.58 crashes per million entering vehicles, and overall crashes were reduced from 1.5 crashes per million entering vehicles to 0.99 crashes per million entering vehicles. However, Klugman et al. caution that the reduction in crash rates at the Golden Valley AWS sites (identified previously) might be attributable to other design modifications that took place at those intersections. Klugman et al. also found that crashes actually increased slightly at AWS sites configured with the Oakdale design identified previously (*27*).

Sayed et al. found a correlation between crash rates and approach volumes at HSSIs (*21*). When minor streets volumes were low, intersections equipped with AWSs performed worse than those without AWSs. Also, when minor street approach volumes

were high, AWS equipped intersections experienced fewer crashes than those intersections without AWSs. AWSs were most effective at reducing crashes at intersections with minor streets having an annual average daily traffic count of 13,000 vehicles or greater (*21*).

Gibby et al., after comparing the intersections in California with the worst and best crash rates, found that HSSIs with AWSs experience significantly lower incidents of left-turn, right-angle, and rear-end crash rates than those without AWSs. However, the results were not statistically significant ().

2.5.4 Effectiveness of AWS Systems Combined with AD Systems

In order to increase the effectiveness of DCZ protection for motorists approaching signalized intersections, dynamic AWSs are often combined with AD systems. An NDOR study looked at the differences between intersections that were equipped with both AD and AWS systems (AD/AWS), an intersections equipped with AD systems only (AD only) (*4*).

The report found that motorists that were upstream of systems combining AD/AWS technologies did not accelerate to try and clear the intersection after the onset of the yellow change interval as much as motorists approaching intersections with AD only systems (*21*). Motorists were also making less abrupt stops on approaches to intersections with AD/AWS systems than on approaches to intersections with the AD only systems. Motorists approaching signalized intersections with AD/AWS systems began accelerating less and making less abrupt stops because the motorists were receiving warning of the impending signal change and would prepare and adjust their driving behavior to account for the information. Motorists approaching intersections with AD only systems did not receive advance warning and were not able to prepare for the yellow change interval (*21*).

The NDOR report also found that their recommended AD/AWS system, which will be described later in this report, also decreased the number of max-outs at signalized intersections (*4, 38*). The AD/AWS system design decreased the number of max-outs by

decreasing the required passage time required for to extend the green thereby decreasing the number of cycles that used all of the allowable green time (*38*).

The NDOR design also provided better DCZ protection for vehicles that were traveling faster than the design speed. The NDOR design did not significantly reduce the amount of vehicles running red-lights, stopping abruptly, or accelerating on yellow; however, it did reduce the percentage of vehicles caught in the DCZ when the green phase was terminated by gap-out. The NDOR design, however, did not provide a higher probability of DMZ protection unless the traffic volumes were high (*4*, *38*).

The NDOR report concluded that the AD/AWS system increased road user time cost savings because of a reduction in wasted time due to a reduction in the number of max-outs. In other words, less green time was being used because gap-out was happening more often (*4*, *38*). A MnDOT study found that when AWSs were used without advance detectors, DCZ protection was only provided for a narrow speed range (*27*).

2.5.5 Summary

Installations of AWSs have been known to mitigate the number of RLR violations, intersection crashes, and other conflicts that are common on approaches to HSSIs. AWSs are also known to reduce vehicle speeds on approaches to HSSIs and to increase the preparedness and likelihood that motorists will stop when the signal turns yellow. However, the results are mixed and sometimes the conditions worsen after AWS installation. AWSs could potentially increase the number of rear-end crashes due to the fact that motorists might be more anxious to stop when they should proceed through the intersection. The literature review suggests that the location and design of AWS systems should be carefully considered before installation. The following section provides guidelines relating the location, installation, and design of AWS systems.

2.6 AWS Installation Guidelines

Although guidelines for installation of AWS systems vary from agency to agency, there are a few guidelines that most agencies have in common. This section discusses common installation guidelines and other less common guidelines used by agencies in North America to determine when and where to install AWS systems.

Most agencies include guidelines for (, , , *39*):

- Approach speed,
- Isolated or unexpected HSSIs, approaches with limited sight distance, including intersections hidden by horizontal or vertical curvature,
- HSSIs with a high number of crashes,
- HSSIs with a high number of red-light runners, and
- Intersections where engineering judgment deems the necessity of AWS installation.

MnDOT recommends that AWS signs be considered for approaches with an 85th percentile speed of 50 mph or greater (*3*), while Agent and Pigman (*36*) recommend that AWSs be considered for 85th percentile approach speeds above 45 mph. Right-angle and rear-end crashes were the number one reason cited in one survey of agencies as a reason to install AWSs (*39*).

The British Columbia Ministry of Transportation recommends that AWS systems be installed when: 1) the posted speed limit is 70 km/h [45 mph] or above; 2) the view of signals is obstructed due to vertical or horizontal alignment; 3) the approach grade requires more than normal braking effort; and 4) the signalized intersection is the first intersection motorists encounter for many miles (*40*).

The City of Calgary recommends that AWS systems be installed when the following conditions exist: 1) a signalized intersection has a posted speed limit of 70 km/h [45 mph] or more; 2) a signalized intersection is the first signalized intersection into a city where the speed limit is greater than 100 km/h [60 mph]; 3) a roadway has a speed limit of 70 km/h [45 mph] and a crash hazard exists that is correctable by using AWS

systems; and 4) when horizontal and vertical alignment causes visibility to be restricted so that the signalized intersection cannot be seen (*41*).

The Manitoba Highways and Transportation Agency guidelines are similar to those of the City of Calgary and British Columbia guidelines but are more specific. For example, the Manitoba Highways and Transportation Agency suggests that AWS systems be installed: 1) at rural intersections at least 2 kilometers [1.2 miles] away from the nearest signalized intersection with approach speeds of 70 km/h [45 mph] or greater; 2) at intersections to urban areas with approach speeds of 70 km/h [45 mph] or greater; 3) at intersections with 1 kilometers [0.6 miles] of 3 percent or greater downgrade with an approach speed of 60 km/h [37 mph] or more; 4) at intersections within 520 meters [1,700 feet] of significant sight restrictions due to horizontal and vertical alignment with 60 km/h [37 mph] or more; and 5) intersections where "fail to stop" right-angle crashes exceed four per year on a three year average (*42*).

Farraher et al. cite MnDOT technical memorandums that caution that AWSs should not be considered a standard signal system component for the following reasons (*37*):

- The lead flash creates a delay for the overall operation of the signal system,
- There exists an ongoing concern that a proportion of motorists use the flashers to "over-drive" the signal timing and "race" the signal system – thereby becoming a hazard,
- Resources for construction, power and maintenance would limit other work,
- The "punch" that the flashers provide would be diminished if used excessively, and
- Such a supplementary system is not perceived by motorists to be needed at an overwhelming majority of signal systems.

A survey of the use of AWS systems, conducted by Eck and Sabra (*39*), found that other guidelines for AWS installations include intersections with steep downgrades and rural expressways with heavy truck traffic. Eck and Sabra suggest that AWS as countermeasures should only be considered after traditional approaches, such as detector placement, yellow time adjustment, and intersection reconfigurations have been

considered or tried; and that most agencies in the survey preferred detector placement followed by red signal ahead signs for crash mitigation steps preceding consideration of AWS installation. Eck and Sabra also discovered that the only measures of effectiveness that some agencies used were subjective assessments based on experience, opinion, and personal preference (*39*).

2.7 Decision Zone Study Methods

DCZ studies are conducted to determine the size and location the area of greatest variation in motorist behavior (*3*). DCZ studies can be conducted at intersections where previous design adjustments and signal timing and/or other safety measures have failed. A DCZ study can aid engineers in determining the area on the approach to an intersection where motorists, for one reason or another, are having a hard time deciding what to do when the signal turns yellow. A DCZ study can also help engineers pinpoint the location where 90 percent of motorists have decided to stop and the location where only 10 percent of motorists have decided to stop after the onset of the yellow change interval.

Some of the most common DCZ study parameters include the following (, *17*, *28*, *30*, *43*):

- The distance from the intersection and speed of approach vehicle at the onset of the yellow signal,
- The location and distance from the intersection when the vehicle's brakes were applied (indicated by the brake lights),
- The time required for the vehicle to stop,
- The motorist's decision to continue through the intersection or come to a stop,
- The time and distance when a vehicle stopped,
- The average number of motorists who run the red-light per cycle,
- The traffic volume or density,
- The time when the vehicle entered and cleared an intersection, and
- The vehicle classification type.

The size and location of DCZ boundaries are usually found through empirical studies of motorist stopping behaviors. Chang et al. utilized time-lapse photography at seven study sites, with speed limits ranging from 30 to 55 mph, and post-processed the video images in a lab (*30*). The post processing parameters used to determine the size of the DCZ were similar to the list of parameters identified previously. Chang et al. found that 99 percent of all motorists would clear the intersection at the onset of the yellow change interval if they were closer than 2 seconds of travel time from the stop bar. Chang et al. also found that 85 percent of motorists who stopped at the onset of the yellow change interval would stop if they were 3 seconds or more away from the intersection at the onset of the yellow change interval and that the distribution of clearing or stopping vehicles was the same regardless of vehicle approach speeds (*30*).

Another finding of the study by Chang et al. was that 90 percent of motorists who cleared the intersection did so if they were within 4.5 seconds of travel time from the intersection at the onset of the yellow change interval suggesting that the yellow change interval may not need to be longer than 4.5 seconds in duration (*30*).

Wortman et al. studied five signalized intersections in the Tucson metropolitan area using time-lapse photography (*43*). Wortman et al. used study parameters similar to those listed and found that the average distance from the intersection at the onset of the yellow change interval that vehicles would be at and still clear the intersection was approximately 131 feet. The average distance from the intersection at the onset of the yellow change interval that vehicles would be at and come to a stop at the intersection was approximately 255 feet. The average values were from observations at seven study site locations with approach speeds ranging from 35 to 45 mph (*43*).

A MnDOT driver simulation study observed motorist reactions to yellow lights on approaches to HSSIs, with speed limits 50 to 60 mph, in an attempt to find the boundaries of a DCZ based on the vehicle proximity to yellow, or the number of seconds of travel time from the intersection. MnDOT researchers considered the boundaries of the DCZ to be between 2 and 5 seconds vehicle proximity to yellow and timed the yellow change interval to begin at various vehicle proximity to yellow times for each motorist during the simulation as they approached signalized intersections (*3*). MnDOT researchers recorded motorist responses on approaches with and without AWSs to determine if AWSs were

effective in reducing variability in driver behavior after the onset of the yellow change interval. The results of the MnDOT driver simulation study found that AWSs decreased the variability of motorists behavior between 2 and 3.5 seconds of vehicle proximity to yellow (*3*). The study also found that the greatest variability in motorist behavior occurred at 2 seconds vehicle proximity to yellow and the least variability occurred at 5 seconds vehicle proximity to yellow (*3*).

2.8 Literature Review Chapter Summary

Safety at signalized intersections is dependant on understanding the yellow change interval, the safe stopping-sight distance requirements of approaching vehicles, and the behavioral characteristics of approaching motorists. Poor yellow change interval timing design can lead to DMZs where motorists are not able to safely stop or safely proceed through the intersection. In order to mitigate the DMZ, signal timing should be adjusted according to the behavioral characteristics of motorists approaching the signalized intersection.

Incorrect design assumptions of motorist behavioral characteristics increase the number of conflicts that motorists face in the DCZ. AD and AWS systems exist to mitigate the negative effects of the DCZ. However, AWS and AD installations will not effectively mitigate DCZ conflicts if designed incorrectly. Guidelines exist to increase the effectiveness of AWS and AD technologies and govern their proper use. Finally, DCZ studies can be conducted to locate the DCZ and determine its extents in an attempt to properly design and install AD and AWS technology.

3 Background

The purpose of this chapter is to provide the reader with more detail relating to the need for the study, the project study site, the theories and philosophies behind the technology that was evaluated, the actual design parameters of the technology, and the metrics chosen to evaluate the effectiveness of the design.

The background chapter is divided into the following sections:

3.1 The Need for BODAWS on Bangerter Highway – outlines the reasons why UDOT chose to install AD/AWS technologies at HSSIs in Utah.

3.2 13400 South Study Site Description – describes the study site location and intersection geometries.

3.3 BODAWS System Design and Configuration – describes the design characteristics, assumptions, and ultimate component configurations.

3.4 BODAWS Evaluation and Evaluation Metrics – describes the evaluation metrics used to evaluate the effectiveness of the BODAWS system.

3.5 Project Background Chapter Summary – summarizes the main points of the chapter.

3.1 The Need for BODAWS on Bangerter Highway

The state of Utah began to consider the installation of AD and AWS systems to address DCZ problems 10 years ago. At that time, research was completed at BYU that compared the safety impacts of AWS systems in Calgary, Alberta, Canada with then current conditions on S.R. 154 (Bangerter Highway) (*44*). In the years that followed,

UDOT maintenance crews began to express concerns about the high concentration of skidding and subsequent load spills that were occurring at HSSIs along this same corridor. The maintenance crews were concerned that repeated exposure to high deceleration rates and subsequent skidding would prematurely damage the pavement on the approaches. At the same time, UDOT traffic and safety officials were concerned with the potential for RLR and related rear-end and right-angle crashes as a result of these conditions.

In response to the concerns of both maintenance and safety engineers, UDOT began to consider possible installations of AWS systems on their HSSI approaches. As a result, a team of UDOT personnel was assembled to identify locations for possible AWS installations. At the same time, UDOT retained a consultant (Project Engineering Consultants, Ltd.) to evaluate and design an effective new system to mitigate the problems identified by their engineers. The consultant recommended a system designed in accordance with research findings published by McCoy and Pesti at the University of Nebraska-Lincoln, for the NDOR, that included both AD and AWS components ().

The consultant used the study findings from the NDOR report to locate the AWS, establish the basis for the lead flash timing, and locate the advance detectors. The consultant also recommended that blank-out dynamic signs mounted over the roadway be used as the AWS sign type. No guidelines exist in the MUTCD for blank-out signs as warning signs or for warning signs to be mounted over the roadway. Therefore, the overhead blank-out component of the design is unique and experimental. The new design is referred to as the BODAWS system, as defined previously, and is illustrated in Figure 3-1. The following subsection describes the components of the BODAWS system and the purposes of the BODAWS configuration.

3.1.1 BODAWS System Components and Purpose

The BODAWS system is comprised of three distinct components. The three components of the system included: 1) an AD component, 2) an AWS component, and 3) a signal timing component. The AD component will be referred to as the BODAWS

detector. The AWS component will be referred to as the BODAWS sign and flashers. The signal timing component will be referred to as the BODAWS signal timing.

Figure 3-1 BODAWS sign and flashers.

The purpose of the new BODAWS system was to increase motorist awareness of impending signal changes and to provide DCZ protection. It was hypothesized that motorist awareness could be increased by mounting the BODAWS signs and flashers over the roadway. The BODAWS signs were dynamic and blanked-out when not in use. The purpose of the dynamic overhead blank-out design of the BODAWS signs was to provide only real-time pertinent information to the motorists so that they would be less inclined to lose respect for the warnings over time.

UDOT determined to install the new BODAWS systems at four locations in Salt Lake County including three on Bangerter Highway and one on S.R. 201 (2100 South). The Bangerter Highway locations included the intersections of Bangerter Highway with S.R. 68 (Redwood Road), 2700 West, and 13400 South. Due to time and equipment constraints, only the installation at the intersection of Bangerter Highway and 13400 South was evaluated for the research as described in the following section.

3.2 13400 South Study Site Description

The study section of Bangerter Highway was a four lane divided highway in a combination of rural and suburban settings. The study site was located in Riverton, Utah, approximately 20 miles south of Salt Lake City, Utah, and is illustrated in Figure 3-2 and Figure 3-3. The land use bordering the Bangerter Highway corridor near the study site was predominately open space with residential and commercial sites located next to each intersection. Bangerter Highway connected on the north to a major east/west interstate freeway (I-80) and on the south to a major north/south interstate freeway (I-15). Bangerter Highway was classified by UDOT as a principal arterial with design speeds ranging from 55 mph to 65 mph along the route. Due to the nature of the corridor and the types of surrounding land uses it was assumed that the majority of motorists traveling on Bangerter Highway lived in the vicinity of Bangerter Highway and routinely traveled the corridor.

The Bangerter Highway approaches to the intersection in the study area consisted of two through lanes in each direction, an exclusive right-turn lane, and two exclusive left-turn lanes. The posted speed limit on Bangerter Highway at 13400 South was 60 mph and UDOT traffic studies determined that the 85th percentile speed was approximately 62 mph on sections of the roadway well outside of the functional area of the study intersection. As can be seen in Figure 3-2, the intersection of Bangerter Highway and 13400 South was skewed because Bangerter Highway crossed 13400 South at approximately 30 degrees counterclockwise from perpendicular. The offset occurred because the intersection was on a portion of Bangerter Highway that was curving towards the north. Motorists approaching from the southeast could not see the main signal heads at the intersection until they are within approximately 700 feet of the stop bar because of a pedestrian overpass that blocked their view. Supplemental signal heads were installed on both sides of the highway, just prior to the pedestrian overpass, to help motorists observe the condition of the signal as they approached the intersection. The 13400 South intersection was chosen as a study site because it was hypothesized that the limited sight distance may have contributed to higher instances of RLR and crashes than the other BODAWS installation sites.

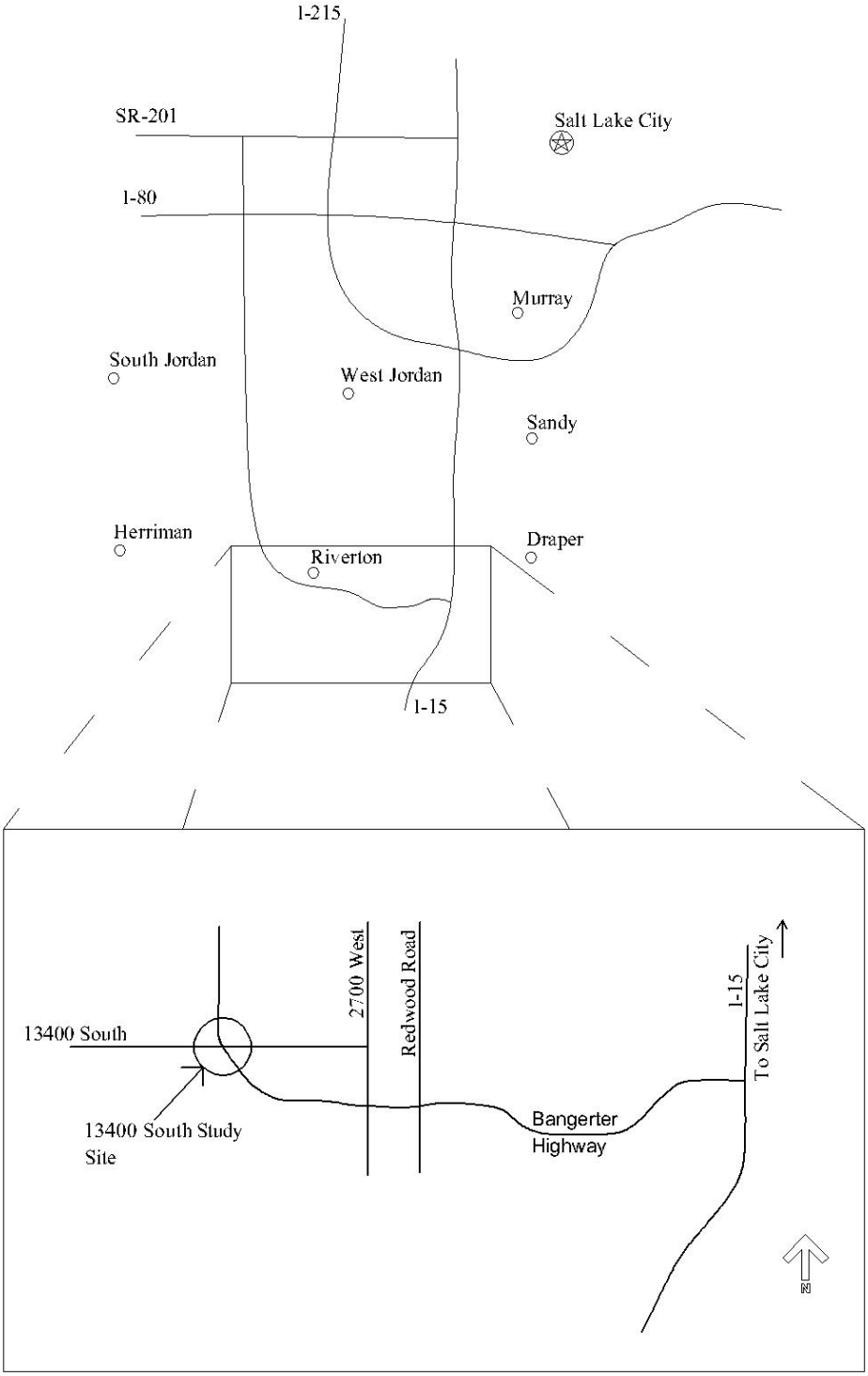

Figure 3-2 Research study site location.

Figure 3-3 Aerial photo of the research study site location (*45*).

Before the BODAWS system was installed the signals and advance detectors were timed according to UDOT guidelines outlined in the UDOT *Design of Signalized Intersections Guideline and Checklist* manual (*46*). The resulting signal timing values are summarized in Table 3-1. The phases of the Bangerter Highway approaches to the study site were set to gap-out or max-out simultaneously.

Table 3-1 Signal Timing Values at the Study Site Before BODAWS Installation

Signal Timing Value	Time (sec)
Minimum Green	15
Unit Extension of Maximum Gap	5
Maximum Green	60
YCI	6
All-Red	2
Time Before Reduction	15
Time to Reduce	10
Minimum Gap	2

3.3 BODAWS System Design and Configuration

This section describes the BODAWS system design and configuration. Each component of the BODAWS system will be discussed including: 1) the BODAWS signs and flashers; 2) BODAWS detector; and 3) the BODAWS signal timing. This section is divided into one subsection for each of the system components.

3.3.1 BODAWS Signs and Flashers Configuration

The AWS component of the BODAWS system deviated somewhat from that outlined in the NDOR report and from any other known installation in that in that the sign itself incorporated a blank-out light emitting diode (LED) sign with two side mounted flashing beacons mounted over the travel lanes of the roadway on a standard 70/75 foot mast arm and signal pole cut back to 55 feet in length. A horizontal damping “wing” (2 foot by 8 foot panel) was installed on the top of the mast arm to reduce horizontal deflection of the mast arm. The blank-out sign included the message “PREPARE TO STOP” when activated in two rows of 14 inch letters as illustrated previously in Figure 3-1. The overall dimensions of the sign are 48 inches tall by 96 inches wide by 12 inches deep, weighing a total of 300 pounds. The blank-out sign, recommended by the consultant, is especially noteworthy as it was the first known AWS installation that incorporated a blank-out sign mounted over the travel lanes as opposed to the more

common prepare to stop when flashing, illustrated previously in Figure 2-6, that is usually mounted on the side of the road.

The location of the BODAWS signs and flashers corresponded to the recommendations of an NDOR study conducted by McCoy and Pesti. The NDOR study recommendations are similar to those of the MUTCD for warning sign placement as discussed previously in Section 2.4. According to McCoy and Pesti, the DCZ boundary at a signalized intersection starts at the upstream boundary of the stopping-sight distance and extends to the stop bar. McCoy and Pesti recommend placing AWS signs at a distance from the signalized intersection equal to the stopping-sight distance minus a sign legibility distance. The equation recommended by McCoy and Pesti for placement of AWS signs and flashers is outlined in Equation 3-1 (*4*, *38*).

$$D_M = t_o v_o + \frac{v_o^{\ 2}}{2a_s} - D_L \tag{3-1}$$

where: D_M = distance between advance warning signs and stop bar (ft),
t_o = perception-reaction time (sec),
v_o = design speed (ft/sec),
a_s = deceleration rate (ft/sec^2), and
D_L = sign legibility distance (ft).

The parameters recommended by McCoy and Pesti for use in Equation 3-1 are based on the assumption that vehicle speed distributions at signalized intersections are normal. McCoy and Pesti developed equations to identify the upper and lower boundaries of the speed range of vehicles that receive DCZ protection based on Equation 3-1. An iterative procedure was used to adjust the parameters of Equation 3-1 until the boundaries of the speed range were maximized. The recommended design speed for sign placement was then correlated to the 85th percentile speed of the approach.

The design speed recommended by McCoy and Pesti was 10 mph less than the 85th percentile speed. McCoy and Pesti found that the 10 mph difference allowed more of the normal speed distribution of approaching vehicles to fall within the limits of the

upper and lower boundaries of the speed range of protected vehicles. The first two columns of Table 3-2 contain the 85th percentile speed and corresponding design speed recommendations of the McCoy and Pesti design (*38*).

Table 3-2 Design Installation Guidelines ()

85th Percentile Speed (mph)	Design Speed (mph)	Distance From Stop Line (ft)		Lead Flasher Timing (sec)
		Advance Detector	AWS	
65	55	755	445	6.5
60	50	655	365	6.0
55	45	560	290	5.5
50	40	470	225	5.0
45	35	385	160	4.5

The 85th percentile used by the consultant to design the BODAWS system was 65 mph. Therefore, based on Table 3-2 the BODAWS signs and flashers were installed 445 feet from the stop bar at the study site.

As indicated previously, the consultant who designed the BODAWS system based the design on the research report prepared by McCoy and Pesti (). The McCoy and Pesti design was established using a rigorous cost-benefit analysis that compared crash and delay costs of a “new” configuration with a “conventional” design based on the then current stopping-sight distance definition of the DMZ.

Although not considered by the consultant at the time of the recommendation, the McCoy and Pesti conventional design was re-evaluated as part of the research conducted for the UDOT project based on revisions to the AASHTO Green Book (*12*) and the MUTCD (*8*). The original analysis completed by McCoy and Pesti for the conventional design was based on a 3.0 second perception-reaction time, a sign legibility distance of 125 feet, and a braking distance as defined in the 1984 AASHTO Green Book (*4*). Using the 2004 AASHTO Green Book and the Millennium Edition of the MUTCD, the relationship outlined in Equation 3-1 was used to calculate the proposed distance from the stop line to the advance warning signal.

The results of this analysis indicate that the distance from the stop line to the advance warning signal for an HSSI approach with a design speed of 65 mph, perception-reaction time of 2.5 seconds, and deceleration rate of 11.2 ft/sec^2, as per AASHTO Green Book standards, is approximately 470 feet. These results are very comparable to the results of the new design identified by McCoy and Pesti using the cost-benefit analysis methodology, thus providing further justification of the proposed design (*4*). The following subsection describes the BODAWS detector design and configuration.

3.3.2 BODAWS Detector Design and Configuration

The AD component of the BODAWS system was installed to reduce the percentage of vehicles in the DCZ at the onset of the yellow change interval. The design of the BODAWS detectors was based on recommendations by McCoy and Pesti outlined in Equation 3-2.

$$D_{PT} = t_p v_o + D_P \tag{3-2}$$

where: D_{PT} = distance between advance detector and AWS (ft),
t_p = controller passage time setting (sec),
v_o = design speed (ft/sec), and
D_P = minimum distance at which AWS can be perceived (ft).

Based on the research by McCoy and Pesti, described in the previous subsection, the design parameters for unit extension or passage time and AWS perception distance were found to be 3 seconds and 70 feet, respectively. The design speed recommendations were also the same as those used to find the setback distance of the AWS signs. The AD component consisted of a single optical detection zone created using a video camera mounted on a standard lamp post installed upstream of the study site intersection. The BODAWS detector was located based on an 85th percentile speed of 65 mph at the study site which, according to Table 3-2, meant that it was located 755 feet from the stop bar.

The lamp post and detector camera were located 705 feet from the stop bar. An illustration of the typical BODAWS system installation is provided in Appendix A.

The advance detectors were not the only detectors at the intersection. Stop bar detectors were also used in the design. The stop bar detectors operated in presence mode during the red interval and during the green interval up until the extendable portion of the green when they were turned off. The following subsection describes BODAWS signal timing component.

3.3.3 BODAWS Signal Timing Plan

Four distinct events occur during the BODAWS system sequence. The first event occurs at the beginning of the extendable portion of the green interval when the stop bar detectors are not active and the BODAWS detectors are active. When the BODAWS detectors are active the signal controller is programmed to identify an appropriate gap in traffic in order to end the major street through phase when as few vehicles are in the DCZ as possible. The maximum unit extension time recommended by McCoy and Pesti is 3 seconds which is long enough to allow a motorist traveling at or greater than the design speed of 55 mph to travel from the BODAWS detector to a point 70 feet from the BODAWS sign and flashers (*4*, *38*). If a motorist is within 70 feet of the BODAWS sign and flashers they will probably not see the variable message “PREPARE TO STOP” or notice the beacons flashing. If 3 seconds pass and no other vehicle actuated the BODAWS detector, the signal can begin the process of phase termination.

The second event of the BODAWS system sequence begins when the BODAWS signs and flashers are activated due to the passage time being met. When the BODAWS signs are active the message “PREPARE TO STOP” appears on the blank-out sign and flashing beacons on each side of the blank-out sign begin to flash in alternating half-second bursts. McCoy and Pesti recommend that the BODAWS sign and flasher lead time be set according to Equation 3-3.

$$t_F = \frac{D_M + D_P}{v_o} \tag{3-3}$$

where: t_F = lead flash time (sec),

D_M = distance from the BODAWS sign to the stop bar (ft),

D_P = minimum perception distance of the BODAWS sign (ft), and

v_o = design speed (ft/sec).

For an approach with an 85th percentile speed of 65 mph, Table 3-2 recommends a lead flasher timing of 6.5 seconds According to Equation 3-3, the lead flash timing is actually 6.36 seconds if McCoy and Pesti's design assumptions are used. However, the consultant rounded the lead flash time down to 6 seconds for the BODAWS installation at the study site. During the lead flash time, the last vehicle that crosses the BODAWS detector before gap-out will have time to travel from 70 feet in front of the BODAWS sign and flashers to the stop bar before the onset of the yellow change interval and will therefore be well clear of the DCZ. Subsequent vehicles will see the warning from the BODAWS sign and flashers and will be prepared at the onset of the yellow change interval to decelerate and stop. Once the BODAWS signs and flashers are activated they remain active through the yellow change interval, the all-red clearance interval, and the red interval and return to inactivity at the beginning of the following green interval.

The third event of the BODAWS system sequence begins with the onset of the yellow change interval. The yellow change interval is at the study site is 6 seconds long. All of the vehicles that approach the intersection after gap-out will receive the warning from the BODAWS sign and flashers and should have sufficient distance and time to safely stop.

The fourth event of the BODAWS system sequence begins with the onset of the 2 second all-red clearance interval and extends through the entire red interval. The BODAWS signs and flashers remain active and warn approaching motorists that the signal is red.

The signal timing of the intersection was altered after installation of the BODAWS system by UDOT employees responsible for the study site intersection.

Signal timing values that were incorporated by UDOT are listed in Table 3-3. Most of the signal timing changes were not related to the BODAWS system design. The only change to signal timing differing from recommendations by McCoy and Pesti was the unit extension time (or passage time or gap time). The value of the unit extension was set to 4.5 seconds instead of 3 seconds. UDOT employees desired to reduce the number of max-outs by gradually reducing the unit extension time from 4.5 seconds to the minimum gap time of 3 seconds. The time before reduction was set to equal the minimum green time of 15 seconds and the time to reduce was set to 15 seconds as shown in Table 3-3.

Table 3-3 Signal Timing Values at the Study Site Before and After BODAWS Installation

Signal Timing Value	Time (sec)	
	Before BODAWS	After BODAWS
Minimum Green	15	15
Unit Extension of Maximum Gap	5	4.5
Maximum Green	60	80
YCI	6	6
All-Red	2	2
Time Before Reduction	15	15
Time to Reduce	10	15
Minimum Gap	2	3

3.4 BODAWS Evaluation Metrics

After the initial design had been completed and the installation locations determined, UDOT retained researchers at BYU to evaluate the effectiveness of the BODAWS system for possible future installations. Possible positive impacts of BODAWS installations that were determined worthy of evaluation at these locations included: 1) crash rate reductions; 2) improved motorist reaction time; and 3) a reduction in RLR violations. Potential negative impacts included increased speeds on intersection approaches following the activation of the BODAWS signs and flashers and increased

collisions due to the fact that some motorists "use the flashers to 'over-drive' the signal timing and 'race' the signal system – thereby becoming a hazard" ().

The metrics proposed for evaluation by the research team included an evaluation of safety impacts of the BODAWS design installation at the study site through crash and RLR differentials, as well as an evaluation of the impact on speed trends immediately following activation of the BODAWS signs and flashers, and before and after the onset of the yellow change interval. In addition, a DCZ study was also conducted for the analysis.

The BODAWS system installation was activated on June 8, 2005. Data were gathered before, immediately after, and eight months after installation of the BODAWS system. Data were collected before BODAWS installation from April 27, 2005 to May 2, 2005, and from May 25, 2005 to June 8, 2005. Data were collected immediately after BODAWS installation from June 8, 2005 to June 23, 2005, and from July 8, 2005 to July 22, 2005. Data were collected approximately eight months after installation from February 8, 2006 to February 15, 2006, and from March 17, 2006 to March 24, 2006.

In order to effectively reference each data collection period in respect to the installation of the BODAWS system, the period before BODAWS installation will be referred to as period 1 (P1), the period immediately after BODAWS installation will be referred to as period 2 (P2), and the period eight months after BODAWS installation will be referred to as period 3 (P3). The data collection metrics are outlined in the following subsections.

3.4.1 Speed Metric

Speeds of approaching vehicles were recorded during on-site spot speed studies conducted by UDOT and through the use of state-of-the-art Digital Wave Radar™ technology during periods P1, P2, and P3. Spot speed studies were conducted for the purpose of verifying and validating the results from the speeds collected by the radar.

Speeds from P1, P2, and P3 were statistically analyzed to identify trends in motorist behavior on approach to the HSSI and to correlate vehicle speeds with the activation of the BODAWS system during each analysis time period. The statistical

analysis included cumulative distribution plots, box plots, and *t*-test comparisons of the speed data. Details of the speed study are explained in Chapter 4 of this report.

3.4.2 RLR Metric

RLR data were collected throughout study periods P1, P2, and P3. RLR data were collected through on-site observations and through the use of Digital Wave Radar™ technology. The on-site observation studies were conducted by UDOT at various times during P1, P2, and P3 to verify the data collected through the use of the electronic equipment. RLR data were compared by calculating the number of RLR violations per 1,000 entering vehicles. The number of vehicles entering the intersection was obtained from the radar sensor detection zone at the stop bar. Details of the RLR study are presented in Chapter 4 of this report.

3.4.3 Crash Metric

Crash data were collected from the law enforcement agency responsible for the study intersection in question (Salt Lake County Sheriff's Department) and for one additional control intersection of similar geometric design and volume. Crash data were gathered for the three years prior to BODAWS installation and for six months following installation. It is understood that to present a more accurate crash data comparison of results, crash data would need to be collected for three years after installation. Due to the contract time period and agreement this was not possible as part of the study. As a result, only a six month crash data analysis is included in this report.

Crash data can be compared according to the number of crashes per million entering vehicles. The number of entering vehicles at the study intersection and a similar control intersection was gathered from UDOT traffic counts. The traffic counts were average annual daily traffic counts that were found on UDOT's traffic studies and statistics website (*47*). Equation 3-4 from the ITE *Traffic Engineering Handbook* was used to calculate the number of crashes per million entering vehicles (*6*).

$$MEV = \frac{(c \times 1{,}000{,}000)}{(b \times 365)} \tag{3-4}$$

where: MEV = crash rate per million entering vehicles,

c = number of crashes in one year,

b = 24-hr total intersection entering volume, and

365 = number of days in a year.

Due to the fact that the crash data at the study site was still being collected at the time of publication of this report, only six months of crash data were available for the period after the BODAWS system had been installed. Therefore, when Equation 3-4 was used to calculate crash rates, the 365 day value was divided by two and only the last six months of data from each available year were used. Further details regarding the crash study are located in Chapter 4 of this report.

3.4.4 DCZ Metric

A DCZ study was conducted on the Bangerter Highway approaches to the intersection study site. The purpose of the DCZ study was to measure the size and location of the greatest variation in motorist behavior and to identify the location where 90 percent of motorists would stop at the onset of the yellow change interval and the location on the approach where only 10 percent of motorists would stop at the onset of the yellow change interval. On-site observations of motorist behavior on each approach to the intersection were collected during periods P1 and P2. Behavioral characteristics of interest included the decision making patterns of motorists at the onset of the yellow change interval, including the decision of the motorists to stop or proceed through the intersection, and the distance from the intersection at which stop or go decisions were manifest. The data output was statistically analyzed using a logit model to determine the regression to the means of the distance data for motorists proceeding through the intersection during the yellow change interval and the motorists who decided to stop at

the onset of the yellow change interval. Details of the DCZ study are available in Chapter 4 of this report.

3.5 Project Background Chapter Summary

UDOT employees and officials identified a need for safety improvements at HSSIs in Utah. UDOT contracted with a private consultant to design a safety system to mitigate DCZ conflicts at four locations in Utah. The consultant created a new system that combined AWS technology with AD technology in a unique configuration based partly on a study conducted by NDOR (*4*). The new system incorporated blank-out signs with flashing beacons that were mounted over the roadway. The use of overhead blank-out signs was unique. UDOT partnered with BYU to evaluate the effectiveness of the BODAWS system at mitigating DCZ conflicts using RLR, crashes, and approach speeds as the evaluation metrics. The following chapter details the implementation of the project including the data equipment designs and configurations.

4 Implementation

This chapter describes the data collection equipment technologies and configurations and the observation studies that were employed to evaluate the effectiveness of the BODAWS system.

The implementation chapter is divided into the following sections:

4.1 Data Collection Equipment Technology and Configuration – describes the data collection equipment and technology that was used to evaluate the BODAWS system.

4.2 Crash Data Analysis Process – describes the equations and calculations used to evaluate the crash data.

4.3 DCZ Study Site Description and Study Methods – describes the DCZ study site layout as well as the methods employed to determine the size and location of the DCZ.

4.4 Implementation Chapter Summary – summarizes the main points of the chapter.

4.1 Data Collection Equipment Technology and Configuration

The data collection equipment at the study site incorporated non-intrusive Digital Wave Radar™ sensor technology developed by Wavetronix LLC, of Lindon, Utah. The radar sensors detected vehicle passage and speed to determine speed trends and trigger the capture of RLR data. Data collection equipment at the study site, as illustrated in, Figure 4-1, Figure 4-2, and Figure 4-3, included the following devices:

- SmartSensor Advance™ Digital Wave Radar™ detectors,
- A data logger,
- A laptop computer,
- Wireless and wired communication devices,
- Surge protectors and power modules, and
- Contact closure devices.

The radar sensors were mounted facing the intersection on the same mast arms as the BODAWS signs and flashers on both the northbound and southbound approaches to the study. As indicated previously, the mast arms for the BODAWS devices were located 445 feet from the intersection.

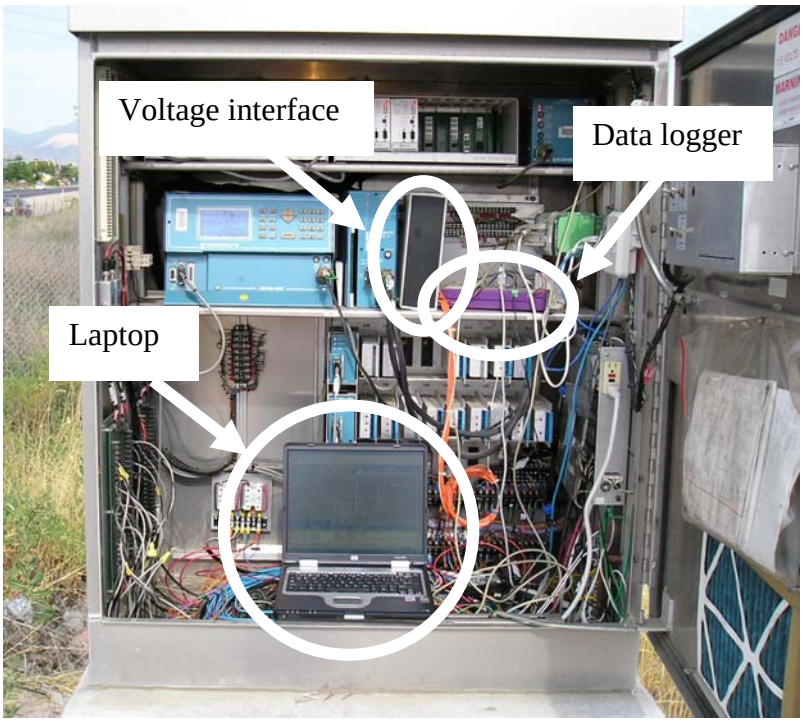

Figure 4-1 Data collection and traffic control devices.

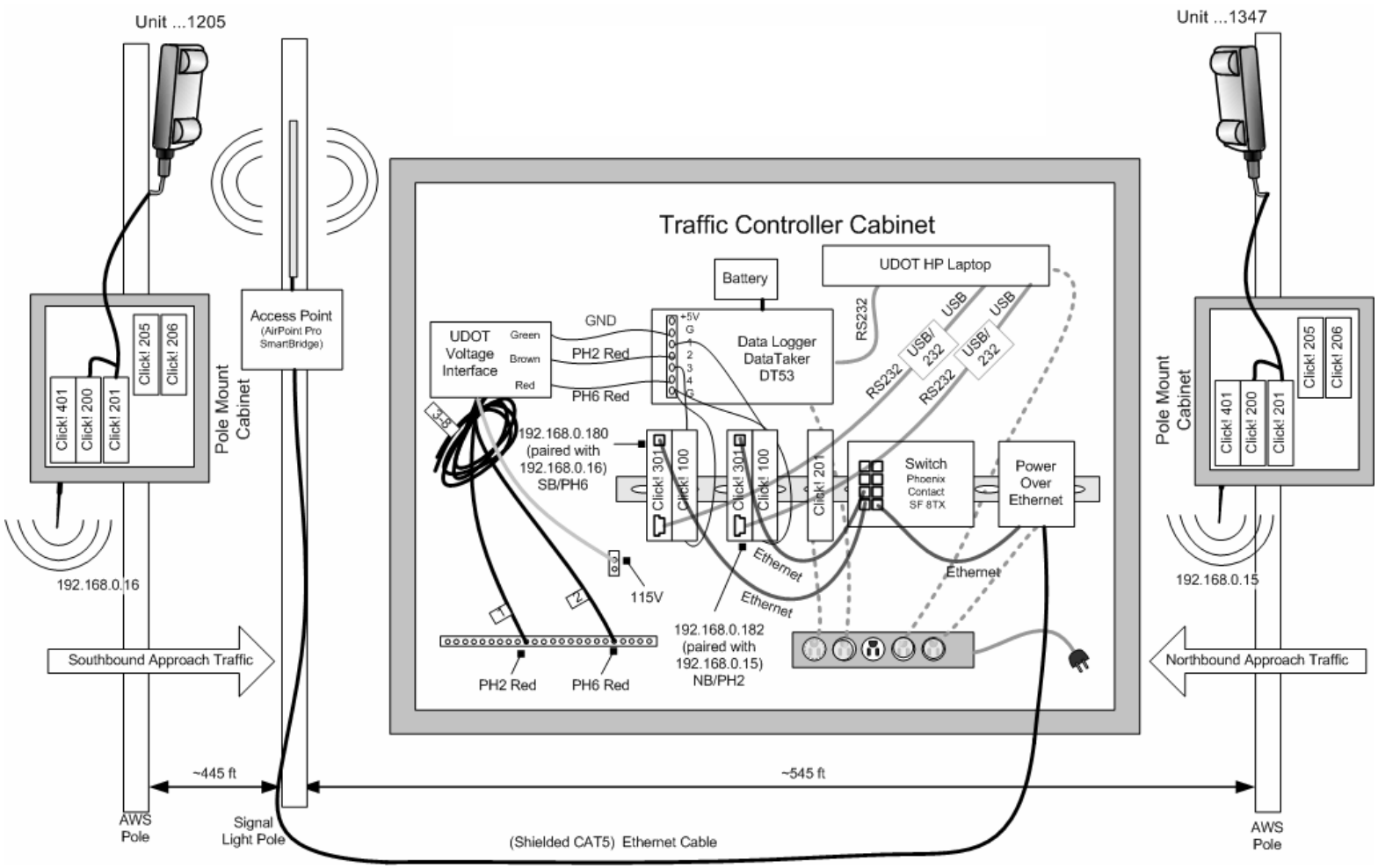

Figure 4-2 Data collection equipment schematic.

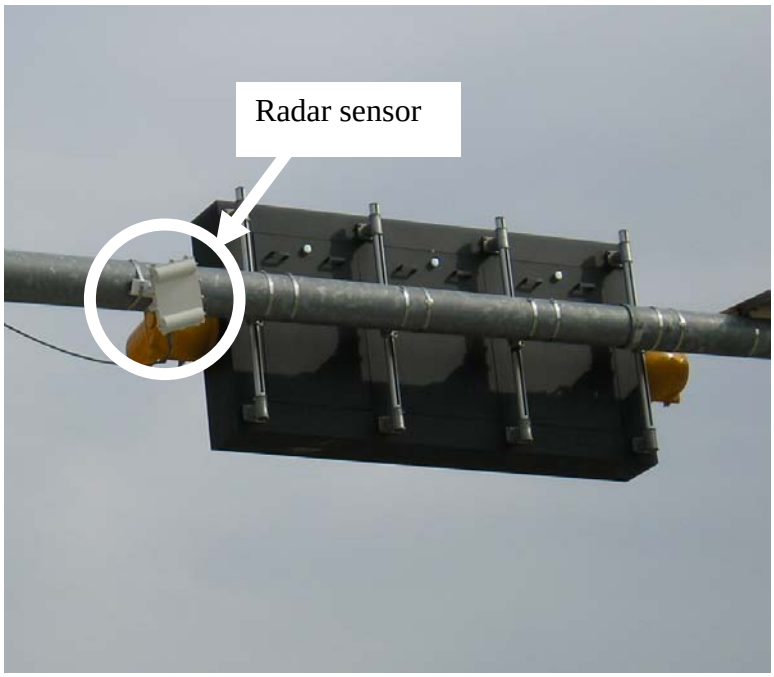

Figure 4-3 Digital Wave Radar™ sensor.

Each radar sensor was configured to detect vehicle passage and speed in seven zones on its corresponding approach. Zone one was located 300 feet from the intersection with the subsequent zones, zones two through seven, located every 50 feet to the stop bar. A plan view of the sensor zones and location of the detection equipment is illustrated in Figure 4-4.

When a vehicle passed through a detection zone, an event was generated by the sensor and stored in a temporary buffer. A laptop computer, located in the traffic signal control cabinet, was used to retrieve and record the real-time event information generated by the sensors. Custom data polling programs running on the laptop computer queried both the northbound and southbound sensors' event data buffers via a hybrid wired/wireless link. The event data were recorded by the data polling programs onto the laptop hard drive in a comma-separated variable (CSV) format.

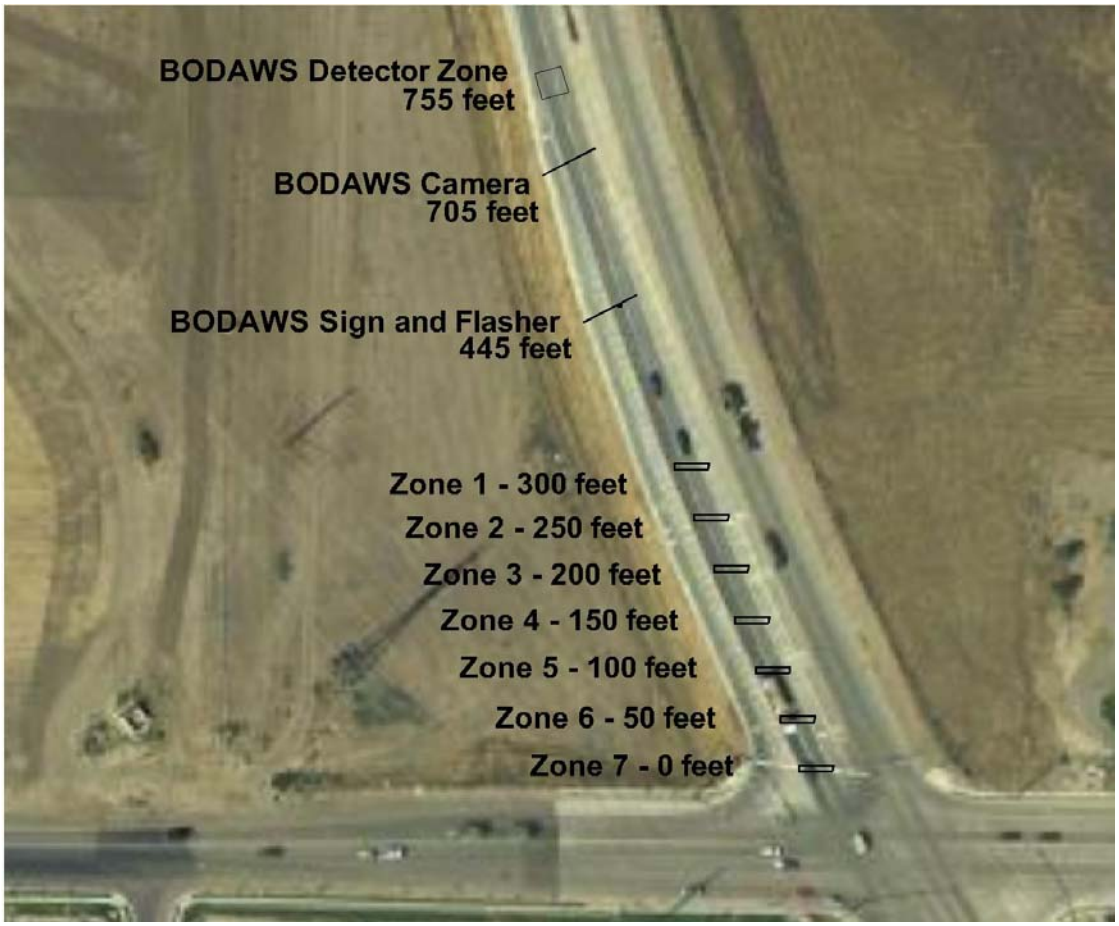

Figure 4-4 Sensor zones and detection equipment layout.

Both wireless links shared one 802.11b wireless access point mounted on the signal pole near the traffic control cabinet. The wired end of the access point was connected to the cabinet via a CAT5 Ethernet cable run through a conduit. The CAT5 cable terminated in a Power-Over-Ethernet module that was connected to an Ethernet switch. Ethernet-to-serial converters connected to the switch were independently paired with 802.11b-to-serial converters housed in pole-mount enclosures on the same masts that the BODAWS signs were mounted on. This pairing of the wireless link serial data converters provided a replacement for the traditional wired connection run through conduit. The final connections to transmit the detection data to the laptop computer were made using RS-232 serial to USB converters.

The data logger located inside the traffic signal controller cabinet connected into the wired portion of the hybrid links via contact closure modules. These modules

listened to the data being retrieved by the laptop data polling programs, using two of the four available digital inputs, to indicate the detection of a vehicle leaving the zones over the northbound and southbound stop bars. The data logger used the remaining two digital inputs to record the beginning and ending time of the northbound and southbound red-light intervals. The red-light interval indications were sent from the traffic controller cabinet and converted from an AC signal to a digital signal suitable for input into the data logger using a voltage interface box.

The data logger was programmed to record both the rising and falling edge of the red interval with a timestamp. It was also programmed to record RLR in both directions of travel by detecting a vehicle leaving one of the detection zones while the signal was red and recording its timestamp. The logic utilized for this RLR detection was a simple if/and statement. If a vehicle crossed the stop bar detection zone and the signal indication was red, the data logger recorded an RLR event.

The data collection equipment at the site generated data files for both the data logger and the sensors as illustrated in Figure 4-5 and Figure 4-6 for the data logger and sensor, respectively. The data files from the data logger included timestamps at the beginning and ending time of the red intervals and for each RLR event. The beginning and ending of the red intervals and the RLR events that occurred from the northbound and southbound through lanes were differentiated by the schedule and state information contained in the data file. Schedules A and B recorded RLR events in the northbound and southbound through lanes, respectively, where state 1 indicated an RLR event. Schedules C and D recorded the red interval changes in the northbound and southbound through lanes, respectively, where state 1 represented the beginning of the red interval and state 0 marked the end of each red interval.

The data files from the sensors contained a timestamp for each vehicle (or vehicle cluster) that passed through each zone, the name of the zone, the duration that each vehicle was in each zone, the speed at which each vehicle traveled through each zone, a vehicle tracker identification (ID) number, and a millisecond count based on the number of 2.5 millisecond intervals from the start of the current day.

Date	Schedule	State
5/2/2005 14:05	D	0
5/2/2005 14:06	C	1
5/2/2005 14:06	D	1
5/2/2005 14:06	B	1
5/2/2005 14:06	A	1
5/2/2005 14:07	C	0
5/2/2005 14:07	D	0
5/2/2005 14:07	C	1
5/2/2005 14:07	D	1
5/2/2005 14:07	A	1

Figure 4-5 Data logger data output file.

```
|                                                                                |
|          DATE         : May 2, 2005                                            |
|          SERIAL NUMBER: SS105 U120001347                                       |
|          DESCRIPTION  : Northbound Approach                                    |
|          LOCATION     : Bangerter & 13400 S                                    |
|________________________________________________________________________________|
|                |              |           |        |            |              |
|   TIMESTAMP    |  ZONE NAME   | DURATION  | SPEED  |  TRACKER   |   COUNT      |
|                |              |           |        |-----------|              |
|  HH:MM:SS:MS   |              | (2.5 ms)  | (MPH)  | Init  |  #  |  (2.5 ms)    |
|________________|______________|___________|________|_______|____|______________|

  14:06:46:002    300            0033       033     1      25   18882401
  14:06:47:177    250            0025       029     0      25   18882871
  14:06:48:535    200            0030       026     0      25   18883414
  14:06:49:935    100            0043       017     1      10   18883974
  14:06:50:062    150            0035       023     0      25   18884025
  14:06:52:052    100            0043       017     0      25   18884821
  14:06:56:567    50             0072       007     0      25   18886627
  14:06:58:730    Stopbar        0066       011     1      18   18887492
  14:06:59:372    Minus 50       0037       014     1      04   18887749
  14:07:01:232    50             0055       006     1      12   18888493
  14:07:01:315    Minus 50       0049       014     0      18   18888526
  14:07:03:605    Minus 50       0049       015     1      14   18889442
  14:07:03:677    Stopbar        0012       018     1      21   18889471
```

Figure 4-6 Sensor data output file format.

The sensor logic was programmed to assign a tracker ID to each vehicle that approached the intersection so that each vehicle could be tracked individually. The tracker ID simplified post-data processing and allowed for a chronological review of the progression of each car traveling through all eight detection zones. By logging speeds in multiple detection zones, acceleration/deceleration patterns were correlated to the activation of the BODAWS signs and flashers and the end-of-green phase using a custom

computer program to determine how the BODAWS system affected motorist performance. The computer program was written to compare the time stamps of sensor data with the data logger data to compare the speed trends of vehicles during each study period and to calculate the number of RLR violations.

4.2 Crash Data Analysis Process

Crash data were collected from both the Salt Lake County Sheriff's Department responsible for the city of Riverton where the study site intersection is located and from the UDOT crash statistics database. UDOT provided crash data in a CSV format that could be manipulated in a spreadsheet program. The Sheriff's Department provided crash data in the form of copies of original crash data reports filed by investigating officers. Crash data were collected from UDOT and the Sheriff's Department for approximately 1,000 feet on either side of the study site intersection. The raw crash data provided by UDOT and the Sheriff's Department were input into a spreadsheet and then organized by crash type. Rear-end and right-angle crashes were categorized individually while all other crash types were counted in a category labeled "other." Data were also separated by approach so that crash statistics could be calculated for crashes that involved one or more participant traveling on Bangerter Highway from crashes that occurred where no participants were traveling on Bangerter Highway. The crash data results are presented in Chapter 5 of this report.

4.3 DCZ Study Site Description and Study Methods

A DCZ study was conducted at the intersection of Bangerter and 13400 South for both the northbound and southbound approaches. Two to three observers were stationed at various distances from the intersection to monitor vehicles as they passed through the study zone illustrated in Figure 4-7. A grid system was created for the observers consisting of marks painted on the pavement at 20 foot intervals and wooden stakes with orange ribbon set up every 100 feet. The grid system extended 1,000 feet from the stop

bar of each approach. During the course of the study observers were asked to record the following information at the onset of the yellow change interval:

- The distance from the intersection for the first vehicle to stop in each lane,
- The distance from the intersection for the last vehicle in each lane that proceeded through the intersection,
- The vehicle classification type (passenger car, passenger truck, or heavy vehicle),
- The time each event occurred, and
- The indication of the signal upon arrival of the observed vehicles at the stop bar.

A sample of the spreadsheet used by observers to record DCZ study events is illustrated in Figure 4-8. Studies were conducted during P1 and P2 on both approaches for the A.M., noon, and P.M. peaks. The same observers were used for each study and were generally placed in the same location during each study to maintain continuity and reduce observer error. In addition to the painted grid system and wooden stake markings, observers were also asked to familiarize themselves with their section of the study area and identify landmarks that would help them determine distances from the intersection.

Observers were equipped with two-way radios so that they could remain in contact with each other throughout the study periods. Two-way radio communication between observers allowed them to avoid duplication of data and helped them to record the location of vehicles that might have been parallel to them when an event occurred. Observers were asked to maintain a low profile and to stay out of sight of approaching vehicles as much as was possible to avoid distracting the motorists and altering their driving behavior patterns.

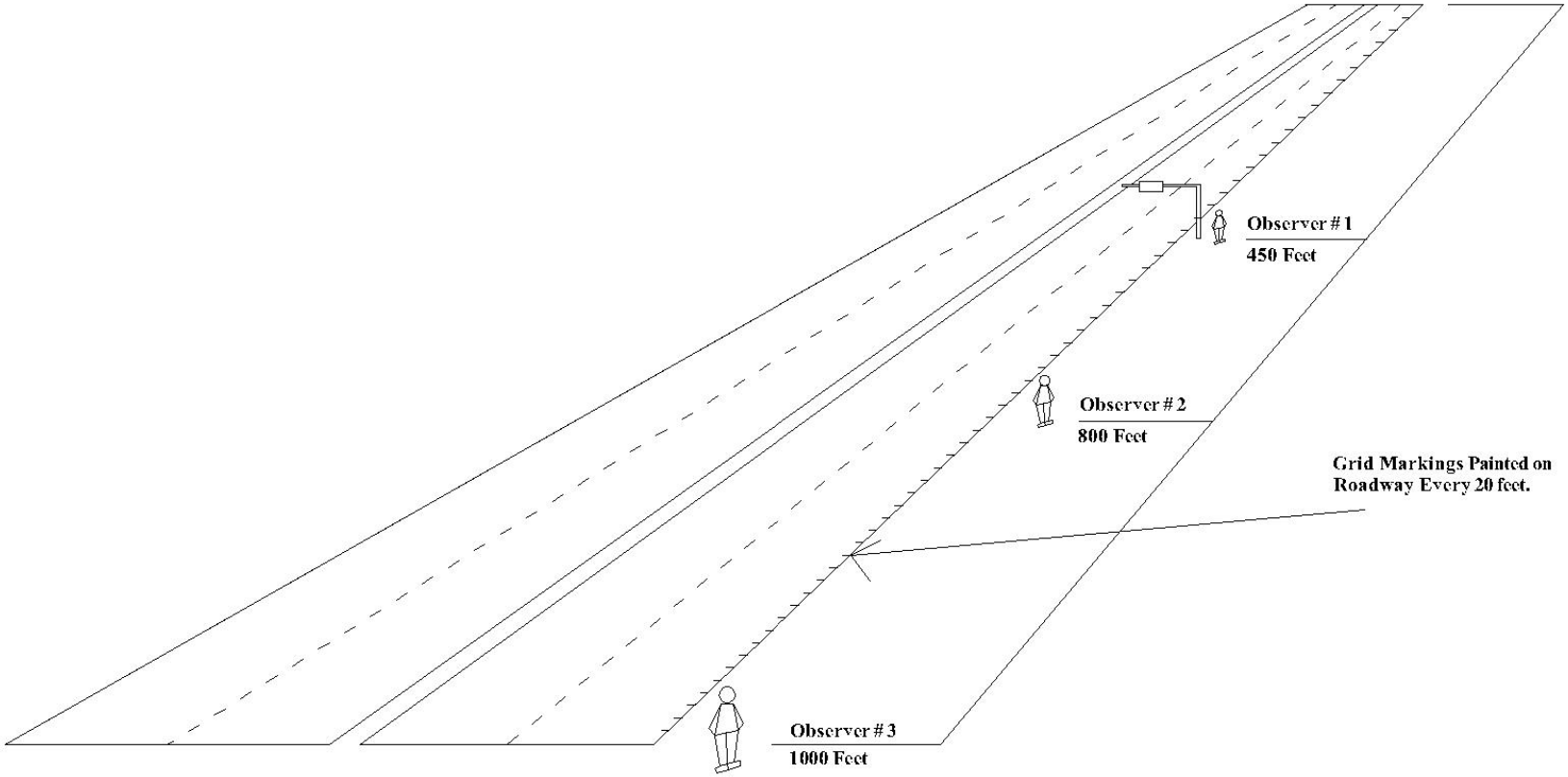

Figure 4-7 Location of DCZ study site observers.

Name:
Date:
Time In:
Time Out:
Intersection:
Approach:
Weather:

	Go/Stop (G/S)	Signal Condition (R/Y)	AWS Condition (F/B)	Distance from Intersection (ft)	Passenger Vehicle (C/T)	Heavy Truck (Y/N)	Time (Hour:Min)	Condition of Signal Upon Arrival at Stop Bar (red = R, yellow = Y)
1								
2								
3								
4								
5								
6								
7								
8								
9								
10								
11								
12								
13								
14								
15								
16								
17								
18								
19								
20								

Figure 4-8 Observation sheet for DCZ observers.

Observation data were collected up to a maximum distance from the intersection corresponding to the location of the observer farthest upstream of the intersection (1,000 feet). Some heavy vehicles already had their brakes on when they reached the observer located at 1,000 feet in which case the observer recorded 1,000 feet as the location that brakes were applied. Empirical evidence suggested that the number of vehicles whose applied braking distance measurements might actually have been farther back than what was recorded was low and only accounted for a handful of vehicles out of thousands of observations.

As outlined in Chapter 2, previously published DCZ studies were conducted on primary approaches to intersections with speed limits of 50 mph or less. Time-lapse or video photography devices were used to observe vehicles and were generally placed within 300 feet to 450 feet of the intersection. The DCZ study site for this research was an HSSI with approach speeds approximately 60 mph and a high volume of heavy

vehicles. Due to the unique high speeds encountered at the study site and a lack of high-tech observation equipment, minor modifications had to be made to the way that data were collected. The higher approach speeds of the study site meant that more than one observation point was needed to account for larger stopping distance requirements of vehicles approaching the intersection.

Based on preliminary observations, researchers determined that it would be necessary to station one of the observers as far back 1,000 feet from the intersection in order to see the application of the brake lights of the vehicles coming to a stop at the HSSI. However, placing an observer at 1,000 feet was still insufficient to collect all of the necessary data. For example, data recorded for the first vehicle in each lane that stopped after the onset of the yellow change interval should have included the distance that the vehicles were at the moment the signal turned yellow. Instead, observers were only able to observe the location of stopping vehicles when their brake lights were applied.

A lack of data on the location of stopping vehicles at the onset of the yellow change interval meant that a true representation of the size and location of the DCZ may not have been possible because the data may have only served to illustrate where motorists were most comfortable applying their brakes. The data may not have been adequate enough to identify the maximum distance from the intersection that motorists determined was sufficient to be able to comfortably and safely stop at the onset of the yellow change interval. On the other hand, recorded brake application distances might have been sure signs that motorists saw and reacted to the yellow change interval; whereas, observations based solely on distance from the intersection at the onset of the yellow change interval might have been unable to provide researchers with a determination of true motorist intention. For example, motorists as far back as 700 feet may not have been paying as much attention to the intersection, during the period between activation of the AWS signs and the actual onset of the yellow change interval, making it difficult to determine where their decision to stop or proceed through the intersection was actually made. Recorded brake application distance data comparisons of P1 and P2 data still allowed for inferences to be drawn between the BODAWS sign and

motorist understanding of the impending signal change. Vehicles that were observed to be coasting before they put their brake lights on were not counted.

4.4 Implementation Chapter Summary

BYU was contracted by UDOT to conduct an analysis of the effectiveness of the BODAWS system at reducing the frequency of RLR and crashes at the intersection of Bangerter Highway and 13400 South. BYU partnered with Wavetronix LLC to use state-of-the-art equipment to gather speed and RLR data at the study sites. The equipment included non-intrusive Digital Wave Radar™ technology that allowed BYU to continuously collect speed and RLR data before and after the BODAWS system was installed. The data collection equipment was installed on both the northbound and southbound approaches to the study site intersection. UDOT crews conducted RLR observations studies to verify the data collected by the radar sensors. Crash data were obtained from the local law enforcement agency with jurisdiction over the study site. Crash data were also collected at a control intersection with similar geometric design and approach volumes.

BYU researchers also conducted a DCZ study to determine the size and location of the DCZ on both approaches to the study site. The study was conducted before and after installation of the BODAWS system. The following chapter details the results of the study data.

5 Results

This chapter contains the output of the research project including the speed, RLR, crash, and DCZ study results in both graphical and quantitative forms. The study results were analyzed according to common statistical procedures. The data presented in this chapter were gathered during time periods P1, P2, and P3. The data gathered during the study were analyzed according to the evaluation metrics proposed by BYU and UDOT. A discussion of the data is provided in Chapter 6.

The results chapter is divided into the following sections:

5.1 Speed Data Results – presents the speed data results for P1, P2, and P3.

5.2 RLR Data Results – describes the results of the RLR data gathered during P1, P2, and P3 through the evaluation equipment and by UDOT.

5.3 Crash Data Results – compares the crash data from the study site with a control site for P1 and P2.

5.4 DCZ Study Results – summarizes the results of the DCZ study conducted during P1 and P2.

5.5 Results Chapter Summary – summarizes the main points of the chapter.

Due to the large number of comparisons and tests that were conducted, the main body of this report only contains a small sample of the results that have been obtained. The remaining study results are contained in the appendices of this report.

5.1 Speed Data Results

Speed data results are organized spatially by detection zone. Chronological data are referenced using the onset of the all-red interval (time zero) as the datum with each 1 second interval prior to the datum recorded as the number of seconds before red (SBR). Cumulative speed data plots, box plots, and statistical t-tests for each sensor zone on both the northbound and southbound approaches to the study site were produced using the program Statistical Analysis Systems (SAS) version six (*48*). The statistical results of the speed data are described in the following subsections.

5.1.1 Speed Data Cumulative Plot Results

Cumulative distribution plots of the speed data were created for each sensor zone on both approaches during P1, P2, and P3. The ordinate of each plot is the cumulative frequency and the abscissa is the vehicle speeds. Each line in the plot represents a cumulative vehicle speed distribution by the number of SBR.

Cumulative distribution plots were created to visually compare the speed trends of vehicles approaching the intersection. A visual comparison of the speed distributions allows trends to be identified that illustrate motorist driving behavioral patterns before and after installation of the BODAWS system. The cumulative distribution plots also make it easier to identify the 85th percentile speeds of approaching motorists.

Cumulative distribution plots of vehicle speeds for the AM peak on the northbound approach to the study site at a detection zone located 100 feet from the intersection during P1, P2, and P3 are illustrated in Figure 5-1, Figure 5-2, and Figure 5-3, respectively. The remainder of the cumulative distribution plots can be found in Appendices B, C, and D of this report. Appendices B, C, and D contain cumulative distribution plots at each sensor zone during P1, P2, and P3, for both the northbound and southbound approaches to the study site during the AM peak, noon peak, and the PM peak, respectively.

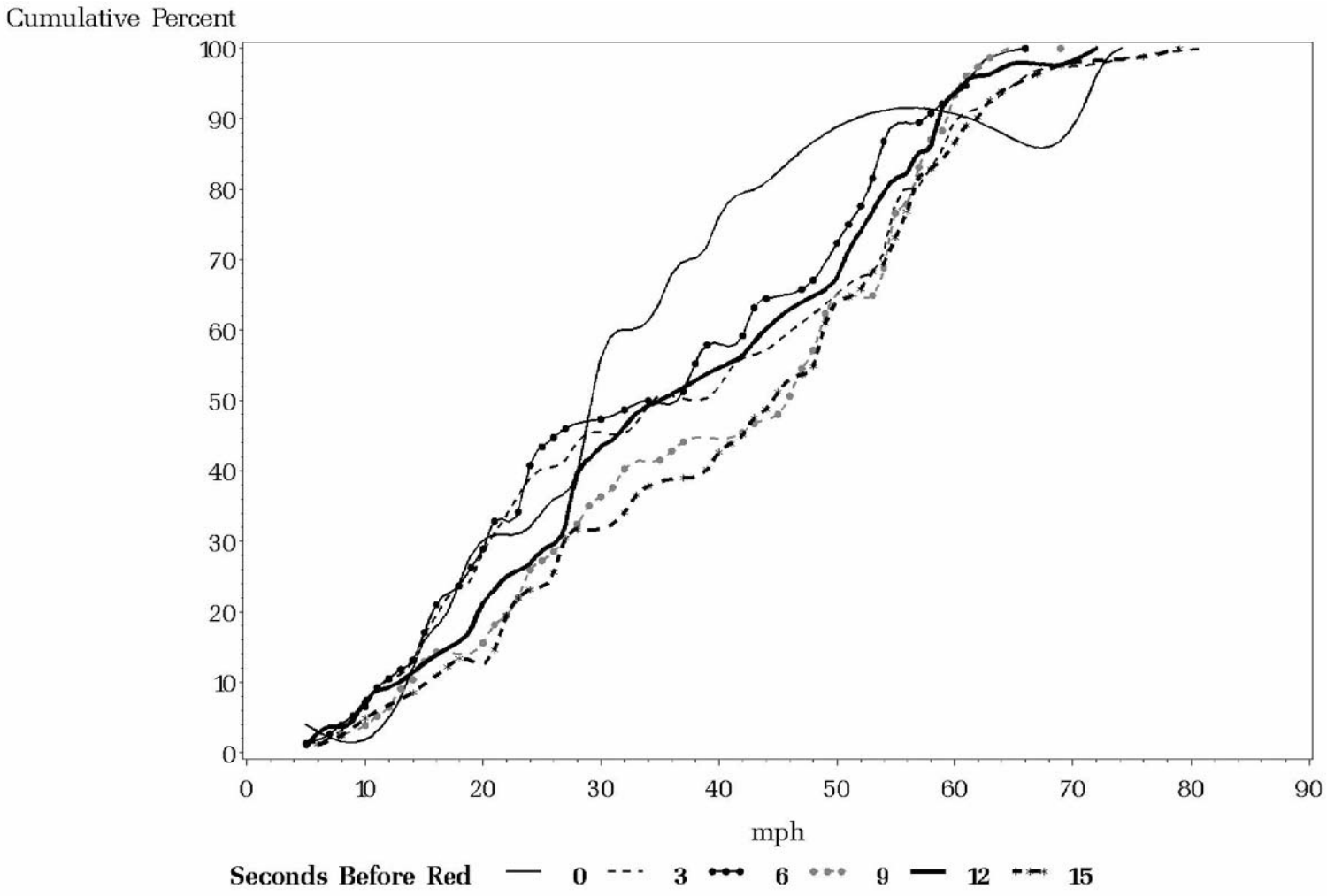

Figure 5-1 Cumulative distribution plot for northbound AM peak speeds at the 100 ft. detection zone for P1.

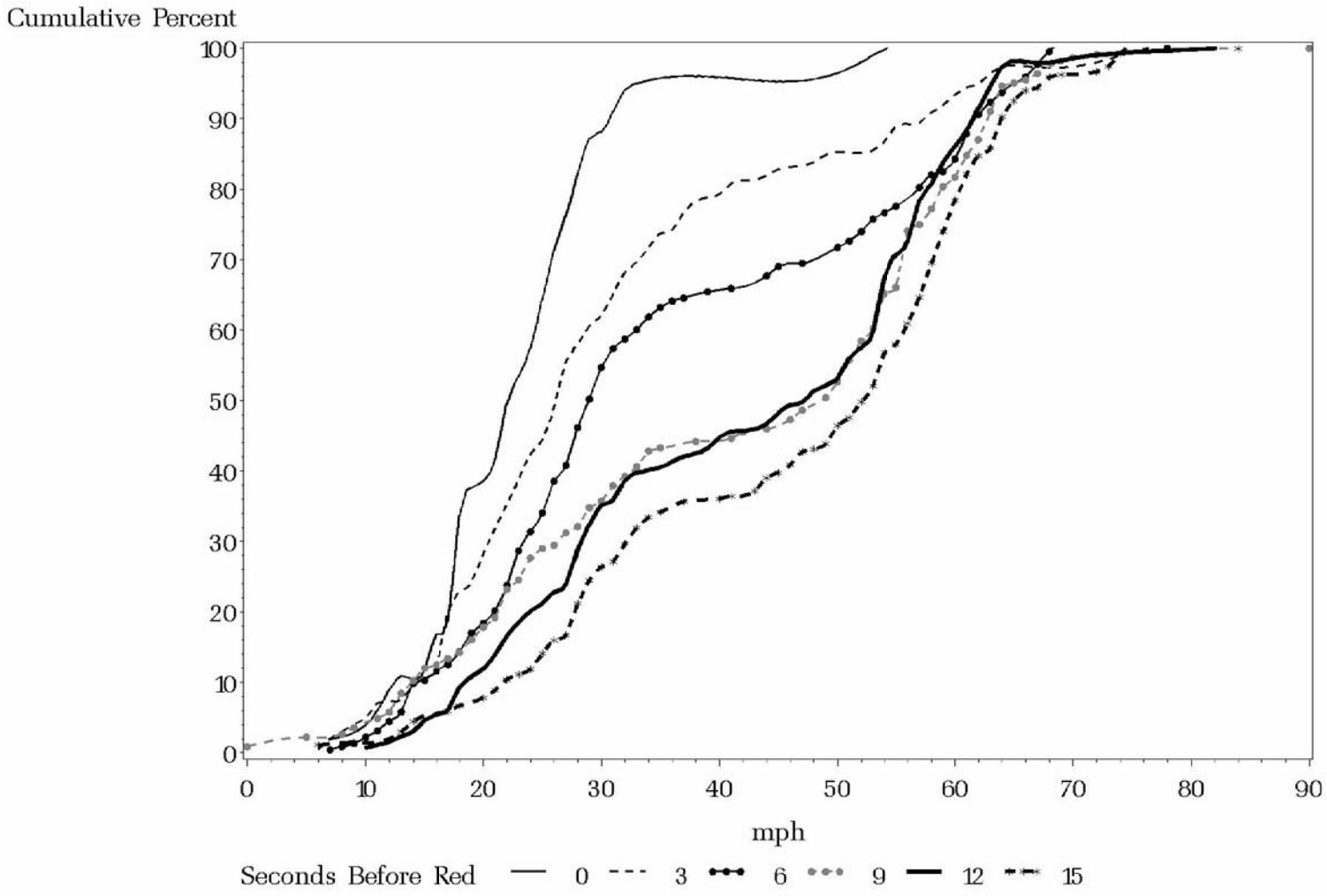

Figure 5-2 Cumulative distribution plot for northbound AM peak speeds at the 100 ft. detection zone for P2.

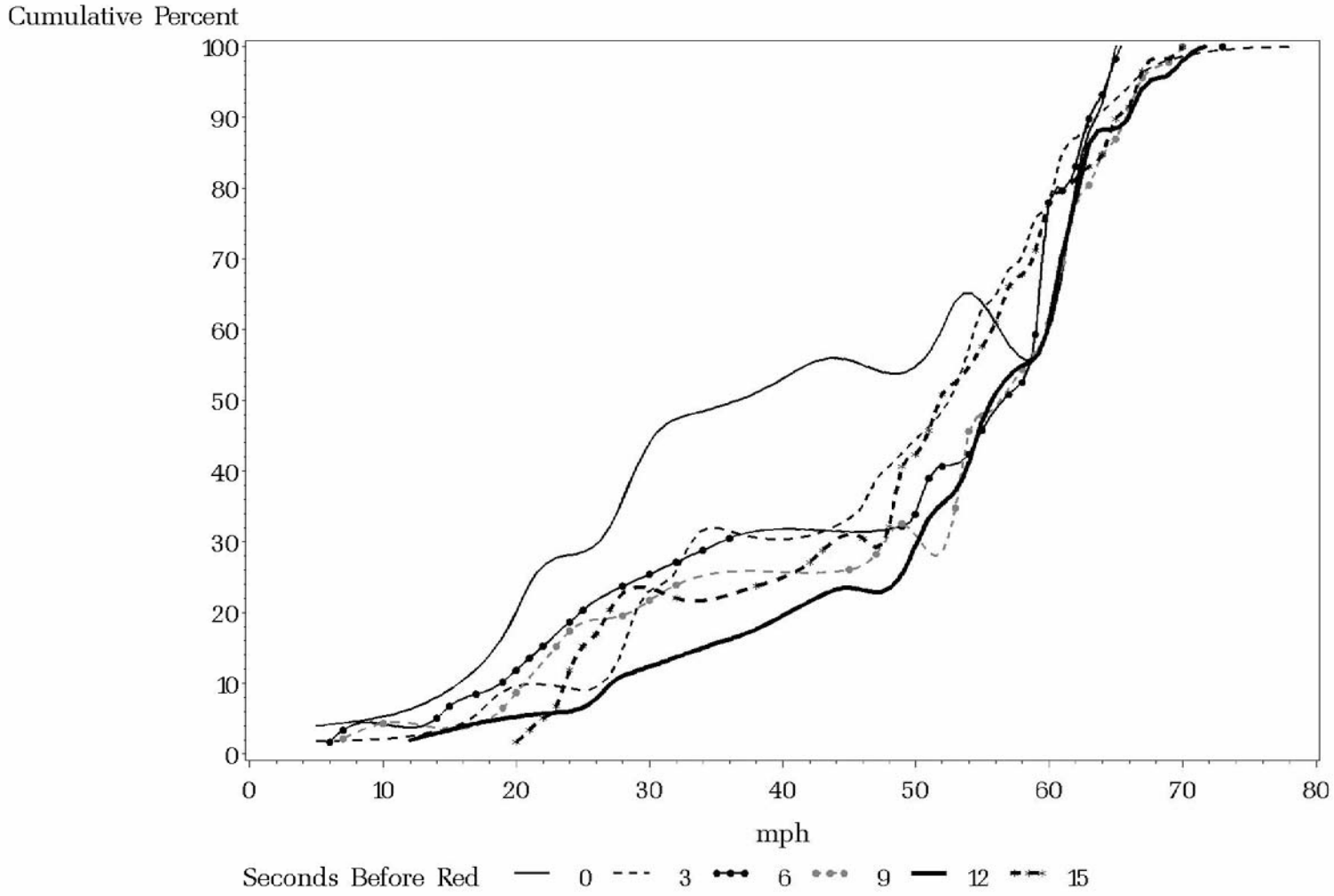

Figure 5-3 Cumulative distribution plot for northbound AM peak speeds at the 100 ft. detection zone for P3.

The cumulative speed data plots from P1, P2, and P3 illustrate that the 85th percentile speed of approaching vehicles varies from study period to study period and by the number of SBR. Table 5-1 lists the highest and lowest 85th percentile speeds from the cumulative speed distribution plots for P1, P2, and P3 at the 100 foot sensor zone on the northbound approach to the study site during the AM peak. The values in Table 5-1 are generated from the results illustrated in Figure 5-1, Figure 5-2, and Figure 5-3.

Table 5-1 Lowest and Highest 85th Percentile Speeds for P1, P2, and P3 at the 100 foot sensor zone on the northbound approach during the AM peak

Study Period	Low 85th Percentile Speed (mph)	High 85th Percentile Speed (mph)
P1	44	60
P2	25	62
P3	60	64

Vehicle speed distributions in Figure 5-1 illustrate that during P1 motorists traveled at lower and more uniform speeds regardless of the number of SBR. The spread of 85th percentile speeds in Table 5-1 for P1 shows that from the lowest to highest values there was a spread of only 16 mph. Similar results were noted for nearly all other detection zones during P1 and are illustrated in Appendices B, C, and D of this report.

Vehicle speed distributions in Figure 5-2 indicate that during P2 motorists traveled at higher speeds before activation of the BODAWS system (16 to 12 SBR), while traveling at lower speeds after the onset of the yellow change interval (6 to 0 SBR). The spread between the lowest and the highest 85th percentile speeds increased to 37 mph. Similar results were noted for nearly all of the other detection zones during P2 and are illustrated in Appendices B, C, and D of this report. Vehicle speed distributions in Figure 5-3 indicate that motorists during P3 increased their speeds throughout all of the detection zones during each SBR. Table 5-1 illustrates that the spread between the lowest and highest 85th percentile speeds is only about 4 mph. Similar results were noted

for nearly all of the other detection zones and time periods and are illustrated in Appendices B, C, and D of this report.

A discussion of the meaning of the trends shown in the cumulative distribution plots can be found in Chapter 6 of this report. While cumulative distribution plots are helpful at identifying speed trends, box plots were also developed to identify the spread of vehicle speeds. The following subsection describes speed data box plot results.

5.1.2 Speed Data Box Plot Results

Box plots are used to visually identify the median vehicle speeds and the spreads of the speeds during each period. The speed data used for the box plot in this subsection is the same data used for the cumulative distribution plots in Section 5.1.1 of this report. The speed data box plots for the other time periods and study periods are similar to the data illustrated in Figure 5-4 and are contained in Appendices E, F, and G for the data collected during P1, P2, and P3 for the AM, noon, and PM traffic peaks, respectively.

Box plots were created for each detection zone on both approaches to the study site to compare speeds during P1, P2, and P3. Box plots are useful to compare the results of multiple samples side by side. A box plot is divided into three main parts. The first part of a box plot is the box itself which represents 50 percent of the data closest to the median. The median is the second part of the box plot which is a line that is drawn horizontally through the box. The area of the box plot from the median to the upper boundary of the box represents the upper quartile (25 percent) of the data. The area of the box plot from the median to the lower boundary of the box represents the lower quartile (25 percent). The third part of the box plot is the whiskers which extend from the upper and lower boundaries of the box plot to the outermost data points graphed with the box plot. The whiskers represent the other 50 percent of the data. The overall shape and size of the box plot as well as the lengths of the whiskers help researchers to see how the data is spread about the median (*49*).

The ordinate of the box plot graph represents vehicle speed and is subdivided into increments of miles per hour. The abscissa of the box plot graph is the number of SBR in 3 second increments starting from 15 SBR and ending 0 SBR. Box plots for P1, P2, and

P3, are illustrated side by side for each 3 SBR interval of the abscissa starting with P1 and ending with P3. Figure 5-4 contains a sample box plot comparison of the P1, P2, and P3 speed data for the northbound approach to the study site at the sensor zone located 100 feet upstream of the intersection during the AM peak.

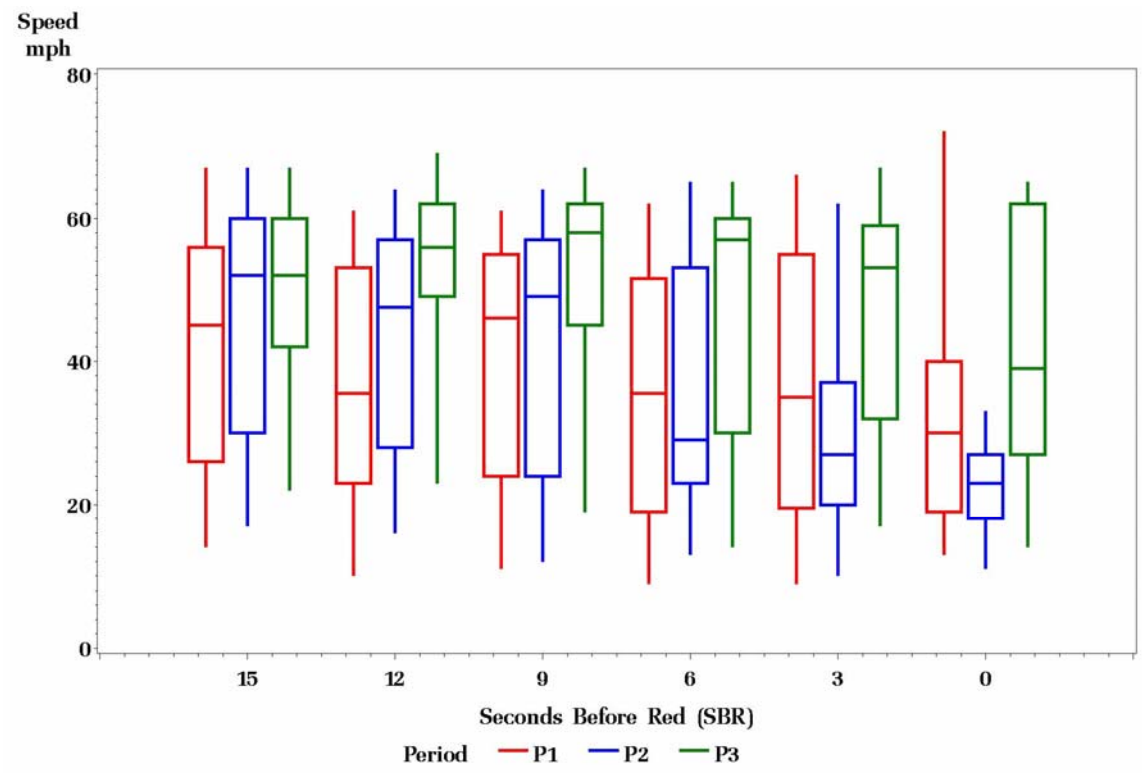

Figure 5-4 Box plots comparison P1, P2, and P3 speed data for the northbound 100 ft. detection zone during the AM peak.

Figure 5-4 illustrates the difference in the speed distributions between P1, P2, and P3. The most notable result illustrated in Figure 5-4 is the increase of vehicle speeds during P3 above the speeds recorded during P1 and P2. Similar results were noted for nearly all detection zones and time periods analyzed and are illustrated in Appendices E, F, and G of this report.

A discussion of the implications of the box plot data results can be found in Chapter 6 of this report. The following subsection deals with speed data probability grids that graphically represent statistically significant changes in speed data.

5.1.3 Speed Data Probability Grids

The cumulative distribution plots were helpful in identifying speed trends while the box plots were helpful in identifying means speeds and spreads. However, the cumulative speed plots and box plots do not identify statistical significances in differences between the speed data between time periods.

A statistical comparison of speed distributions between P1 and P2 was completed by analyzing and comparing nearly 410 *t*-test combinations. The output of statistical data for the northbound approach to the study site is summarized in Table 5-2 for the AM peak period, Table 5-3 for the noon peak period, and Table 5-4 for the PM peak period. Each row in the grids represents the location of a sensor zone on the approach. Each column in the grids represents the number of SBR that a speed comparison was made. Grids that are shaded gray represent an increase in speeds between P1 and P2 that are statistically significant at a 95 percent confidence level. Grids that are shaded black represent a decrease in speeds between P1 and P2 that are statistically significant at a 95 percent confidence level. Grids that are not shaded signify no statistical difference in speeds between P1 and P2 although they do not communicate whether speeds increased, decreased, or remained the same.

There appears to be a statistical significance between P1 and P2 speeds on both approaches to the intersection at nearly every sensor zone except when vehicles are approaching the intersection between 12 and 7 SBR (the period between activation of the BODAWS signs and flashers and the onset of the yellow change interval). A discussion of the statistical significance of the speed data can be found in Chapter 6 of this report.

Table 5-2 Statistical Significance Grid for Northbound Speed Data for Weekday AM Peak Traffic (P1 vs. P2)

	Seconds Before Red (SBR)																
Zone (feet)	16	15	14	13	12	11	10	9	8	7	6	5	4	3	2	1	0
300																	
200																	
100																	
50																	

Note: Gray squares = P2 > P1 and black squares = P2 < P1

Table 5-3 Statistical Significance Grid for the Northbound Speed Data for Weekday Noon Peak Traffic (P1 vs. P2)

	Seconds Before Red (SBR)																
Zone (feet)	16	15	14	13	12	11	10	9	8	7	6	5	4	3	2	1	0
300																	
200																	
100																	
50																	

Note: Gray squares = P2 > P1 and black squares = P2 < P1

Table 5-4 Statistical Significance Grid for the Northbound Speed Data for Weekday PM Peak Traffic (P1 vs. P2)

	Seconds Before Red (SBR)																
Zone (feet)	16	15	14	13	12	11	10	9	8	7	6	5	4	3	2	1	0
300																	
200																	
100																	
50																	

Note: Gray squares = P2 > P1 and black squares = P2 < P1

Similar trends in vehicle speeds were discovered in the southbound data and can be found in the Appendix H of this report. The recorded southbound speed data contains a high number of right-turning vehicles that were recorded by the radar due to the skewed geometry of the intersection. The radar sensors were not able to distinguish the right-turning vehicles from the through vehicles and the sensors could not be rotated to avoid recording the right-turning vehicles. Therefore, the speeds of the slower right-turning vehicles were combined with the speeds of the through vehicles which lowered the overall speeds recorded on the approach. However, P1 and P2 southbound data were still analyzed in the same manner as the northbound approach because it was assumed that the number of right turning vehicles remained constant during P1 and P2. Therefore, trends in southbound vehicle speeds were analyzed for statistical significance and found to be similar to the northbound speed results illustrated in the northbound statistical significance grids. The northbound approach did not have the same problem as the

southbound approach because the number of right-turning vehicles on the northbound approach was insignificant during the entire day.

A statistical comparison of speed distributions between P1 and P3 was also conducted. Table 5-5, Table 5-6, and Table 5-7 contain speed probability grids for the northbound approach to the study site during the AM, noon, and PM peaks, respectively. The purpose of the comparison of speed distributions between P1 and P3 was to determine if vehicle speeds during P3 reverted back to what was seen during P1. Statistical significance between speeds from P1 and P3 predominately occur between 11 and 0 SBR which means that vehicles speeds have increased during P3. The speeds between 16 to 11 SBR, however, tend not to be statistically different between P1 and P3 for the noon and PM peak traffic. Appendix H contains statistical probability grids for the southbound data. A discussion of the speed data changes or similarities between P1 and P3 can be found in Chapter 6 of this report.

Table 5-5 Statistical Significance Grid for the Northbound Speed Data for Weekday AM Peak Traffic (P1 vs. P3)

	Seconds Before Red (SBR)																
Zone (feet)	16	15	14	13	12	11	10	9	8	7	6	5	4	3	2	1	0
300																	
200																	
100																	
50																	

Note: Gray squares = P3 > P1 and black squares = P3 < P1

Table 5-6 Statistical Significance Grid for the Northbound Speed Data for Weekday Noon Peak Traffic (P1 vs. P3)

	Seconds Before Red (SBR)																
Zone (feet)	16	15	14	13	12	11	10	9	8	7	6	5	4	3	2	1	0
300																	
200																	
100																	
50																	

Note: Gray squares = P3 > P1 and black squares = P3 < P1

Table 5-7 Statistical Significance Grid for the Northbound Speed Data for Weekday PM Peak Traffic (P1 vs. P3)

	Seconds Before Red (SBR)																
Zone (feet)	16	15	14	13	12	11	10	9	8	7	6	5	4	3	2	1	0
300																	
200																	
100																	
50																	

Note: Gray squares = P3 > P1 and black squares = P3 < P1

5.1.4 Speed Data Results Section Summary

Cumulative distribution plots, box plots, and probability grids were created to visually and statistically compare the speed data results from P1, P2, and P3. Overall, it appears from the speed comparisons that motorists slowed down immediately after installation of the BODAWS system (P2) but increased their speeds to speeds that are in some instances higher than before installation of the BODAWS system after eight months (P3). It also appears that motorists during P2 slowed down the most between 6 and 0 SBR, or the time period between the onset of the yellow change interval and the all-red clearance interval. However, during P3 motorists were traveling through the intersection at higher and more uniform speeds during each SBR. The following section details the RLR data results.

5.2 RLR Data Results

RLR data were collected using the radar sensors and the data logger data at the study site and from hand counts conducted by UDOT technicians. This section is divided into two subsections describing the results of the RLR data collected from the electronic equipment and the RLR data collected by UDOT, respectively.

5.2.1 Electronic RLR Data Results

RLR data were collected from the sensors and the data logger data at the study site. In order to determine if a RLR event occurred, timestamps from the speed data at the stop bar sensor zone of each approach were compared to the timestamps for the onset of the red signal in each direction. Due to the high number of right turning vehicles on the southbound approach that were recorded by the radar sensors, a filter was applied to the data using a 4 second time-after-red limit and a 20 mph minimum speed. A statistical analysis of the RLR data was performed using the statistical software program SAS (*48*). Results of a comparison of P1, P2, and P3 RLR data are illustrated in Table 5-8. More data can be found in Appendix I of this report.

Table 5-8 RLR Data Results

Study Intersection Approach	Study Period	Number of Through Vehicles	Number of RLR Violations	RLR Violations (Per 1,000 Entering Vehicles)
NB	P1	23,855	125	5.24
	P2	126,351	142	1.12
	P3	23,409	193	8.24
SB	P1	12,285	21	1.71
	P2	56,096	39	0.70
	P3	12,435	9	0.72

The number of through vehicles in Table 5-8 varies from period to period because the number of days that the radar sensors were collecting data also varied in each period. Although the number of through vehicles is not equal period to period, the number of RLR violations was calculated per thousand entering vehicles so that a comparison among rates could take place. As illustrated in Table 5-8, the number of RLR events per thousand entering vehicles on the northbound approach decreased from P1 to P2 but increased between P2 and P3. The number of RLR events per thousand entering vehicles

on the southbound approach decreased from P1 to P2 with the number of events rising slightly during P3. Each change in the number of RLR events between P1, P2, and P3 on both the northbound and southbound approaches was statistically significant at a 95 percent confidence level. Overall, the number of RLR violations per thousand entering vehicles is higher on the northbound approach than it is on the southbound approach.

Figure 5-5 contains a cumulative distribution comparison of the speeds of RLR violations during P1, P2, and P3 for both northbound and southbound RLR vehicles. Figure 5-5 illustrates that the speeds of RLR motorists decreased during P2 and continued to decrease during P3.

Figure 5-6 illustrates a cumulative distribution of the number of seconds of time-after-red that vehicles that run the red-light are entering the intersection during P1, P2, and P3. Figure 5-6 illustrates that more motorists started committing RLR violations farther into the red during P2, and P3. Because the all-red clearance interval at the intersection is only 2 seconds long it appears that almost 30 percent of RLR violations occurred during P1 and P2 when the conflicting traffic had already received the green.

A discussion of the RLR data results can be found in Chapter 6 of this report. The following subsection discusses the hand counts that were conducted by UDOT to verify the RLR data collected by the radar sensors.

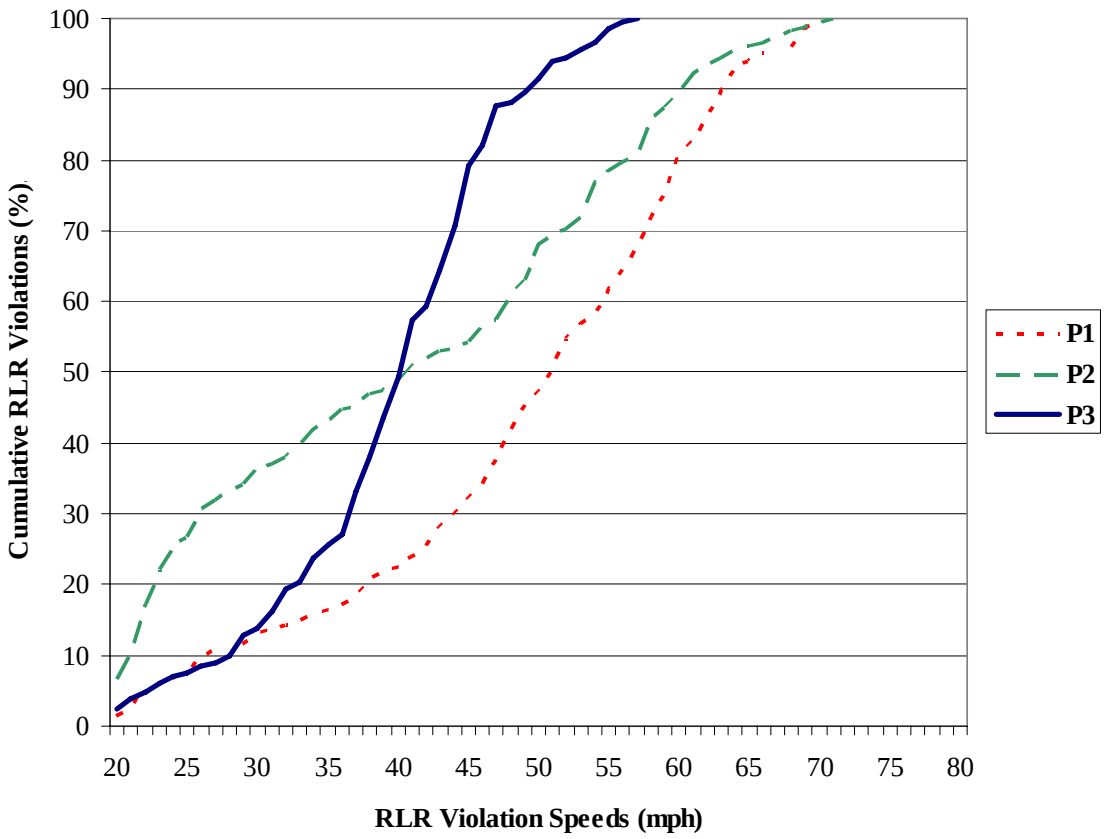

Figure 5-5 Cumulative distribution of RLR violation speeds during P1, P2, and P3.

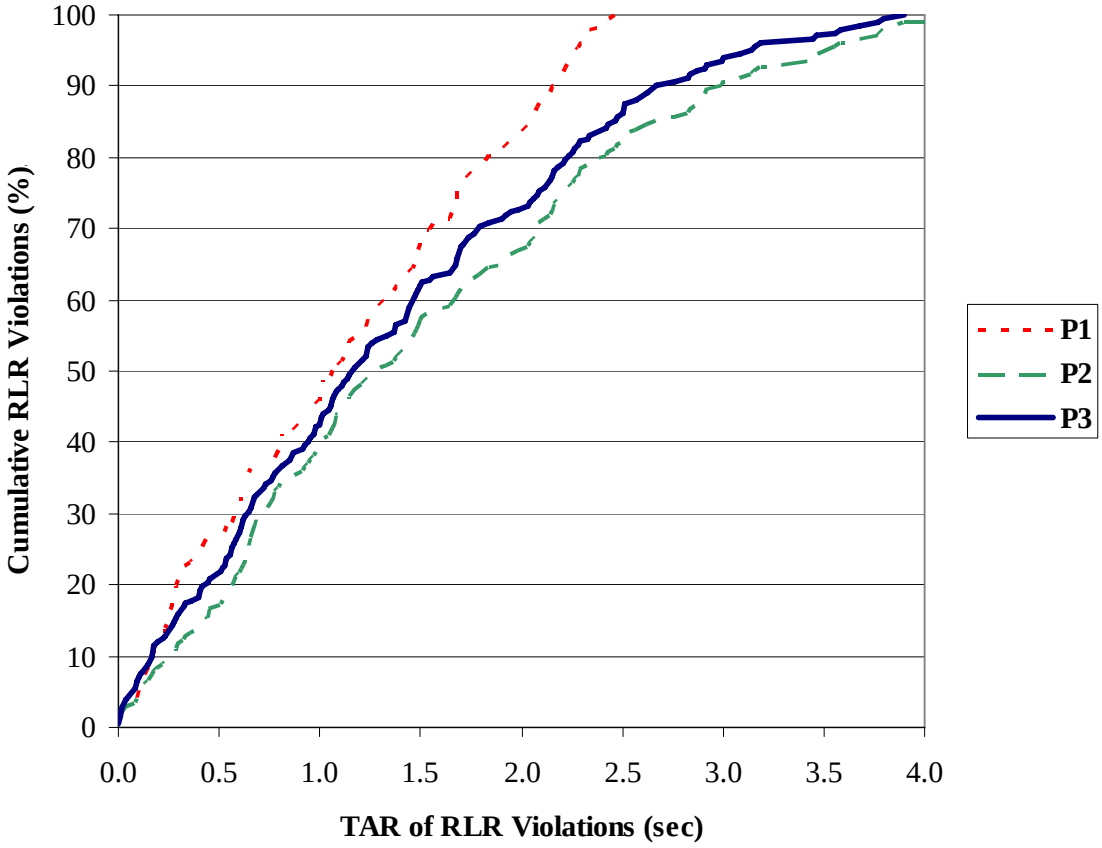

Figure 5-6 Cumulative distribution of the time-after-red of RLR vehicles during P1, P2, and P3.

5.2.2 UDOT RLR Hand Count Data Results

The UDOT Traffic and Safety Division conducted RLR observation studies at the study site intersections in order to verify the RLR data collected by the radar sensors. UDOT conducted studies at random peak periods during P1, P2, and P3. UDOT employees counted RLR in the through lanes of the Bangerter Highway approaches to the study site. The counts were conducted in 15 minute intervals for one hour at a time. The RLR data at each intersection were then summarized as the percentage of total through vehicles that entered the intersection on red for each hour. Table 5-9 contains a sample of the hand count data collected by UDOT.

Table 5-9 UDOT RLR Hand Counts

Study Period	Number of Thru Vehicles	Number of RLR Violations	RLR Violations (Per 1,000 Entering Vehicles)
P1	530	4	7.55
P2	946	6	6.34
P3	407	5	12.29

Because the RLR data collected by the radar sensors was collected by approach, the RLR results for each period were listed by approach in Table 5-8. However, the UDOT hand counts did not distinguish between RLR events by approach but counted the total number of RLR events on Bangerter Highway. Therefore, to compare the sensor data with the UDOT hand counts, the sensor data RLR results for each approach were added together and combined into counts for both directions of the study site and are listed in Table 5-10.

Table 5-10 Sensor RLR Counts

Study Period	Number of Thru Vehicles	Number of RLR Violations	RLR Violations (Per 1,000 Entering Vehicles)
P1	36,140	146	4.04
P2	182,447	181	0.99
P3	35,844	202	5.64

The number of RLR events recorded by the sensor data and from the UDOT hand counts show that the number of RLR violations decreased from P1 to P2 but increased from P2 to P3 to levels that were higher than before the BODAWS system was installed. As indicated previously, a discussion of the RLR data results can be found in Chapter 6

of this report. The following section presents the crash data results for the study site and a control intersection of similar geometry and approach volumes.

5.3 Crash Data Results

Crash data were collected from UDOT and from the Salt Lake County Sheriff's Department branch office responsible for the study intersection and a similar control intersection also located on Bangerter Highway at 12600 South. Crash rates from the study site and the control intersection are listed in Table 5-11 and Table 5-12, respectively. The data in the tables are reported in crashes per million entering vehicles (MEV) and are based on crash statistics from the last six months of each year due to the fact that only six months of crash data after BODAWS installation were available for the year 2005. Crash results are separated into three crash type categories including rear-end, right-angle, and other crashes as outlined previously. The crash data are also separated in Table 5-11 and Table 5-12 into crashes that involve at least one vehicle on Bangerter Highway (Bangerter Only Crash Rates) and all of the crashes that occurred at the intersection regardless of whether or not a vehicle on Bangerter Highway was involved (Intersection Crash Rates).

Detailed crash data including: 1) crash statistics for right-angle, rear-end, and other crashes; 2) crash rates per million entering vehicles calculated using Equation 3-4; and 3) UDOT average annual daily traffic counts for all of the approaches to the study site and the control intersection are provided in Appendix J of this report. The UDOT average annual daily traffic counts were found on the UDOT traffic statistics website (*47*). When average annual daily traffic counts for one of the study period years was not available for an intersection approach the average annual daily traffic of the previous and following years was averaged.

A comparison of the crash rates between the study site and the control intersection show that the number of crashes at both the study site and the control intersection increased during the last six months of 2003 and 2004 but decreased slightly in the last six months of 2005. Table 5-13 and Table 5-14 combine the crash rate data for the last six months of 2002, 2003, 2004 under the "Before" column so that the it can easily be

compared to the data from the last six months of 2005 listed in the “After” column for both the Bangerter Highway only crash rates and the crash rates of the entire intersection. A discussion of the crash data results can be found in Chapter 6 of this report.

Table 5-11 Yearly Crash Rates at the Study Site

Collision Type	Bangerter Only Crash Rates (MEV)				Intersection Crash Rates (MEV)			
	2002	2003	2004	2005	2002	2003	2004	2005
Rear-end	0.55	0.29	1.26	0.25	1.17	1.20	1.26	0.32
Right-angle	0.55	0.57	0.25	0.25	0.67	0.34	0.16	0.47
Other	0.28	0.57	0.25	0.00	0.33	0.17	0.32	0.32
Total	1.38	1.43	1.76	0.50	2.17	1.71	1.73	1.10

Table 5-12 Yearly Crash Rates at the Control Intersection

Collision Type	Bangerter Only Crash Rates (MEV)				Intersection Crash Rates (MEV)			
	2002	2003	2004	2005	2002	2003	2004	2005
Rear-end	0.00	0.00	0.20	0.00	0.55	0.14	0.42	0.00
Right-angle	0.23	0.95	1.01	0.81	0.41	0.69	0.69	0.87
Other	0.23	0.48	0.40	0.00	0.14	0.28	0.28	0.12
Total	0.47	1.43	1.62	0.81	1.10	1.11	1.39	1.00

Table 5-13 Before and After Crash Rates at the Study Site.

Collision Type	Bangerter Only Crash Rates (MEV)		Intersection Crash Rates (MEV)	
	Before	After	Before	After
Rear-end	1.14	0.25	3.15	0.32
Right-angle	0.72	0.25	1.00	0.47
Other	0.57	0.00	0.72	0.32
Total	2.43	0.50	4.86	1.10

Table 5-14 Before and After Crash Rates at the Control Intersection.

Collision Type	Bangerter Only Crash Rates (MEV)		Intersection Crash Rates (MEV)	
	Before	After	Before	After
Rear-end	0.11	0.00	1.23	0.00
Right-angle	1.12	0.81	0.45	0.87
Other	0.56	0.00	0.45	0.12
Total	1.79	0.81	2.12	1.00

5.4 DCZ Study Results

The DCZ study data were analyzed using the statistical analysis software program SAS (*48*). The 90 percent and 10 percent probability braking distances were calculated for heavy vehicles and passenger vehicles on both the northbound and southbound approaches during P1 and P2. The probability of stopping distances were calculated using a logit model of statistically significant parameters. Independent relationships between study parameters were evaluated using a Chi-Square test. Parameters with relationships found to be statistically significant at 95 percent confidence level included:

- Distance,
- Distance and passenger cars,
- Distance and passenger trucks, and
- Distance and heavy vehicles.

The function used to determine the probability that a vehicle will stop based on distance from the intersection at the onset of the yellow change interval is shown in Equation 5-1. The logit utility estimate can be calculated using Equation 5-2.

$$P = \left(1 - \frac{e^U}{1 + e^U}\right) \times 100 \qquad (5\text{-}1)$$

where: P = probability that vehicle will stop, and

U = logit utility estimate.

$$U = -0.0214D - 0.00018DC_d + 0.00243DT_d - 0.00206DH_d + 0.00182P + 7.313 \quad (5\text{-}2)$$

where: U = logit utility estimate,

D = distance from intersection at the onset of the yellow change interval (ft),

C_d = passenger car utility (passenger car = 1, else 0),

T_d = passenger truck utility (passenger truck = 1, else 0),

H_d = heavy vehicle utility (heavy vehicle = 1, else 0), and

P = study period (P1 = 0, else 1).

The parameter estimates for Equation 5-2 were obtained using maximum likelihood estimates in SAS Proc Logistic (*48*). To calculate the probability that a motorist in a passenger car would choose to stop when the motorist was only 237 feet from the intersection at the onset of the yellow change interval before BODAWS installation, the following values would be input into Equation 5-2: 1) D = 237; 2) C_d = 1; 3) T_d = 0; 4) H_d = 0; and 5) P = 0. Equation 5-2 would yield the logit utility estimate (U) as 2.199. The logit utility estimate of 2.199 could then be used as input into Equation 5-1 which for this example would yield a probability of stopping of 10 percent.

Stopping probabilities for P1 and P2 were calculated for the study site using Equations 5-1 and 5-2 for each foot of distance from the intersection. The results of the logit model analysis output for the study site for stopping probabilities of 10 percent and 90 percent during P1 and P2 are displayed in Table 5-15. Table 5-15 shows the 10 percent and 90 percent stopping probability boundaries of the DCZ by distance from the intersection in feet. Table 5-16 shows the 10 percent and 90 percent stopping probability boundaries of the DCZ by the number of seconds of travel time from the intersection assuming an approach speed of 60 mph and using the stopping probability distances from Table 5-15. No statistical relationship was found between stopping probabilities and intersection approach or time of day therefore the data represented in Table 5-15 and Table 5-16 are for both approaches to Bangerter Highway throughout the entire day. Appendix K contains plots of the cumulative distributions of stopping probabilities for

passenger cars, passenger trucks, and heavy vehicles during P1 and P2 as well as a table of probability stopping distances for each vehicle classification type every 100 feet from the intersection and tables of passenger car probability of stopping for various speeds (55 mph, 60 mph, and 65 mph).

Table 5-15 Probability of Stopping Distances for the Study Site

Vehicle Type	10% Probability of Stopping		90% Probability of Stopping	
	P1	P2	P1	P2
	Distance (feet)	Distance (feet)	Distance (feet)	Distance (feet)
Passenger Car	237	259	440	481
Passenger Truck	218	236	405	439
Heavy Vehicle	270	298	501	554

Table 5-16 Probability of Stopping Travel Times for the Study Site

Vehicle Type	10% Probability of Stopping		90% Probability of Stopping	
	P1	P2	P1	P2
	Time (sec)	Time (sec)	Time (sec)	Time (sec)
Passenger Car	2.7	2.9	5.0	5.5
Passenger Truck	2.5	2.7	4.6	5.0
Heavy Vehicle	3.1	3.4	5.7	6.3

Note: Times are based on a 60 mph approach speed assumption.

The stopping probability results by distance and by time show that that boundaries of the DCZ changed after installation of the BODAWS system. For example, the 10 percent and 90 percent probability braking distances moved 20 feet further from the intersection after BODAWS system installation and the travel time boundaries increased

by 0.2 to 0.5 seconds of travel time from the intersection. A discussion of the significance of the DCZ study data and the reason why the DCZ was not analyzed during P3 can be found in Chapter 6 of this report.

5.5 Results Chapter Summary

Speed, RLR, and crash data were collected during P1, P2, and P3 while DCZ boundary data were collected during P1 and P2. The speed, RLR, crash, and DCZ study data were statistically analyzed to determine if the BODAWS system was effective in reducing RLR violations and crashes and to determine if the speeds of vehicles and the boundaries of the DCZ were affected by the BODAWS system. Speed data were collected by radar sensors in seven different zones on each approach to the study site. Cumulative distribution plots, box plots, and probability graphs were created for the speed data.

RLR violations were recorded by a stop bar radar sensor detection zone on each approach to the study site. UDOT employees also conducted on-site RLR observations in an attempt to verify the sensor data results. RLR violations results were calculated by the number of RLR violations per 1,000 entering vehicles.

Crash data were collected from the agencies responsible for the study site intersection and a similar control site. The crash data consisted of copies of investigating officers' crash reports as well as CSV file outputs from UDOT's crash statistics personnel. Crash data results were calculated per million entering vehicles and were based on average annual daily traffic counts provided by UDOT.

A DCZ study was conducted to determine if installation of the BODAWS system affected the size and location of the DCZ boundaries. The DCZ study occurred during P1 and P2 and was conducted on-site and by observation of the braking habits of vehicles approach the intersection. The DCZ boundaries results were provided in terms of both distance from the intersection and the number of seconds of travel time from the intersection. The following chapter discusses the implications of the results of the study.

6 Discussion of Results

This chapter relates the results of the study data to the evaluation metrics of the study and compares the results to what was expected. The results that will be discussed include: 1) speed trends; 2) RLR trends; 3) crash data results; and 4) DCZ study results. The chapter also presents hypotheses to explain the significance of the results and the outcomes produced by the results.

Due to the sheer volume and variety of data that was collected during this study, only a small portion of the results will be discussed in this report. The discussions and analysis in this report relate more to the overall trends that were observed in the data and will not attempt to answer in detail the dozens of questions that might be answered if time permitted. Further analysis of the data is suggested and future analysis ideas are presented in Chapter 7.

The Discussion of Results chapter is divided into the following sections:

6.1 Speed Trends – discusses the speed trends and comparisons for P1, P2, and P3.

6.2 RLR Trends – compares the results of the RLR data gathered during P1, P2, and P3.

6.3 Crash Comparisons – compares the crash data from the study site with a control site for P1 and P2.

6.4 DCZ Study Results – discusses the results of the DCZ study conducted during P1 and P2.

6.5 Discussions of Results Chapter Summary – summarizes the main points of the chapter.

6.1 Speed Trends

The speed data results indicate that the BODAWS system affected the speeds of motorists approaching the study site intersection. The purpose of this section is to discuss the significance of the speed data results and hypothesize the causes of the changes in speeds before and after BODAWS system installation. A discussion of the speed data leans more towards a qualitative discussion of changes in approach speed trends due to the fact that no quantitative evaluation metrics were specified by UDOT.

The section is divided into three subsections: 1) the speed trends of motorists during P1, P2, and P3 between 16 and 12 SBR; 2) the speed trends between 12 to 6 SBR; and 3) the speed trends between 6 to 0 SBR. As outlined in Chapter 5, Appendices B through G illustrate the speed trends of vehicles during P1, P2, and P3 and can be referred to throughout the discussion.

Before the discussion begins it should be noted that the estimated detection distances of the radar sensor zones were not equal to the exact physical distance from the intersection as outlined. As described previously, the geometry of the intersection was skewed because the section of the roadway that the study site was located on was a long and gentle curve. The radar waves that were emitted by the sensors emanated in a spherical pattern and did not travel parallel to the stop bar. Figure 6-1 illustrates the offset that might have existed between the estimated sensor zone detection area representing the stop bar and the actual physical location of the stop bar. It is possible that the speed data might be skewed higher due to the offset between the location that the radar designated as the stop bar and the location of the actual physical stop bar. The reason that the data may be skewed higher is because if the detector is reporting speeds (by calculating the change in distance that the vehicle makes every 2.5 ms) at some point upstream from the stop bar, the speeds may be slightly higher than if the detector were reporting the speeds at the actual physical stop bar location. The offset is estimated to be between 15 and 25 feet depending on the approach lane.

Figure 6-1 Spherical path of radar waves and skewed intersection geometry.

The geometry of the radar waves did not make it possible to completely filter out right-turning and left-turning vehicles. The southbound speed data, as recorded by the southbound sensors, were 10 to 20 mph lower than the northbound approach speeds due in theory to the large number of right-turning vehicles on the southbound approach, which during most of the day was almost equal to the number of through vehicles on the southbound approach. The northbound approach right-turning vehicle count was negligible.

The radar sensors were not capable of filtering out the right turning vehicles from the through vehicles even though the right turn lane was separated from the through lanes. Speed averages of the approach were lower because the right turning vehicles had to slow down to navigate the turn. However, the southbound speed data will still be presented in this report because the speed trends can still be analyzed assuming that the number of right turning vehicles remained constant throughout the study.

6.1.1 Speed Trends and Data Results between 16 and 12 SBR

The data results between 16 and 12 SBR indicate that the average speeds of motorists approaching the study site intersection during P2 and P3 increased between 5 to 20 mph over the speeds of P1 motorists. The large speed increases during P2 and P3 between 16 and 12 SBR were not anticipated; however, the speed increases are one of the key criteria to determine that the BODAWS system affects motorist behavior.

During P1, P2, and P3 motorists approaching the intersection between 16 and 12 SBR were approaching the intersection while the signal was still green. Motorists approaching the intersection during P1 did not on receive advance warning of impending signal changes. It is hypothesized that the large increase in the average speeds of motorists during P2 and P3 show that although the motorists approaching the intersection still did not know how long it would be before the signal was going to turn yellow, they understood that the BODAWS signs and flashers were going to provide them with advance warning of the onset of the yellow change interval before it happened.

Motorists traveling towards the intersection during P1 did not receive warning before the onset of the yellow change interval and probably approached the intersection at slower speeds because they were trying to anticipate the yellow signal and were exercising caution. The results suggest that BODAWS signs and flashers may be helpful at increasing vehicle speeds in congested corridors with HSSIs even when the signs and flashers are not active.

6.1.2 Speed Trends and Data Results between 12 and 6 SBR

Speed trends of motorists approaching the study site intersection between 12 and 6 SBR varied the least during P1, P2, and P3 of the three time periods analyzed at the intersection. The lack of variability is best explained in terms of the events that occur during this time period as will be explained.

P1 motorists, between 12 and 6 SBR, were traveling toward the intersection while the signal was green and received no warning of impending signal changes. The speeds of P1 motorists between 12 and 6 SBR remained fairly similar to the speeds of motorists

between 16 and 12 SBR and were probably lower than speeds of P2 and P3 motorists because they were exercising more caution in anticipation of the onset of the yellow change interval.

Less variability exists in the speeds of P2 and P3 motorists because of a fundamental time shift that occurs due to the design and location of the data collection equipment. For example, it is anticipated that some of the P2 and P3 motorists were between the intersection and the BODAWS signs and flashers between 12 and 6 SBR and were not affected by the activation of the BODAWS signs and flashers at 12 SBR because they did not see them activate. Furthermore, it is anticipated that other P2 and P3 motorists saw the BODAWS signs and flashers between 12 and 6 SBR and decided to accelerate through the intersection before the onset of the yellow change interval. Finally, some of the P2 and P3 motorists may have noticed the BODAWS signs and flashers between 12 and 6 SBR and decelerated to stop at the intersection.

Less variability of vehicle speeds for P2 and P3 motorists exists between 12 and 6 SBR because the speeds of motorists from each of the three categories were averaged throughout all of the sensor zones. In order to more accurately analyze the speed trends of motorists during this time slice, one would have to analyze each of the box plots starting at the 300 foot detection zone and move towards the 50 foot detection zone while also shifting the number of SBR being analyzed.

6.1.3 Speed Trends and Data Results between 6 and 0 SBR

The greatest variation in vehicle speeds between P1, P2, and P3 occurred between 6 to 0 SBR. The average speeds generally decreased between P1 and P2 but increased during P3 to levels higher that during P1 or P2. Not only did the average speeds of vehicles vary greatly between 6 and 0 SBR, but the spread of vehicles speeds grew larger, especially for the northbound approach.

It is theorized that speeds were lower during P2 than they were during P1 between 6 to 0 SBR because motorists were still adapting to the new BODAWS system and were exercising more caution after the onset of the yellow change interval. The speeds may also have been lower during P2 because motorists had already determined that they

would proceed through the intersection or stop at the onset of the yellow change interval because they had been warned with sufficient time to make such a determination.

The average speeds of motorists approaching the intersection were greater during P3 than during P2 and even increased to speeds higher than those recorded during P1. It is possible that in the months between P2 and P3 that motorists became accustomed to the BODAWS signs and flashers and used the extra information provided to them in the form of an advance warning of the impeding signal change to speed up and try to "beat" the signal as was identified in the literature review as a potential concern. Data from other studies tend to support the observation that motorists on straight-aways might gun their engines to beat the signal (*23*,). In other words, motorists may have begun to accelerate once the BODAWS signs and flashers were activated in order to enter the intersection before the signal turned red.

It is also theorized that speeds may have increased between P1 and P3 because the BODAWS signs and flashers may have been activated too far in advance of the onset of the yellow change interval. The BODAWS signs and flashers are activated 6 seconds prior to the onset of the yellow change interval. Adding to the 6 second lead time of the BODAWS signs and flashers to the 6 second duration of the yellow change interval allowed a motorist traveling at 55 mph (the design speed of the BODAWS system) or faster to be 12 seconds (1,000 feet or more) from the intersection and still be able to make it to the stop bar before the signal turned red. The 12 seconds of warning time provided by the BODAWS system may have been long enough that motorists knew they did not have to "PREPARE TO STOP" as the sign suggested. A motorist would only have had to stop at the stop bar while the signal was still yellow and wait for a few seconds before the signal actually turned red a few times before they realized that the information provided by the BODAWS system could be used to beat the light.

As described previously, the geometry of the intersection was skewed because the section of the roadway that the study site was located on is a long and gentle curve. Due to the offset that existed between the estimated sensor zone detection area representing the stop bar and the actual physical location of the stop bar, the sensor could potentially have counted vehicles as having passed the stop bar when those vehicles were still 15 to 20 feet from the actual physical stop bar painted on the road. The speeds recorded at the

sensor designated stop bar may have been higher than they would actually have been if they were recorded at the physical stop bar due to the fact the vehicles were not stopped at that point.

6.2 RLR Trends

As indicated in Chapter 5, fewer motorists committed RLR violations during P2 than during P1. A reduction in RLR may be tied to the fact that motorists were exercising more caution immediately after BODAWS installation as they became accustomed to the system. The BODAWS signs contained bright LED lenses that could be seen from hundreds of feet away and commanded more attention and respect from motorists when the "PREPARE TO STOP" message appeared and the beacons started flashing. The visibility of the BODAWS system may have contributed to increased caution and respect for the yellow change interval immediately after installation. Multiple hours of on-site observations during P2 tend to support this theory. Although no data have been gathered on RLR by vehicle classification, more than 12 hours of on-site observations also indicated that heavy vehicles benefited from the presence of the BODAWS signs and often stopped before the yellow change interval was complete.

However, as indicated in Chapter 5, the number of RLR violations increased during P3 on the northbound and southbound approaches. The P3 speed data tend to support the increase in RLR violations seen during P3 because the average speeds of vehicles recorded 50 feet from the intersection at 0 SBR were high enough that motorists would not have been able to brake in time to avoid running the red-light. As indicated previously it is possible that the lead flash of the BODAWS may was too long and that motorists abused the advance warning to try and beat the signal.

Another plausible explanation to the increase of the RLR violations during P3 is tied to the land use. A new Home Depot was opened on the northeast corner of the intersection between P2 and P3 that potentially attracts more right-turning vehicles from the northbound approach. Because the radar waves traveled in spherical bands from the radar sensor they could not be channeled to avoid detecting vehicles in the northbound right-turn lane; and, as discussed in Section 6.1, the sensor detection zone representing

the stop bar may actually have been located 15 to 20 feet in front of the actual physical stop bar. Empirical evidence suggests that most motorists did not come to a complete stop before making a right-turn which would mean that motorists in the right-turn lane may still have been traveling faster than the 20 mph speed filter after the signal turned red.

Although a speed filter of 20 mph and a 4 second time-after-red filter were applied to the RLR data in an attempt to filter out the northbound right-turning vehicles, some vehicles may have been counted among the RLR violation data. If the number of right-turning vehicles had remained constant during P1, P2, and P3 there may not have been any appreciable change in the number of RLR violations. However, because the new Home Depot was completed between P2 and P3, the number of right-turning vehicles could have increased the chance that more vehicles would not fit the filter criteria.

6.3 Crash Comparisons

Due to a lack of adequate crash data it is difficult to draw any concrete comparisons between the installation of the BODAWS system and the crash data at this time. The crash data appears to suggest that the number of crashes at the study site decreased during the last six months of 2005 in comparison to the last six months of the previous years. However, a similar decrease in crashes during 2005 was also apparent at the control intersection. Although both UDOT and the Sheriff's Department do all they can to provide accurate data, it is possible that not all crashes were reported or filed and reported correctly in the UDOT and the Sheriff's Department systems. The researchers for this study had to rely on UDOT and Sheriff's Department employees to provide adequate data. UDOT employees had to know how to effectively query their crash statistic database to provide the crash data that related to each site. Sheriff's Department employees had to pull the crash data by hand and may not have been consistent over the year and a half that this study was conducted. It is recommended that crash data be continually monitored at this location by UDOT or their contracted research team.

6.4 DCZ Study Discussion

The DCZ study results indicate that the BODAWS signs affected both the location of the DCZ, or the zone of greatest motorist variation. Although the results are statistically significant, they do not appear to be practically significant. A change of approximately 20 to 40 feet, as seen in the change of size and location of the DCZs by vehicle type, was not a large enough change that would indicate that motorists used the advance warning to give themselves more distance to come to a stop than they might have previously used. A lack of practical change in the size and location of the DCZ also suggests that motorists may not have been using the BODAWS signs as a landmark to determine if they needed to stop or proceed through the intersection. Because no practically significant change in the size and location of the DCZ was observed during P2, BYU decided not to conduct a DCZ study during P3. BYU researchers assumed that if the speeds and RLR events changed during P2 but the size and location of the DCZ did not change during P2, that it would be less likely that the size and location would change eight months later during P3.

The motorists who traveled through the intersection on a regular basis were probably more familiar and comfortable making space-time decisions at high speeds. The significance of their familiarity would indicate that even with the BODAWS signs and flashers most motorists felt comfortable stopping or proceeding through the intersection based on their own perceptions of space and time.

Another factor may also have biased the data collected during the DCZ study. As discussed previously in Section 4.3.1, the BYU researchers were unable to record the distance from the intersection of the vehicles that stopped at the onset of the yellow change interval. Instead, researchers were only able to record the location where motorists applied their brakes. The discrepancy in data collection may have negated any changes in the 90 percent stopping distance location. Future DCZ studies (for other BODAWS installations) should be conducted using technologies that will allow observers to monitor vehicles farther back from the intersection.

6.5 Discussion of Results Chapter Summary

The speed data results tend to suggest that the BODAWS system was effective in increasing motorist awareness and caution for a limited time after installation. It also appears that motorists may have lost respect for the advance warning as the study went on. It is even possible that motorists may have begun to take advantage of the system as they became more familiar with it. The RLR data tends to confirm the possibility of motorist abuse of the advance warning due to the increase in RLR events after BODAWS installation. The crash data proved inconclusive due to the lack of adequate data and the fact that crash rates dropped proportionately at the control intersection as well. The DCZ data results seem to indicate that the BODAWS system did not affect the locations that motorists deemed as safe to proceed from or stop at after the onset of the yellow change interval.

It is possible that the signal timing, or more specifically the lead flash time of the BODAWS signs and flashers, was too generous and may need to be adjusted. Reducing the amount of time between activation of the BODAWS signs and flashers and the onset of the yellow change interval might reduce the number of motorist who are able to speed up to beat the red light. A reduction in the lead flash time may cause greater respect for the BODAWS advance warning if motorists know they have to stop when the signs and flashers are activated. The following chapter contains conclusions and recommendations that might increase the effectiveness of the BODAWS system.

7 Conclusions and Recommendations

The preceding chapters have outlined the background of the blank-out overhead dynamic advance warning (BODAWS) system and identified the results of the system based on a number of analysis metrics including: 1) speed variations; 2) red-light running (RLR) compliance; 3) crash data analysis; and 4) decision zone (DCZ) evaluations. The results, in terms of effectiveness, of the BODAWS system are varied depending on the measures of effectiveness and the time period analyzed. Based on the speed and crash data analysis results and multiple hours of on-site observations it is apparent that too many motorists may be attempting to clear the intersection after the activation of the BODAWS signs and flashers instead of preparing to stop as the BODAWS sign recommends. The purpose of this chapter is to provide recommendations that may increase the effectiveness of the BODAWS system at reducing RLR and crash while increasing motorist respect for the advance warning.

The Recommendations and Conclusion chapter is divided into the following sections:

7.1 Conclusions – summarizes the main points of the project.

7.2 Recommendations for BODAWS System Improvement – suggests changes to the BODAWS lead flash timing.

7.3 Future Research – provides recommendations for future research possibilities and study areas.

7.1 Conclusions

The Utah Department of Transportation (UDOT) has been concerned about safety on the approach to high-speed signalized intersections (HSSIs) on Bangerter Highway and at other locations in the state for many years. UDOT determined that skid marks and spilled loads at HSSIs indicated that motorists were not reacting properly to the yellow change interval at several intersections. UDOT hired a consultant to design an advance warning (AWS) system that would mitigate the safety problems that were manifest at the intersections.

The consultant designed a unique AWS system, incorporating advance detection (AD) technology, based partly on the recommendations of a study conducted by the Nebraska Department of Roads (NDOR). The new AWS design was unique because it incorporated blank-out AWS signs and flashing beacons mounted over the roadway of the intersection approach. The new design was named the BODAWS system.

UDOT contracted with researchers at Brigham Young University (BYU) to evaluate the effectiveness of the BODAWS system. BYU researchers decided to study the BODAWS installation located at the intersection of 13400 South and Bangerter Highway in Riverton, Utah because of the sight-distance and geometric abnormalities that were possibly contributing to motorist indecision on the approaches to the intersection. The measures of effectiveness specified by UDOT to be used in the analysis of the BODAWS system included: 1) speed trends; 2) RLR violations; and 3) crash frequencies. BYU also collected data to determine the size and location of the DCZ, or the zone of greatest variation in motorist behavior on the approaches to the study site intersection.

BYU collected speed and RLR data using sophisticated Digital Wave Radar™ sensors provided by a private technology company. Crash data were collected from the jurisdictions responsible for the study site accounting for three years prior to BODAWS installation and for at least six months after installation. The BODAWS speed and RLR evaluation data were collected for a few weeks at a time before, immediately after, and eight months after installation. The data were then statistically analyzed and compared to determine if the BODAWS system contributed to the safety of the study intersection.

The speed data trend results suggest that motorists decreased their speeds by 5 to 10 mph during the yellow change interval immediately after BODAWS installation while they were adapting to the new system. At the same time, speeds during the green interval increased to levels closer to the posted speed limit while the BODAWS signs and flashers were inactive suggesting that motorists understood they would receive warning before the onset of the yellow change interval and felt comfortable traveling through the intersection closer to the speed limit. Eight months after BODAWS installation, motorists increased their speeds on average by 5 to 10 mph greater than before installation. The greatest speed increase occurred during the yellow change interval, eight months after installation, suggesting that motorists may have become accustomed to the sign and knew that they could use the advance warning of impending signal changes to try and clear the intersection before the signal turned red.

The RLR data tends to support the speed data assumptions. The RLR data showed that motorists were not running the red-light immediately after BODAWS installation as much as they were before installation suggesting that the BODAWS system contributed to a decrease of as much as 4 violations per 1,000 entering vehicles as motorists were adapting to the new system. However, eight months after installation the number of RLR violations increased to frequencies in some instances higher than before BODAWS installation by as much as 3 violations per 1,000 entering vehicles. The RLR data verifies the increased speeds recorded during the yellow change interval indicating that motorists may have been trying to beat the red light.

The crash data results are somewhat inconclusive due to the fact that only six months of crash data from the time period after BODAWS installation were available for the study. Although the crash data at the study site suggests that the number of crashes was reduced following BODAWS installation by more than half, the number of crashes at the control intersection also decreased by a similar amount. Further data should be collected during the next year to 18 months and the data analyzed statistically to determine if any significant or practical change in the number of crashes is apparent.

The DCZ study found that the size of and location of the DCZ changed slightly after installation of the BODAWS system. The location of the 10 percent probability stopping distance moved from 237 feet before BODAWS installation to approximately

259 feet after BODAWS installation while the 90 percent probability stopping location moved from 440 feet to 481 feet during the same time period. Although the change in the size and location of the DCZ boundaries was determined to be statistically significant, it did not appear to be practically significant. The DCZ study results suggest that motorists feel comfortable proceeding through an intersection or stopping after the onset of the yellow change interval based on distance and travel time from the intersection regardless of the advance warning or information they are provided by the BODAWS system.

The speed, RLR, crash, and DCZ data results suggest that motorists might have been more cautious immediately after BODAWS installation but by eight months after installation they may have adapted to the system and used the advance warning to try and proceed through the intersection when it would have been safer for them to stop.

The BODAWS system has proven that it affects motorist behavior and changes driving patterns. However, not all of the changes in motorist behavior were positive. It is possible that the warning that motorists received came too far in advance of the onset of the yellow change interval allowing motorists to abuse the system and persuading motorists that the warning did not have to be obeyed to safely proceed through the intersection. Therefore, UDOT and BYU should continue to monitor driver behavior patterns at the study site and collect data for at least another 10 months which would provide them with 18 months of data after BODAWS installation. In the meantime, UDOT should consider reducing the BODAWS lead flash time. The following section provides recommendations regarding BODAWS system improvement including recommendations to reduce the lead flash time.

7.2 Recommendations for BODAWS System Improvement

The purpose of the BODAWS system is to provide DCZ protection through the use of AD and AWS technology. The BODAWS detectors were installed to extend the green until a gap in traffic could be found that would allow the onset of the yellow change interval to occur when there were no vehicles in the DCZ. The BODAWS signs and flashers were installed to provide advance warning of the impending yellow change interval to motorists who were not able to pass over the advance detector and extend the

green. The location of the BODAWS signs and flashers and the BODAWS detector, as well as the amount of lead flash time required to warn motorists of the yellow change interval, were designed based on NDOR assumptions of motorist reaction capabilities such as a 3 second perception-reaction time, and a 10 ft/sec^2 deceleration rate. The design speed recommended by NDOR also allowed for adjustments to be made that would provide DCZ protection to a wider range of vehicle approach speeds. For example, NDOR recommended using a 55 mph design speed to calculate the stopping-sight distance and lead flash time for the study site intersection even though the actual 85th percentile speeds were closer to 65 mph. Furthermore NDOR advance detector timing was designed to extend the green long enough to allow a vehicle to travel from the advance detector to the stop bar before the onset of the yellow change interval. The amount of time provided for a motorist to travel from the detector to the stop bar before the onset of the yellow change interval, however, may be too long because the DCZ study found that most motorists will proceed through the intersection if they are within approximately 235 feet at the onset of the yellow change interval (*4*, *38*).

Furthermore, the NDOR perception-reaction time, deceleration rate, design speed, and green extension recommendations may have made the design too conservative thereby providing motorists with too much warning time. The extended warning may have even worked to counteract the meaning of the BODAWS warning because motorists found that they did not have to prepare to stop as the BODAWS sign indicated. In fact, many motorists may have discovered that to stop their vehicle after activation of the BODAWS signs and flashers would have been unreasonable because they would have come to a stop at the intersection with a few seconds of yellow change interval still remaining.

To illustrate this point, consider the intersection as it is currently designed as illustrated in Figure 7-1 with a 60 mph posted speed limit. The hatched area of the figure represents the 10 percent and 90 percent boundaries of the DCZ as measured by BYU immediately after BODAWS installation with the 10 percent probability of stopping boundary located 260 feet (3.0 seconds) and the 90 percent probability of stopping boundary located 480 feet (5.5 seconds) from the signalized intersection illustrated on the left side of the figure. Vehicle 1 (V1) has just crossed over the leading edge of the

BODAWS detector and is traveling toward the signalized intersection approximately 15 seconds before red (SBR). Vehicle 2 (V2) is approximately 3.5 seconds behind V1. At the instant that V1 crosses over the edge of the detector, the signal controller begins looking for a 3 second gap in traffic to end the green phase.

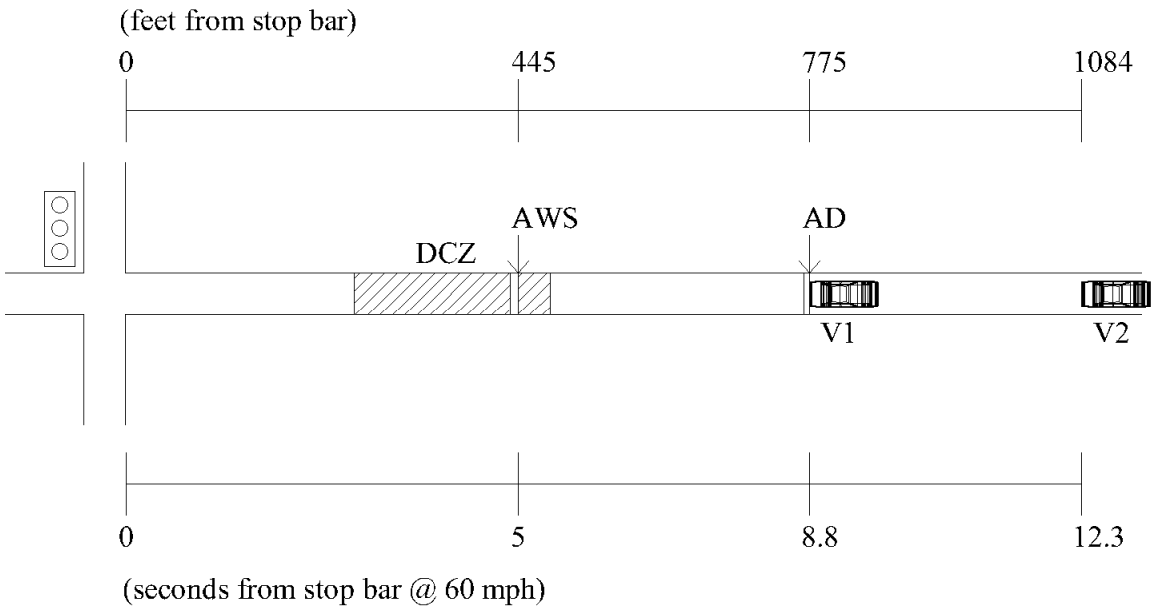

Figure 7-1 Vehicles approaching study site intersection during green (15 SBR).

Figure 7-2 illustrates vehicles V1 and V2 approximately 3 seconds after V1 initially crossed the leading edge of the detector. At this point in time, V2 is just about to cross the leading edge of the detector but has not done so before the signal controller detected the 3 second gap in traffic. A third vehicle (V3) also traveling at 60 mph approaches the intersection 2.7 seconds behind V2 and is approximately 1,057 feet from the stop bar. The signal controller begins the process of gap-out and the BODAWS signs and flashers are activated. V1 is not within the 125 feet sign legibility distance from the BODAWS AWS sign and likely cannot see the BODAWS signs and flashers but V2 and V3 receive the warning.

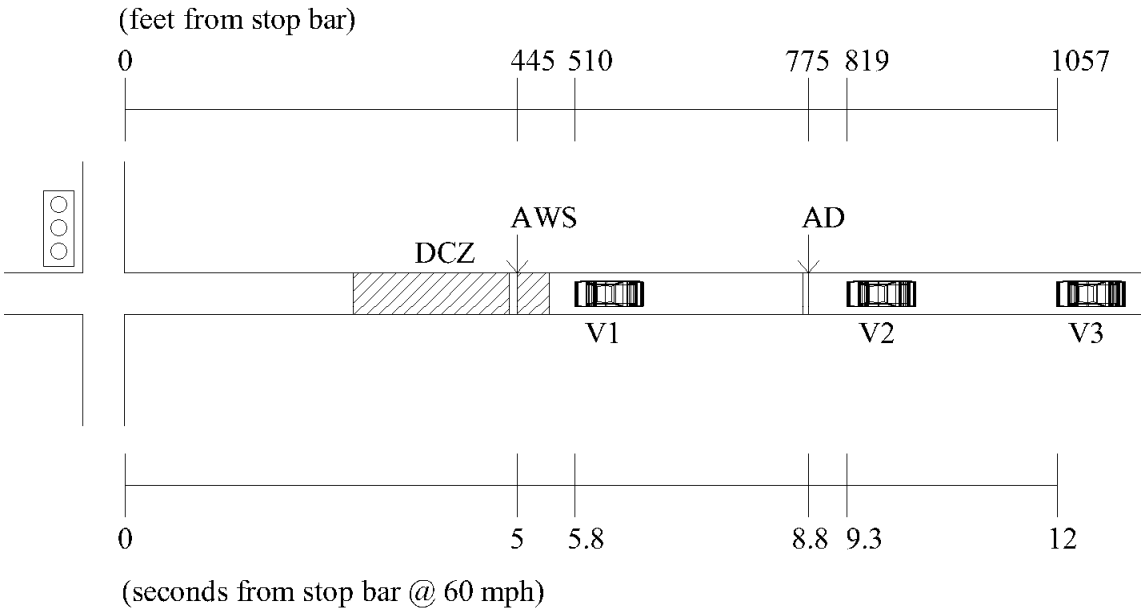

Figure 7-2 Vehicles at the beginning of BODAWS activation (12 SBR).

Figure 7-3 illustrates vehicles V1, V2, and V3, 6 seconds after the BODAWS signs and flashers were activated. The signal has just turned yellow and V1 just passed the stop bar and will proceed through the intersection. V2 has 6 seconds to proceed through the intersection and is only 3.3 seconds from the stop bar. V2 needs 425 feet to stop comfortably and safely according to the American Association of State Highway and Transportation Officials (AASHTO) recommended stopping-sight distance equation and assumptions calculated using Equation 2-3. The DCZ study found that only 10 percent of motorists within 237 feet of the intersection will actually stop so there is only a slightly greater chance that V2 will decide to stop. Most likely V2 will proceed through the intersection. V3 is only 2.7 seconds behind V2. At this moment in time V3 is still 6 seconds from the intersection a distance of approximately 528 feet from the stop bar. It is possible for V3 to safely and reasonably clear the intersection during the remainder of the yellow change interval. In fact, if V3 were to accelerate, it could reach the stop bar sooner. The DCZ study found that 90 percent of passenger cars further than 440 feet from the intersection at the onset of the yellow change interval will stop. At this point in time the probability that V3 will stop is high. However, speed and RLR data at the study

intersection show that too many motorists may be accelerating in an attempt to enter the intersection before the signal turns yellow.

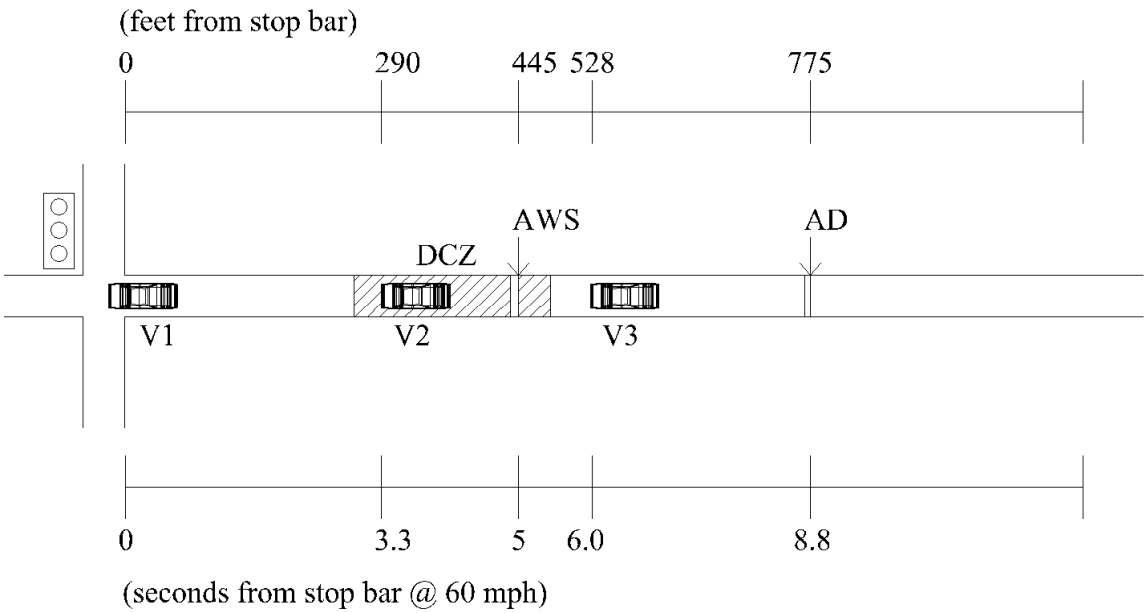

Figure 7-3 Vehicles at the beginning of the yellow change interval (6 SBR).

The potential problem with the situation outlined using Figure 7-1, Figure 7-2, and Figure 7-3 is that motorists like those in V2 and V3 receive the BODAWS sign and flasher warning and do not have to stop. If the motorist in V2 had decided to stop, he/she potentially would have come to a complete stop when there were 2 or more seconds of yellow time remaining. Most motorists will likely do this only once or twice before they realize that the BODAWS signs and flashers really did not signify that they must stop and they realize that they could have easily made it through the intersection. Such motorists will lose respect for the BODAWS system. V3 motorists will also learn that they can beat the system if they try with very little effort. Another alternative is needed.

It is recommended that UDOT consider adjusting the BODAWS lead flash time such that when the signs and flashers are activated any vehicle that can see the signs will be required to stop (based on design assumptions). Suppose that the situation of vehicles V1, V2, and V3 remains similar to the situation described using Figure 7-1 and Figure

7-2 in regards to vehicle speeds, following distances, and proximity to the intersection. Now suppose that instead of a 6 second lead flash time between the activation of the BODAWS signs and flashers and the onset of the yellow change interval there is only a 3 second lead flash time. At the onset of the yellow change interval vehicles V1, V2, and V3 will have moved from their positions illustrated in Figure 7-2 to the position illustrated in Figure 7-4.

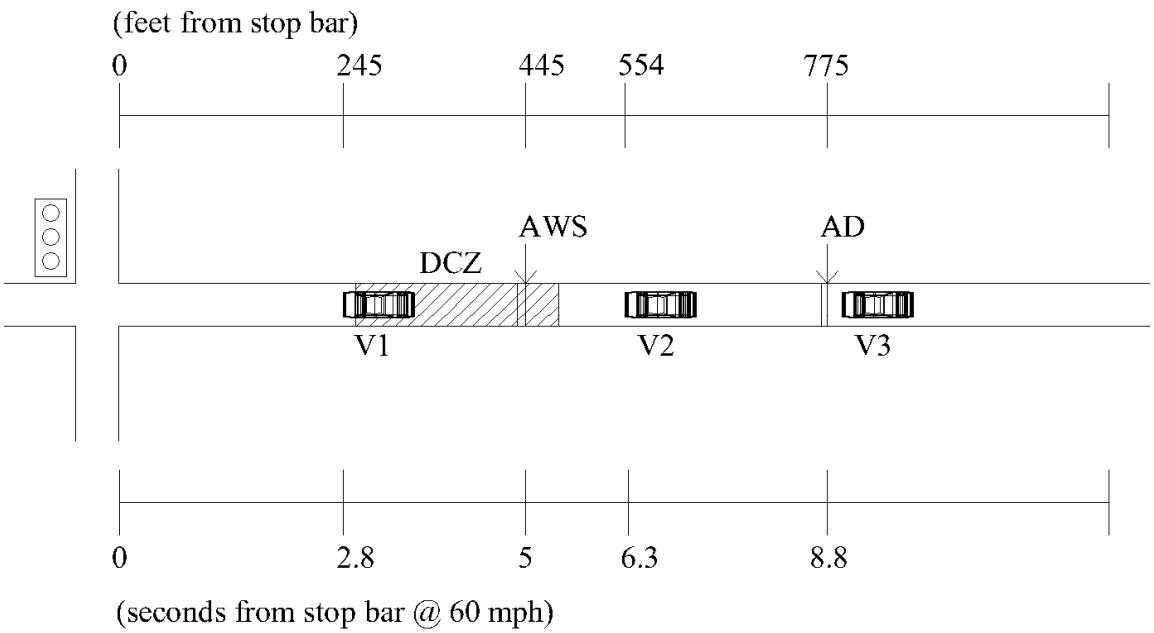

Figure 7-4 Alternative scenario of vehicles at the beginning of the yellow change interval (6 SBR).

Although V1 has not crossed the stop bar, the vehicle will still be close to the 10 percent probability of stopping DCZ boundary where most motorists will decide to proceed through the intersection. V2 will be far enough back from the intersection that it would be well past the 440 foot DCZ boundary where 90 percent of motorists decide to stop with sufficient distance to safely stop. V3 will also be far enough back from the intersection that it will be required to stop. Under this scenario, fewer motorists will be in a position where they could accelerate and beat the signal. The motorists in V2 and V3 will have received the warning and will be required to prepare to stop.

It is recommended that UDOT consider adjusting the lead flash time as a function of sign location, the perception distance of the sign, and the probability of stopping time as outlined in Equation 7-1. It is also important to note the advance detection location should be located as a function of the design speed and gap time as identified previously.

$$t_{DCZ} = \frac{D_M + D_P}{v_0} - DCZ_n \tag{7-1}$$

where: t_{DCZ} = lead flash time as a function of DCZ (sec),

D_M = distance from BODAWS sign to stop bar (ft),

D_P = minimum sign recognition distance of BODAWS sign (ft),

v_0 = design speed (ft/sec),

DCZ_n = n percent probability of stopping time (sec), and

n = desired probability of stopping (1, 5, or 10 percent).

Using the design criteria from the NDOR study, assuming an 85th percentile speed of 65 mph, a resulting design speed of 55 mph, 445 feet from the BODAWS sign to the intersection, 70 foot sign recognition distance (based on NDOR study), and a probability of stopping time equal to 2.5 seconds (5 percent probability of stopping based on the design outlined in Appendix K); the lead flash would be set at 3.9 seconds. Using a design speed of 60 mph and holding all other variables constant, the lead flash would be set at 3.4 seconds, while a design speed of 65 mph (holding all other variables constant) would set the lead flash at 2.9 seconds.

As identified previously, the 85th percentile of this segment of Bangerter Highway is approximately 62 mph. Using a range of design speeds from 50 to 65 mph and lead flash times between 3.0 and 4.0 seconds, the distance traveled during this time can be calculated as summarized in Table 7-1.

Table 7-1 Distance Traveled as a Function of Lead Flash and Design Speed

Lead Flash (sec)	Distance Traveled (ft)			
	50 mph	55 mph	60 mph	65 mph
3.00	660	726	792	858
3.25	678	746	814	882
3.50	697	766	836	906
3.75	715	787	858	930
4.00	733	807	880	953

The total time for a vehicle to travel in Table 7-1 includes both the lead flash (variable) and the yellow change interval of 6 seconds as defined previously. It is recommended that UDOT consider changing the lead flash time from the current 6 seconds to somewhere between 3.9 and 4.5 seconds to decrease the amount of time that motorists have to clear the intersection. The BODAWS system should be designed in such a way that it only provides motorists with pertinent real-time information that can be relied upon as accurate and that commands attention and respect.

7.3 Future Research

Due to the massive amount of information that was acquired through the use of state-of-the-art Digital Wave Radar™ technology and due to the time constraints of this report, many questions about the effectiveness of the BODAWS system were left unanswered. Furthermore, recommended design changes should be analyzed to see if the changes improve the system and increase safety at the study site. Future research could include a consideration of the following topics:

- The frequency and types of crashes occurring at night;
- The effects of the BODAWS system on weekend drivers;
- The frequency of max-outs of the green interval before and after BODAWS installation;

- The frequency of max-outs of the green interval after recommended lead flash time changes;
- The correlation between volumes and max-outs;
- The frequency of RLR violations occurring during the early morning hours;
- The speed trends of vehicles between the beginning of the green interval and 16 SBR;
- The speeds of vehicles after the onset of the red interval;
- The size of the actual offset between the radar sensor detection zone representing the stop bar and the actual physical location of the stop bar to determine if the offset is large enough to contribute to a misrepresentation of RLR and speeds trends;
- The effects of recommended lead flash time changes on RLR, speeds, and crashes;
- The number of vehicles caught in the DCZ at the onset of the yellow change interval before and after BODAWS installation; and
- The number of vehicles caught in the DCZ after recommended lead flash time changes.

Many other research questions may also arise as discussions between UDOT and BYU continue, and further analysis may contribute to improvements to the BODAWS system in general. Finally, studies may also need to be conducted to determine what motorists, especially heavy vehicle operators, think of the BODAWS system and to see if they understand the purpose of and need for the system. If the BODAWS system ultimately proves effective at increasing safety, the system may yet become a standard installation at HSSIs across Utah.

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Appendix A

Typical Advance Warning Signal Layout

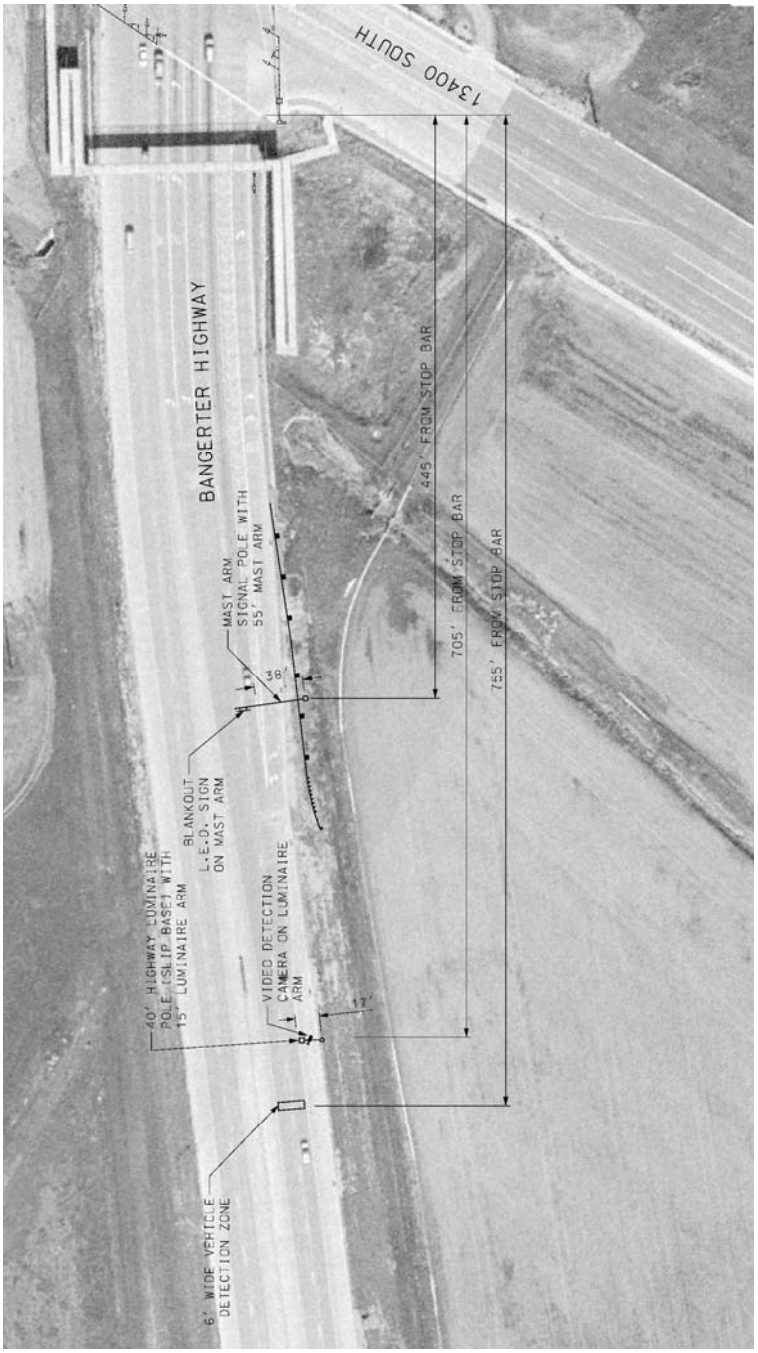

Figure A-1 Typical advance warning signal layout.

Appendix B

Cumulative Distribution of the Speed Data Results for the AM Peak

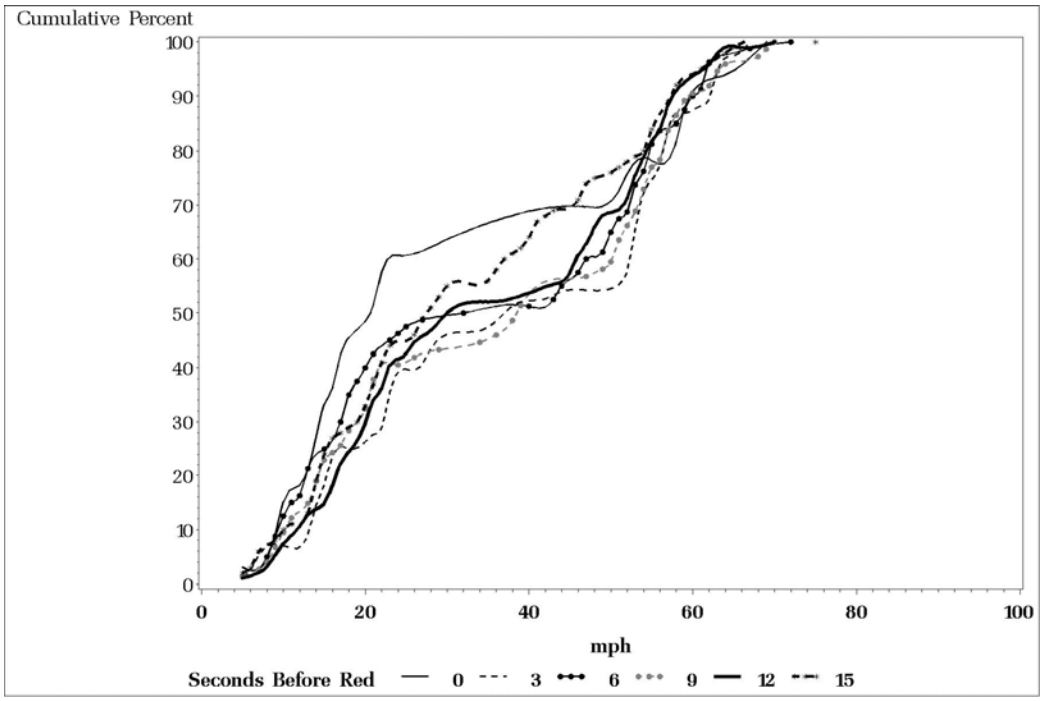

Figure B-1 Cumulative speeds for the northbound AM peak 50 ft. sensor for P1.

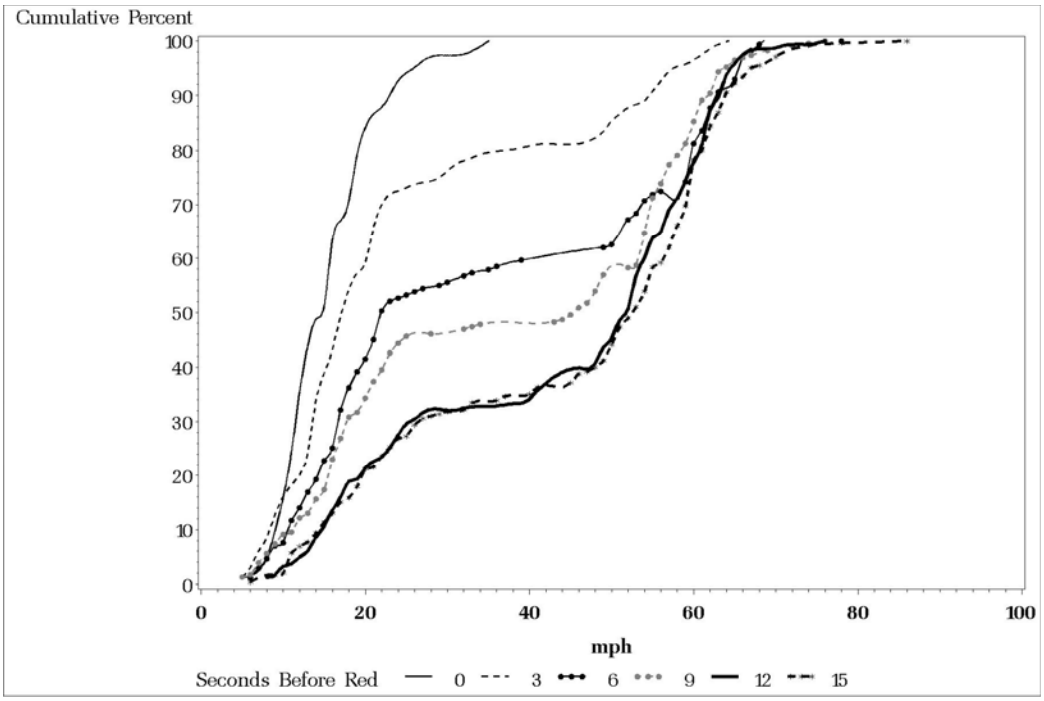

Figure B-2 Cumulative speeds for the northbound AM peak 50 ft. sensor for P2.

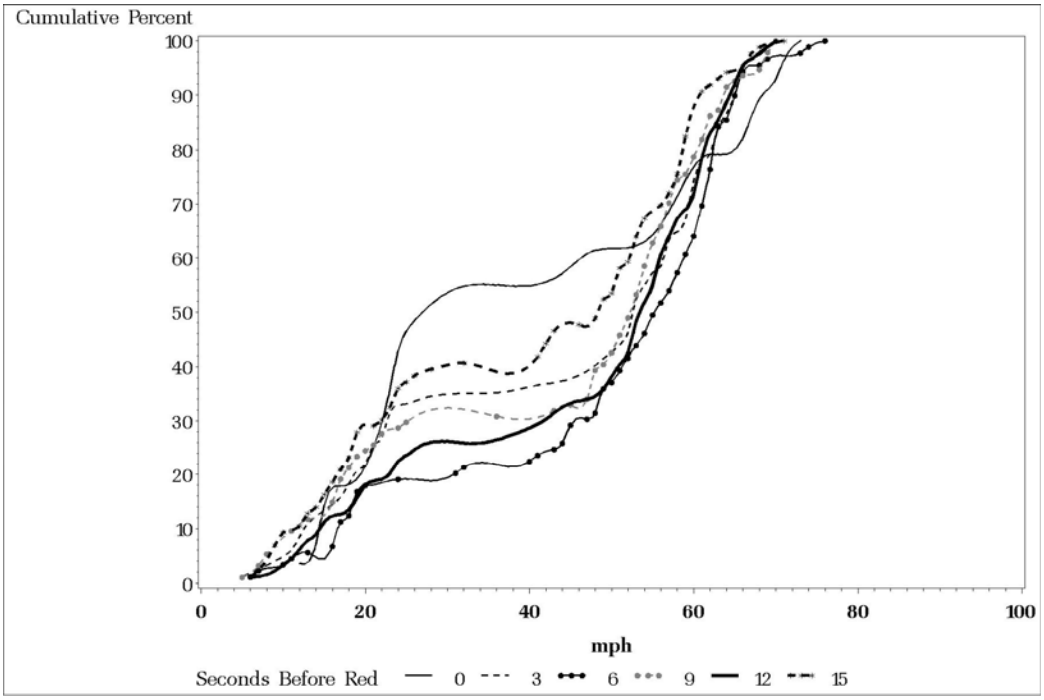

Figure B-3 Cumulative speeds for the northbound AM peak 50 ft. sensor for P3.

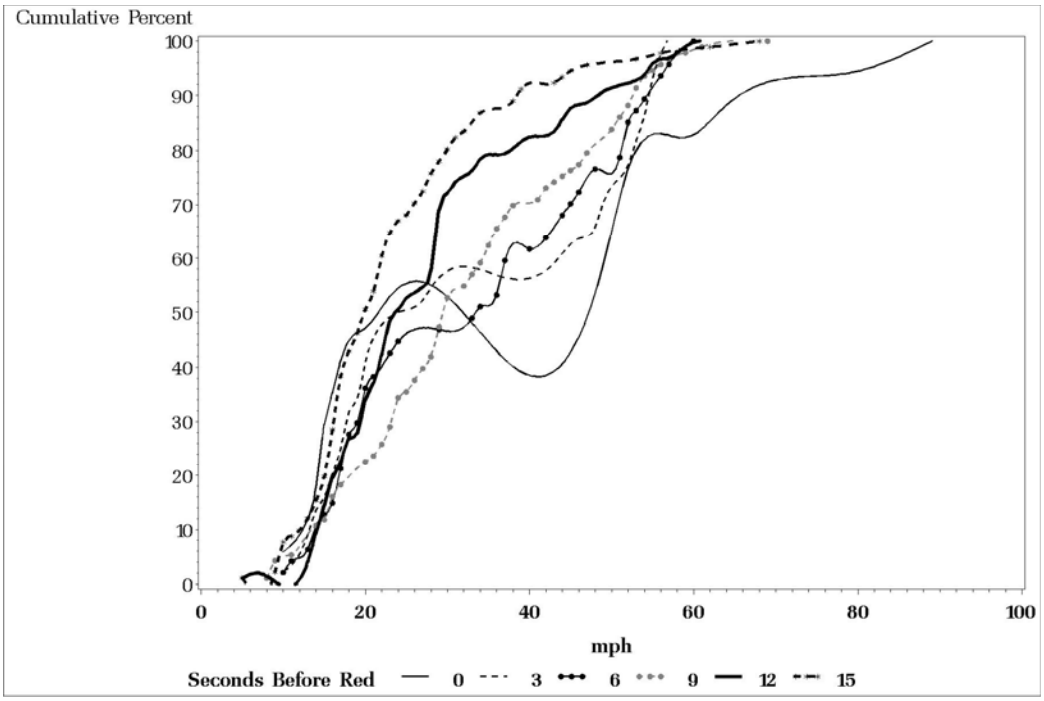

Figure B-4 Cumulative speeds for the southbound AM peak 50 ft. sensor for P1.

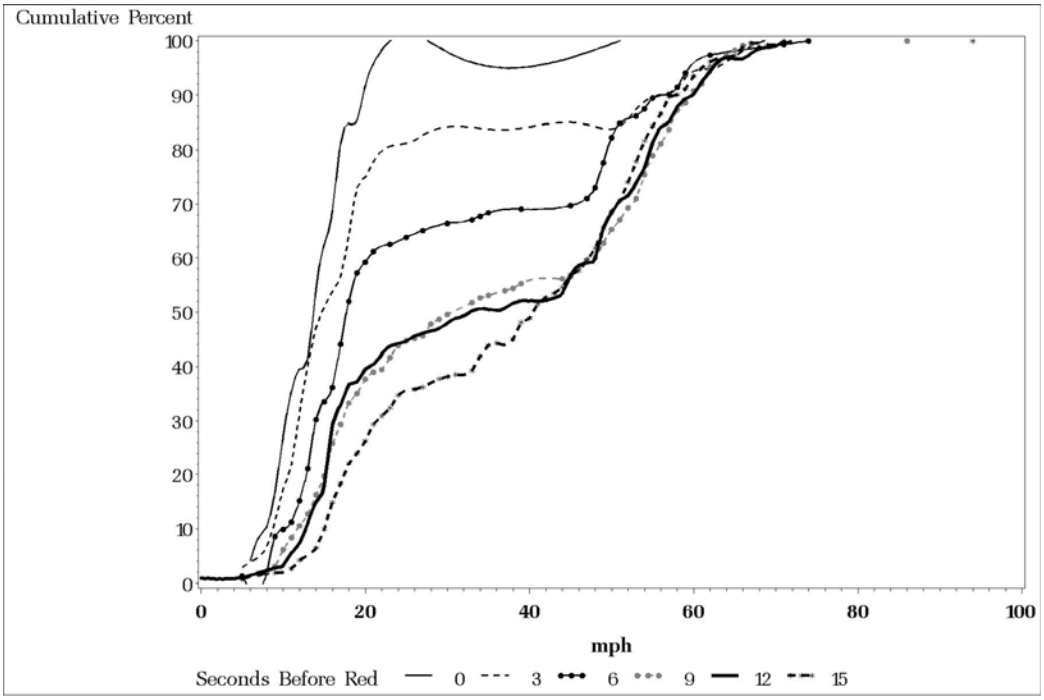

Figure B-5 Cumulative speeds for the southbound AM peak 50 ft. sensor for P2.

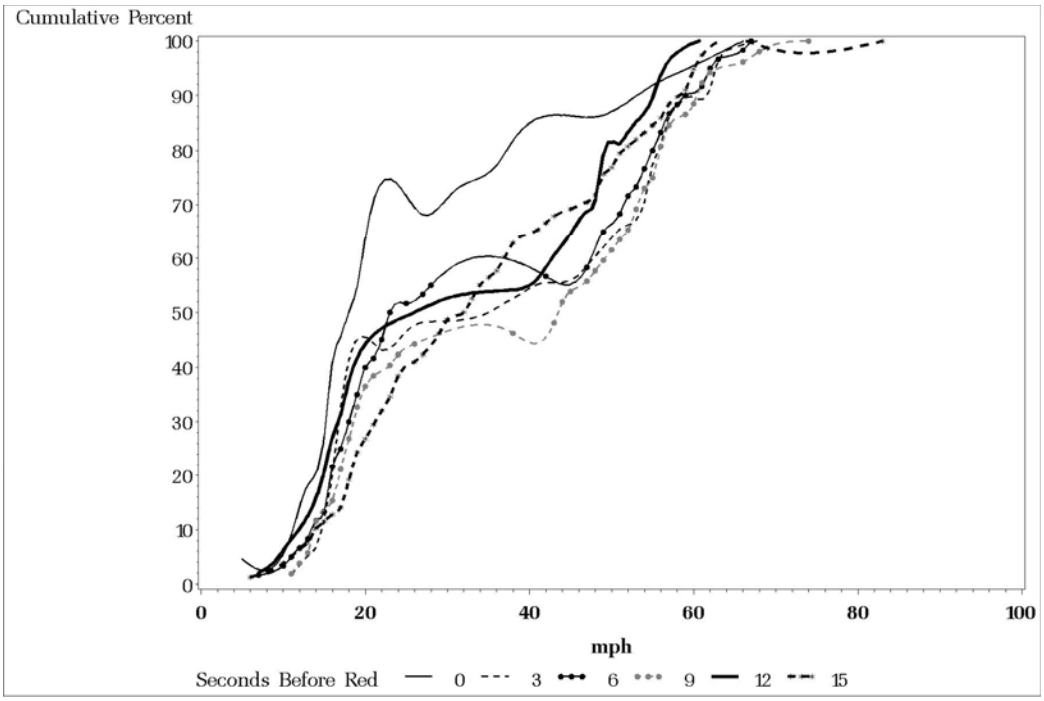

Figure B-6 Cumulative speeds for the southbound AM peak 50 ft. sensor for P3.

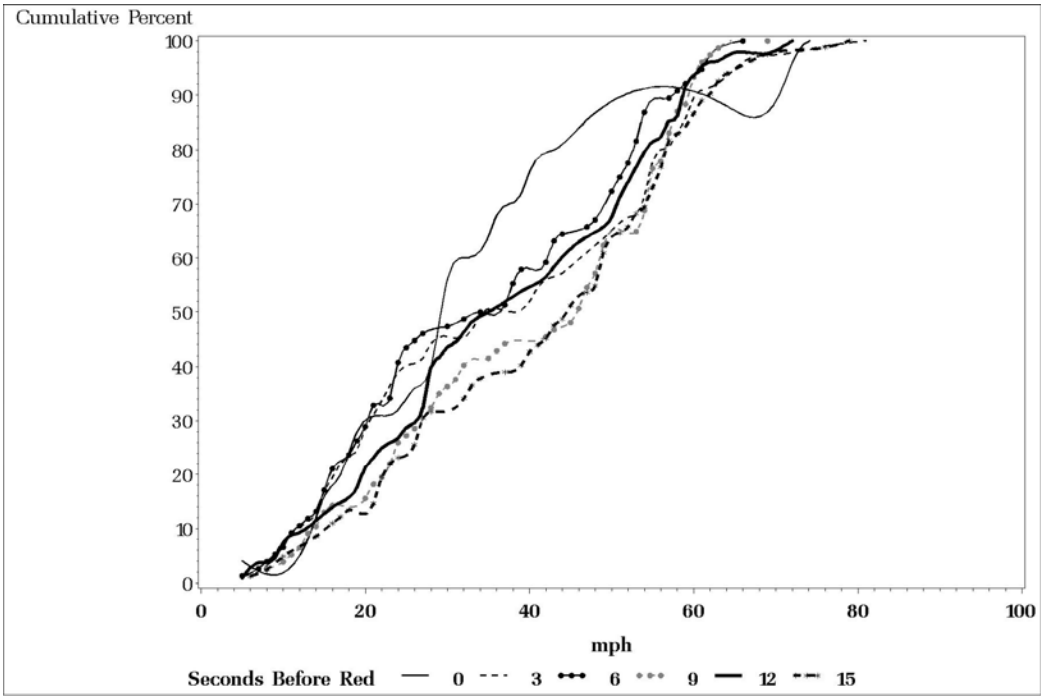

Figure B-7 Cumulative speeds for the northbound AM peak 100 ft. sensor for P1.

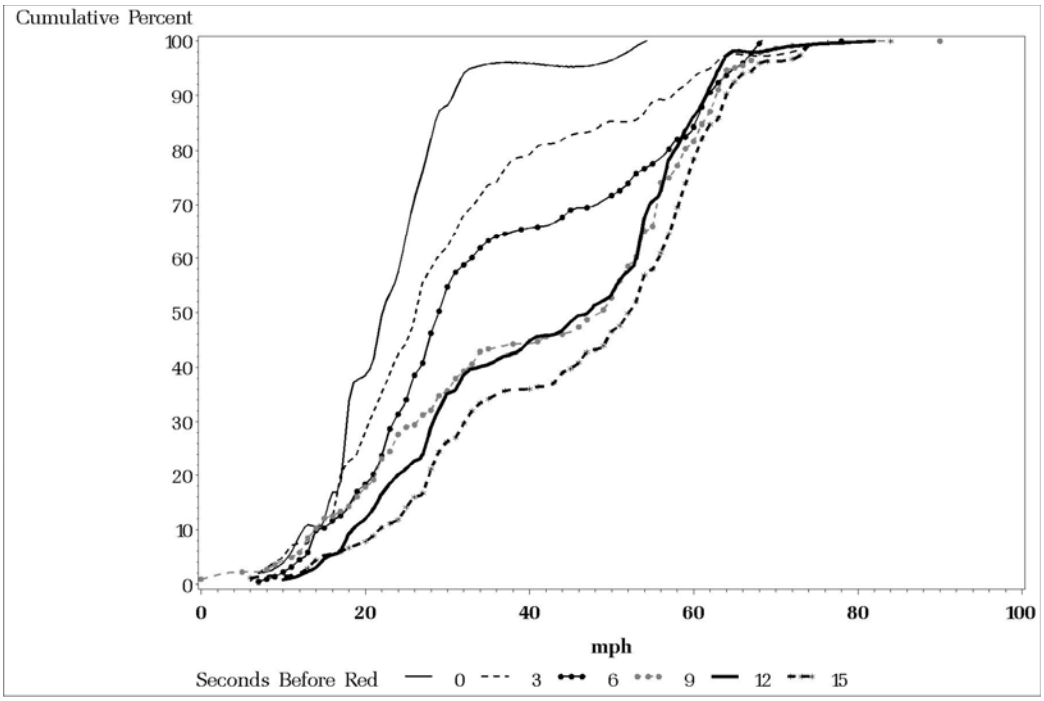

Figure B-8 Cumulative speeds for the northbound AM peak 100 ft. sensor for P2.

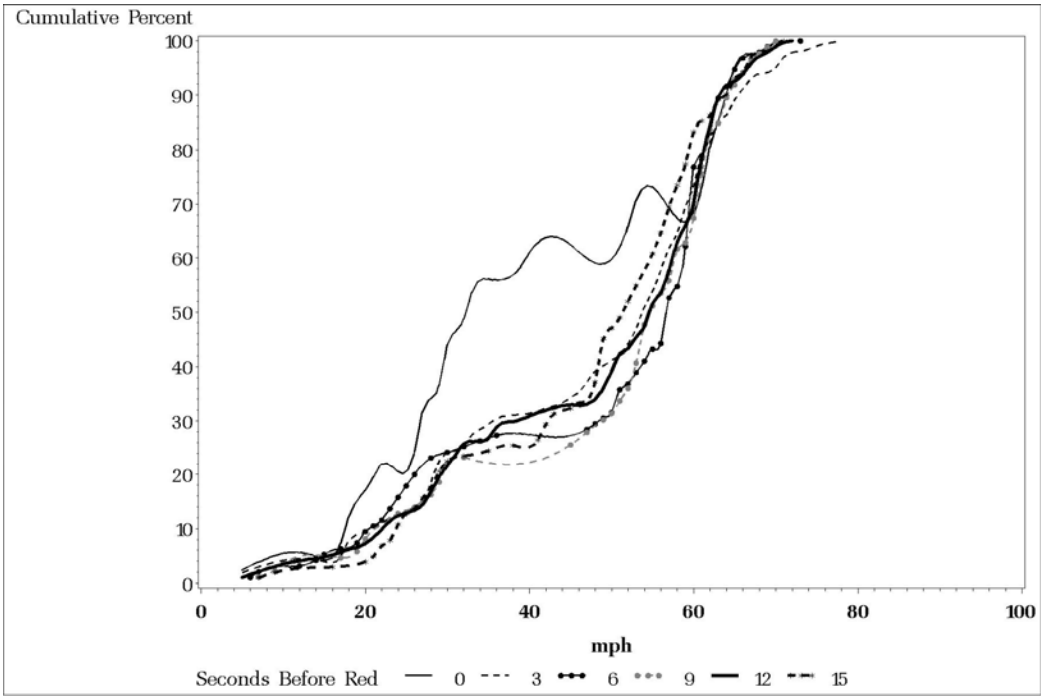

Figure B-9 Cumulative speeds for the northbound AM peak 100 ft. sensor for P3.

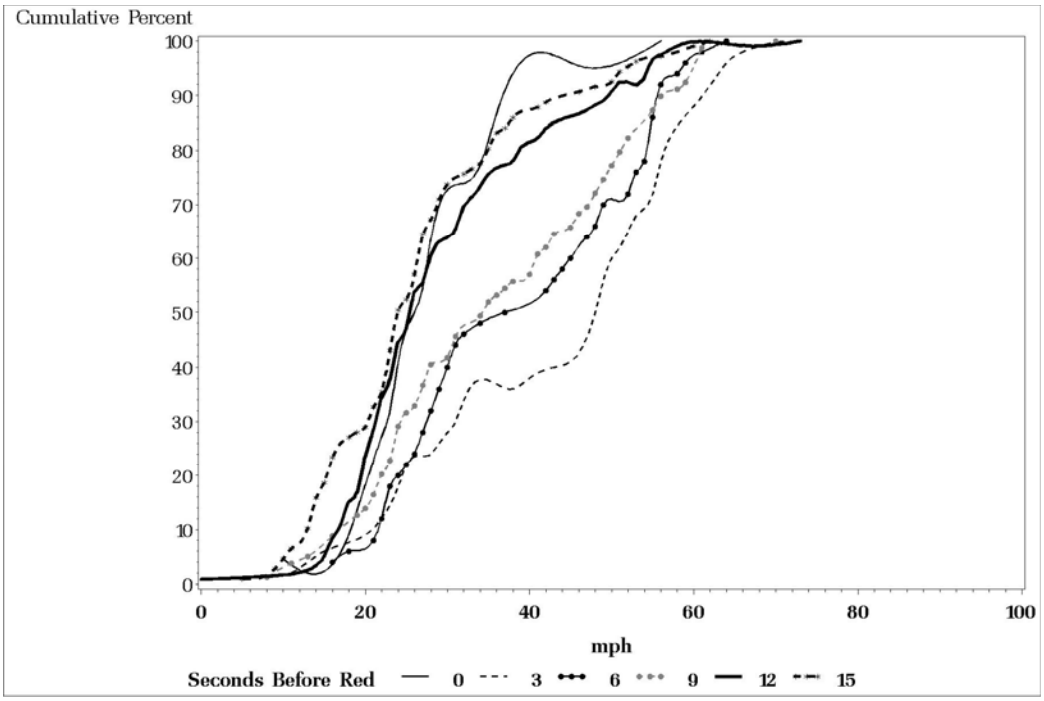

Figure B-10 Cumulative speeds for southbound AM peak 100 ft. sensor for P1.

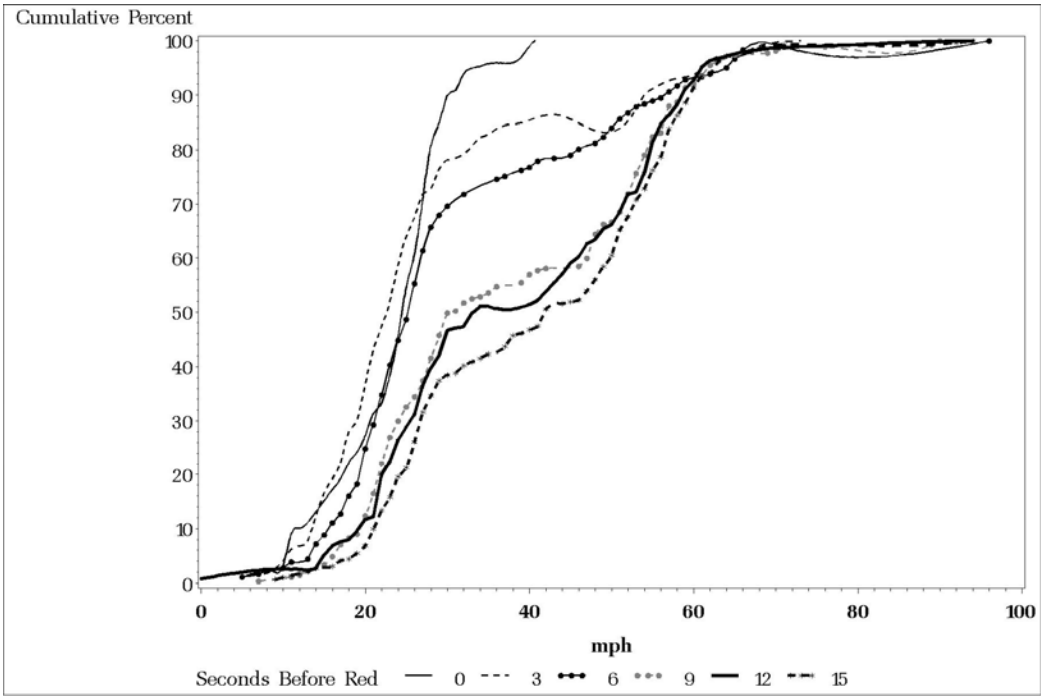

Figure B-11 Cumulative speeds for the southbound AM peak 100 ft. sensor for P2.

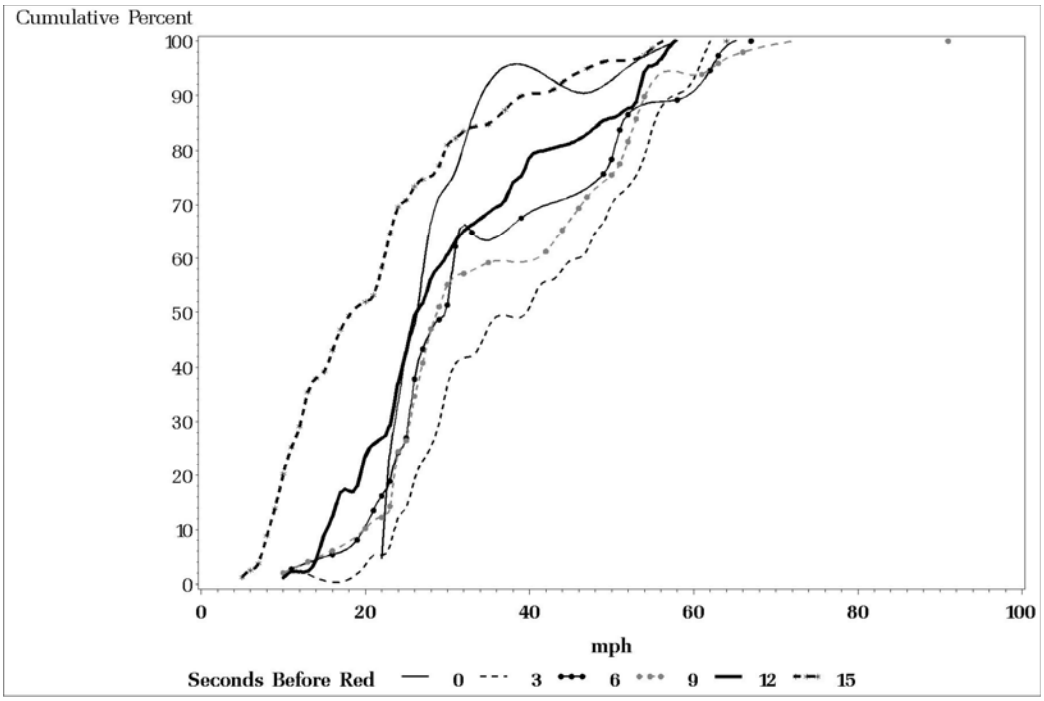

Figure B-12 Cumulative speeds for the southbound AM peak 100 ft. sensor for P3.

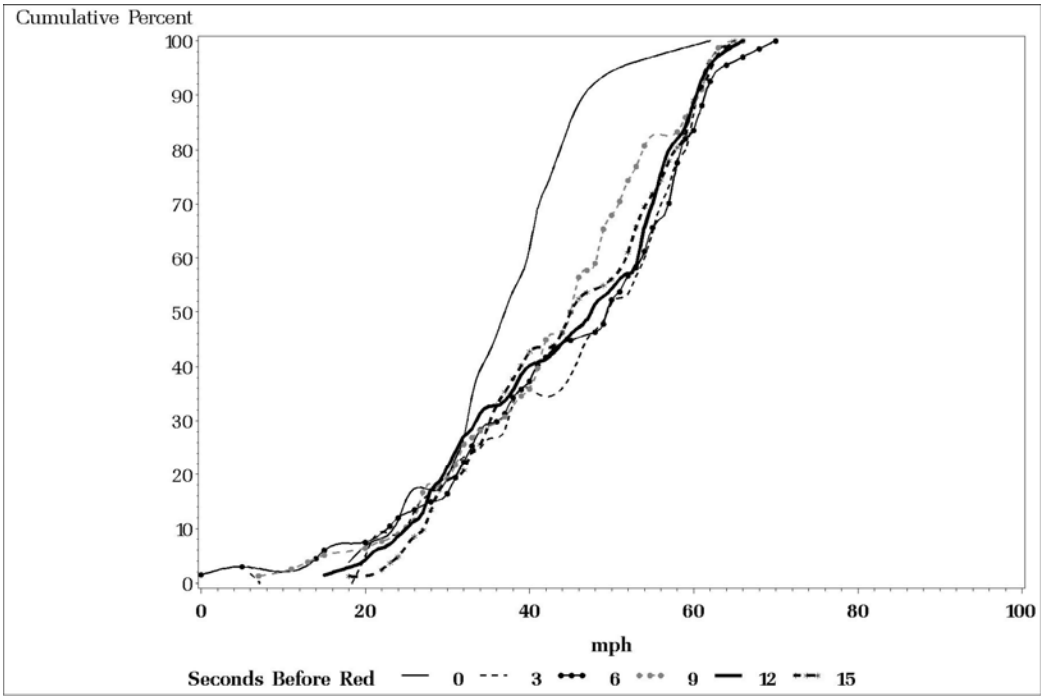

Figure B-13 Cumulative speeds for the northbound AM peak 200 ft. sensor for P1.

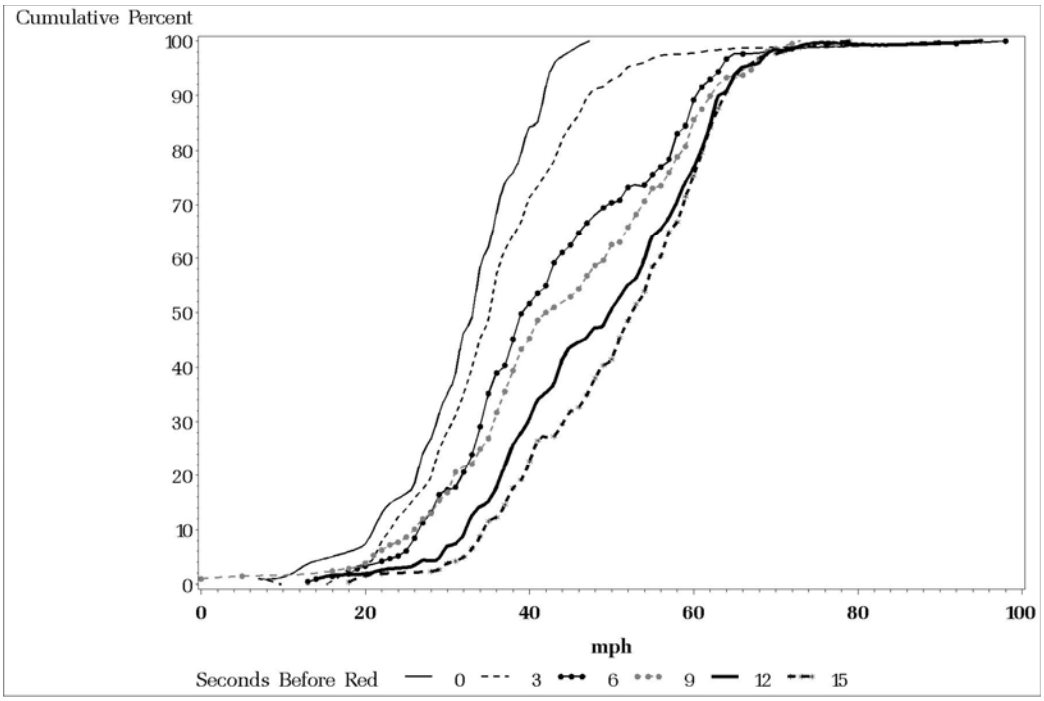

Figure B-14 Cumulative speeds for the northbound AM peak 200 ft. sensor for P2.

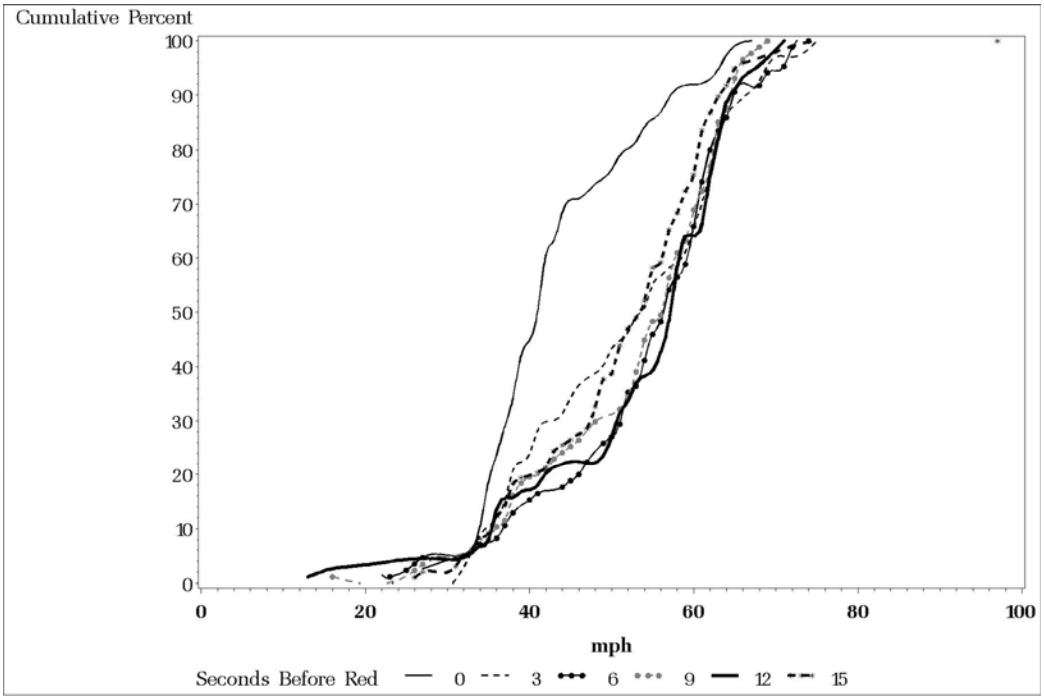

Figure B-15 Cumulative speeds for the northbound AM peak 200 ft. sensor for P3.

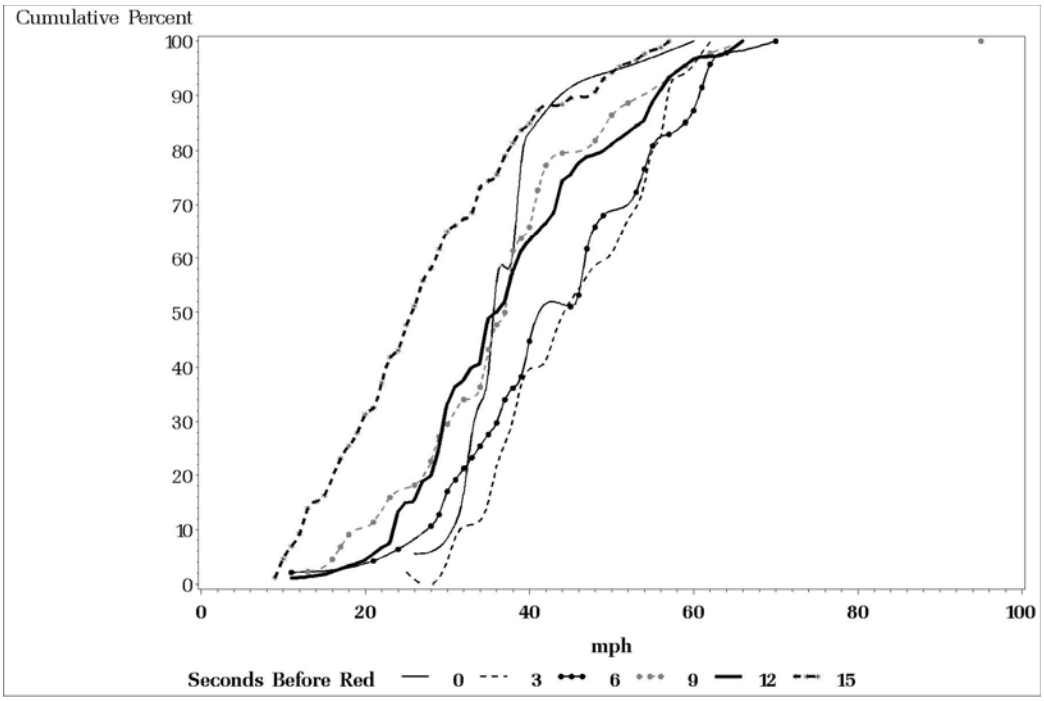

Figure B-16 Cumulative speeds for the southbound AM peak 200 ft. sensor for P1.

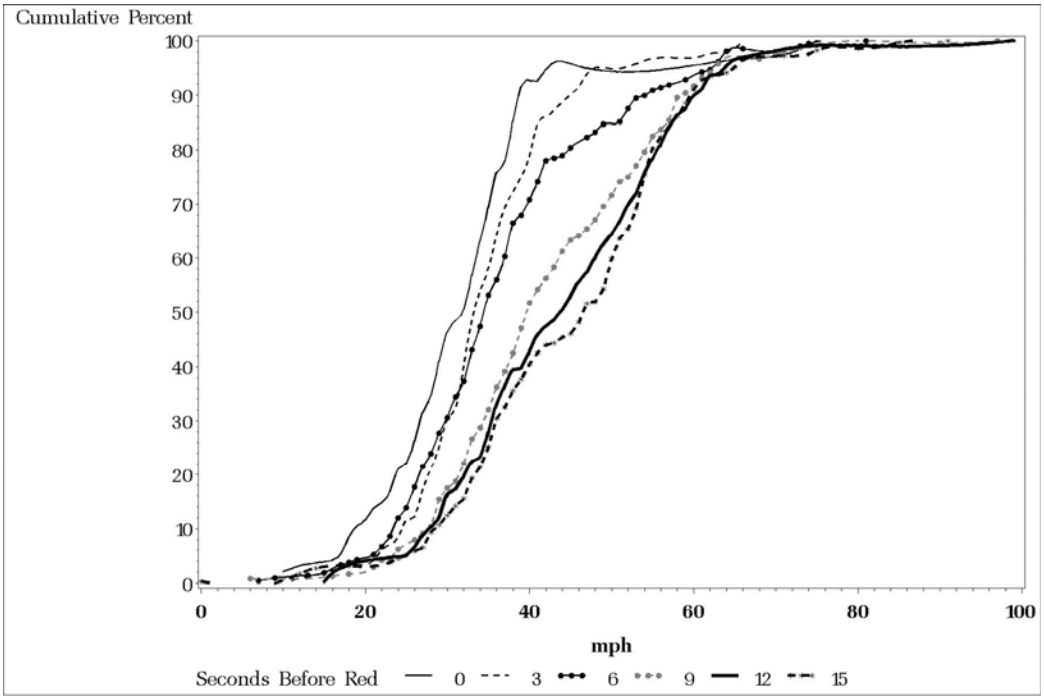

Figure B-17 Cumulative speeds for the southbound AM peak 200 ft. sensor for P2.

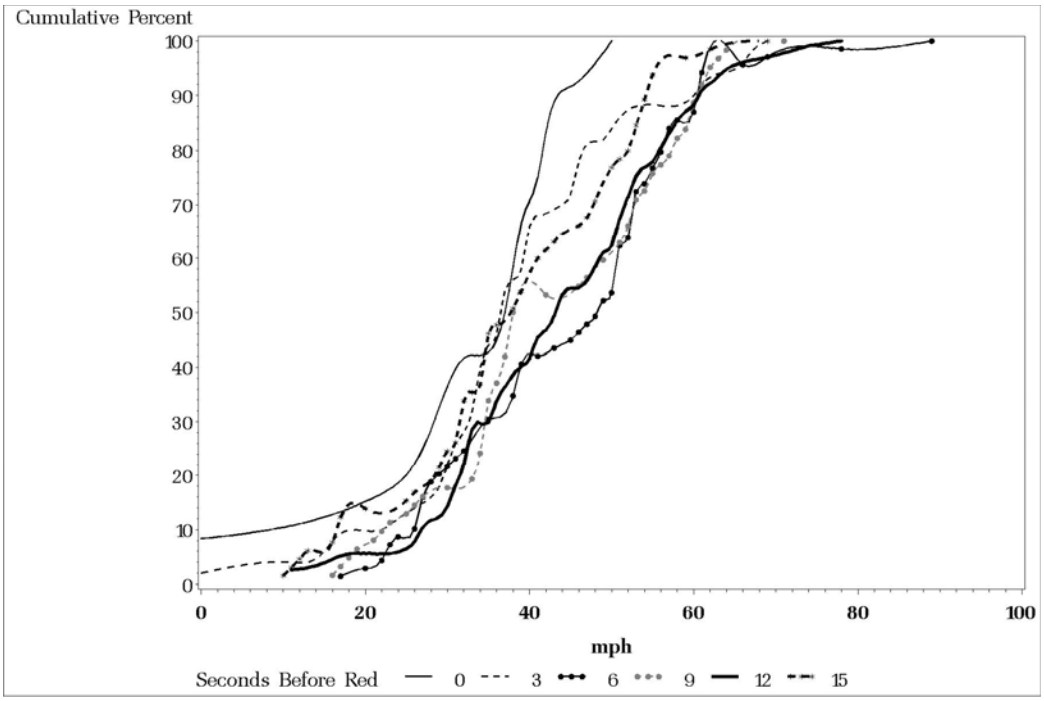

Figure B-18 Cumulative speeds for the southbound AM peak 200 ft. sensor for P3.

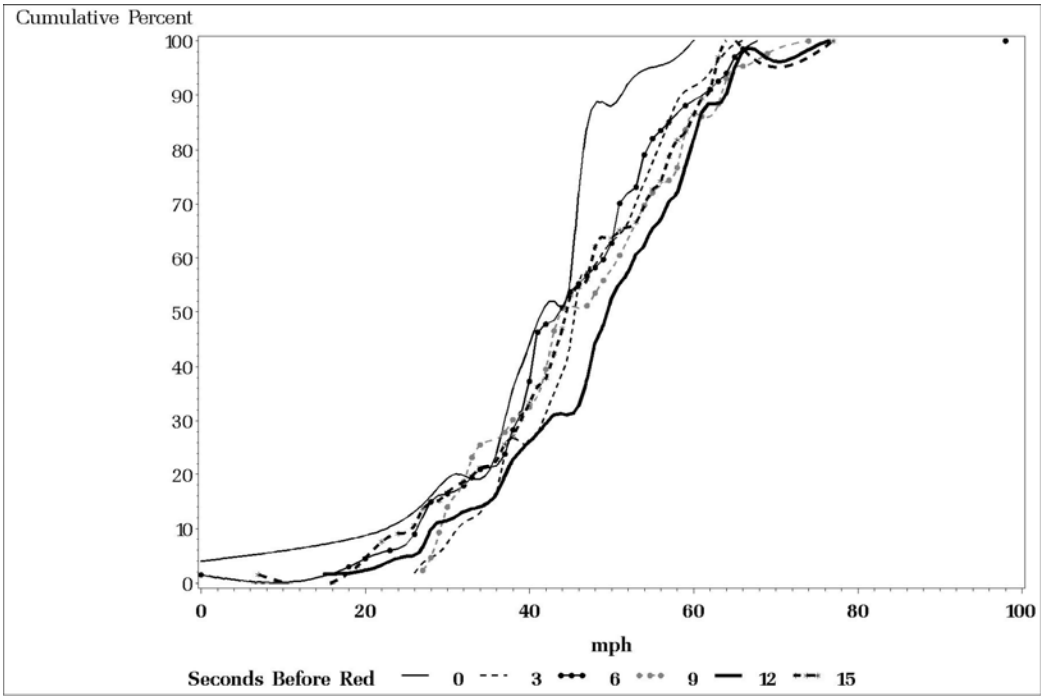

Figure B-19 Cumulative speeds for the northbound AM peak 300 ft. sensor for P1.

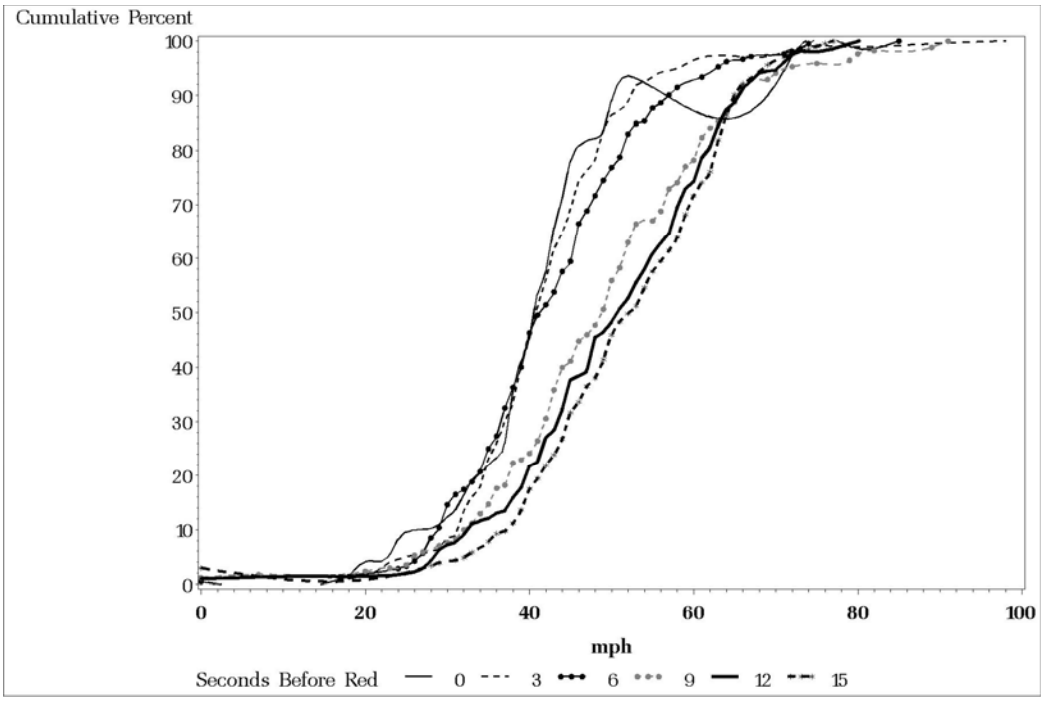

Figure B-20 Cumulative speeds for the northbound AM peak 300 ft. sensor for P2.

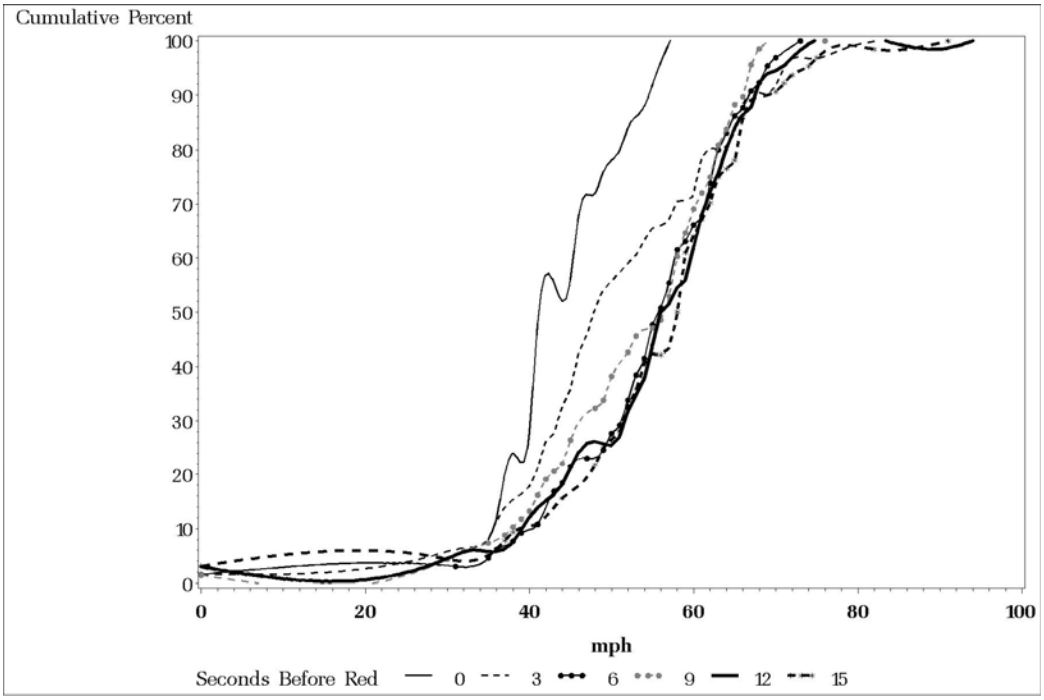

Figure B-21 Cumulative speeds for the northbound AM peak 300 ft. sensor for P3.

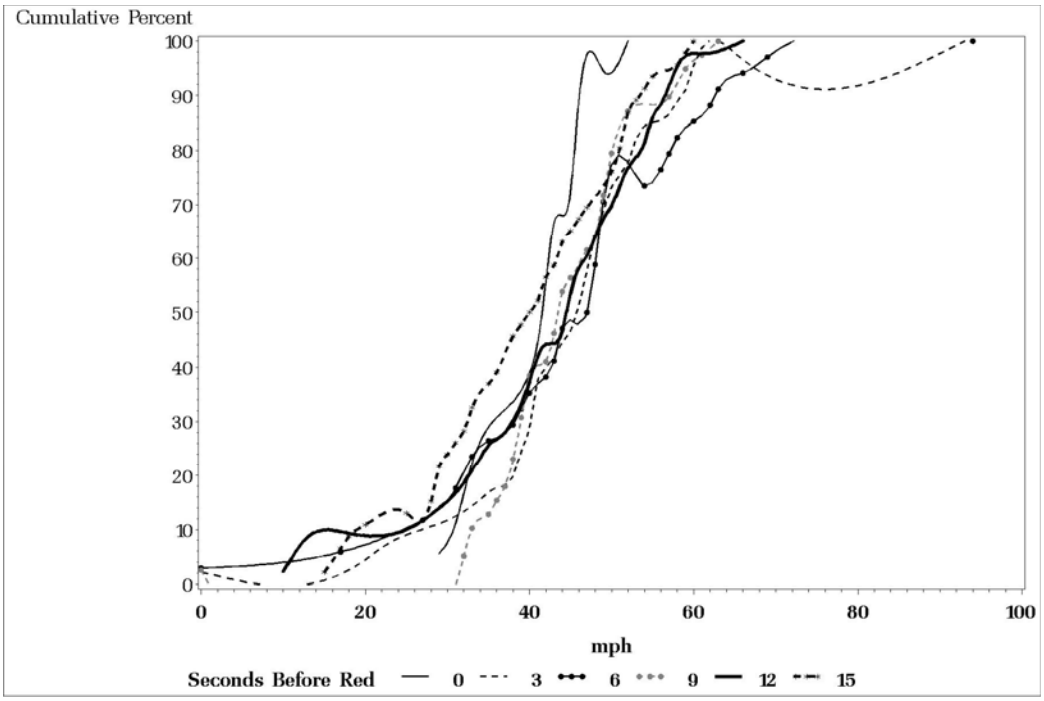

Figure B-22 Cumulative speeds for the southbound AM peak 300 ft. sensor for P1.

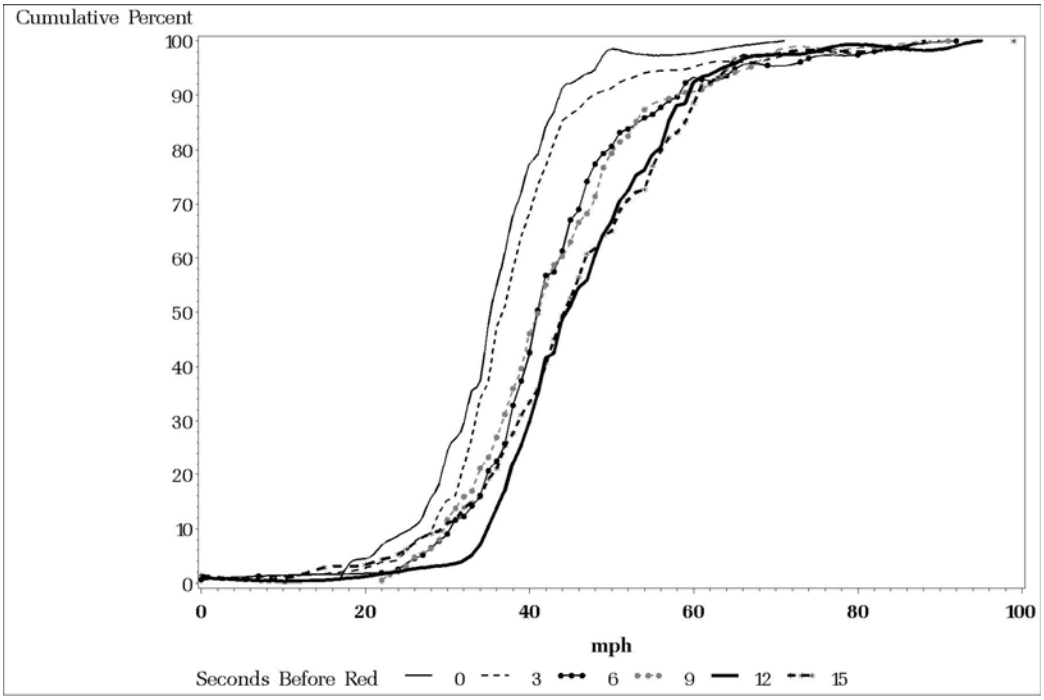

Figure B-23 Cumulative speeds for the southbound AM peak 300 ft. sensor for P2.

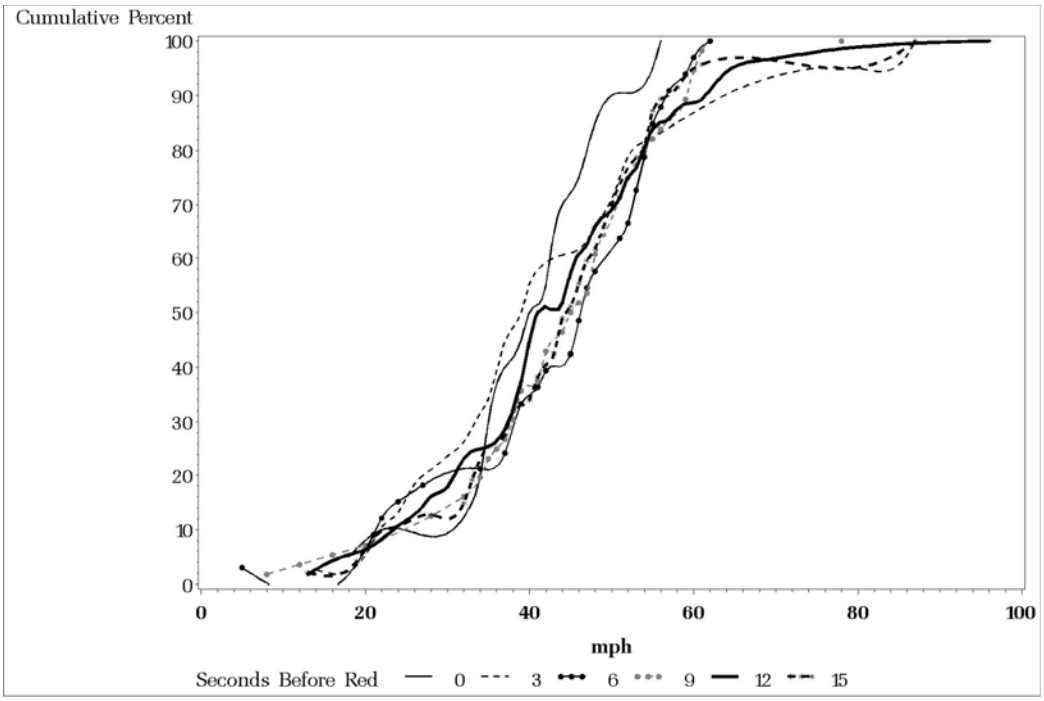

Figure B-24 Cumulative speeds for the southbound AM peak 300 ft. sensor for P3.

Appendix C

Cumulative Distributions of the Speed Data Results for the Noon Peak

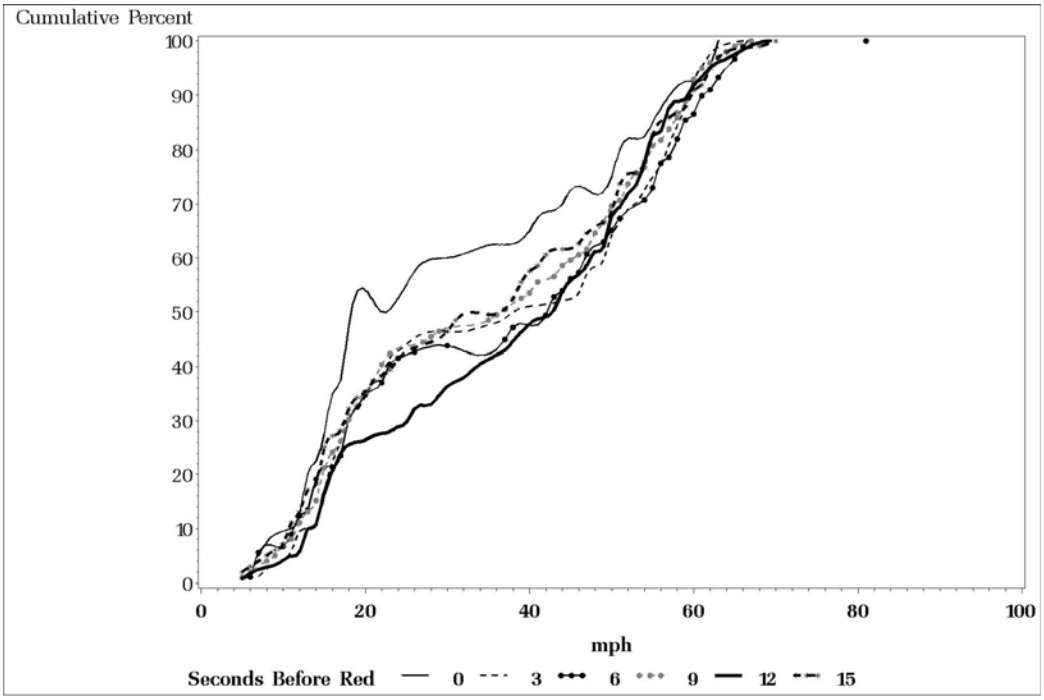

Figure C-1 Cumulative speeds for the northbound noon peak 50 ft. sensor for P1.

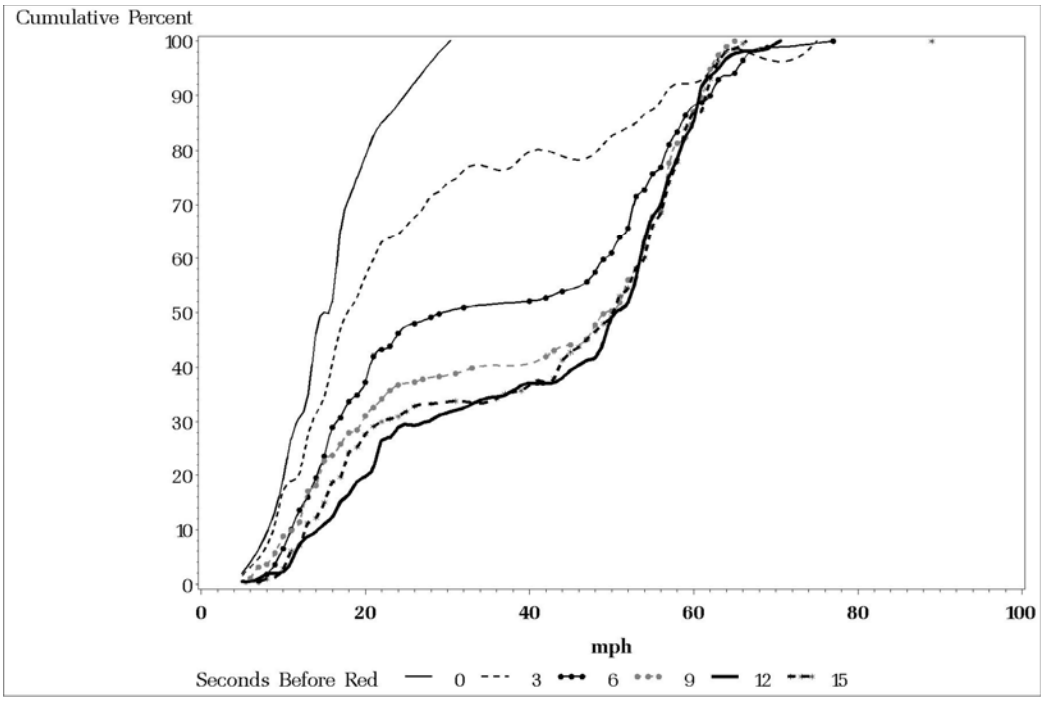

Figure C-2 Cumulative speeds for the northbound noon peak 50 ft. sensor for P2.

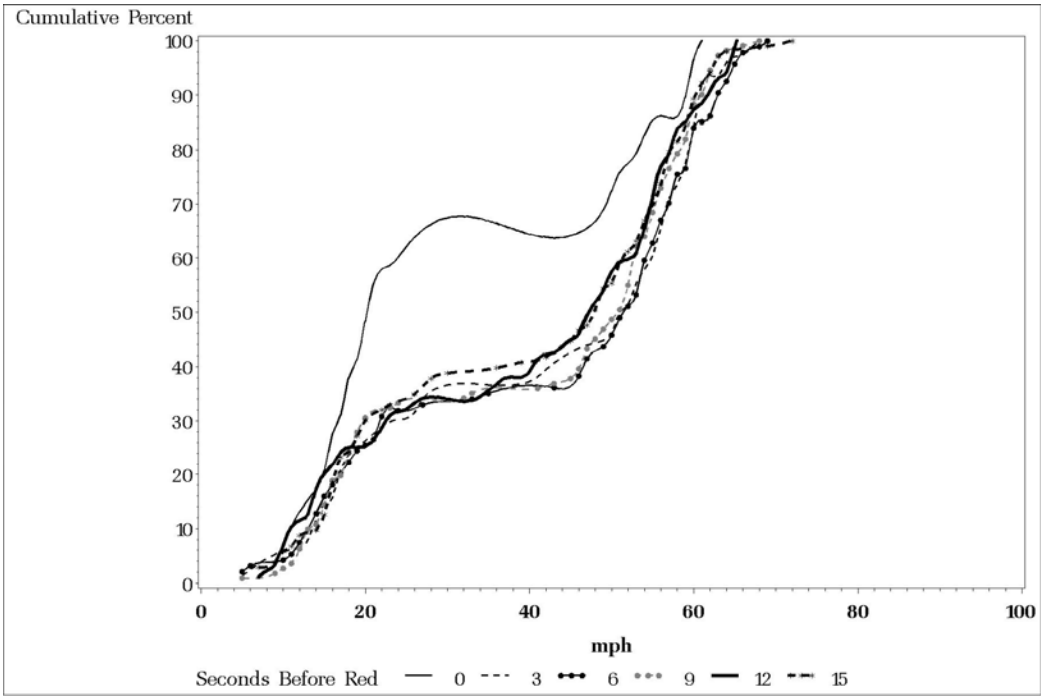

Figure C-3 Cumulative speeds for the northbound noon peak 50 ft. sensor for P3.

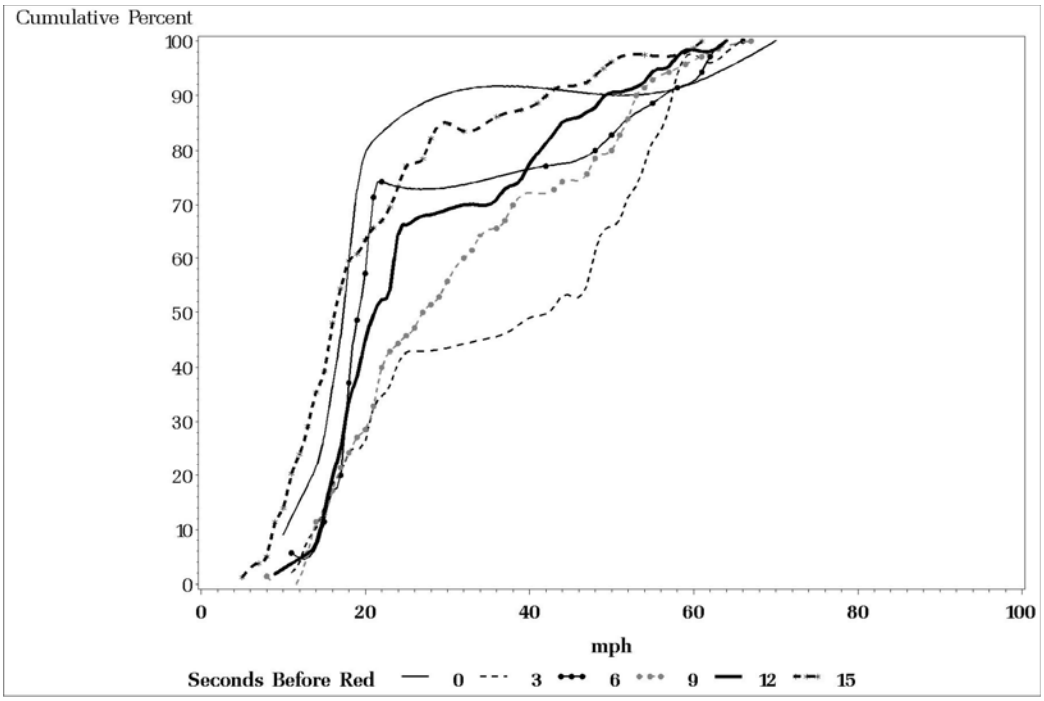

Figure C-4 Cumulative speeds for the southbound noon peak 50 ft. sensor for P1.

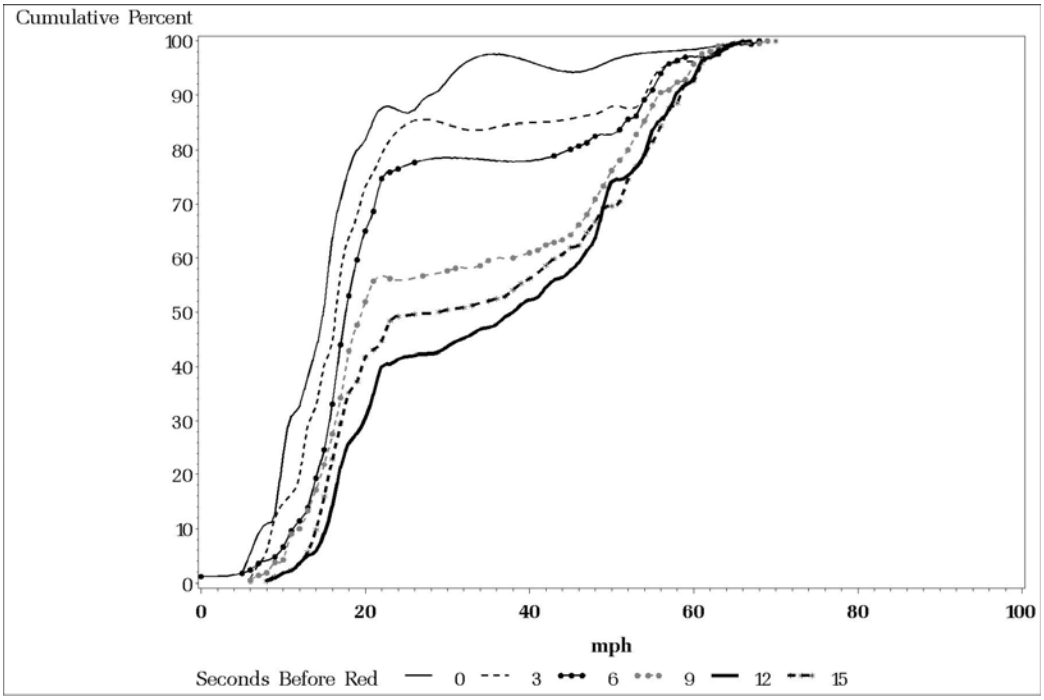

Figure C-5 Cumulative speeds for the southbound noon peak 50 ft. sensor for P2.

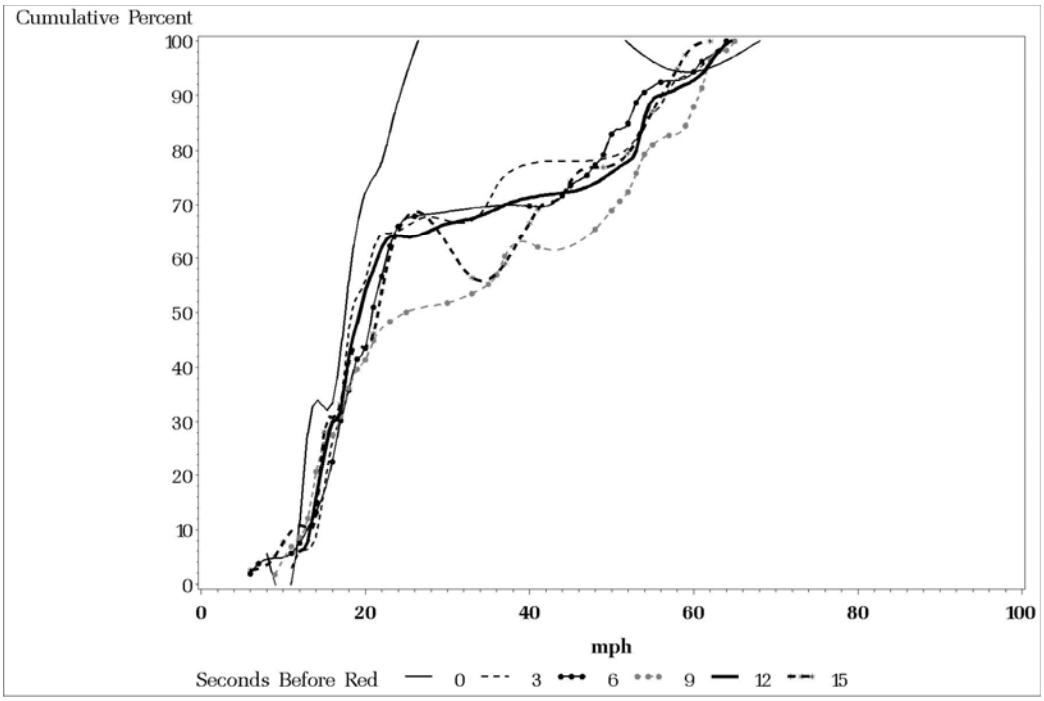

Figure C-6 Cumulative speeds for the southbound noon peak 50 ft. sensor for P3.

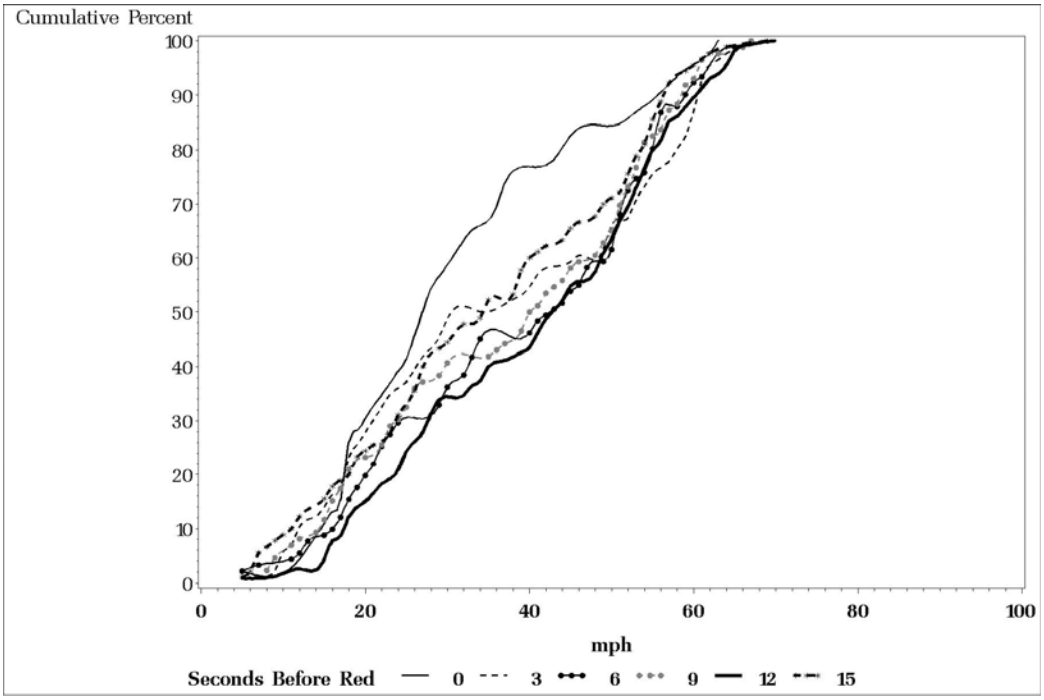

Figure C-7 Cumulative speeds for the northbound noon peak 100 ft. sensor for P1.

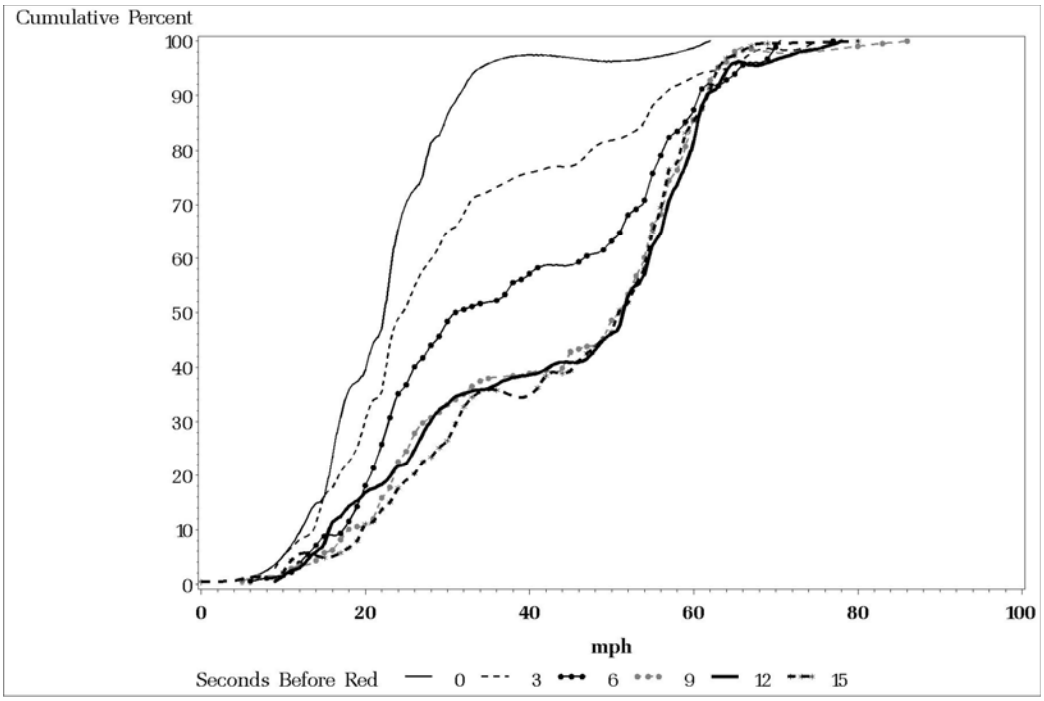

Figure C-8 Cumulative speeds for the northbound noon peak 100 ft. sensor for P2.

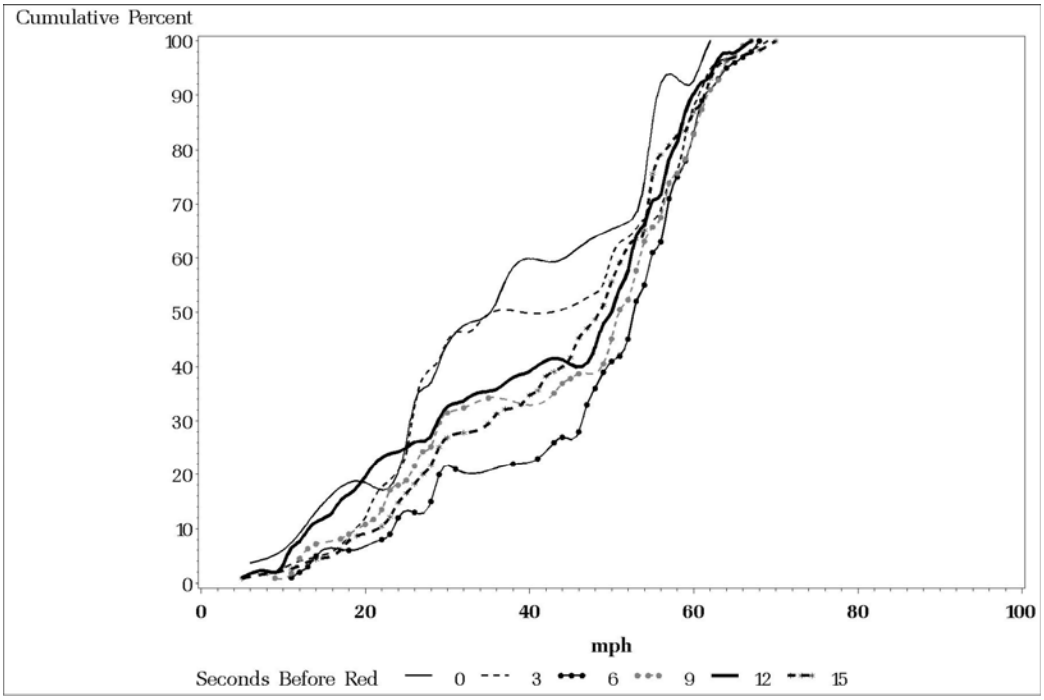

Figure C-9 Cumulative speeds for the northbound noon peak 100 ft. sensor for P3.

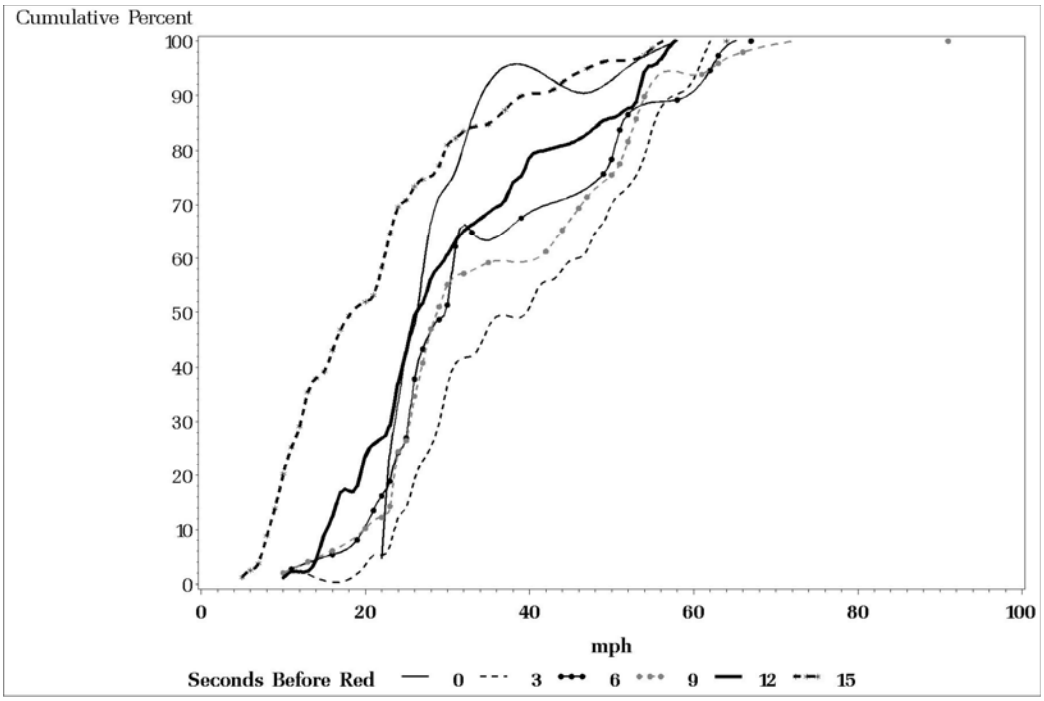

Figure C-10 Cumulative speeds for the southbound noon peak 100 ft. sensor for P1.

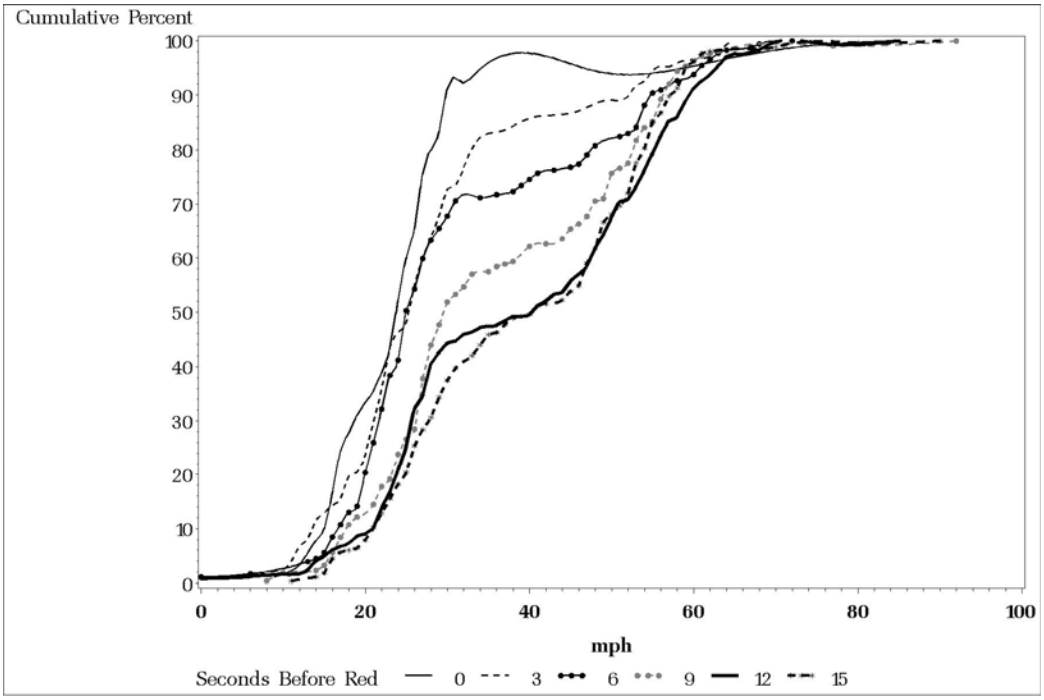

Figure C-11 Cumulative speeds for the southbound noon peak 100 ft. sensor for P2.

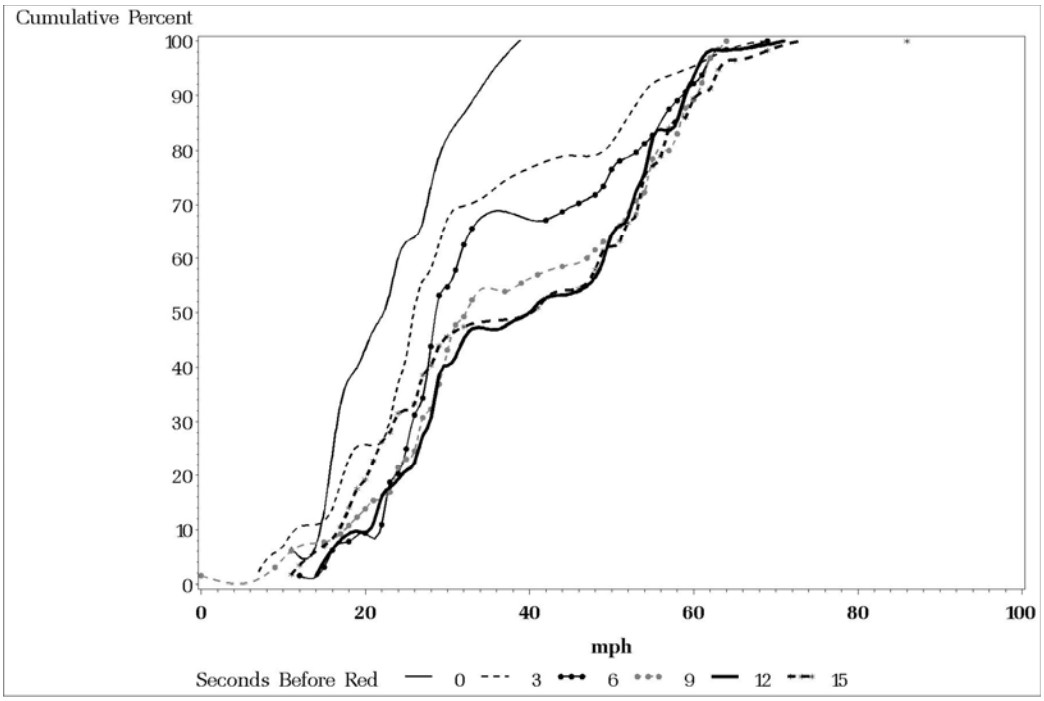

Figure C-12 Cumulative speeds for the southbound noon peak 100 ft. sensor for P3.

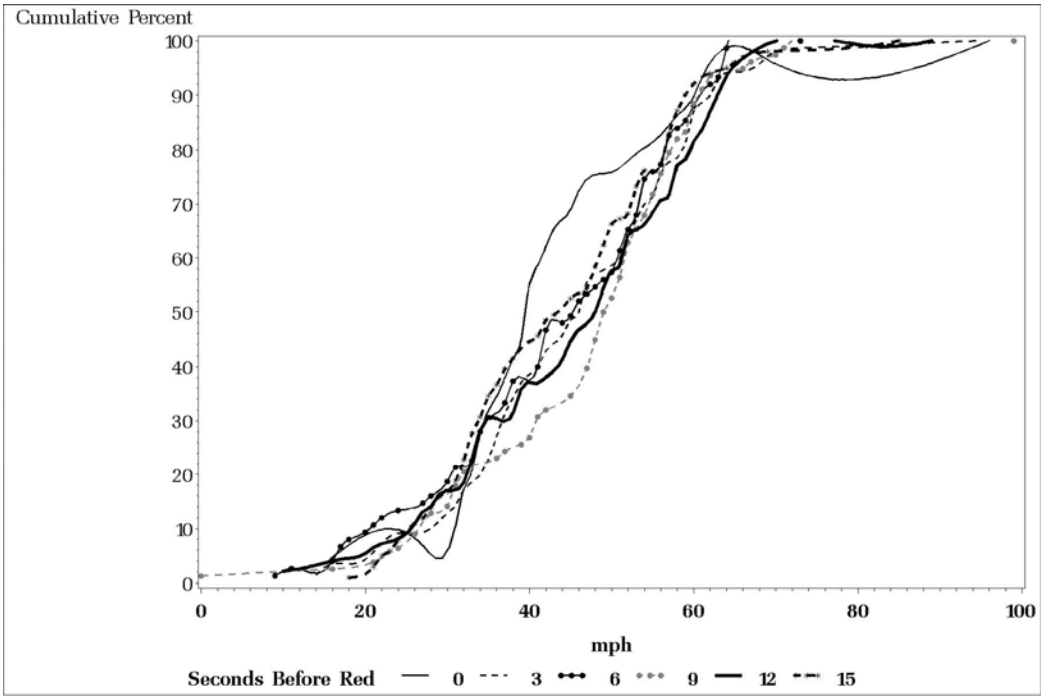

Figure C-13 Cumulative speeds for the northbound noon peak 200 ft. sensor for P1.

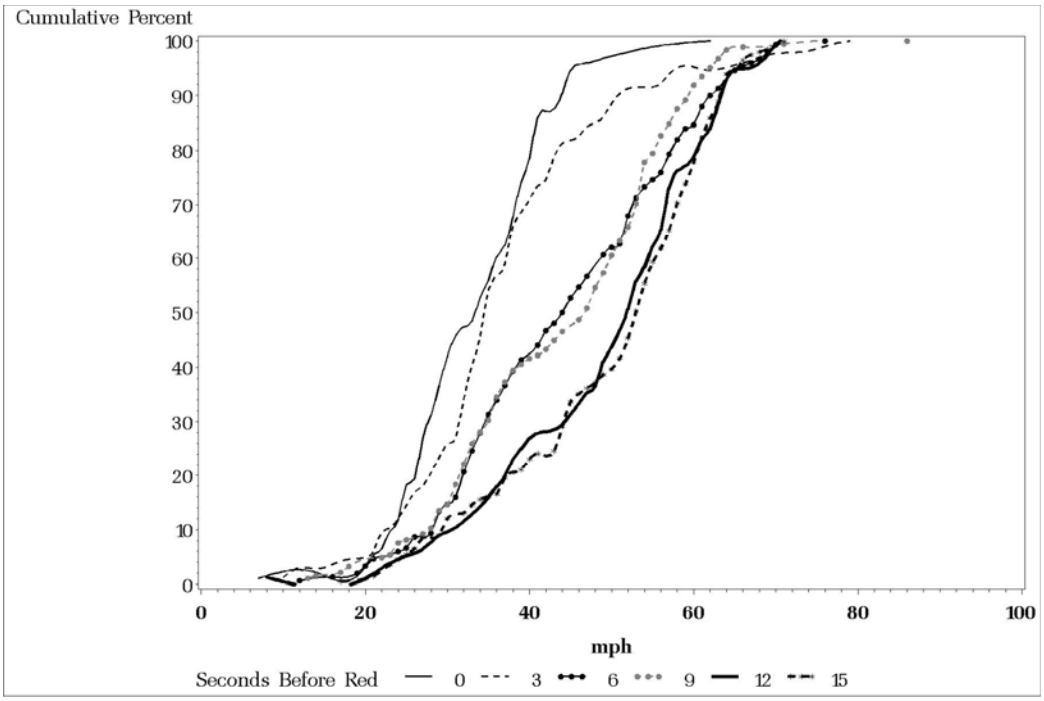

Figure C-14 Cumulative speeds for the northbound noon peak 200 ft. sensor for P2.

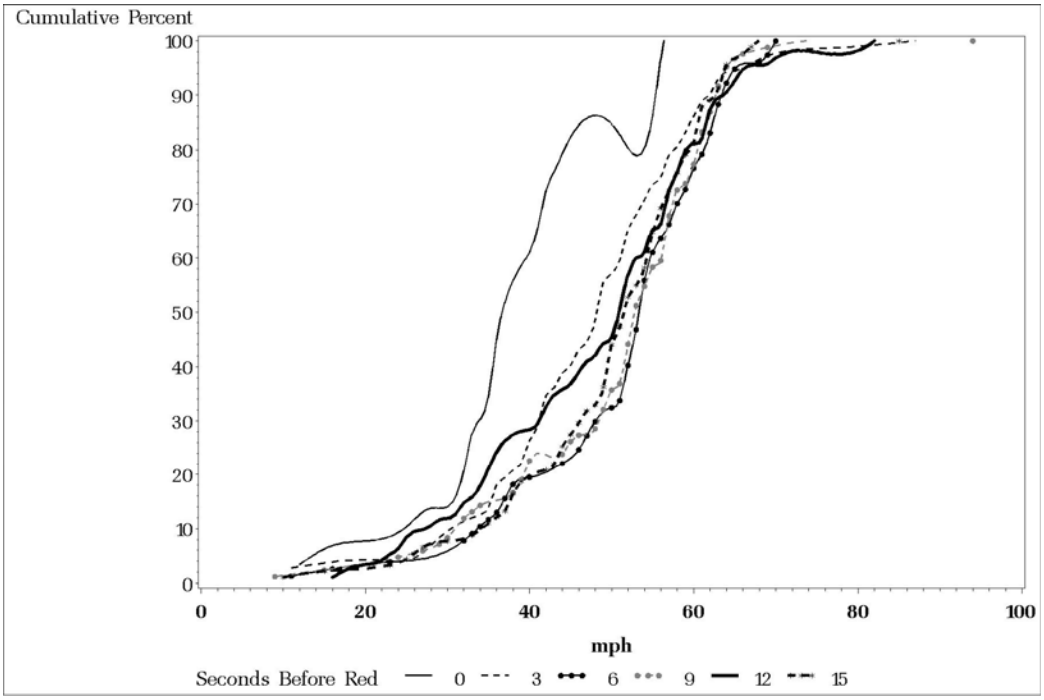

Figure C-15 Cumulative speeds for the northbound noon peak 200 ft. sensor for P3.

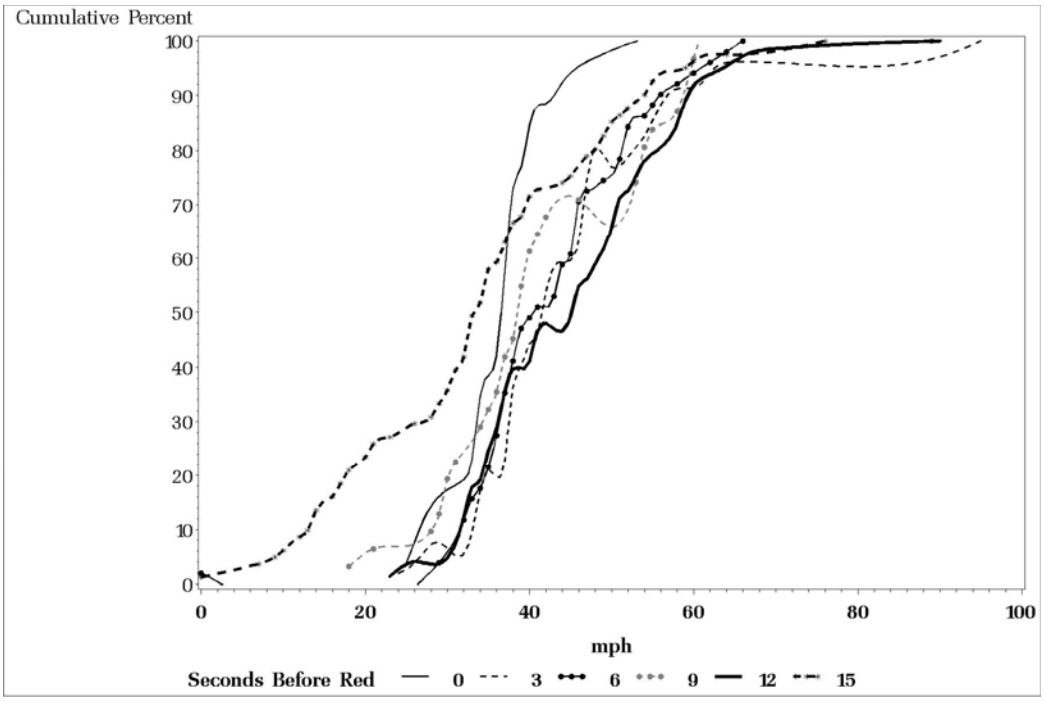

Figure C-16 Cumulative speeds for the southbound noon peak 200 ft. sensor for P1.

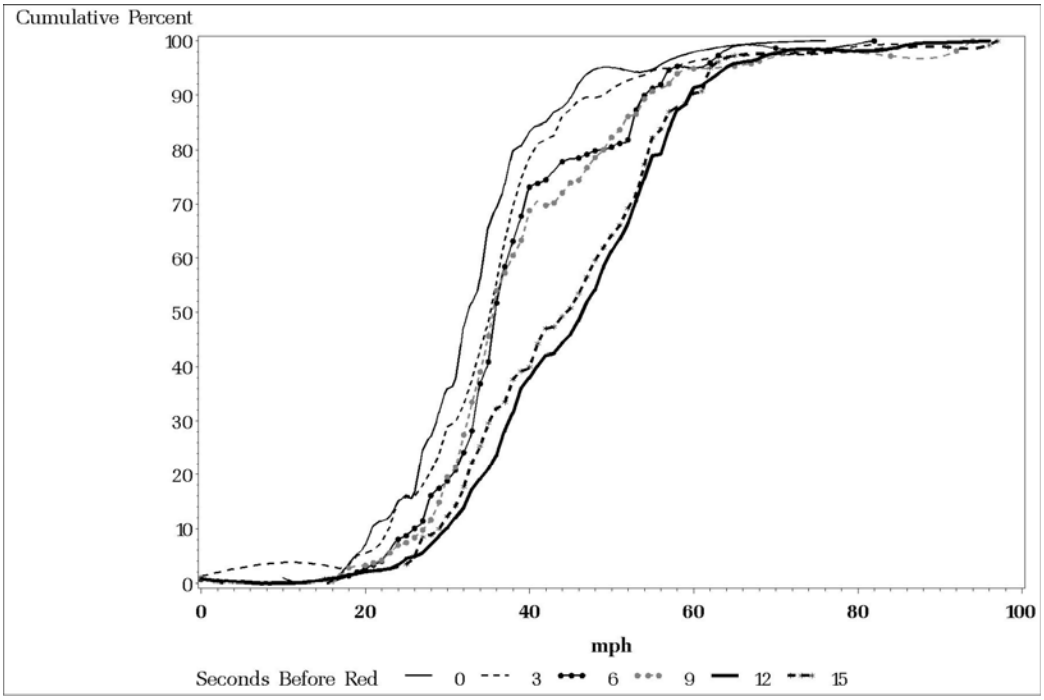

Figure C-17 Cumulative speeds for the southbound noon peak 200 ft. sensor for P2.

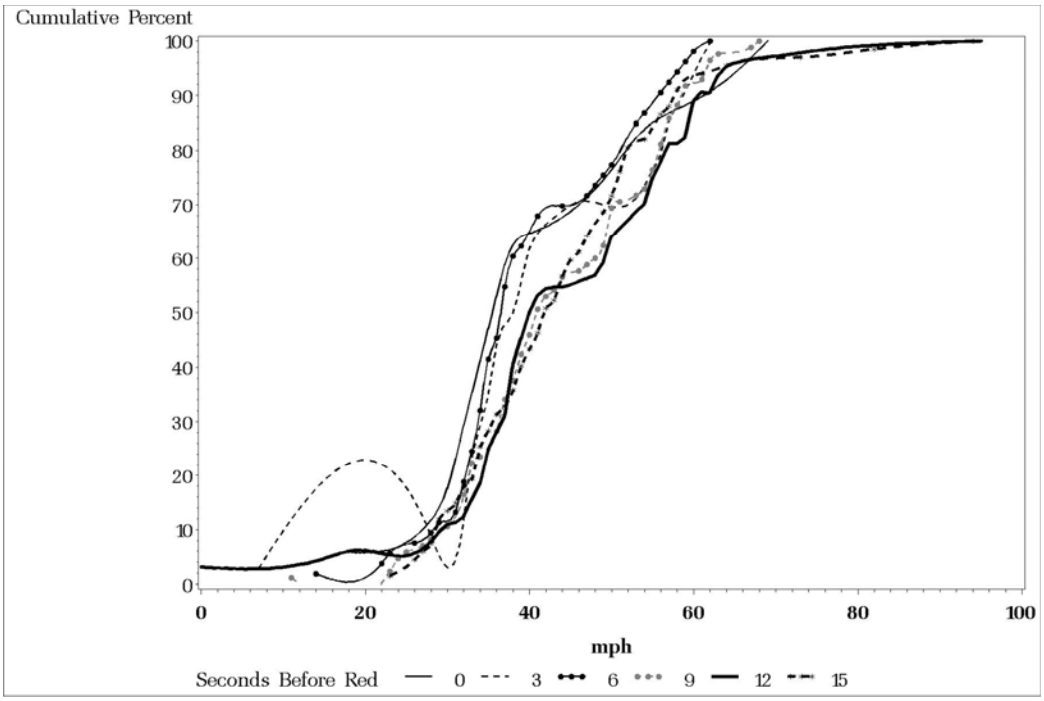

Figure C-18 Cumulative speeds for the southbound noon peak 200 ft. sensor for P3.

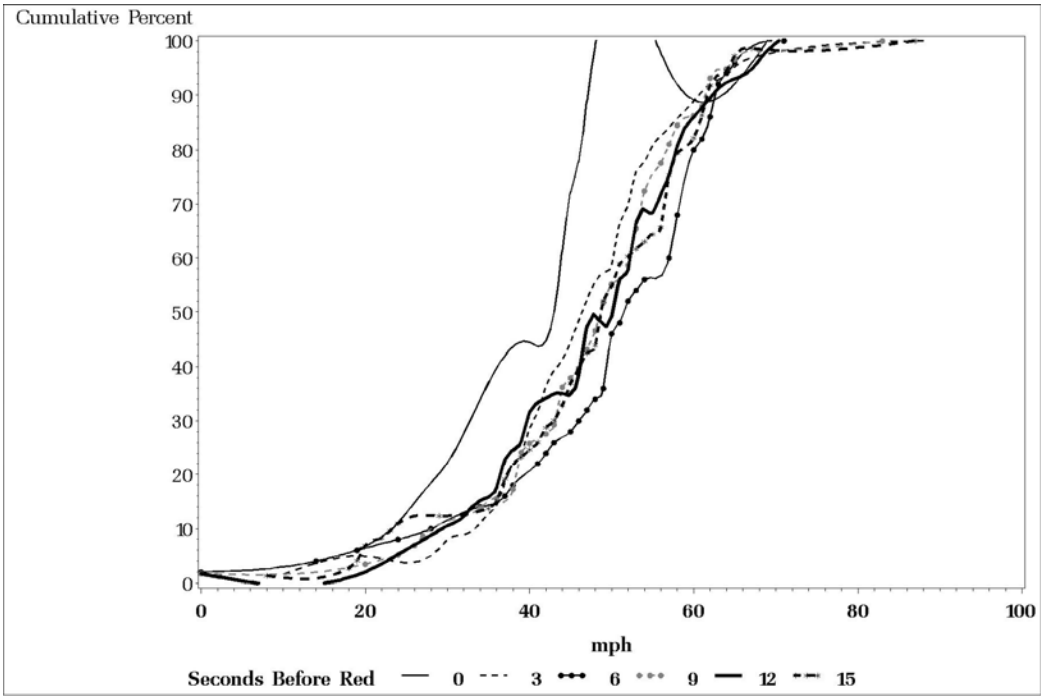

Figure C-19 Cumulative speeds for the northbound noon peak 300 ft. sensor for P1.

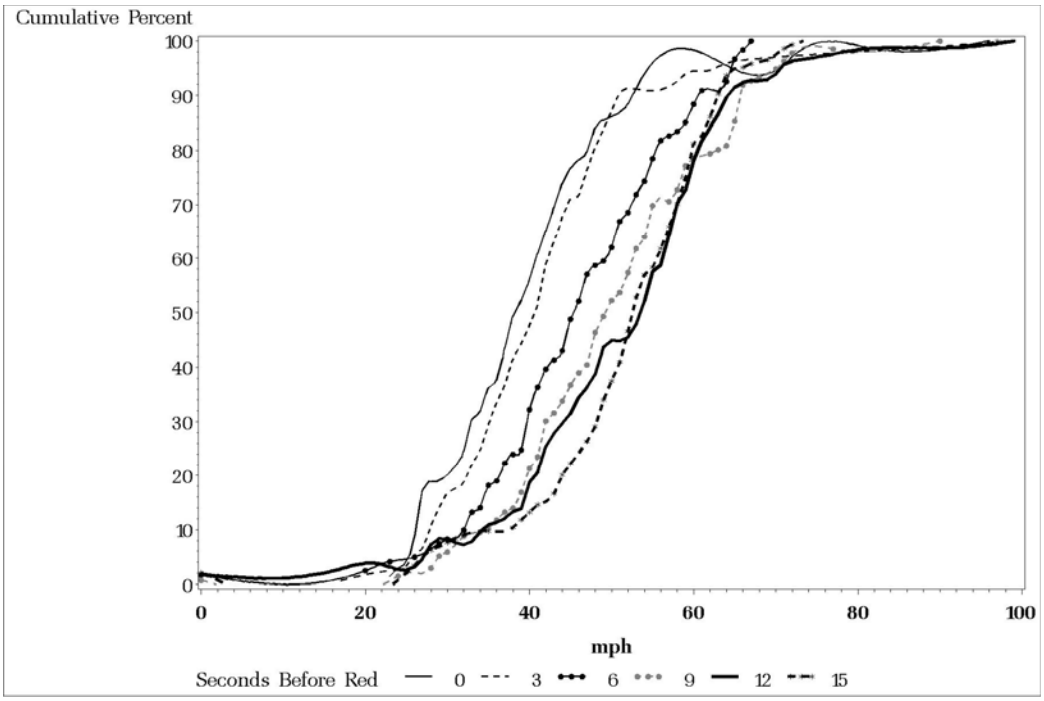

Figure C-20 Cumulative speeds for the northbound noon peak 300 ft. sensor for P2.

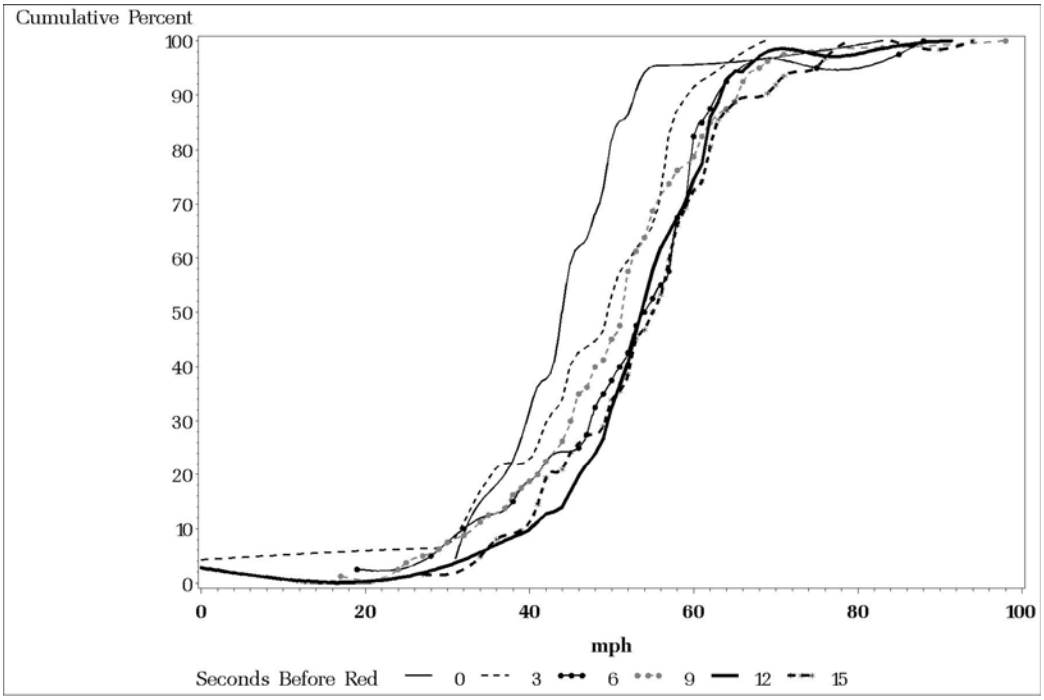

Figure C-21 Cumulative speeds for the northbound noon peak 300 ft. sensor for P3.

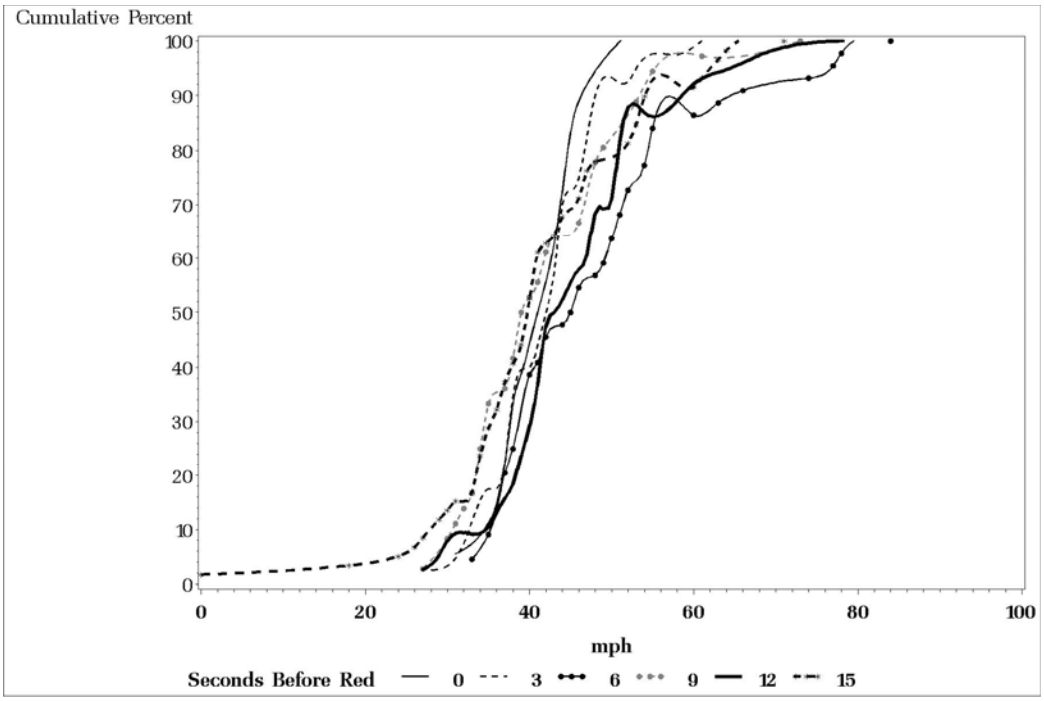

Figure C-22 Cumulative speeds for the southbound noon peak 300 ft. sensor for P1.

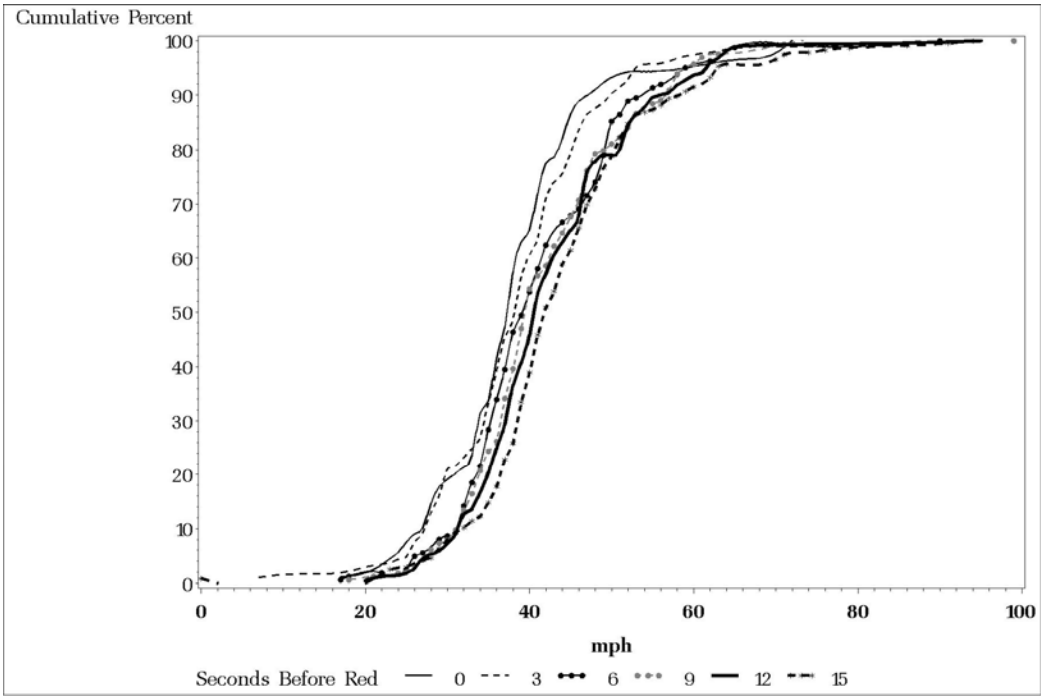

Figure C-23 Cumulative speeds for the southbound noon peak 300 ft. sensor for P2.

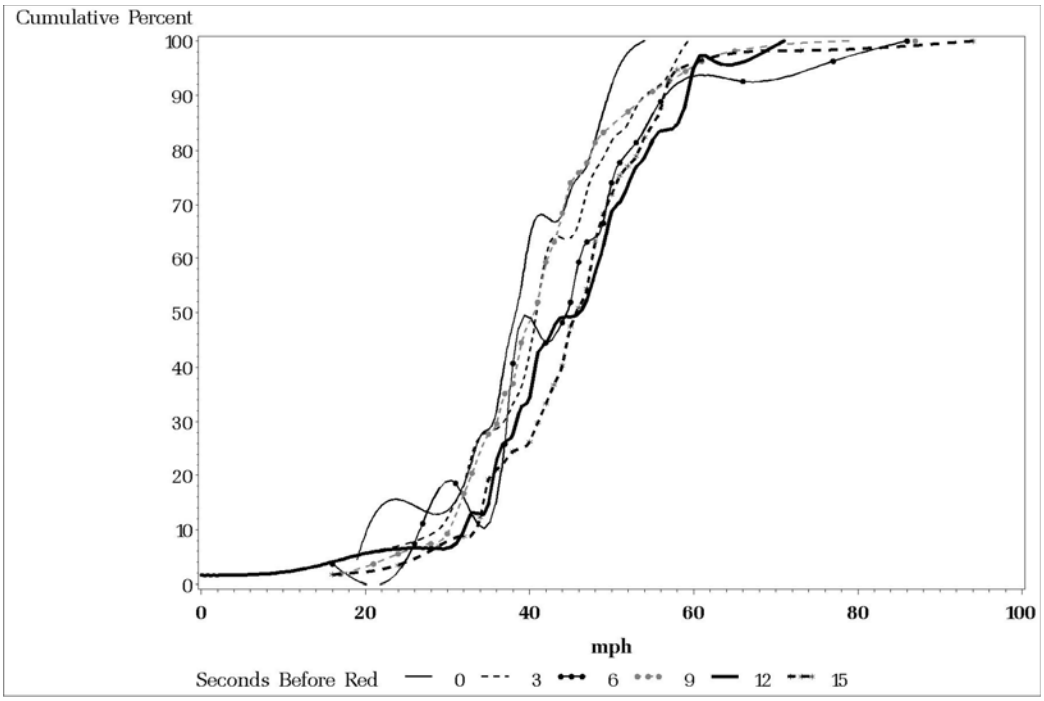

Figure C-24 Cumulative speeds for the southbound noon peak 300 ft. sensor for P3.

Appendix D

Cumulative Distributions of the Speed Data Results for the PM Peak

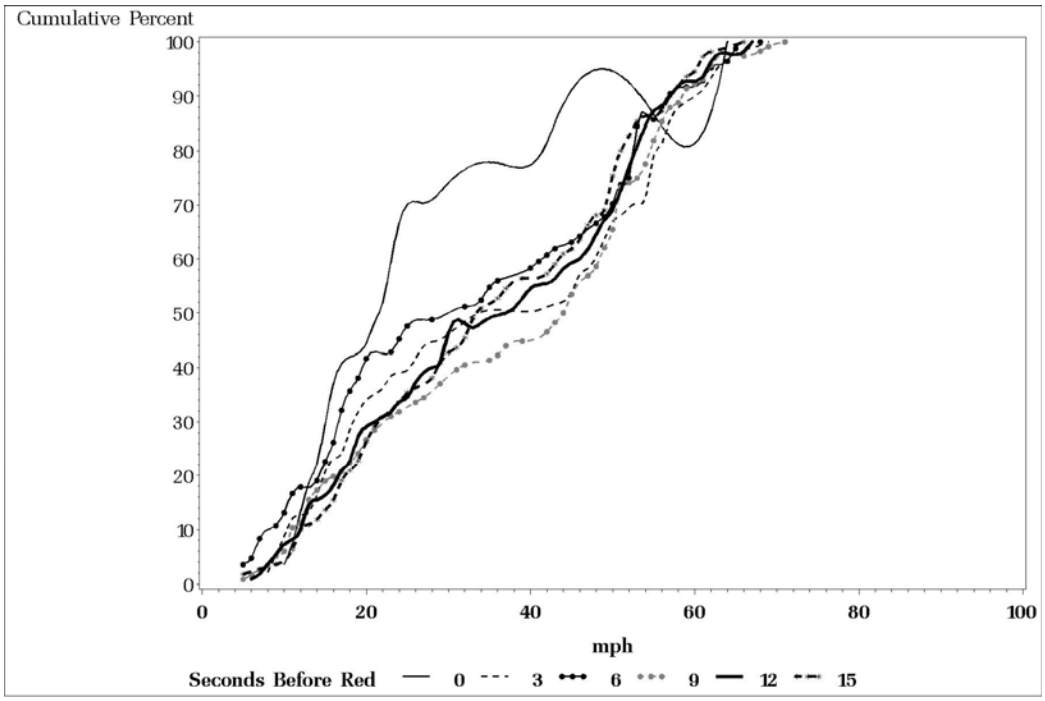

Figure D-1 Cumulative speeds for the northbound PM peak 50 ft. sensor for P1.

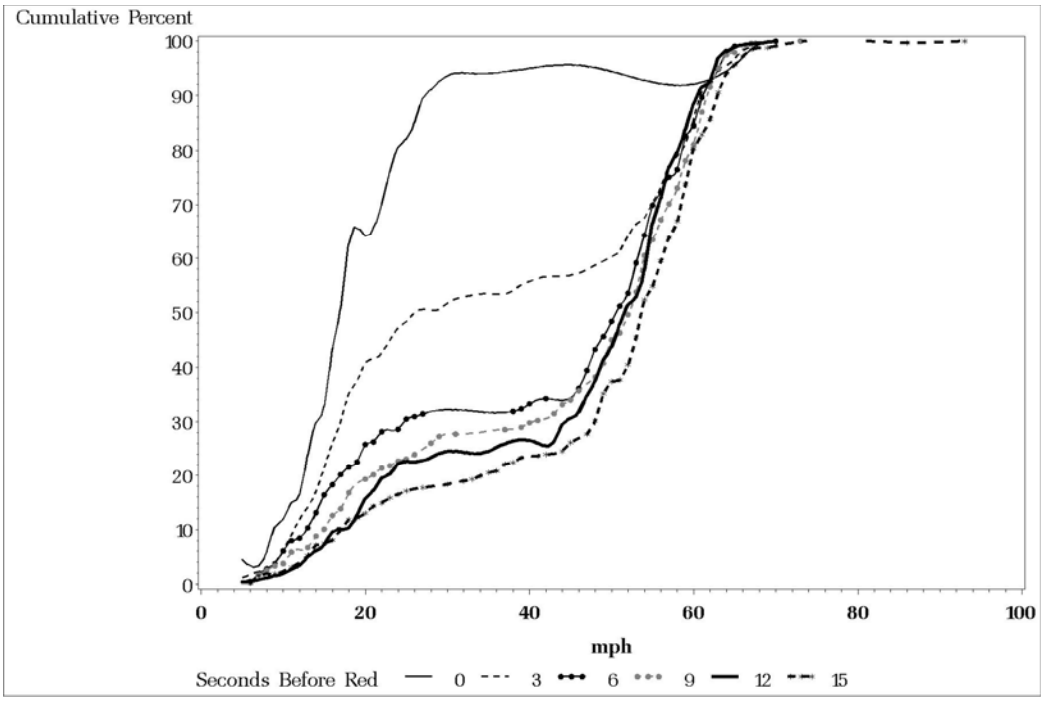

Figure D-2 Cumulative speeds for the northbound PM peak 50 ft. sensor for P2.

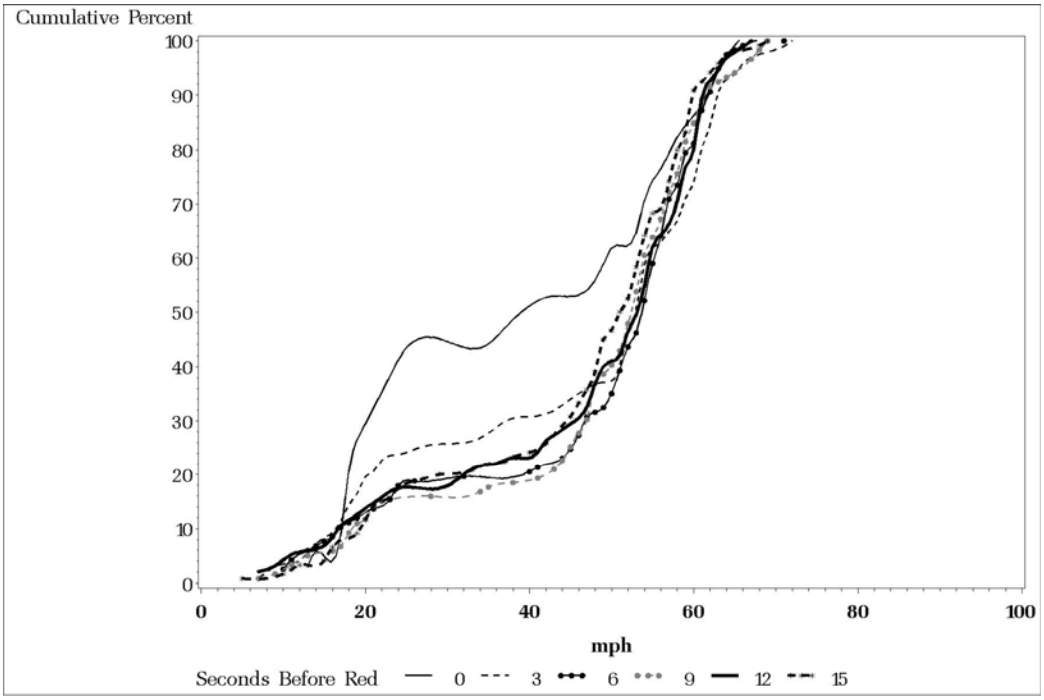

Figure D-3 Cumulative speeds for the northbound PM peak 50 ft. sensor for P3.

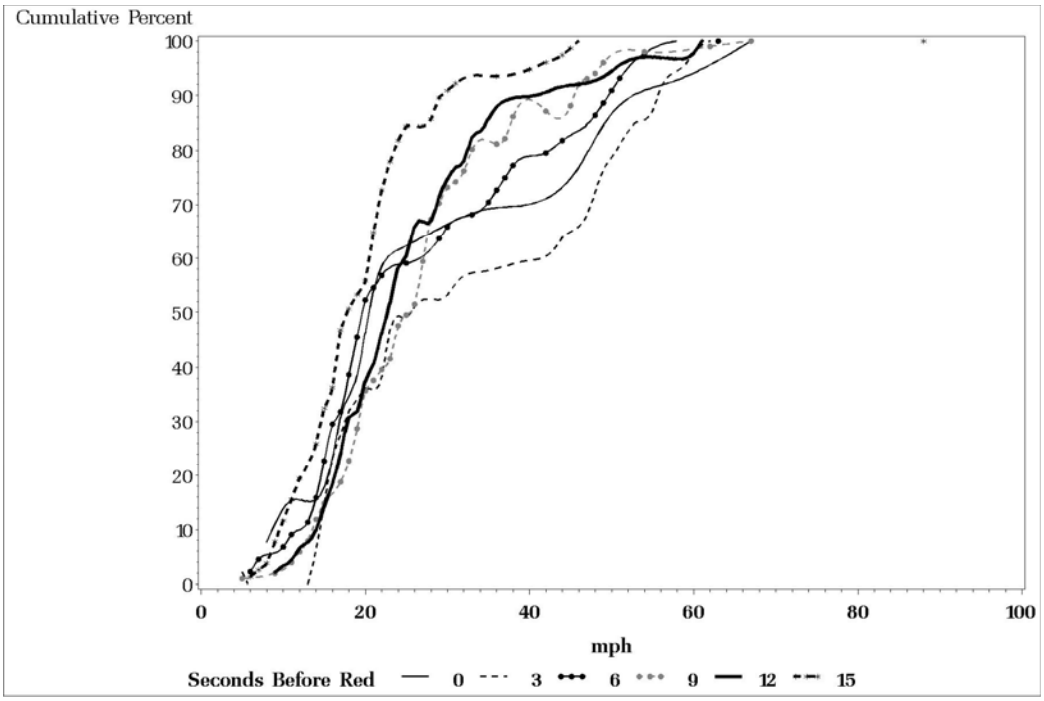

Figure D-4 Cumulative speeds for the southbound PM peak 50 ft. sensor for P1.

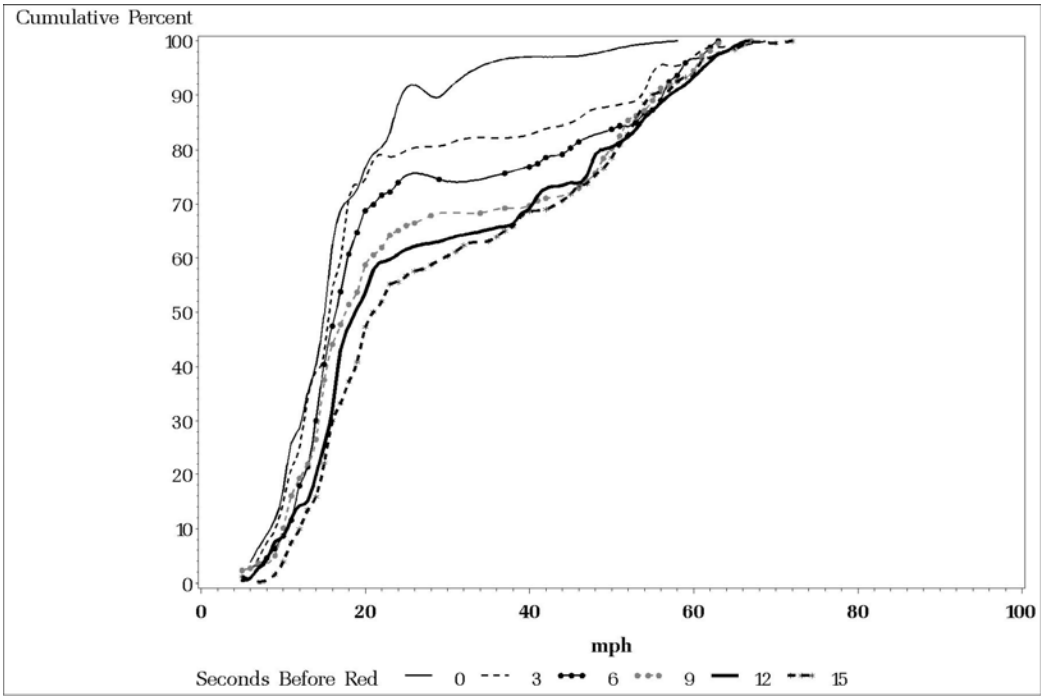

Figure D-5 Cumulative speeds for the southbound PM peak 50 ft. sensor for P2.

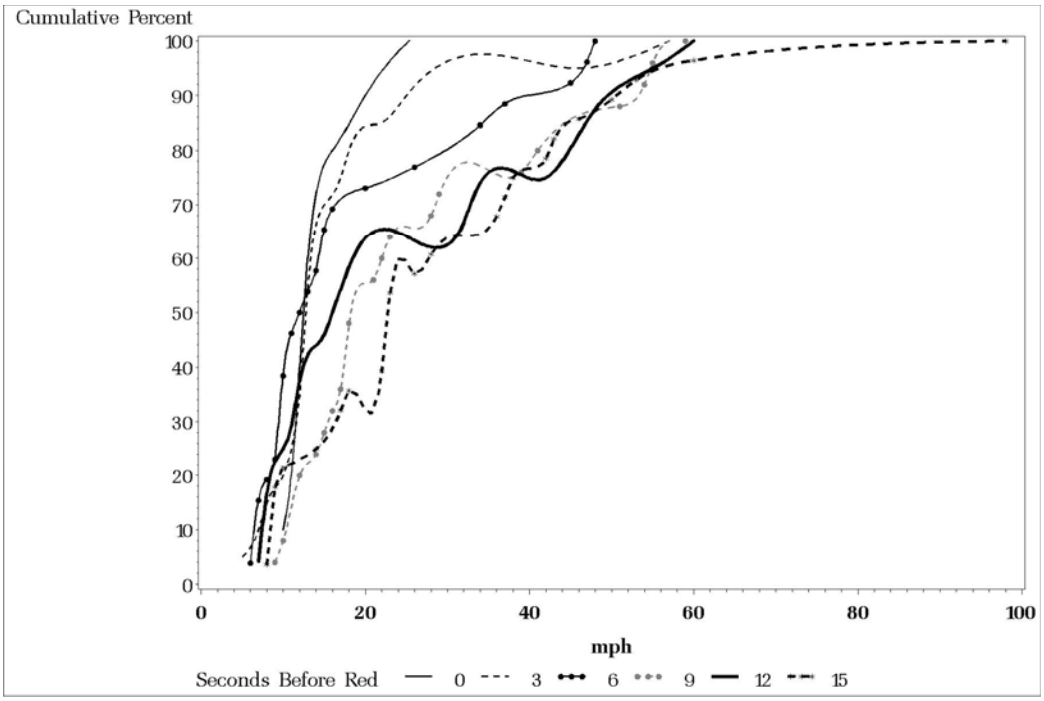

Figure D-6 Cumulative speeds for the southbound PM peak 50 ft. sensor for P3.

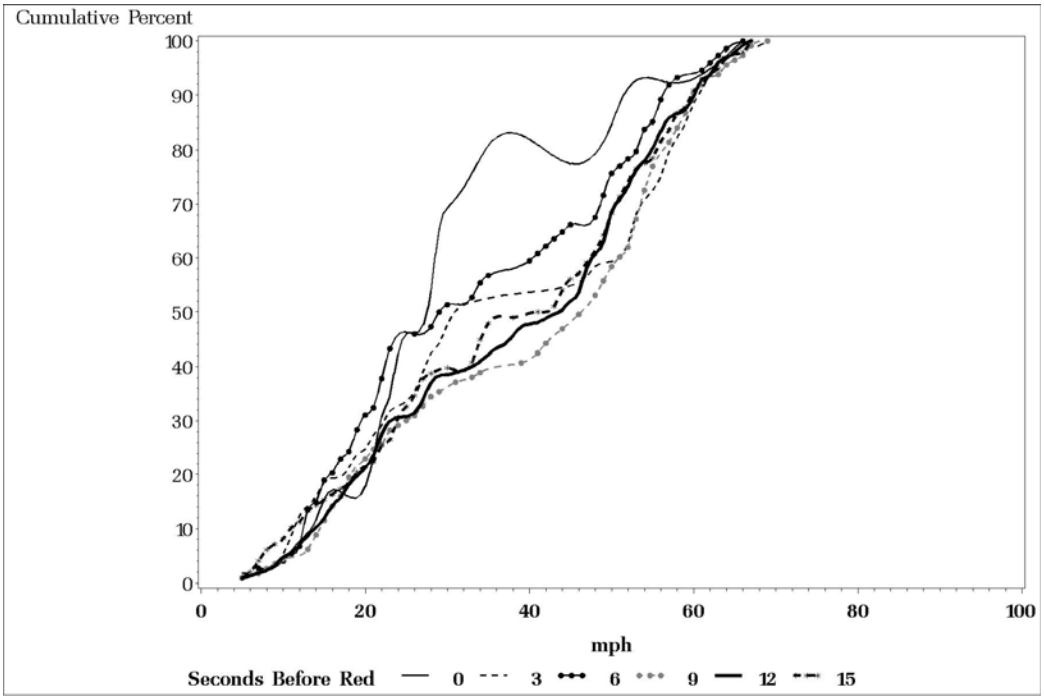

Figure D-7 Cumulative speeds for the northbound PM peak 100 ft. sensor for P1.

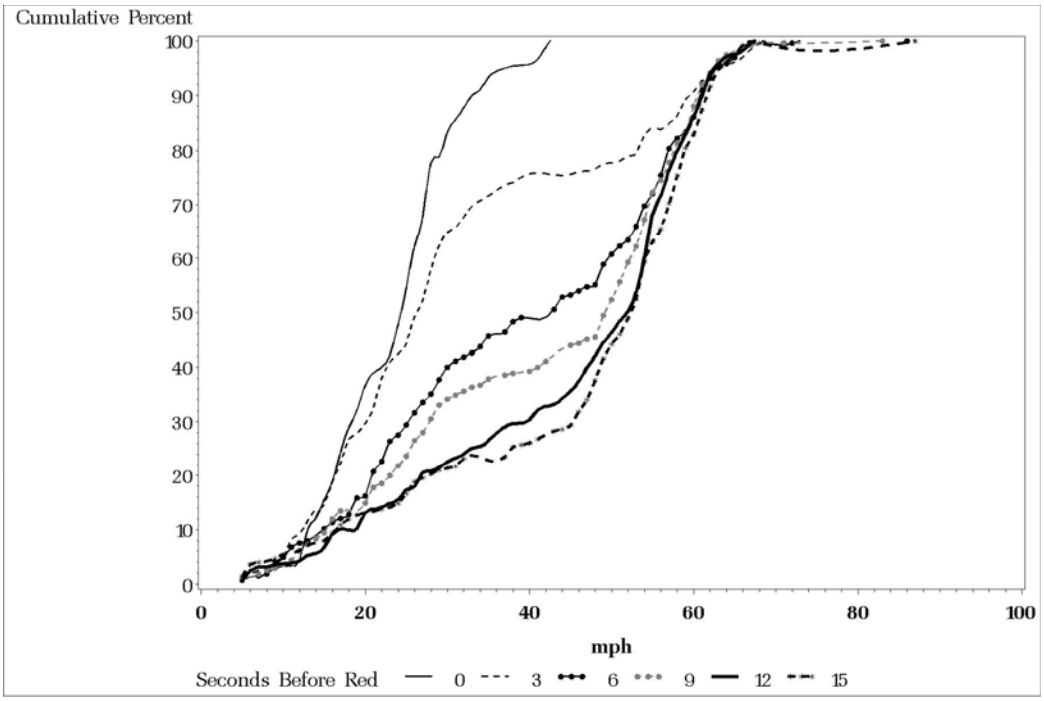

Figure D-8 Cumulative speeds for the northbound PM peak 100 ft. sensor for P2.

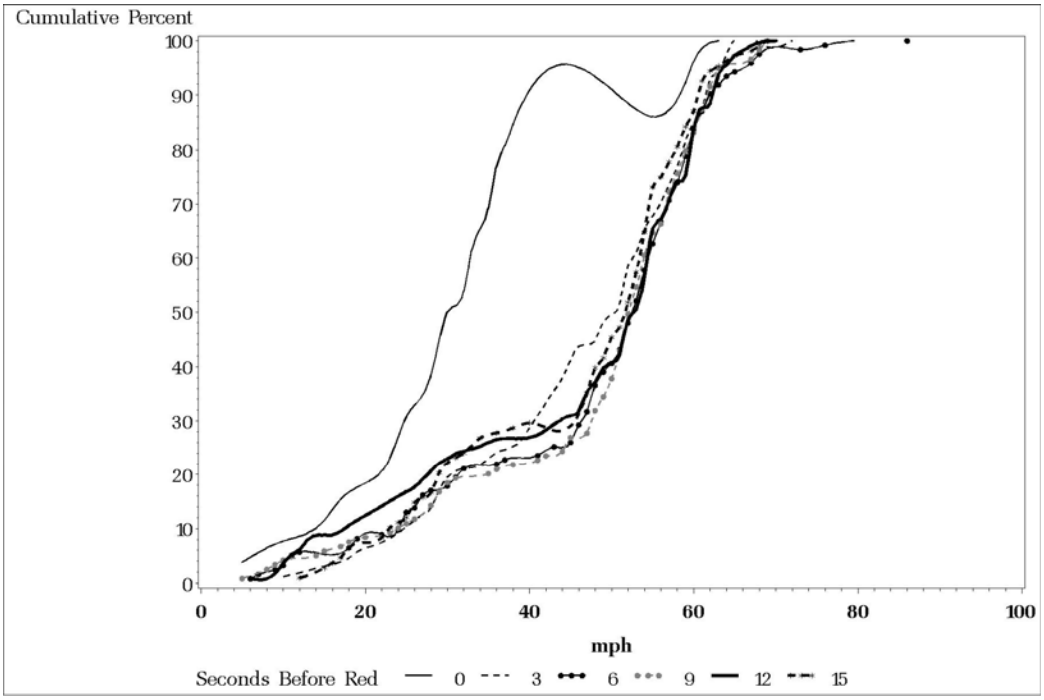

Figure D-9 Cumulative speeds for the northbound PM peak 100 ft. sensor for P3.

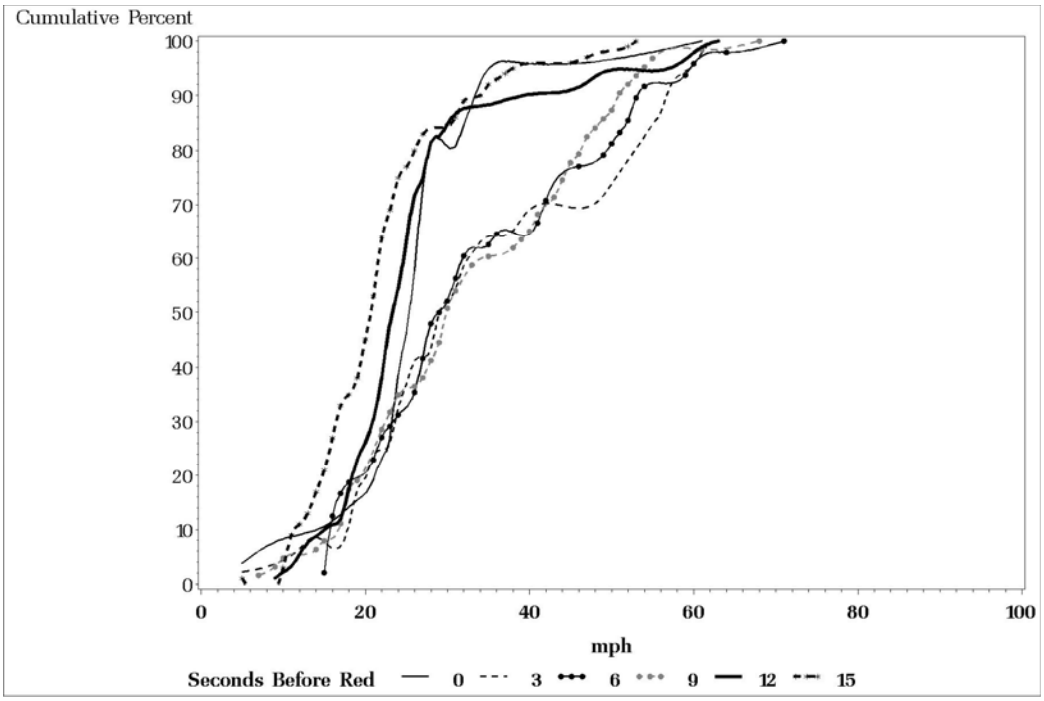

Figure D-10 Cumulative speeds for the southbound PM peak 100 ft. sensor for P1.

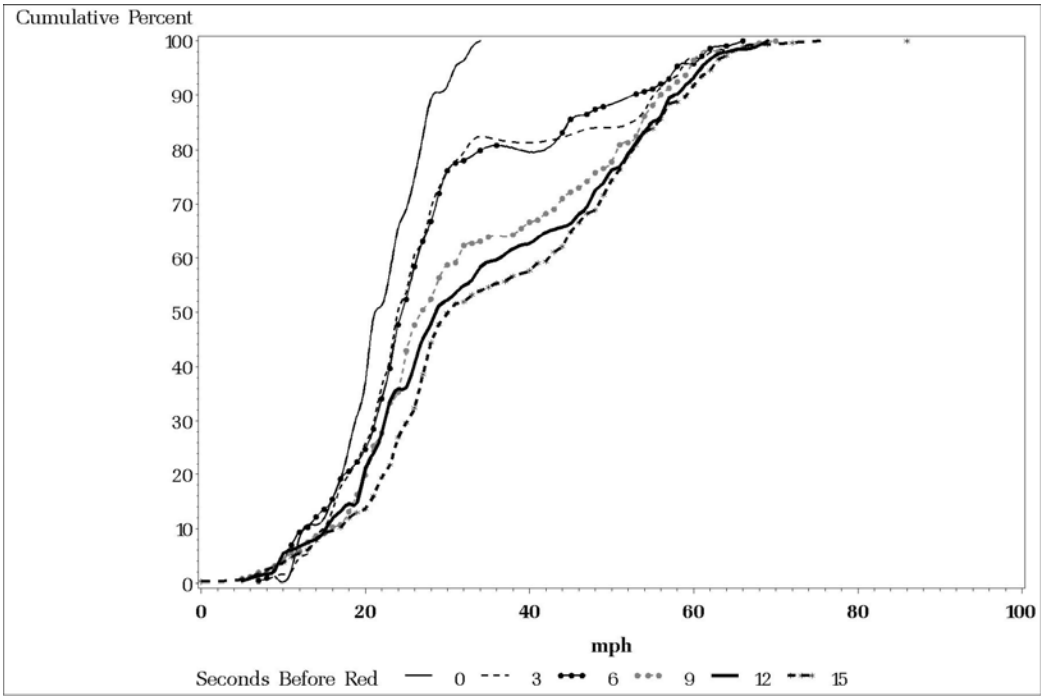

Figure D-11 Cumulative speeds for the southbound PM peak 100 ft. sensor for P2.

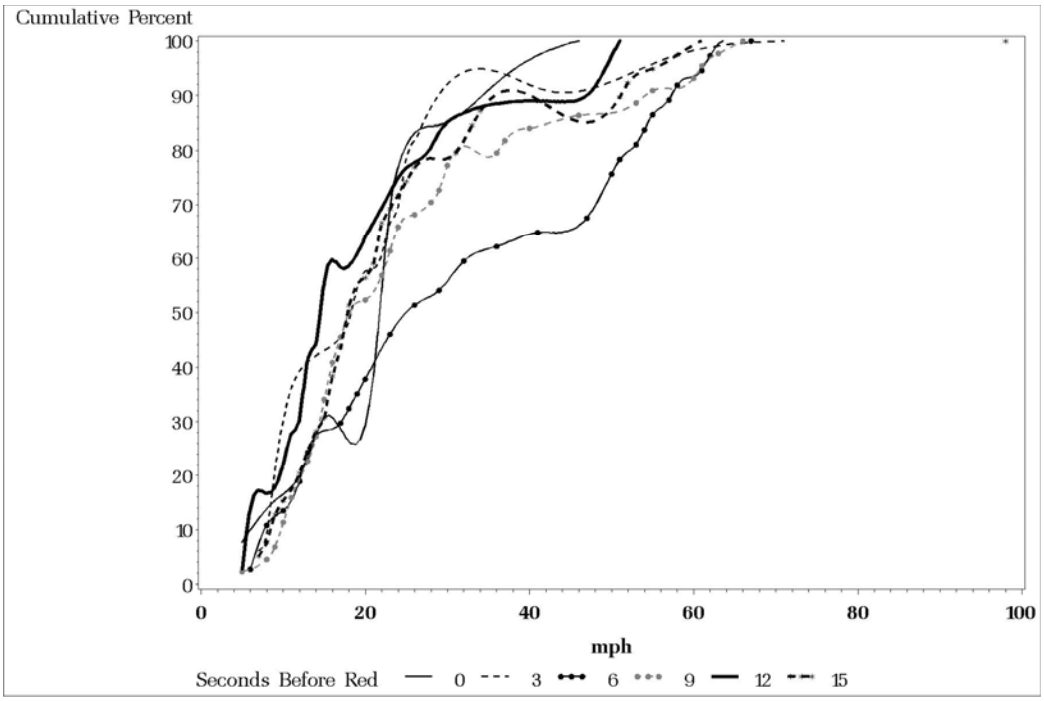

Figure D-12 Cumulative speeds for the southbound PM peak 100 ft. sensor for P3.

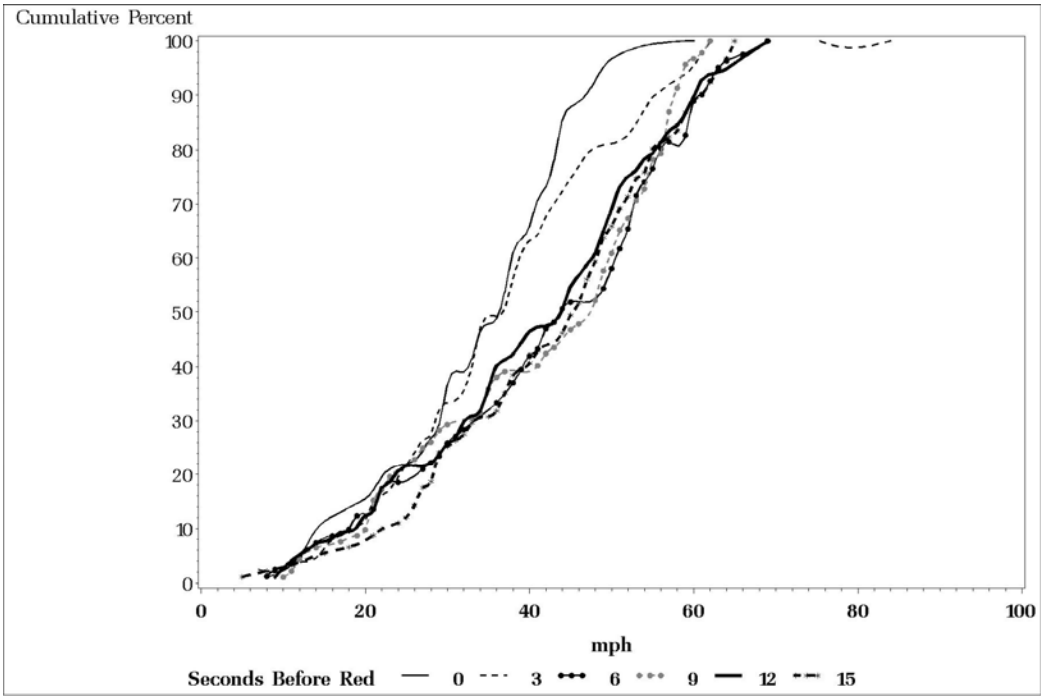

Figure D-13 Cumulative speeds for the northbound PM peak 200 ft. sensor for P1.

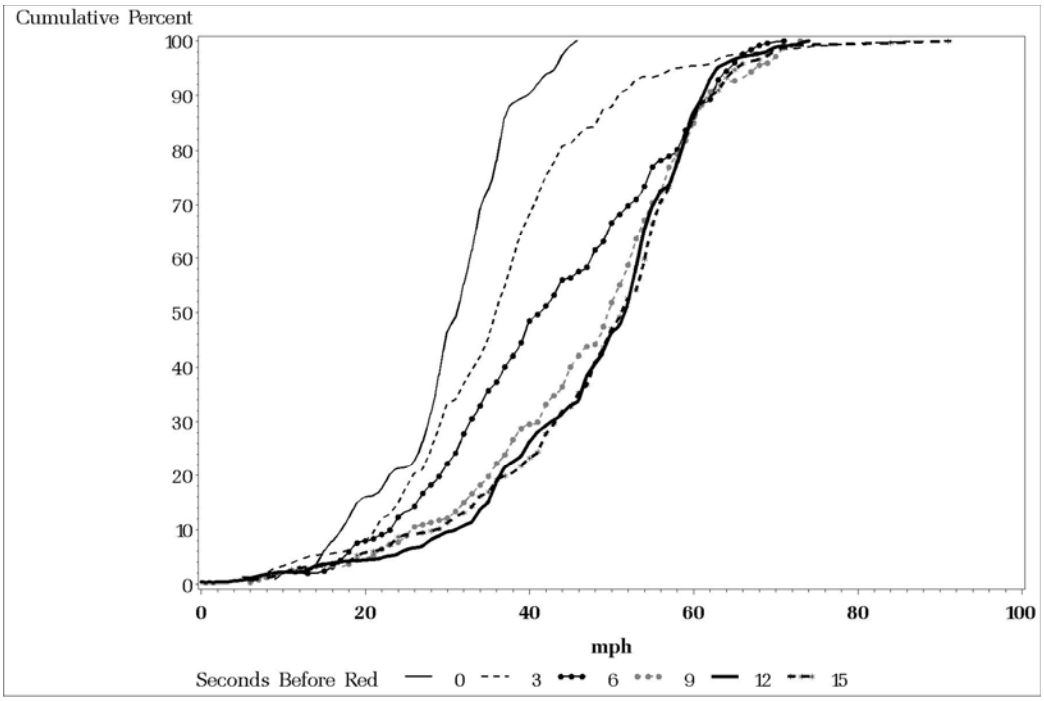

Figure D-14 Cumulative speeds for the northbound PM peak 200 ft. sensor for P2.

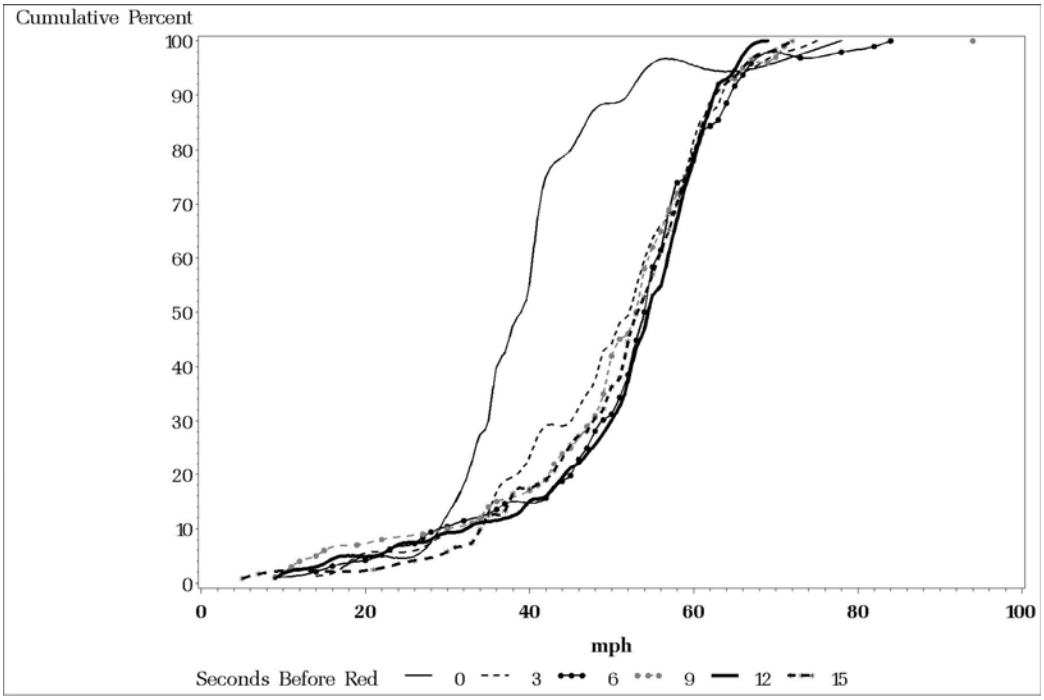

Figure D-15 Cumulative speeds for the northbound PM peak 200 ft. sensor for P3.

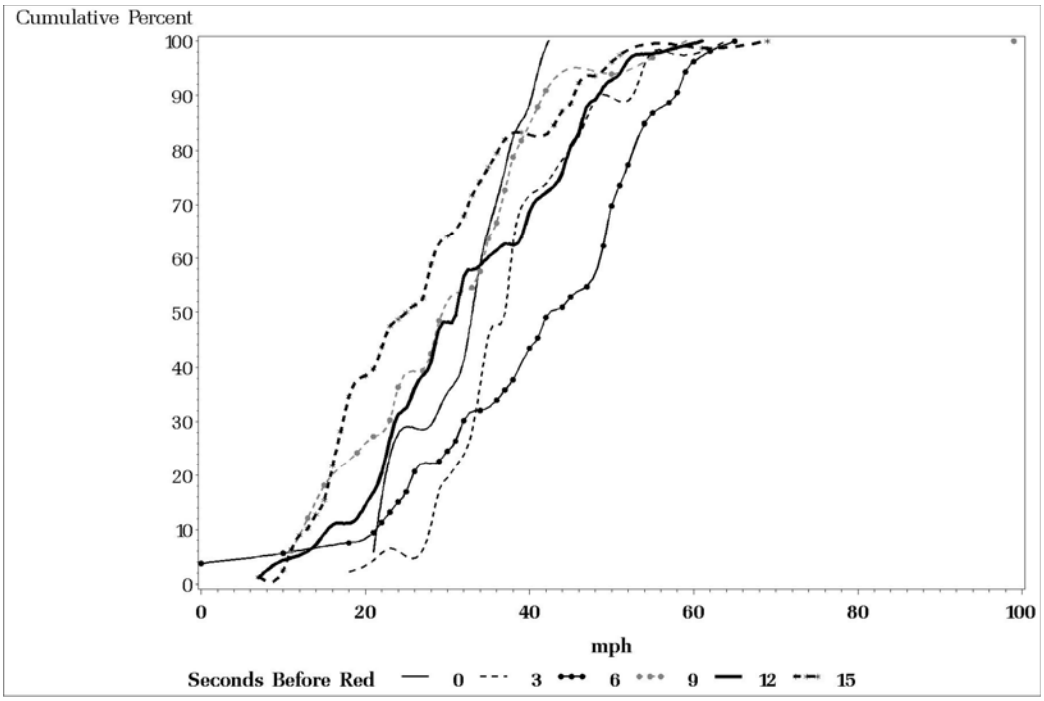

Figure D-16 Cumulative speeds for the southbound PM peak 200 ft. sensor for P1.

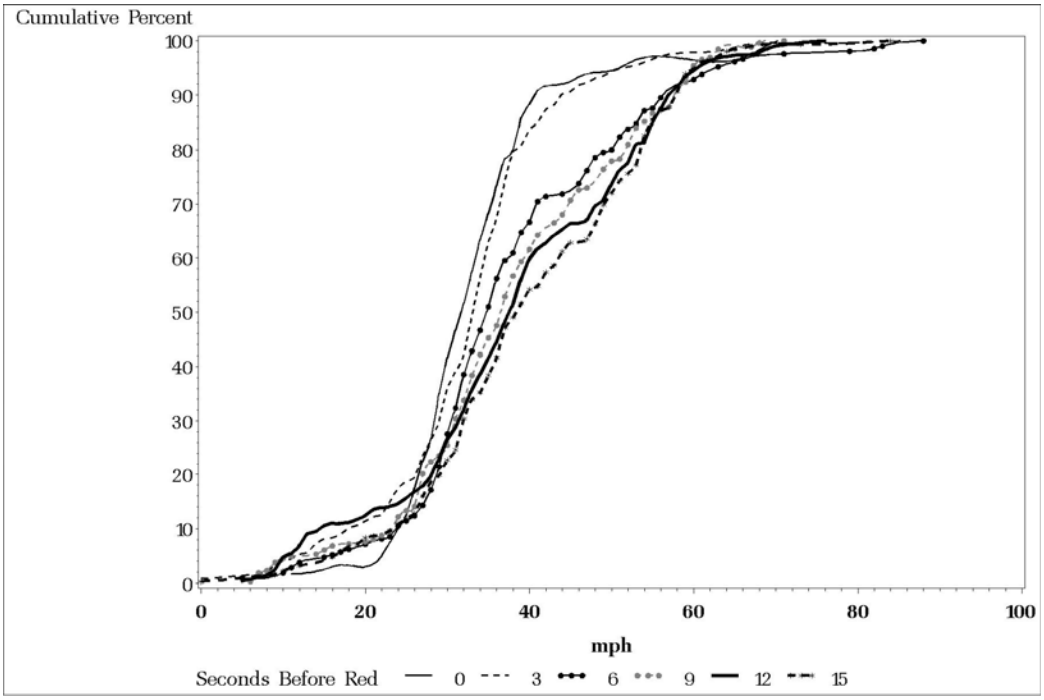

Figure D-17 Cumulative speeds for the southbound PM peak 200 ft. sensor for P2.

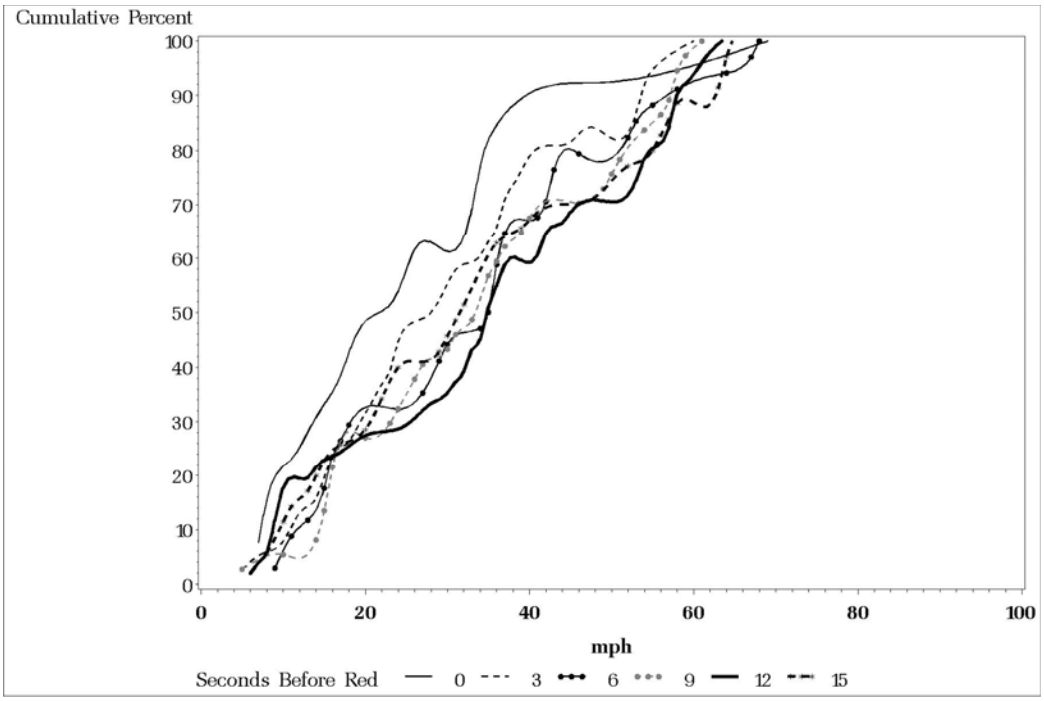

Figure D-18 Cumulative speeds for the southbound PM peak 200 ft. sensor for P3.

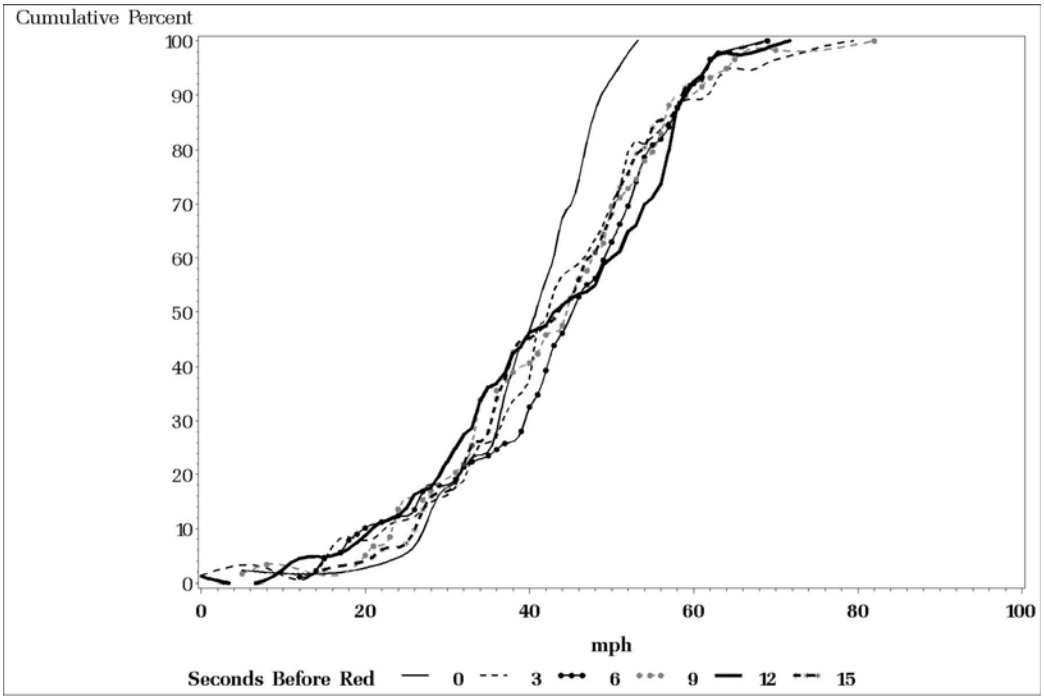

Figure D-19 Cumulative speeds for the northbound PM peak 300 ft. sensor for P1.

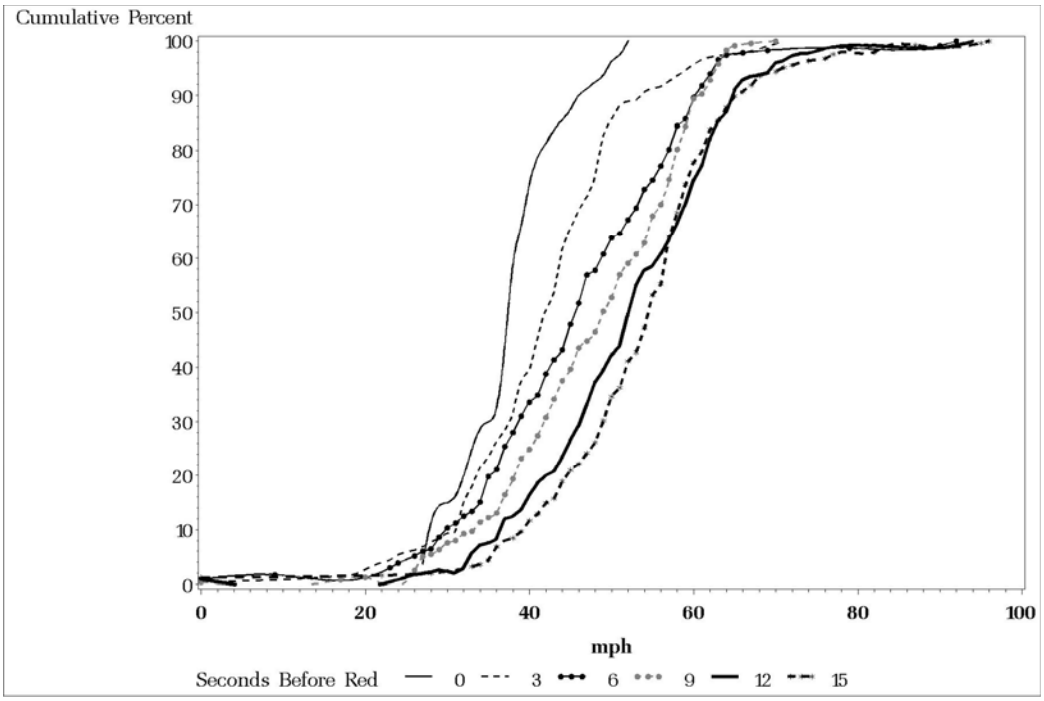

Figure D-20 Cumulative speeds for the northbound PM peak 300 ft. sensor for P2.

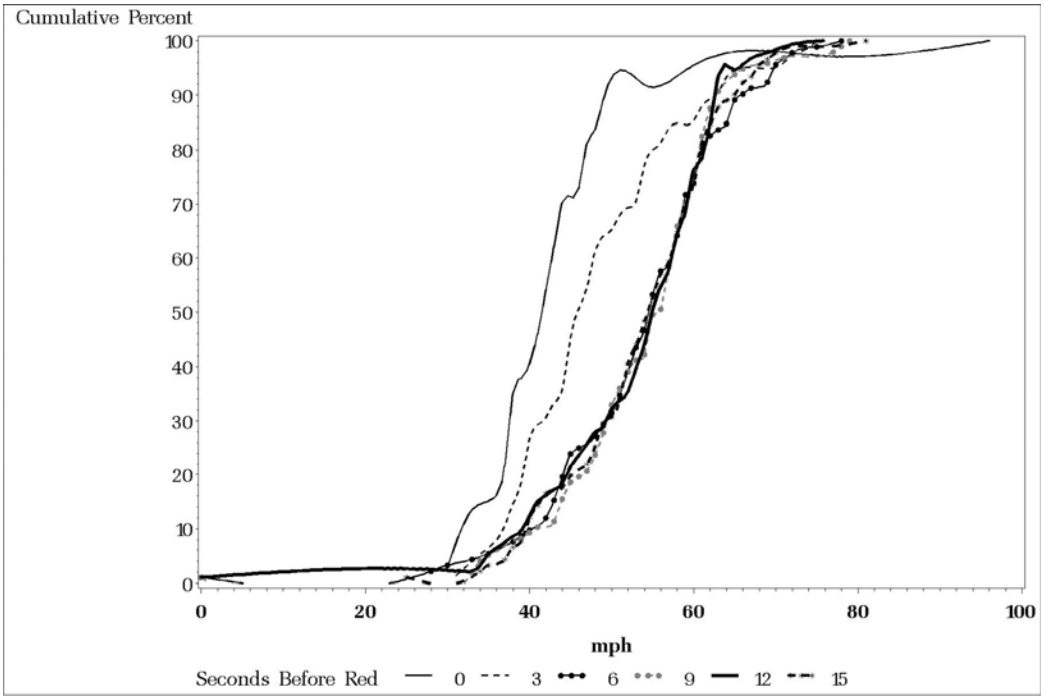

Figure D-21 Cumulative speeds for the northbound PM peak 300 ft. sensor for P3.

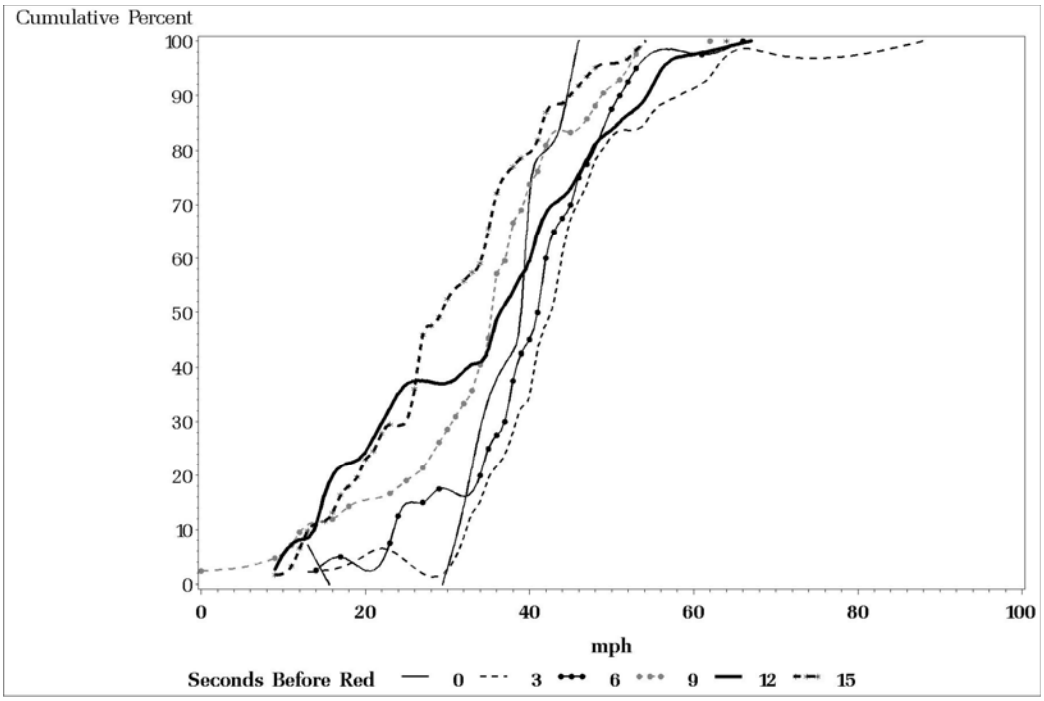

Figure D-22 Cumulative speeds for the southbound PM peak 300 ft. sensor for P1.

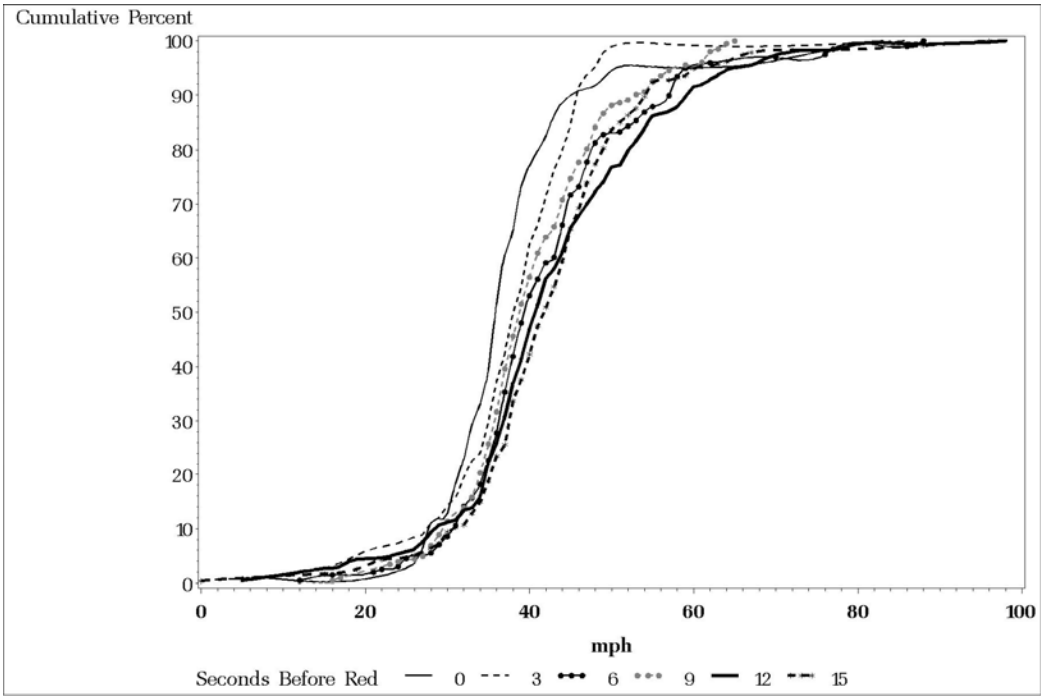

Figure D-23 Cumulative speeds for the southbound PM peak 300 ft. sensor for P2.

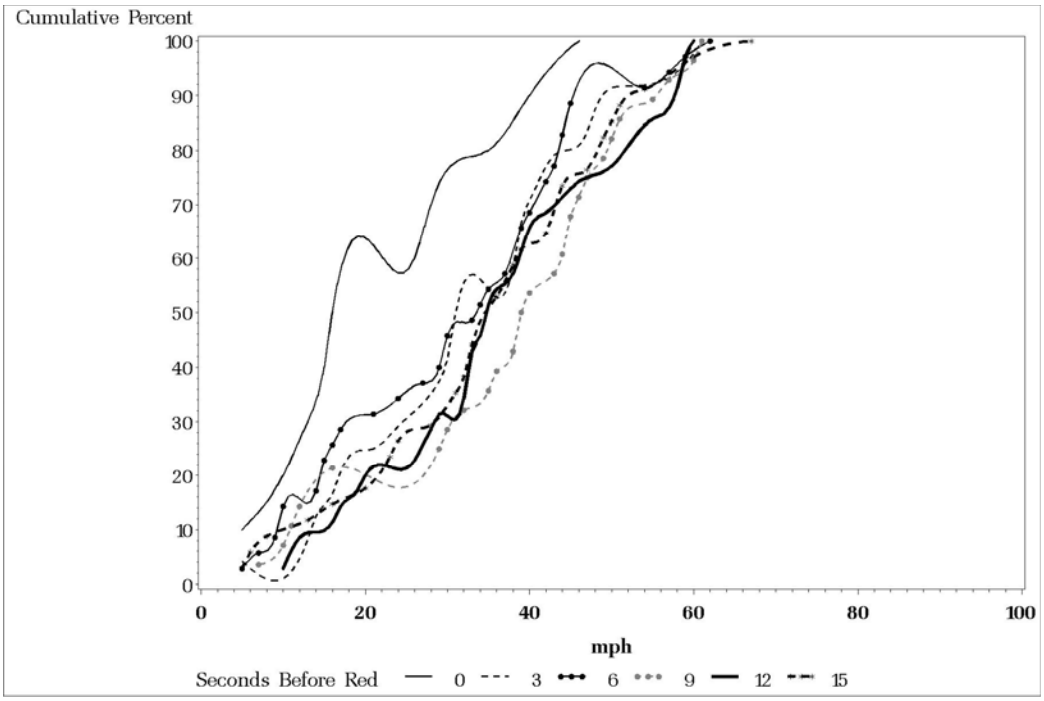

Figure D-24 Cumulative speeds for the southbound PM peak 300 ft. sensor for P3.

Appendix E

Box plots of Speed Data Results for the AM Peak

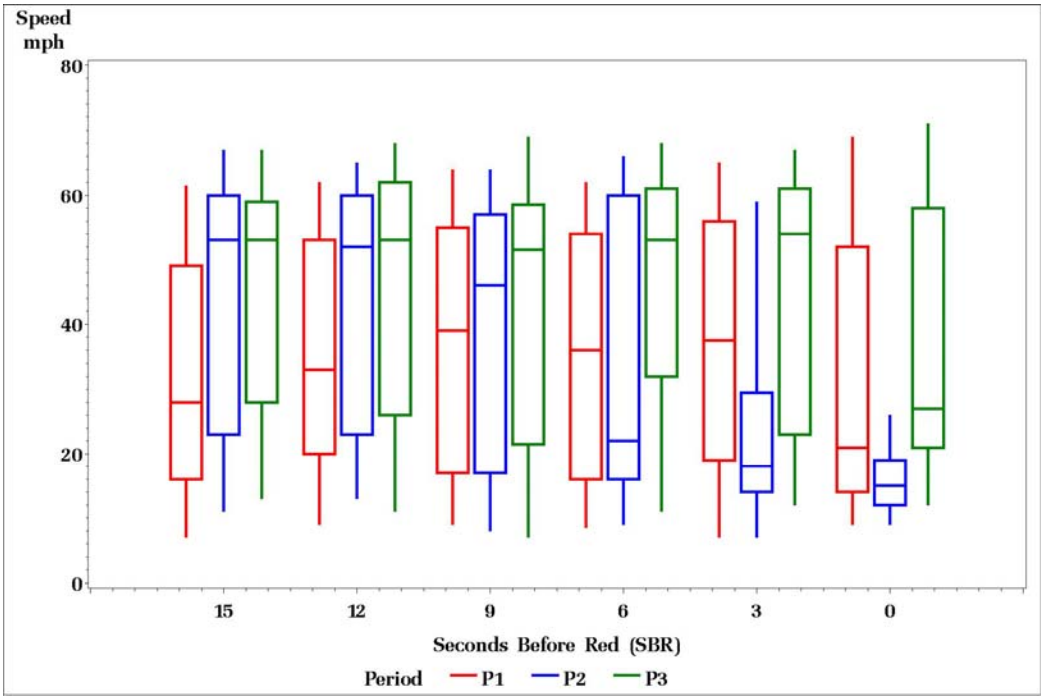

Figure E-1 Box plots of speeds for the northbound AM peak 50 ft. sensor zone.

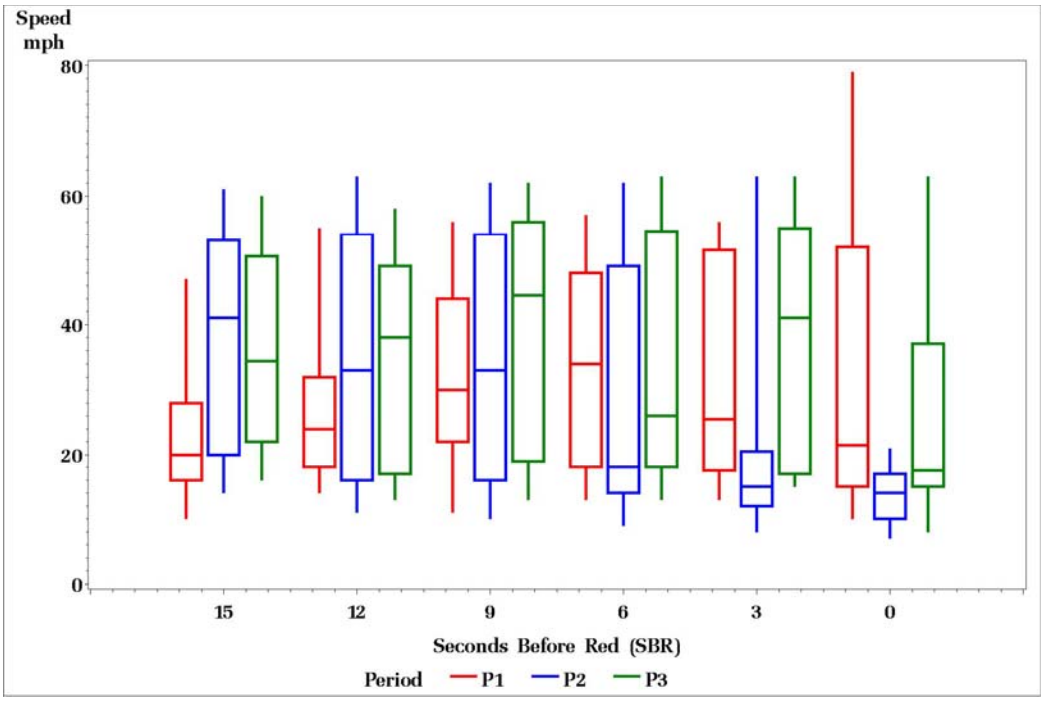

Figure E-2 Box plots of speeds for the southbound AM peak 50 ft. sensor zone.

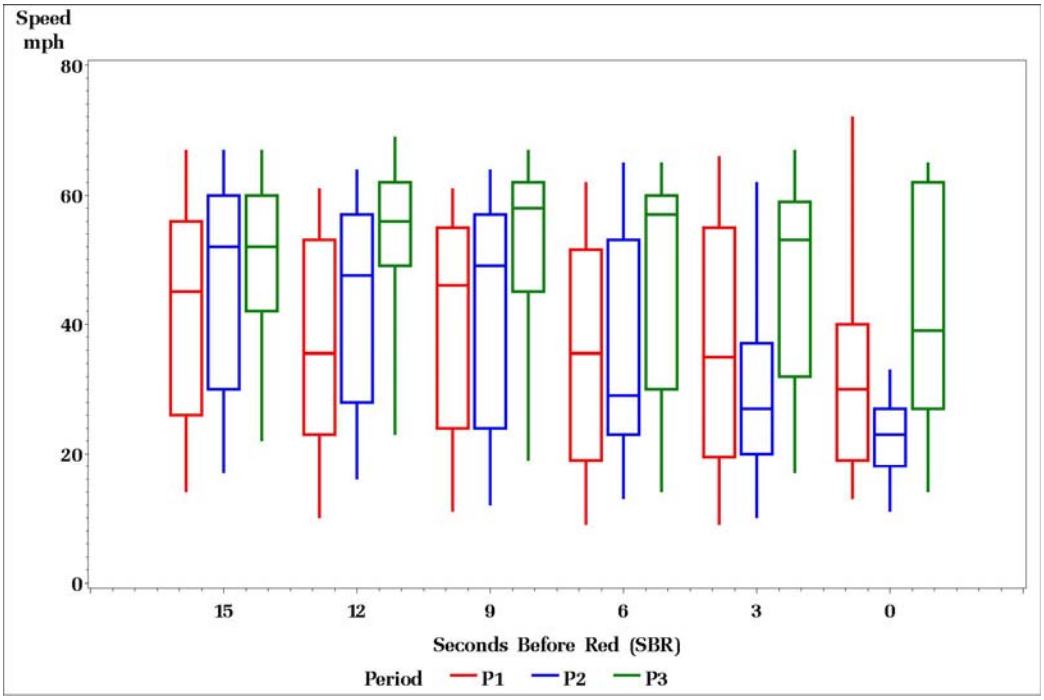

Figure E-3 Box plots of speeds for the northbound AM peak 100 ft. sensor zone.

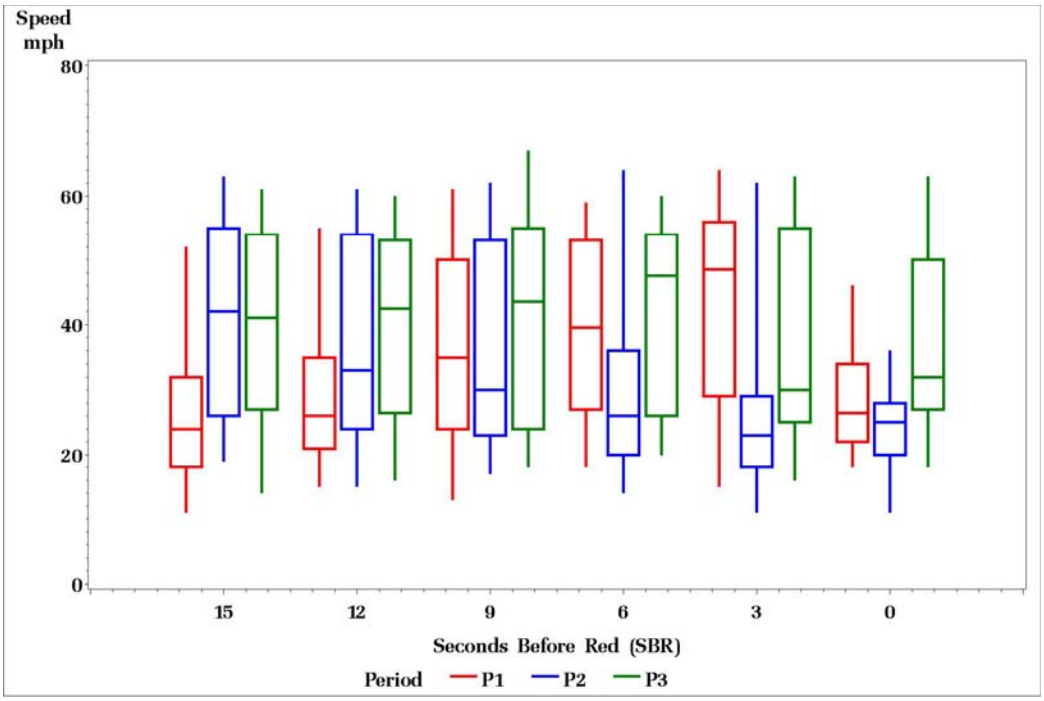

Figure E-4 Box plots of speeds for the southbound AM peak 100 ft. sensor zone.

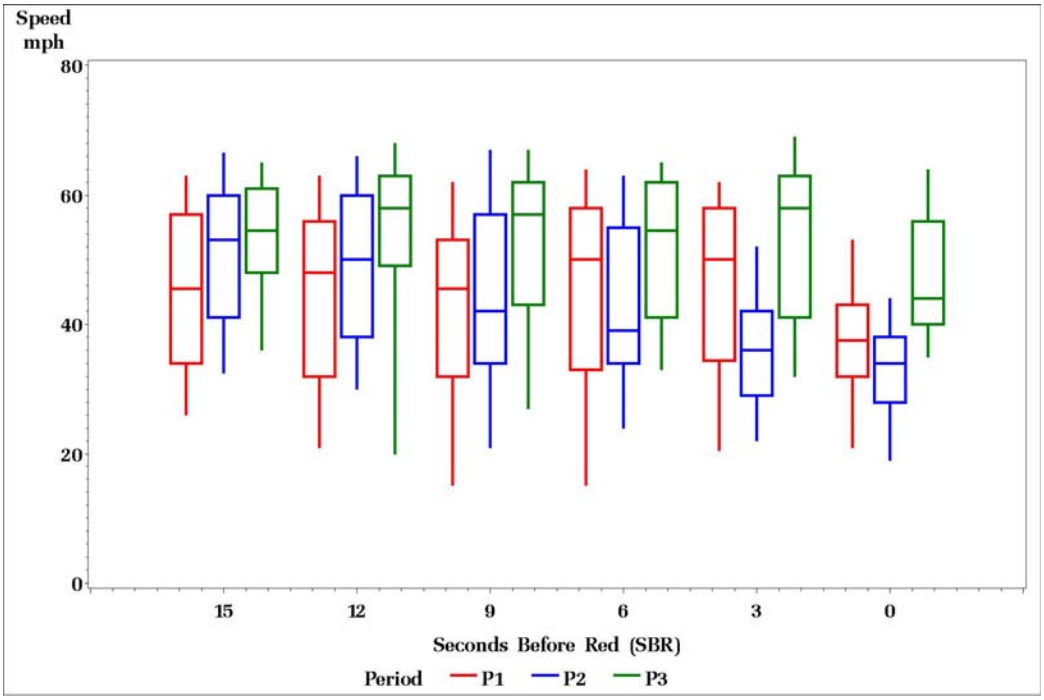

Figure E-5 Box plots of speeds for the northbound AM peak 200 ft. sensor zone.

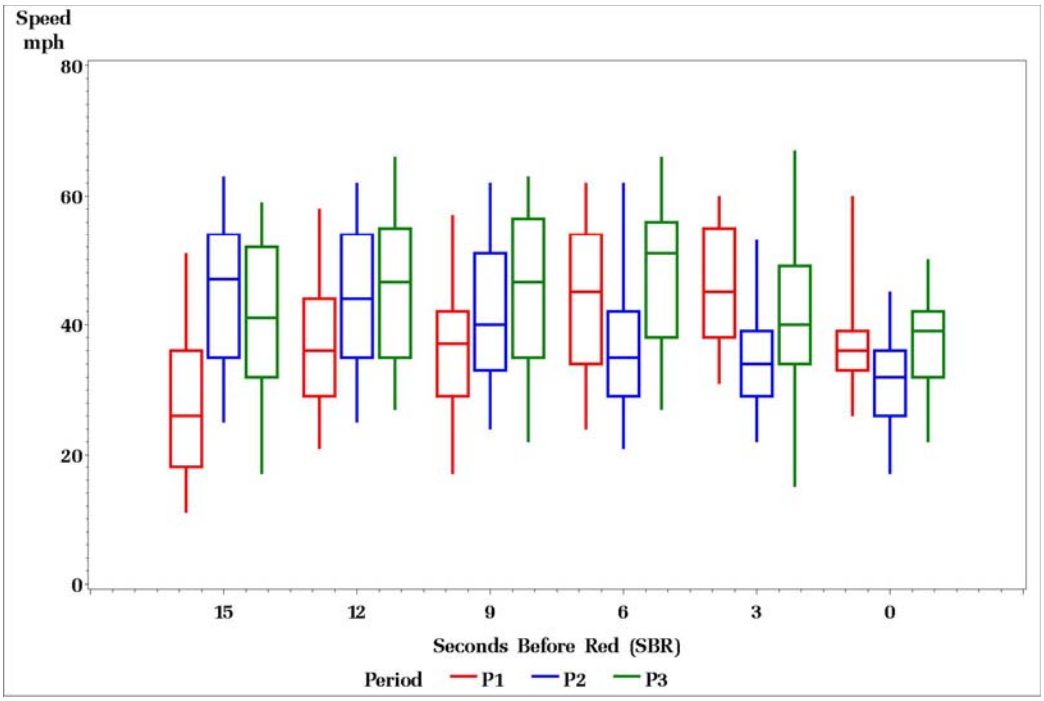

Figure E-6 Box plots of speeds for the southbound AM peak 200 ft. sensor zone.

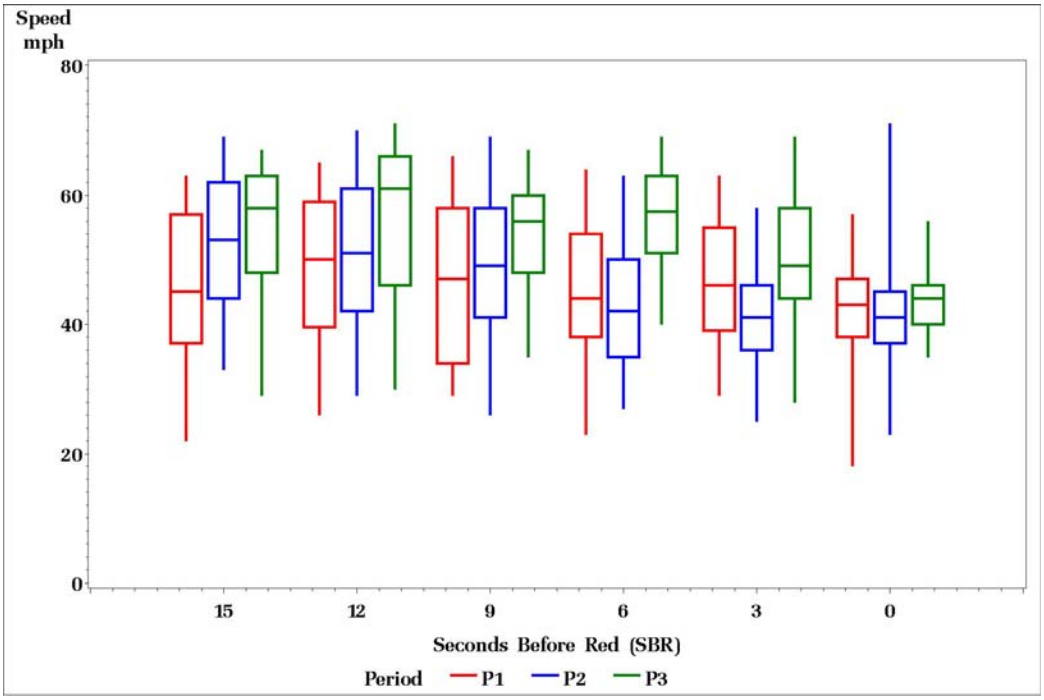

Figure E-7 Box plots of speeds for the northbound AM peak 300 ft. sensor zone.

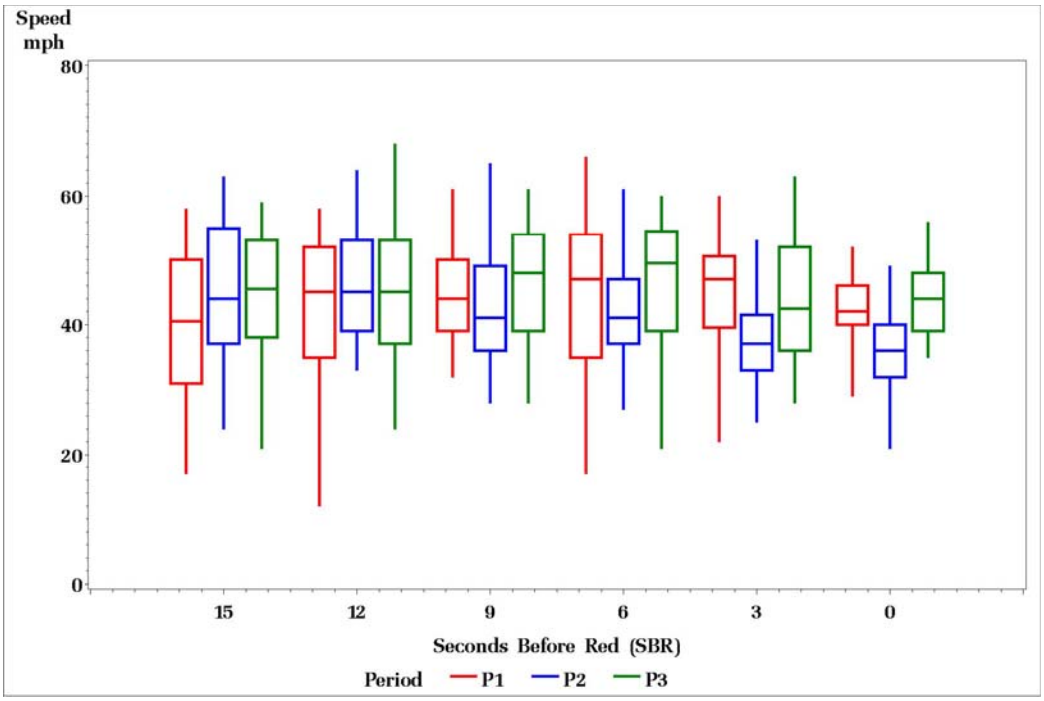

Figure E-8 Box plots of speeds for the southbound AM peak 300 ft. sensor zone.

Appendix F

Box plots of Speed Data Results for the Noon Peak

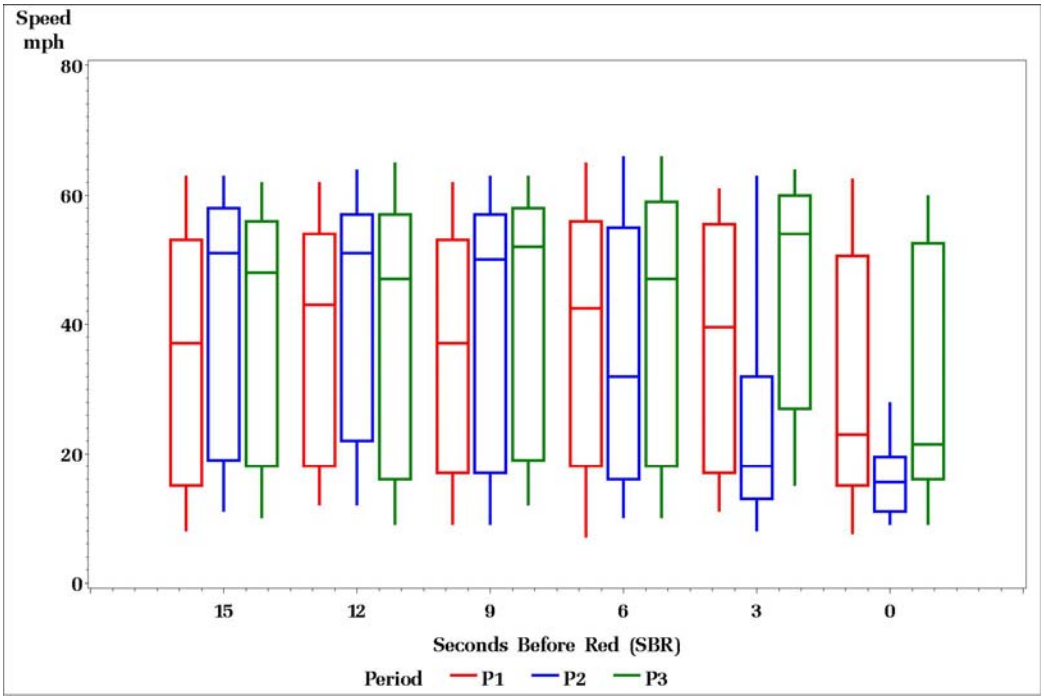

Figure F-1 Box plots of speeds for the northbound noon peak 50 ft. sensor zone.

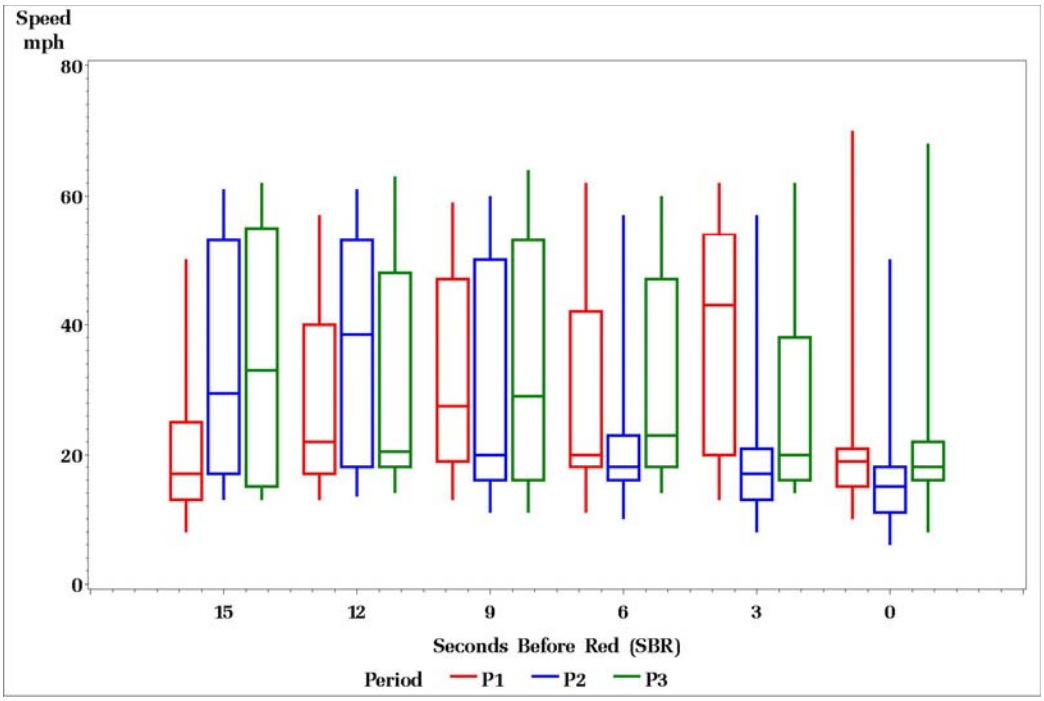

Figure F-2 Box plots of speeds for the southbound noon peak 50 ft. sensor zone.

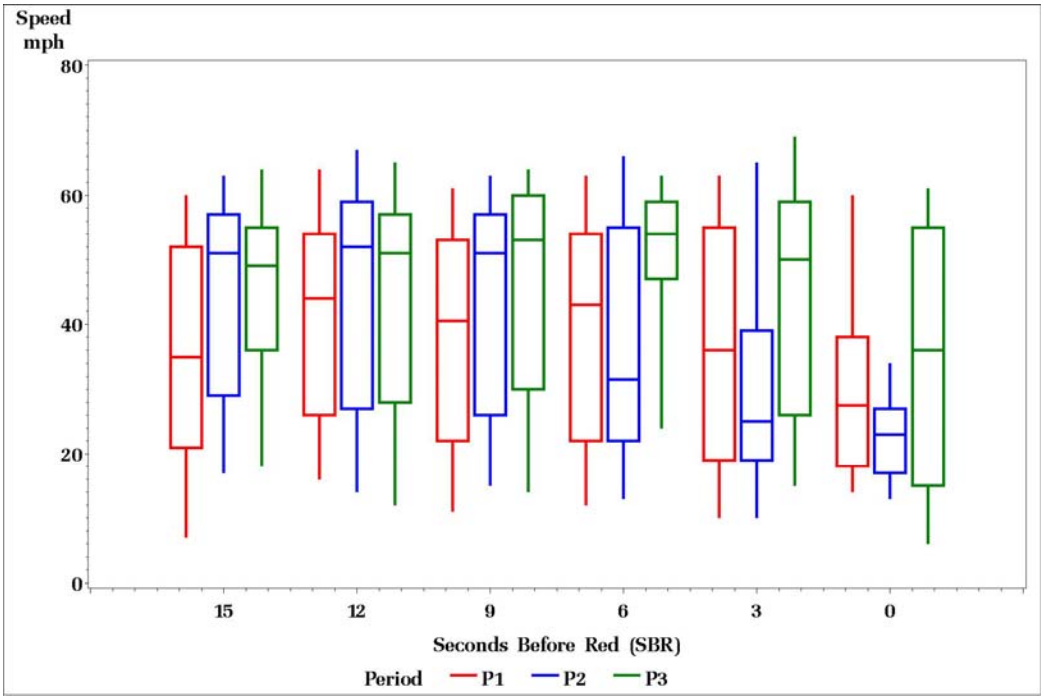

Figure F-3 Box plots of speeds for the northbound noon peak 100 ft. sensor zone.

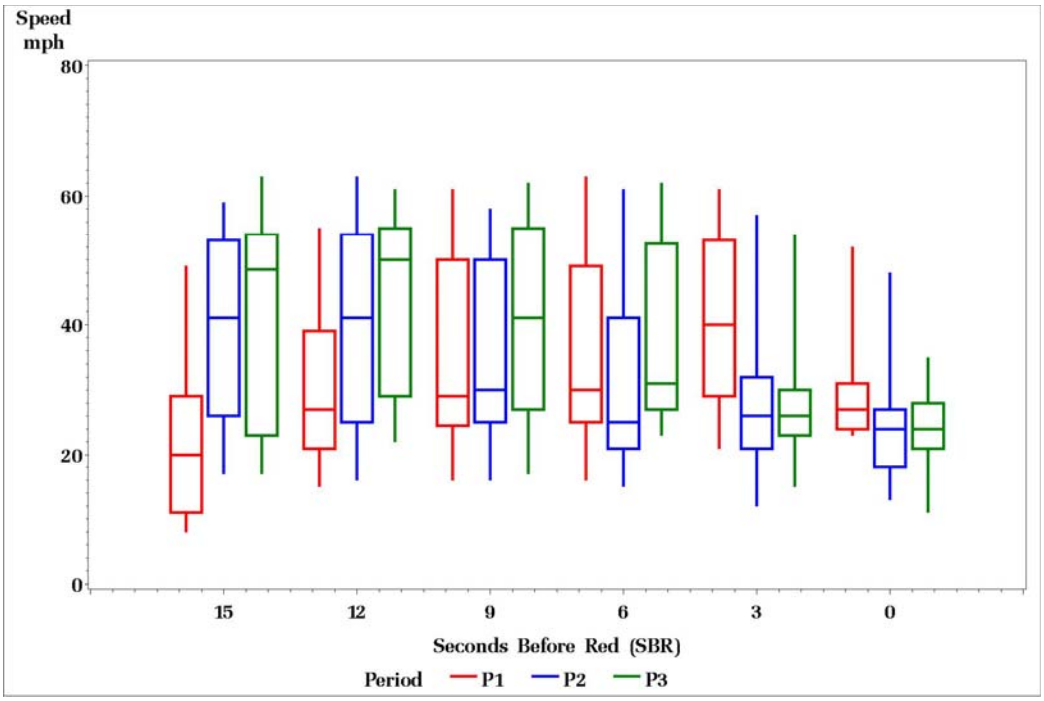

Figure F-4 Box plots of speeds for the southbound noon peak 100 ft. sensor zone.

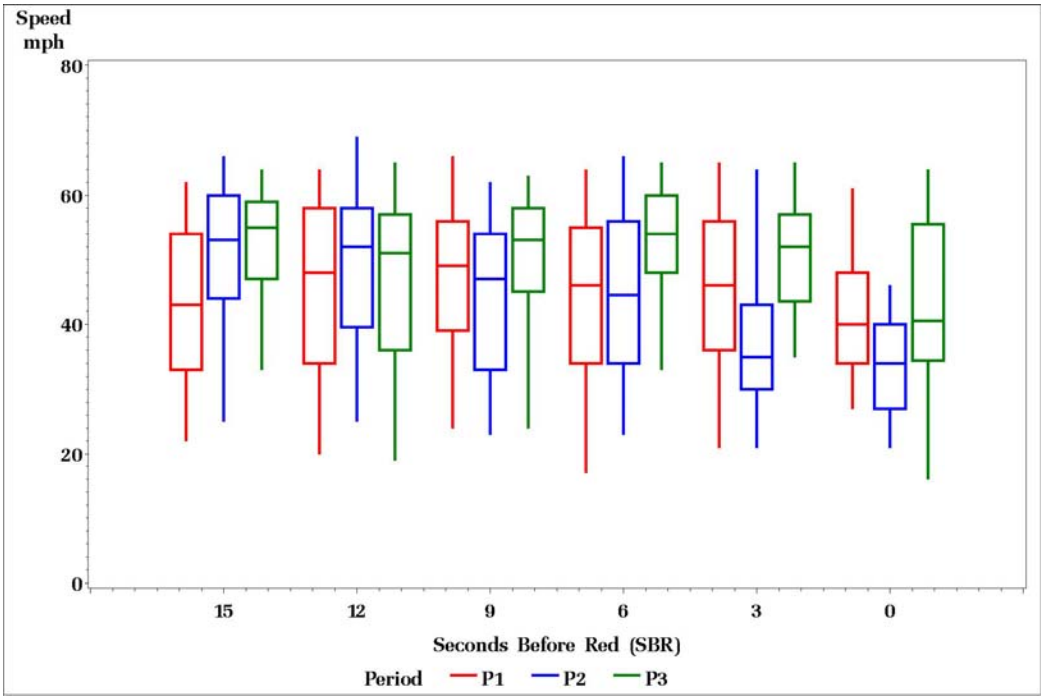

Figure F-5 Box plot of speeds for the northbound noon peak 200 ft. sensor zone.

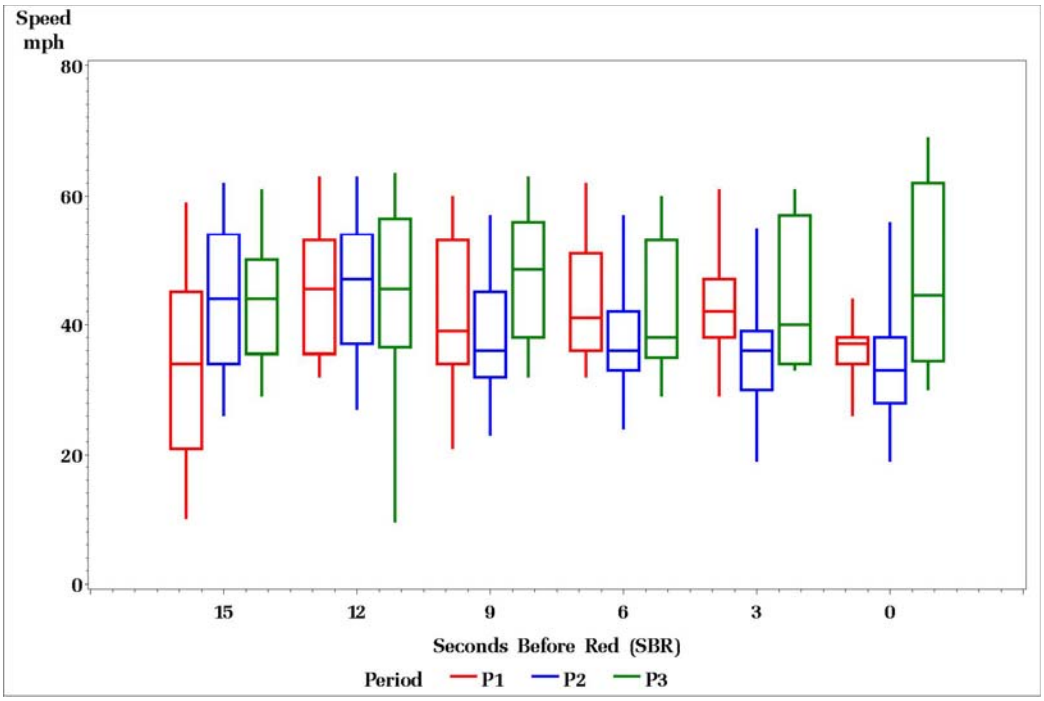

Figure F-6 Box plot of speeds for the southbound noon peak 200 ft. sensor zone.

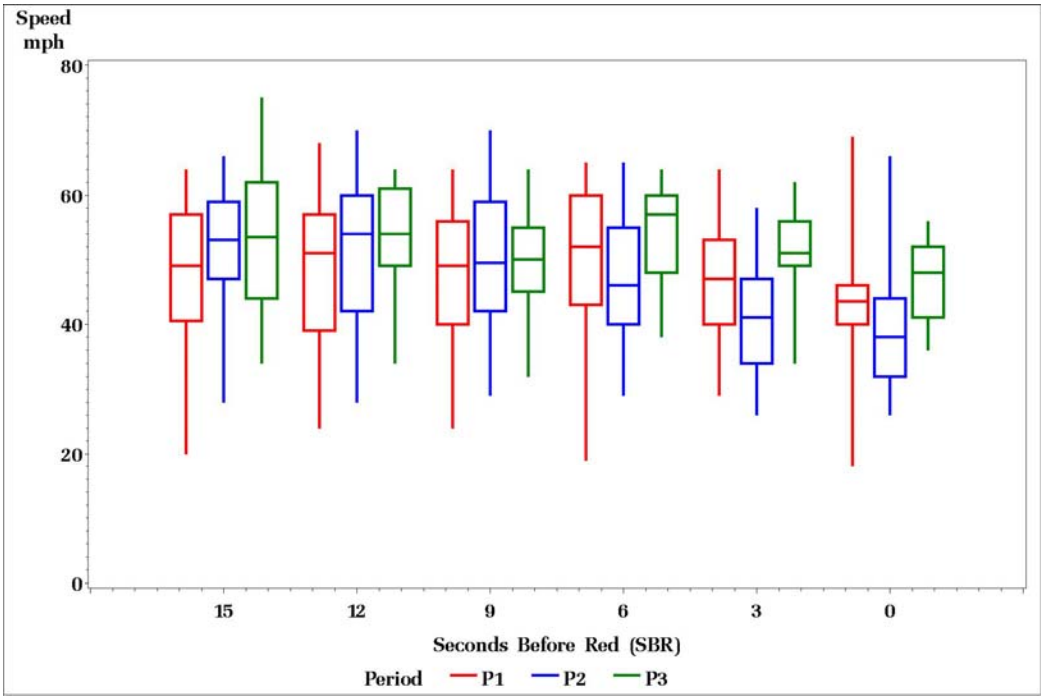

Figure F-7 Box plot of speeds for the northbound noon peak 300 ft. sensor zone.

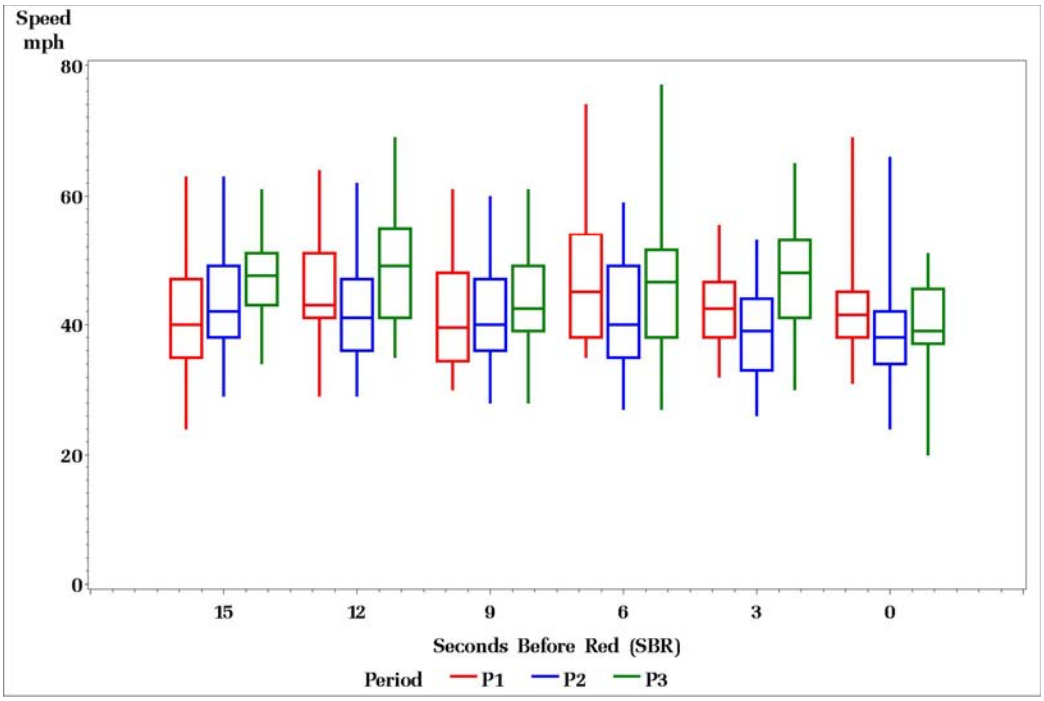

Figure F-8 Box plot of speeds for the southbound noon peak 300 ft. sensor zone.

Appendix G

Box plots of Speed Data Results for the PM Peak

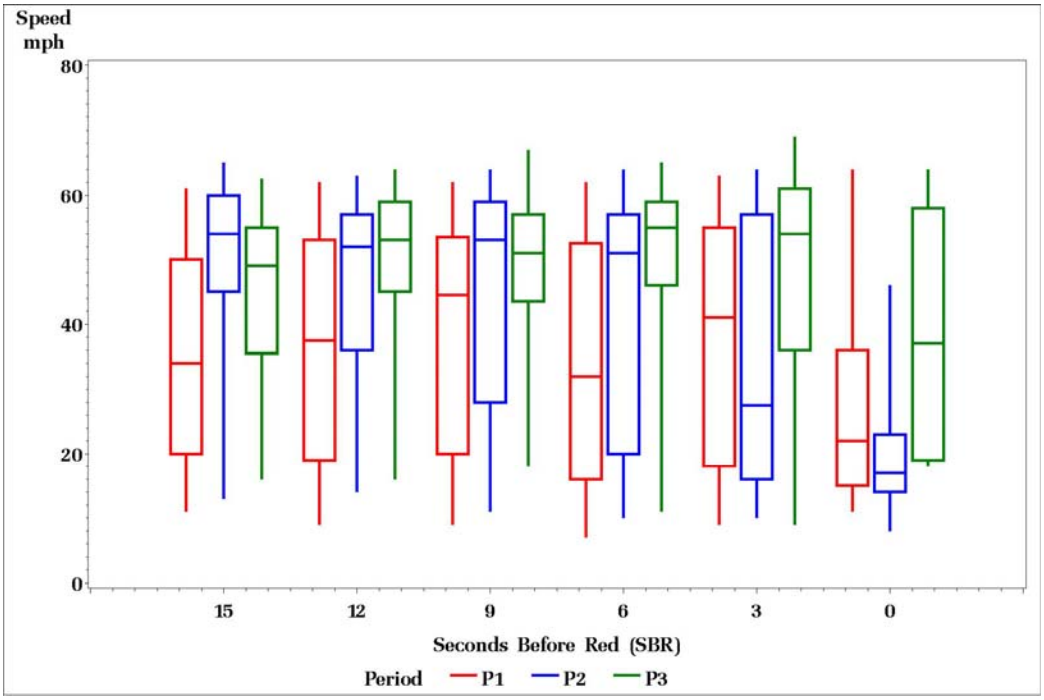

Figure G-1 Box plot of speeds for the northbound PM peak 50 ft. sensor zone.

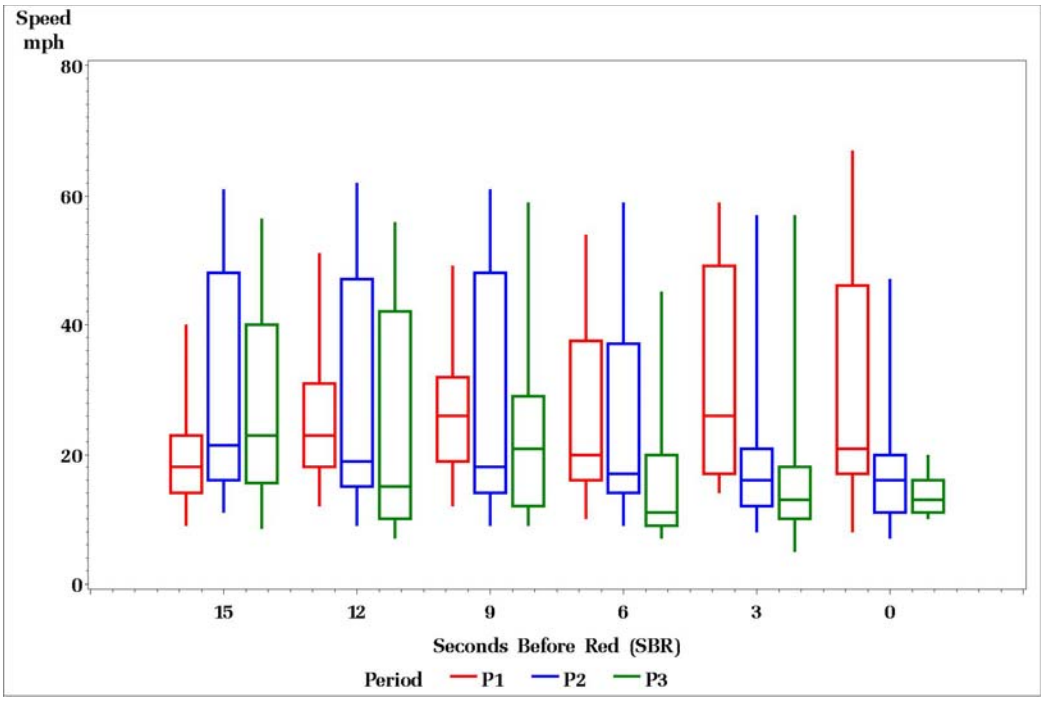

Figure G-2 Box plot of speeds for the southbound PM peak 50 ft. sensor zone.

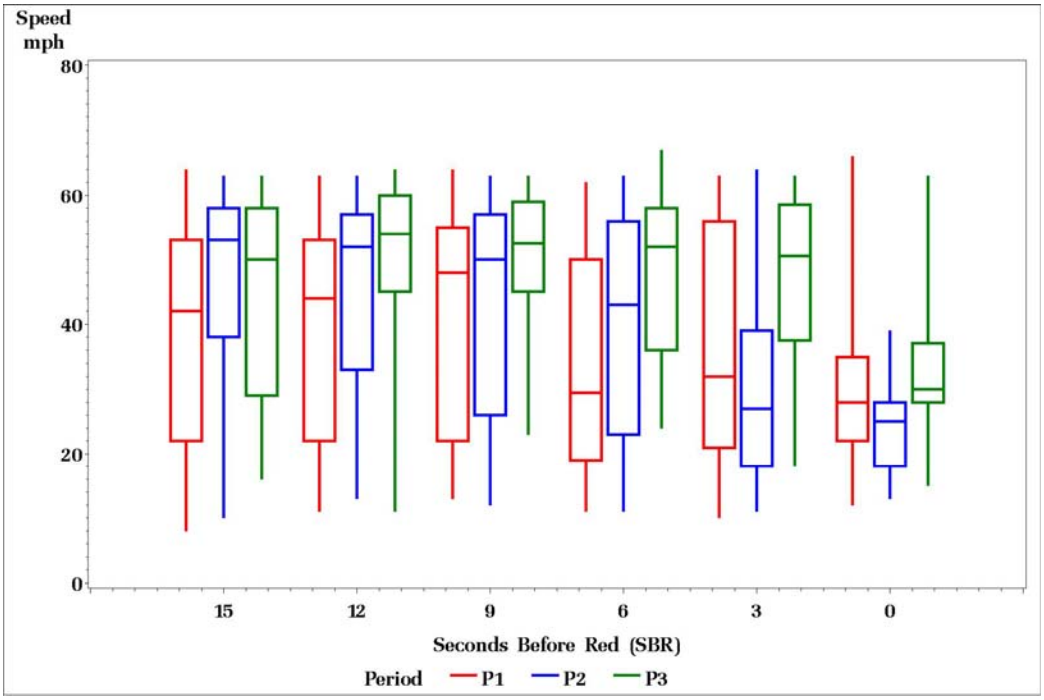

Figure G-3 Box plot of speeds for the northbound PM peak 100 ft. sensor zone.

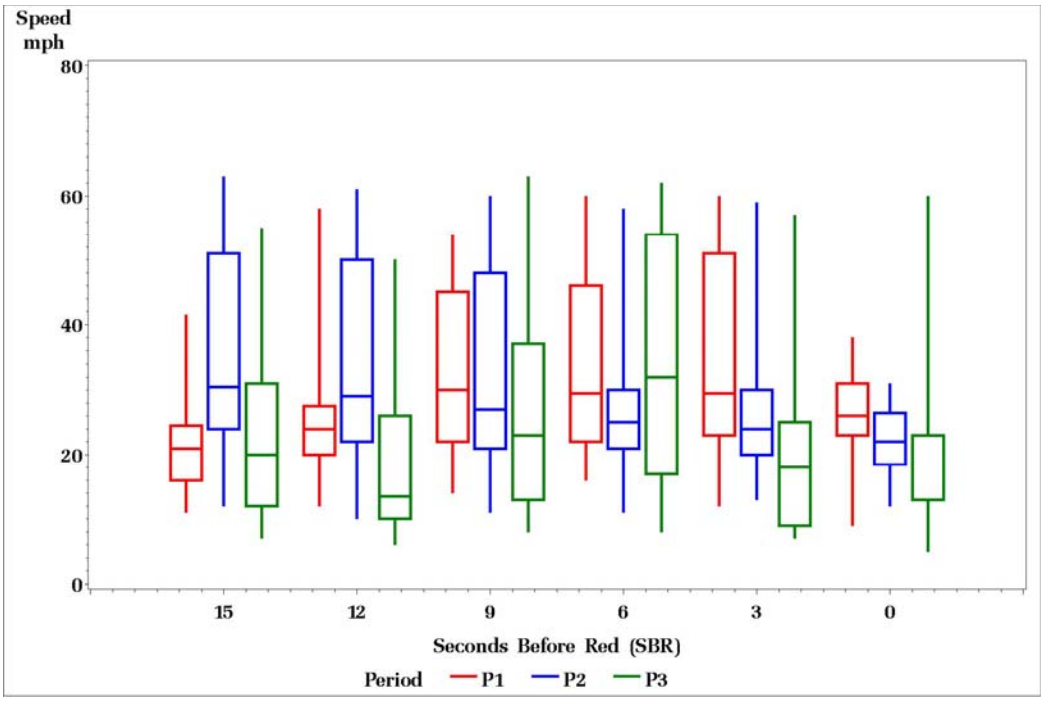

Figure G-4 Box plot of speeds for the southbound PM peak 100 ft. sensor zone.

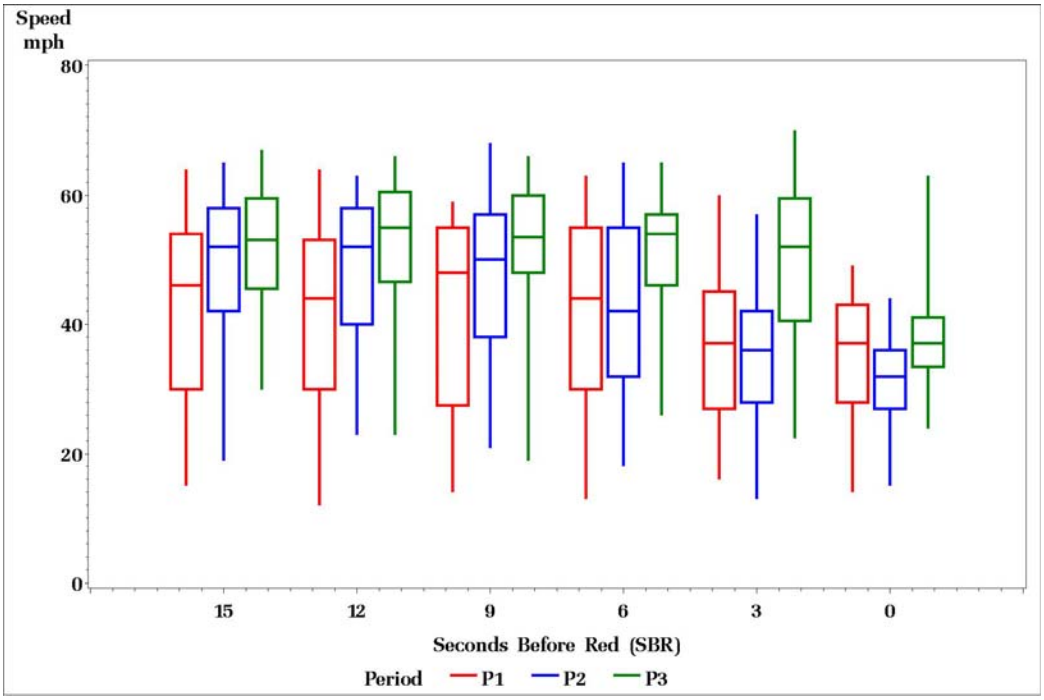

Figure G-5 Box plot of speeds for the northbound PM peak 200 ft. sensor zone.

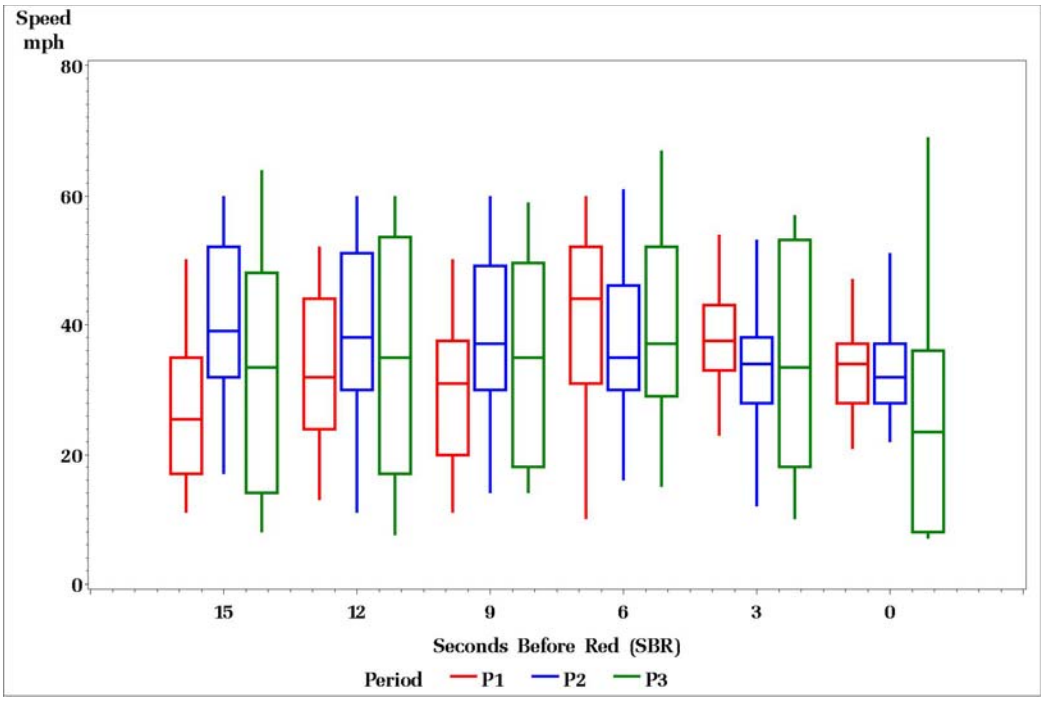

Figure G-6 Box plot of speeds for the southbound PM peak 200 ft. sensor zone.

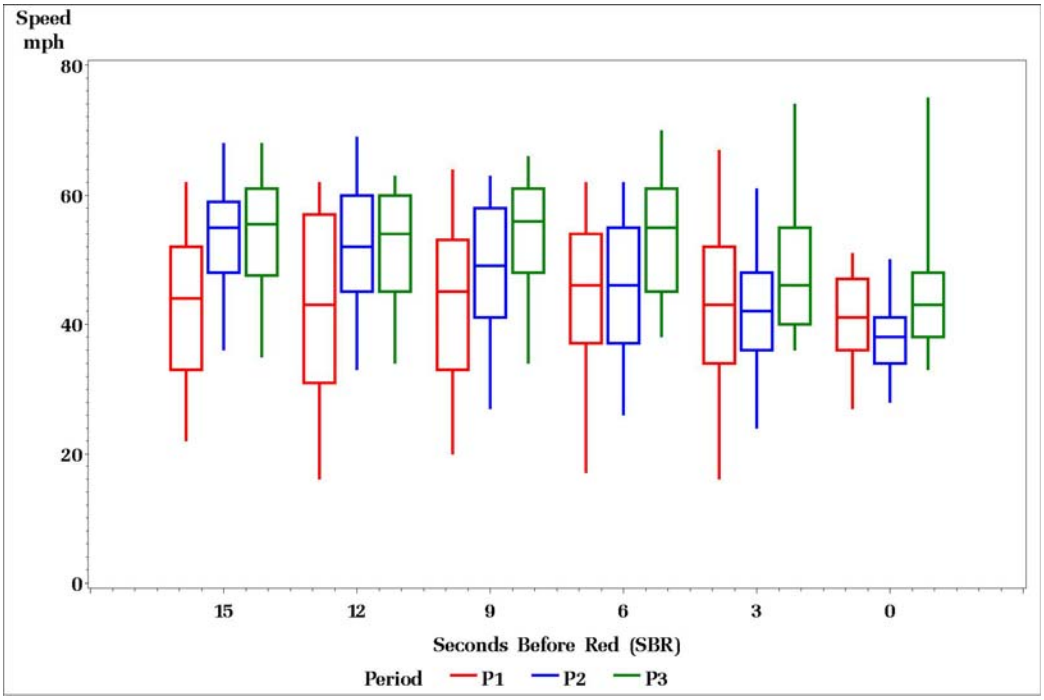

Figure G-7 Box plot of speeds for the northbound PM peak 300 ft. sensor zone.

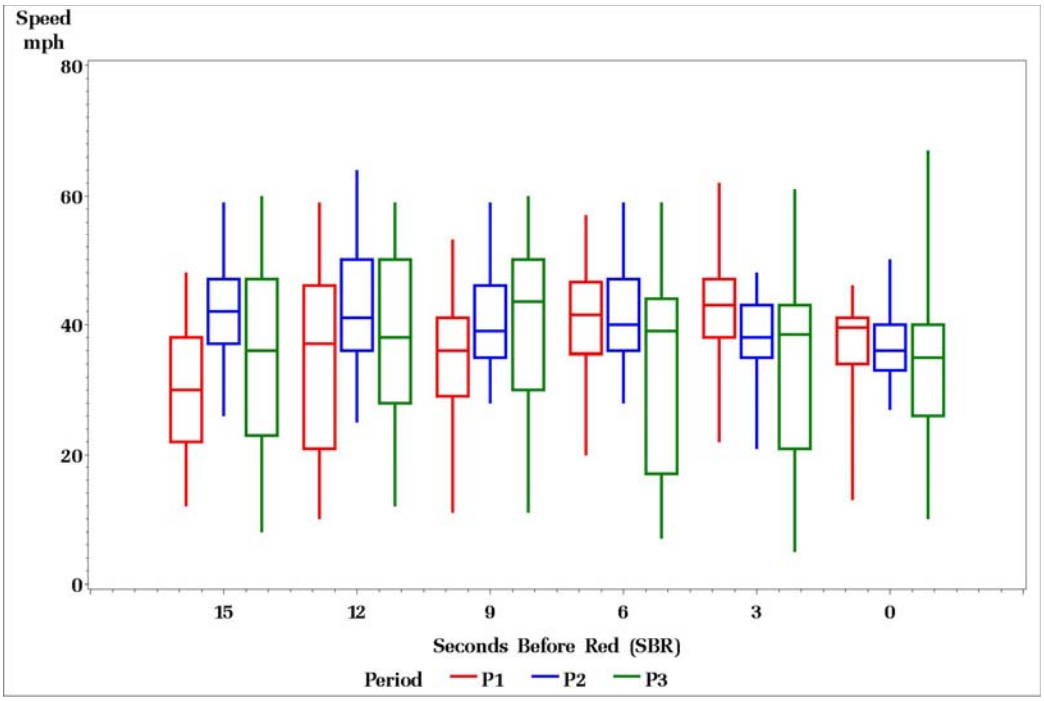

Figure G-8 Box plot of speeds for the southbound PM peak 300 ft. sensor zone.

Appendix H

Southbound Speed Probability Grids

Table H-1 Statistical Significance Grid for Southbound Speed Data for Weekday AM Peak Traffic (P1 vs. P2)

	Seconds Before Red (SBR)																
Zone (feet)	16	15	14	13	12	11	10	9	8	7	6	5	4	3	2	1	0
300																	
200																	
100																	
50																	

Note: Gray squares = P2 > P1 and black squares = P2 < P1

Table H-2 Statistical Significance Grid for the Southbound Speed Data for Weekday Noon Peak Traffic (P1 vs. P2)

	Seconds Before Red (SBR)																
Zone (feet)	16	15	14	13	12	11	10	9	8	7	6	5	4	3	2	1	0
300																	
200																	
100																	
50																	

Note: Gray squares = P2 > P1 and black squares = P2 < P1

Table H-3 Statistical Significance Grid for the Southbound Speed Data for Weekday PM Peak Traffic (P1 vs. P2)

	Seconds Before Red (SBR)																
Zone (feet)	16	15	14	13	12	11	10	9	8	7	6	5	4	3	2	1	0
300																	
200																	
100																	
50																	

Note: Gray squares = P2 > P1 and black squares = P2 < P1

Table H-4 Statistical Significance Grid for the Southbound Speed Data for Weekday AM Peak Traffic (P1 vs. P3)

	Seconds Before Red (SBR)																
Zone (feet)	16	15	14	13	12	11	10	9	8	7	6	5	4	3	2	1	0
300																	
200																	
100																	
50																	

Note: Gray squares = P3 > P1 and black squares = P3 < P1

Table H-5 Statistical Significance Grid for the Southbound Speed Data for Weekday Noon Peak Traffic (P1 vs. P3)

	Seconds Before Red (SBR)																
Zone (feet)	16	15	14	13	12	11	10	9	8	7	6	5	4	3	2	1	0
300																	
200																	
100																	
50																	

Note: Gray squares = P3 > P1 and black squares = P3 < P1

Table H-6 Statistical Significance Grid for the Southbound Speed Data for Weekday PM Peak Traffic (P1 vs. P3)

	Seconds Before Red (SBR)																
Zone (feet)	16	15	14	13	12	11	10	9	8	7	6	5	4	3	2	1	0
300																	
200																	
100																	
50																	

Note: Gray squares = P3 > P1 and black squares = P3 < P1

Appendix I

UDOT RLR Hand Count Data

Table I-1 UDOT RLR Hand Counts on Bangerter Highway for P1

Study Date	Study Intersection	Study Time Period	Number of Thru Vehicles	Number of RLR Violations	(Per 1,000 Entering Vehicles)
11/18/2004	Redwood	12 PM - 1 PM	934	2	2.14
11/18/2004	Redwood	3 PM - 4 PM	1421	1	0.70
11/18/2004	2700 West	12 PM - 1 PM	1045	1	0.96
11/18/2004	2700 West	3 PM - 4 PM	1333	8	6.00
4/25/2005	13400 South	12 PM - 1 PM	530	4	7.55
4/25/2005	2700 West	3 PM - 4 PM	2580	7	2.71
5/2/2005	13400 South	12 PM - 1 PM	578	4	6.92
5/11/2005	Redwood	12 PM - 1 PM	909	4	4.40

Table I-2 UDOT RLR Hand Counts on Bangerter Highway for P2

Study Date	Study Intersection	Study Time Period	Number of Thru Vehicles	Number of RLR Violations	(Per 1,000 Entering Vehicles)
6/23/2005	13400 South	8 AM - 9 AM	946	6	6.34
6/23/2005	Redwood	11 AM - 12 PM	1046	2	1.91

Table I-3 UDOT RLR Hand Counts on Bangerter Highway for P3

Study Date	Study Intersection	Study Time Period	Number of Thru Vehicles	Number of RLR Violations	(Per 1,000 Entering Vehicles)
3/6/2006	13400 South	12 PM - 1 PM	407	5	12.29
3/6/2006	Redwood	2 PM -3 PM	378	7	18.52

Appendix J

Crash Data Figures, Rates, and Traffic Counts

Table J-1 Crash Figures for the Study Site Bangerter Highway Approaches.

Type	Bangerter Only									
	2000	2001	2002B	2002A	2003B	2003A	2004B	2004A	2005B	2005A
Rear-end	2	1	0	2	0	1	3	5	4	1
Right-angle	1	1	1	2	1	2	0	1	1	1
Other	3	2	2	1	0	2	2	1	2	0
Total	6.00	4.00	3.00	5.00	1.00	5.00	5.00	7.00	7.00	2.00

Note: The B represents the first six months of the year (the before period) while the A represents the last six months of the year (the after period) in accord with the installation of the BODAWS system.

Table J-2 Crash Figures for the Study Site for All Intersection Approaches.

Type	Intersection Totals									
	2000	2001	2002B	2002A	2003B	2003A	2004B	2004A	2005B	2005A
Rear-end	3	3	0	7	2	7	4	8	10	2
Right-angle	2	2	2	4	2	2	1	1	1	3
Other	3	2	1	2	0	1	2	2	2	2
Total	8.00	7.00	3.00	13.00	4.00	10.00	7.00	11.00	13.00	7.00

Note: The B represents the first six months of the year (the before period) while the A represents the last six months of the year (the after period) in accord with the installation of the BODAWS system.

Table J-3 Crash Figures for the Control Intersection Bangerter Highway Approaches.

Type	Bangerter Only									
	2000	2001	2002B	2002A	2003B	2003A	2004B	2004A	2005B	2005A
Rear-end	--	--	1	0	2	0	2	1	--	0
Right-angle	--	--	1	1	4	4	4	5	--	4
Other	--	--	0	1	1	2	0	2	--	0
Total			2.00	2.00	7.00	6.00	6.00	8.00	0.00	4.00

Note: The B represents the first six months of the year (the before period) while the A represents the last six months of the year (the after period) in accord with the installation of the BODAWS system.

Table J-4 Crash Figures for the Control Intersection for All Intersection Approaches.

Type	Intersection Totals									
	2000	2001	2002B	2002A	2003B	2003A	2004B	2004A	2005B	2005A
Rear-end	--	--	1	4	2	1	2	3	--	0
Right-angle	--	--	2	3	5	5	4	5	--	7
Other	--	--	0	1	1	2	0	2	--	1
Total			3.00	8.00	8.00	8.00	6.00	10.00	0.00	8.00

Note: The B represents the first six months of the year (the before period) while the A represents the last six months of the year (the after period) in accord with the installation of the BODAWS system.

Table J-5 Crash Rates for the Study Site Bangerter Highway Approaches.

Type	Bangerter Only (MEV)									
	2000	2001	2002B	2002A	2003B	2003A	2004B	2004A	2005B	2005A
Rear-end	0.27	0.12	0.00	0.55	0.00	0.29	0.75	1.26	1.01	0.25
Right-angle	0.13	0.12	0.28	0.55	0.29	0.57	0.00	0.25	0.25	0.25
Other	0.40	0.24	0.55	0.28	0.00	0.57	0.50	0.25	0.50	0.00
Total	0.81	0.49	0.83	1.38	0.29	1.43	1.26	1.76	1.76	0.50

Note: The B represents the first six months of the year (the before period) while the A represents the last six months of the year (the after period) in accord with the installation of the BODAWS system.

Table J-6 Crash Rates for the Study Site for All Intersection Approaches.

Type	Intersection Crash Rates (MEV)									
	2000	2001	2002B	2002A	2003B	2003A	2004B	2004A	2005B	2005A
Rear-end	0.25	0.23	0.00	1.17	0.34	1.20	0.63	1.26	1.58	0.32
Right-angle	0.16	0.15	0.33	0.67	0.34	0.34	0.16	0.16	0.16	0.47
Other	0.25	0.15	0.17	0.33	0.00	0.17	0.32	0.32	0.32	0.32
Total	0.66	0.54	0.50	2.17	0.68	1.71	1.10	1.73	2.05	1.10

Note: The B represents the first six months of the year (the before period) while the A represents the last six months of the year (the after period) in accord with the installation of the BODAWS system.

Table J-7 Crash Rates for the Control Intersection for Bangerter Highway Approaches.

Type	Bangerter Only (MEV)									
	2000	2001	2002B	2002A	2003B	2003A	2004B	2004A	2005B	2005A
Rear-end	--	--	0.23	0.00	0.48	0.00	0.40	0.20	--	0.00
Right-angle	--	--	0.23	0.23	0.95	0.95	0.81	1.01	--	0.81
Other	--	--	0.00	0.23	0.24	0.48	0.00	0.40	--	0.00
Total			0.47	0.47	1.67	1.43	1.21	1.62		0.81

Note: B represents the first six months of the year (the before period) while the A represents the last six months of the year (the after period) in accord with the installation of the BODAWS system.

Table J-8 Crash Rates for the Control Intersection for All Intersection Approaches.

Type	Intersection Crash Rates (MEV)									
	2000	2001	2002B	2002A	2003B	2003A	2004B	2004A	2005B	2005A
Rear-end	--	--	0.14	0.55	0.01	0.14	0.25	0.42	--	0.00
Right-angle	--	--	0.27	0.41	0.03	0.69	0.50	0.69	--	0.87
Other	--	--	0.00	0.14	0.01	0.28	0.00	0.28	--	0.12
Total			0.41	1.10	0.06	1.11	0.75	1.39		1.00

Note: B represents the first six months of the year (the before period) while the A represents the last six months of the year (the after period) in accord with the installation of the BODAWS system.

Table J-9 UDOT Traffic Statistics Annual Average Daily Traffic (AADT) Data for the Study Site Intersection.

Year	13400 South AADT	Bangerter Highway AADT
2000	12,950	20,415
2001	12,950	22,550
2002	12,950	19,815
2003	12,950	19,120
2004	12,950	21,800
2005	12,950	21,800

Table J-10 UDOT Traffic Statistics Annual Average Daily Traffic (AADT) Data for the Control Intersection.

Year	12600 South AADT	Bangerter Highway AADT
2000	--	--
2001	--	--
2002	16,587	23,405
2003	16,587	22,990
2004	16,757	27,110
2005	16,757	27,110

Appendix K

DCZ Study Graphs and Stopping Probability Tables

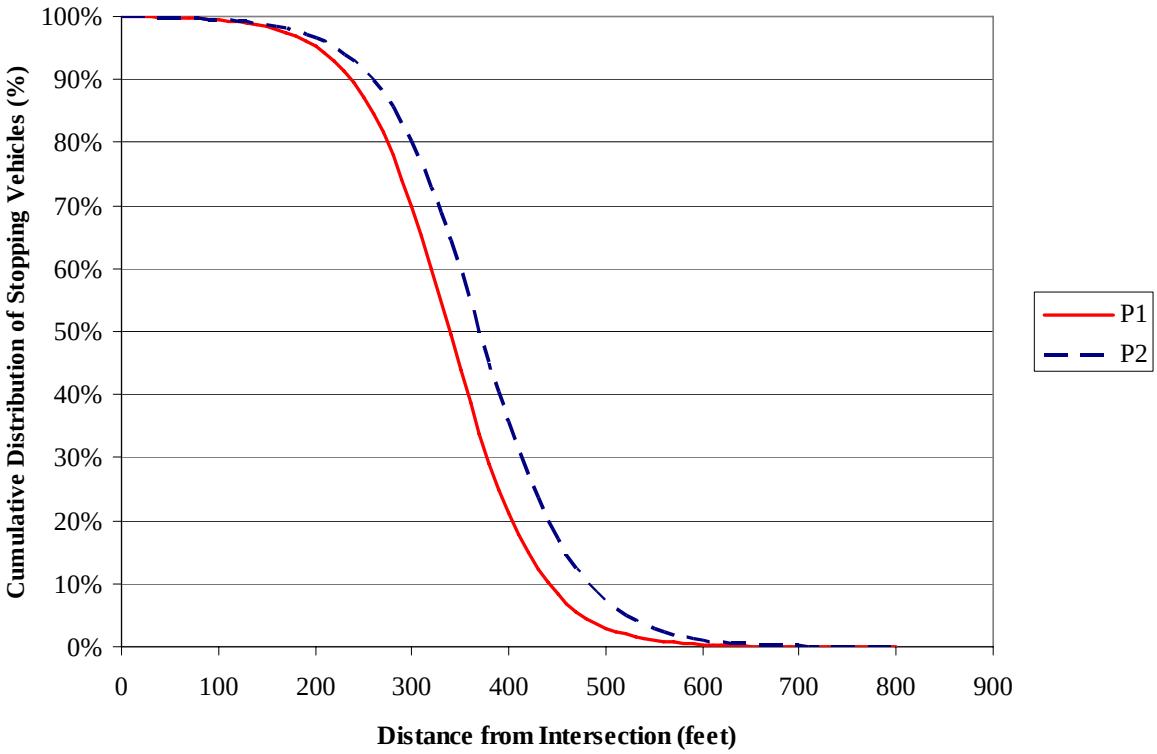

Figure K-1 Cumulative distribution probability stopping distances of passenger cars.

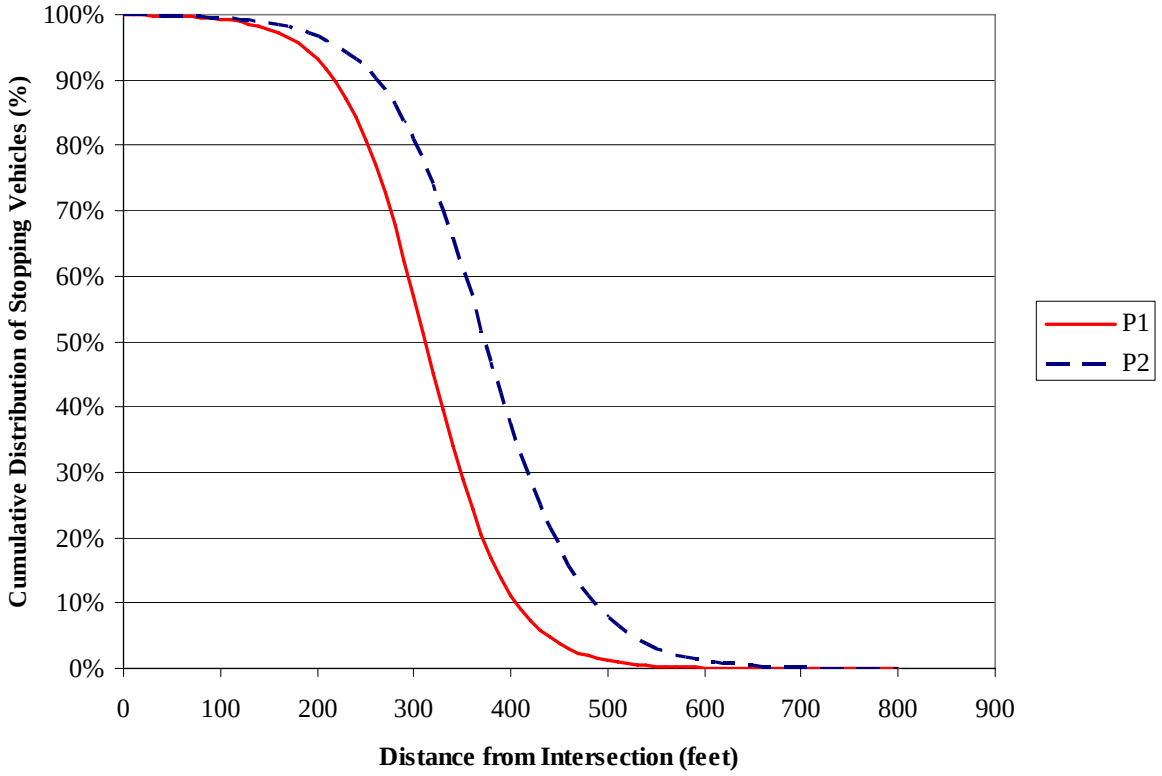

Figure K-2 Cumulative distribution probability stopping distances of passenger trucks.

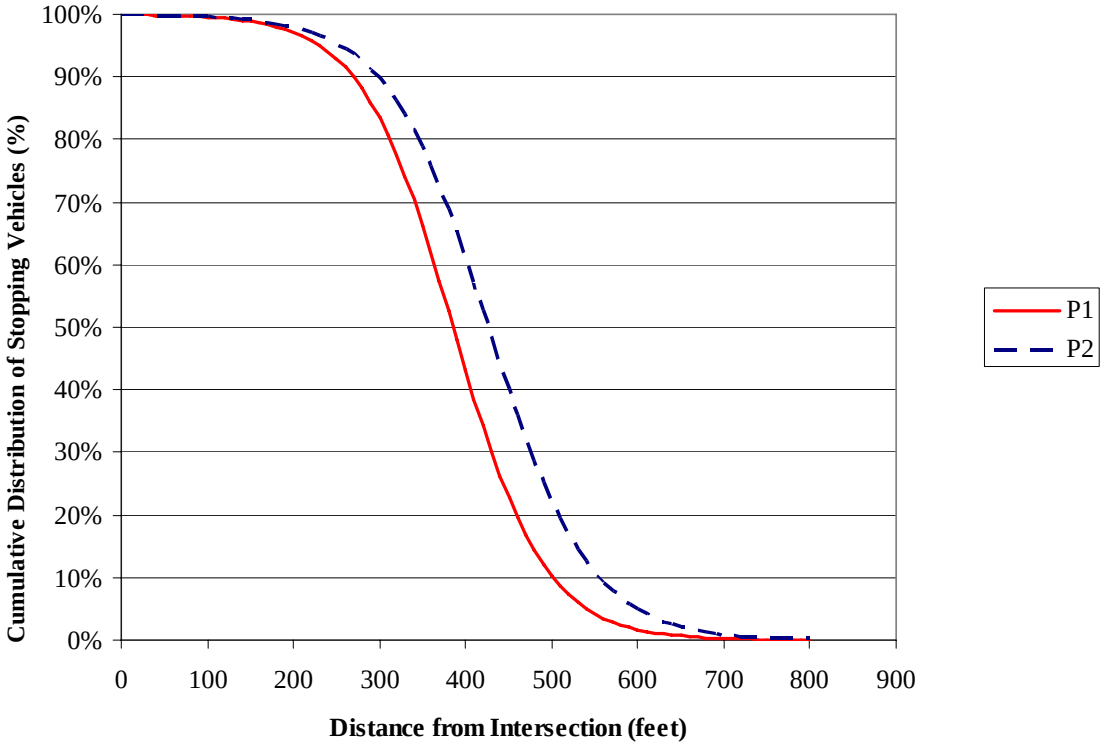

Figure K-3 Cumulative distribution of probability stopping distances of heavy vehicles.

Table K-1 Stopping Distance Probability Values

Distance from Intersection (feet)	Probability of Stopping (%)					
	Passenger Cars		Passenger Trucks		Heavy Vehicles	
	P1	P2	P1	P2	P1	P2
100	99.4	99.5	99.3	99.5	99.6	99.6
200	95.2	96.6	93.2	96.8	97.1	98.0
300	69.8	80.0	56.8	80.8	83.5	89.7
400	21.1	35.6	11.2	37.3	43.2	61.1
500	3.0	7.1	1.2	7.7	10.2	22.1
600	0.4	1.1	0.1	1.2	1.7	4.8
700	0.0	0.1	0.0	0.2	0.3	0.9
800	0.0	0.0	0.0	0.0	0.0	0.2
900	0.0	0.0	0.0	0.0	0.0	0.0

Table K-2 Passenger Car Probability Summary (55 mph)

Distance from Intersection (feet)	Time from Intersection (sec)	Passenger Car Utility	Probability of Stopping (%)
126	1.56	4.60	1
202	2.50	2.96	5
237	2.94	2.20	10
339	4.20	0.00	50
440	5.45	-2.18	90
475	5.89	-2.94	95

Table K-3 Passenger Car Probability Summary (60 mph)

Distance from Intersection (feet)	Time from Intersection (sec)	Passenger Car Utility	Probability of Stopping (%)
126	1.43	4.60	1
202	2.30	2.96	5
237	2.69	2.20	10
339	3.85	0.00	50
440	5.00	-2.18	90
475	5.40	-2.94	95

Table K-4 Passenger Car Probability Summary (65 mph)

Distance from Intersection (feet)	Time from Intersection (sec)	Passenger Car Utility	Probability of Stopping (%)
126	1.32	4.60	1
202	2.12	2.96	5
237	2.49	2.20	10
339	3.56	0.00	50
440	4.62	-2.18	90
475	4.98	-2.94	95