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16. Abstract
The primary objective of this research was to validate the new TxDOT procedures for loop detector placement on high-speed approaches to signalized intersections. The study approach involved conducting a field study at selected sites to compare the proposed new loop configuration to the existing configuration. The data analysis included investigating approach speeds to the intersection, driver actions in response to a yellow indication, and vehicle location at the onset of yellow.

Results from the field study revealed that the new loop configuration is as good as, and in some cases better than, the old loop configuration. Because the new loop configuration can detect vehicles further upstream from the intersection (at the beginning of the dilemma zone), it results in fewer vehicles being caught in the dilemma zone at the onset of yellow. The new loop configuration resulted in fewer vehicles running the red light, a major cause of accidents. Also, because the new loop configuration typically resulted in more vehicles using the yellow light instead of stopping, fewer rear-end accidents may result.

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EVALUATION OF DETECTOR PLACEMENT FOR HIGH-SPEED APPROACHES TO SIGNALIZED INTERSECTIONS

by

Dan Middleton, P.E.
Associate Research Engineer
Texas Transportation Institute

R. Lewis Nowlin
Assistant Research Scientist
Texas Transportation Institute

Mark Shafer
Assistant Research Scientist
Texas Transportation Institute

Angelia H. Parham, P.E.
Assistant Research Engineer
Texas Transportation Institute

and

Debbie Jasek
Research Associate
Texas Transportation Institute

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The Texas A&M University System
College Station, Texas 77843-3135

IMPLEMENTATION STATEMENT

TxDOT's new procedures for detector placement address all high-speed approaches from 72 km/h (45 mph) to 113 km/h (70 mph). The results of this research indicate that the new TxDOT detector placement performs as expected in detection of vehicles at much greater distances from the intersection. This provides more distance (or time) for drivers to make the appropriate decision upon the onset of yellow, then red. The new detector placement plan has already been implemented in a few districts where 113 km/h (70 mph) approaches exist. With the successful outcome of this project, this detector scheme should be implemented elsewhere at intersections which are otherwise safe for these speeds. Based on the findings from this study, it is recommended that the detector layout be based on the 85th percentile approach speed to the intersection (as opposed to the posted speed limit). Deliverables for this project include a standard sheet for implementation of the new detector layout on future construction projects.

DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Texas Department of Transportation (TxDOT). This report does not constitute a standard, specification, or regulation, nor is it intended for construction, bidding, or permit purposes. The principal investigator for the project was Dan Middleton, P.E. #60764.

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SUMMARY

The restoration of the 113 km/h (70 mph) speed limit in Texas has created a concern for signalized intersections with the higher approach speeds. Current TxDOT recommended procedures do not address approach speeds above 89 km/h (55 mph), and therefore, the high-speed approach intersections may not have adequate *dilemma zone* detection. TxDOT has proposed a new procedure for detector placement which addresses all high-speed approaches 72 to 113 km/h (45 to 70 mph).

The researchers' primary objective was to validate the new TxDOT procedures for loop detector placement on high-speed approaches to signalized intersections. The goal of the new procedures is to increase the safety at high-speed approach intersections above that of existing procedures. The study approach involved the following five main tasks: literature search and review, survey of other state practices, data collection at existing field sites with high-speed approaches, data analysis, and development of recommendations.

The field study involved conducting a before/after analysis at selected sites to compare the proposed new loop configuration to the existing configuration. The data analysis included investigating approach speeds to the intersection, driver actions in response to a yellow indication, and vehicle location at the onset of yellow.

Results from the field study revealed that the new loop configuration is as good as, and in some cases better than, the old loop configuration. Because the new loop configuration can detect vehicles further upstream from the intersection (at the beginning of the dilemma zone), it results in fewer vehicles being caught in the dilemma zone at the onset of yellow. The new loop configuration resulted in fewer vehicles running the red light, a major cause of accidents. Also, because the new loop configuration typically resulted in more vehicles running the yellow light instead of stopping, fewer rear-end accidents may result.

The new detector placement plan has already been implemented in a few districts where 113 km/h (70 mph) approaches exist. Based on the findings from this study, the researchers recommend that the new procedures for loop detector placement should be implemented elsewhere at intersections which are otherwise safe for these speeds. In addition, it is recommended that the detector layout be based on the 85^{th} percentile approach speed to the intersection (as opposed to the posted speed limit).

1.0 INTRODUCTION AND METHODOLOGY

1.1 OVERVIEW

The restoration of the 113 km/h (70 mph) speed limit in Texas has created a concern for signalized intersections with the higher approach speeds. TxDOT recommended procedures do not address approach speeds above 89 km/h (55 mph), and therefore, the high-speed approach intersections may not have adequate *dilemma zone* detection. The term dilemma zone refers to either a physical segment of the intersection approach, or it can be defined in terms of the decision-making process. The "physical segment" refers to a physical length of the approach in which a driver cannot go through the intersection or stop legally. The "decision-making" definition refers to the area where the probability of drivers attempting to stop is between 10 and 90 percent. TxDOT has developed a new procedure for detector placement which addresses all high-speed approaches 72 to 113 km/h (45 to 70 mph). Two TxDOT districts are field testing the recommended procedure and are preparing plans for intersections with 113 km/h (70 mph) approaches.

The problem at hand concerns traffic actuated control in which demand varies throughout the day or main street traffic which is heavier than side street traffic. The quality of service provided by the controller/detector system is dependent upon three items: 1) controller settings, 2) detector unit operation, and 3) detector layout. The third item is the primary subject matter of this research. Optimum performance of the detector layout requires a detector design "tuned" to the geometry of the intersection and its traffic demand. It should also be noted that the detector of choice is still the inductive loop detector (ILD), although other detection technologies such as video imaging could also be used. However, the increased detection distances of 183 m (600 ft) or more required for high-speed approaches challenges the currently available products using the typically available camera optics and mounting heights.

This research study was used to evaluate the new recommended procedure to ensure that it accomplishes the intended goal of providing adequate safety at high-speed approach intersections. The new procedure, if successful, will be implemented statewide for intersections with high-speed approaches.

1.2 RESEARCH FOCUS

This research study focused on maximizing traffic safety as opposed to emphasizing efficiency on high-speed approaches, even though efficiency is still an important topic to be considered. Typically, detector designs which avoid the onset of yellow when the intersection approach is occupied are less likely to be associated with rear-end crashes. In this context, it should be noted that there are differences in driver responses to the onset of the yellow indication. With two or more drivers on an approach presented with the yellow, it is likely that some drivers will decide to stop while others will continue through the intersection. These conflicting responses create the potential for rear-end crashes when stopping drivers are ahead of those choosing to proceed. There is also the potential for right-angle crashes within the intersection for vehicles proceeding through upon the onset of the yellow.

1.3 RESEARCH OBJECTIVES

The work plan for this study initially consisted of six specific research objectives including: a literature search and review, survey of other state practices, data collection at high-speed approaches, data analysis, simulation of selected speed categories, and preparation of reports. However, a modification of the study eliminated the simulation of selected speed categories and replaced it with additional field data collection.

1.4 METHODOLOGY

A detailed description of the approach the research team used to accomplish the objectives addressed in this report is presented below.

1.4.1 Literature Search and Review

A comprehensive literature search, which is fundamental for any research project, was conducted to identify publications and reports on state-of-the-art technologies and current knowledge concerning traffic signal detector placement, high-speed intersections and dilemma zones. This search, using key words and phrases, utilized the following catalogs and databases: Texas A&M University's Sterling C. Evans Library NOTIS (local library database), Wilson's Periodical Database, FirstSearch, National Technical Information System (NTIS), and Transportation Research Information Service (TRIS).

Sterling C. Evans Library is a major local source of information with holdings of more than two million volumes of books, 4.3 million documents and microforms, 12,000 current periodical titles and holdings for more than 28,000 serial titles. FirstSearch is an electronic information system designed to provide access through the Online Computer Library Center (OCLC) national database. The database contains more than 34 million bibliographic records representing the holdings of 22,000 libraries in more than 63 countries and to Article First and Contents First which index 11,000 journals. NTIS is a CD-ROM database which provides bibliographic records of published scientific and technical information. TRIS is a worldwide source of information on various modes and aspects of transportation including planning, design, finance, construction, equipment, traffic, operations, management, marketing, safety, and other topics. It contains more than 315,000 abstracts of completed research, summaries of research projects in progress, and selected articles from more than 1,000 journals. TRIS also includes access to TLIB (Transportation Library Subfile) which is the bibliographic citations of the new acquisitions of the Institute of Transportation Studies Library at the University of California, Berkeley, and the Northwestern University Transportation Library at Evanston. TLIB covers all modes of transportation and provides an annual input of more than 9,500 records to TRIS.

Key words and key word combinations were selected to conduct a systematic search of the above databases. Some of the key words and key word combinations used in the search included: dilemma zone, high-speed signalized intersections, detectors, traffic detectors, detector placement, signal timing, and high-speed intersections.

Approximately 175 documents were identified as possible sources and were reviewed for relevance. The literature review is discussed in detail in Chapter 2.

1.4.2 Survey of Other States

Several states were identified through the literature search process and from the knowledge of project staff. The TTI research team conducted a telephone survey with a number of these states. The survey included questions about the procedures used in each state and quality of the data used for evaluation. The survey of states is discussed in Chapter 2, and a copy of the survey can be found in Appendix A.

1.4.3 Data Collection at High-Speed Approaches

The primary goal of this research study was to validate the new TxDOT procedures for high-speed approaches for speeds from 72 km/h (45 mph) through 113 km/h (70 mph). TxDOT provided support for TTI data collection at four intersections with 113 km/h (70 mph) approach speeds for field data collection: three in the Houston district and one in the Odessa district. A fifth intersection, located in the Brownwood district with a 80 km/h (50 mph) speed limit, was included in the crash analysis portion. The primary interest in the field data collection activity was determining whether dilemma zone protection is adequate and how it compares with procedures used today for slower approach speeds. The goal of the new procedures is increasing the safety at high-speed approach intersections above that of existing procedures. Data collection for accomplishing this evaluation was directed at both vehicle crashes and erratic maneuvers in a before/after study scenario. Given the short duration of the study plus the fact that speed limits were recently increased, availability of crash data during both the "before" and "after" periods (constant speed limit) was limited. The "after" data were also limited by the typical delay involved in accident record keeping.

The field data collection portion of this research took a two-pronged approach. The first step was to evaluate the performance of the detector system as vehicles approach the intersection. The second step included an evaluation of how well the overall signal system (including detectors) performed in terms of dilemma zone protection.

For monitoring conflicts at study sites, research staff utilized color video cameras to monitor high-speed approaches during the data collection phase. For this task, technicians mounted a camera on a trailer equipped with a telescoping pole that can be extended up to 9.1 m (30 ft). This field procedure required visible "targets" along the pavement to help data reducers determine exact locations of detectors and limits of the dilemma zone during video replay.

1.4.4 Data Analysis

The data analysis included investigating approach speeds to the intersection, driver actions in response to a yellow indication, and vehicle location at the onset of yellow. The evaluation followed a "before-after" scenario in which the existing (89 km/h [55 mph]) detector placement and signal timing plan represented the "before" period. Once sufficient data were collected under the existing situation, the study team evaluated the proposed TxDOT detector placement. Data collection followed a statistically sound plan in order to make an accurate comparison from the "before" to "after" scenarios. Evaluation used the *t*-test and chi-square test to study driver actions and vehicle locations within the dilemma zone. For the crash rate analysis, the original intent was to research statistically significant changes in crash rates or severity between the "before" and "after" time periods. However, evaluation was limited by a shortage of data.

2.0 LITERATURE REVIEW AND SURVEY OF EXISTING PRACTICES

2.1 OVERVIEW

Current detector designs used on Texas roadways do not consider recent changes in the maximum speed limits that were increased to 113 km/h (70 mph). Higher approach speeds increase the length of the dilemma zone and increase the probability of crashes. The term dilemma zone refers to either a physical segment of the intersection approach, or it can be defined in terms of the decision-making process. The "physical segment" refers to a physical length of the approach in which a driver cannot go through the intersection or stop legally. The "decision-making" definition refers to the area where the probability of drivers attempting to stop is between 10 and 90 percent *(1)*.

An examination of the literature revealed that there is a broad range of design philosophies being used for detector placement. Some agencies locate advance detectors based on stopping sight distance for a specified design speed. The design speed is decreased by 16 km/h (10 mph) for each successive detector on the approach. Other agencies locate detectors based on having a constant travel time between successive detector pairs. Some agencies choose to extend the green until the vehicle is fully within the intersection. Other agencies prefer to extend the green until the vehicle clears its dilemma zone. Yet other agency approaches vary based on controller options (e.g., locking versus non-locking memory).

2.2 DILEMMA ZONES

As previously noted, the dilemma zone is a term that refers to either a physical segment of the intersection approach or to the decision-making process. In both early and current research on dilemma zones there is some disagreement as to the location of dilemma zone boundaries. Some of this disparity can be explained by differences in driver/vehicle populations at the various test sites. A recent study by Bonneson et al. *(2, 3)* noted a trend toward increased length of dilemma zone boundaries compared to older study findings. It suggested that the reason for the increase is a trend toward decreasing driver respect for the change interval.

In the early dilemma zone analyses, Parsonson et al. *(1)* examined and summarized existing research on the probability of stopping from various speeds *(4, 5, 6)*. Comparison of data collected by Zegeer of the Kentucky Department of Transportation *(7)* revealed that his dilemma zones (10 and 90 percent probabilities of stopping) were 28 to 38 percent longer than those measured by Parsonson et al. *(10)* for speeds of 72 to 80 km/h (45 to 50 mph). Since Zegeer's data were collected under closely controlled conditions, many practitioners have used his data.

Zegeer *(7)*, using the parameter of passage time, found that five seconds was sufficient for vehicles to travel from the initial upstream detector to the intersection for speeds below 97 km/h (60 mph). Other methods used before these analyses involved kinematic analyses of either stopping or clearing the stop bar. Some early investigators used AASHTO (then AASHO) minimum stopping sight distances, while others used a one second driver reaction time and an emergency stop on dry pavement *(8)*. One of the detector-controller design scenarios that looked very promising to these investigators used a green extension system, apparently similar to that used today but probably more primitive. One example used a 21 m (70 ft) loop detector at the stop bar for normal detector output supplemented by an extended call detector five seconds before the stop bar. Zegeer *(7)* reported on the effectiveness of five locations in Kentucky, concluding that there was an overall crash reduction of approximately 50 percent compared to previously used detection scenarios. Another parameter measured by Zegeer in dilemma zone studies was traffic conflicts *(9)*. In studies before and after installation of green-phase extension systems (GES), he used the following six types of conflicts: red light runs, abrupt stops, swerve to avoid collision, vehicle skidded, acceleration through yellow, and brakes applied before passing through the intersection. Zegeer's findings included reductions in conflicts at two test sites with the use of GES. Mean values of conflict rates reduced from 4.34 to 2.64 conflicts per 15-minute interval at one site and from 4.22 to 0.66 at another site.

2.3 TRAFFIC CONTROL AT HIGH-SPEED INTERSECTIONS

In a study by Parsonson reported in an NCHRP Synthesis entitled *Signal Timing Improvement Practices (10)*, information is provided on traffic signal phase change interval practice in various states. He found that at least half of the states follow the "permissive yellow rule" that permits vehicles to enter an intersection on a yellow signal and to be in the intersection when the signal changes to red. Parsonson noted that the *Manual on Uniform Traffic Control Devices* (MUTCD) *(11)* provides the following guidance on change intervals, "Yellow vehicle change intervals should have a range of approximately 3 to 6 seconds. Generally, the longer intervals are appropriate for higher approach speeds."

Parsonson conducted a survey regarding yellow time and approach speed; Table 2-1 provides the results. Findings of the study included: 1) there is a need for uniform timing practices and procedures, 2) there is a need for field observations prior to setting signal timing, and 3) there is a tendency for computer program generated cycles to be too short.

In a study entitled "Traffic Control and Accidents at Rural High-Speed Intersections," *(12)* Agent examined the effect of traffic control on crashes at high-speed rural intersections. The objectives of this study were:

- To determine the types of traffic control measures used at rural high-speed intersections,
- To establish the type of crashes that occur at rural high-speed intersections,
- To discover the factors that contribute to these crashes, and

Table 2-1. Change Intervals Used by Various Jurisdictions.

City, County or State	Speed km/h (mph)	Yellow Time (seconds)
New York State	89 (55)	5.0
	97 (60)	5.4
Iowa DOT	>64 (40)	5
Montgomery County, Maryland	>72 (45)	5
Lakewood, Colorado	89 (55)	5.5

- To recommend the traffic control measures that could most effectively decrease potential crashes.

Agent conducted the study by using a sample of rural high-speed intersections in Kentucky. He evaluated crash records using the following site specific factors as variables: geometry, traffic control measures, speed, sight distance, channelization, pavement markings, and intersection type. Agent found that providing the driver adequate warning of the intersection, providing proper change intervals, and maximizing visibility of the signal heads were important to minimizing the crash risk at high-speed rural intersections. He also noted that a red clearance interval should always be provided for both roadways.

Agent and Pigman *(13)*, in their study entitled *Evaluation of Change Interval Treatments for Traffic Signals at High-Speed Intersections*, found that a large number of traffic crashes at signalized intersections on high-speed roadways occur during or just after the change interval. The green extension system is extensively used in Kentucky as a way of alleviating the problem related to the dilemma zone. This study evaluated both the green extension system and an advanced warning flasher system. The study evaluated how these systems could be used in diminishing problems associated with dilemma zones at signalized intersections, with a specific focus on high-speed roadways.

2.4 RED CLEARANCE INTERVALS

The all-red clearance interval proposed by Agent (*12*) in his study of rural high-speed intersections has been debated by traffic engineers as a safety measure. A number of studies have been conducted on the subject as it relates to intersection safety. These studies include research by Newby (*14*), the ITE Technical Council Committee 4A-16 (*15*), Benioff et al. (*16*), and Roper et al. (*17*).

The Newby study documented four years of research, two years before and two years after, on 12 intersection sites in England. The study showed that there was a definite decrease in crashes. This decrease was attributed to the introduction of all-red clearance intervals (*14*).

Benioff et al. conducted a comprehensive study which examined all-red clearance intervals at 45 sites. This research, which used crash rates as the primary measure of safety, found that the total crash rate decreased mainly due to a reduction of the right-angle crash rate (*16*). Research by Benioff et al., in 1980, concluded that when an intersection has a right-angle crash rate greater than 0.8 right-angle crashes per million entering vehicles, implementing an all-red clearance interval should be considered. A report by the ITE Technical Council Committee 4A-16, in 1985, found a 21 percent reduction in right-angle crashes during the first year after the implementation of all-red clearance intervals in the Hamilton-Wentworh Regional Municipality in Ontario, Canada (*15*). It should be noted that Roper et al. observed that none of these previous studies used a comparison group to measure the all-red clearance interval's success relative to intersections without the all-red clearance interval (*17*). The study by Roper et al. examined 50 intersections in Indiana; 25 intersections were "treated" with the all-red clearance interval, and 25 were comparison or control intersections. The study concluded that there was no significant decrease in crashes by the use of the all-red clearance interval.

2.5 DILEMMA ZONE PROTECTION

In a recent *ITE Journal* article entitled, "Traffic Detector Designs for Isolated Intersections," Bonneson and McCoy *(2)* provided some insights based on their recent research on detector design *(3)*. They stated that the overall objective in properly designing detection at actuated high-speed approaches is to minimize delay without compromising safety. This is typically accomplished by proper coordination of detector size and location with the various timing features of the detector unit and controller. The authors discuss dilemma zone protection and describe it as the prevention of phase termination while a vehicle is in the dilemma zone. This protection may be achieved by strategically locating detectors on the intersection approach and adjusting the detector unit settings such that a vehicle can "hold" the green while it travels through the dilemma zone. As vehicles approach the dilemma zone, drivers face a decision upon onset of yellow to either stop or proceed through the intersection. Intuition suggests a correlation between the number of vehicular crashes (typically rear-end) and frequency of "max-out." This is primarily due to a leading vehicle that attempts to stop followed by a vehicle in the same lane that attempts to proceed. The authors promote the idea of dilemma zone protection through proper design of advance detectors.

Bonneson and McCoy discussed recommended detector designs for both urban and rural actuated signalized intersections. Advance detector design is determined by the range of speeds on the approach. Each advance loop has its own design speed, with the highest design speed for the detector farthest from the stop bar. Each subsequent detector has a design speed of approximately 16 km/h (10 mph) less than the one just upstream. One indicator of design performance is the maximum allowable headway (MAH) produced by a particular detector design. The MAH represents the maximum time headway that can occur between successive calls to the controller such that the green is extended in spite of demand on a conflicting movement. There is no set MAH that is best for all detector designs due to the many possible variables. In general, shorter MAHs reduce the frequency of max-out and delay to waiting traffic. The authors suggested using the *Manual of Traffic Detector Design (18)* for determining a design's MAH.

Woods and Koniki, in a report entitled, *Optimizing Detector Placement for High Speed Isolated Signalized Intersections Using Vehicular Delay as the Criterion, (19)* noted a negative aspect of providing dilemma zone protection. On high-speed approaches to an isolated intersection, providing dilemma zone protection may result in sluggish operations and possibly higher delays. A trade-off analysis of detector placement is essential for optimization of dilemma zone protection and reducing delays. They utilized the TEXAS Model (Version 3.2) to determine optimal detector placement strategies on high-speed isolated intersections. Traffic volumes varied between 200 vehicles per hour per approach to 800 vehicles per hour per approach. Mean speeds of 89 km/h (55 mph), 72 km/h (45 mph), and 55 km/h (35 mph) were simulated. Detector placements were developed for both the mean and 85th percentile speeds.

The authors used a regression analysis on delays and cycle lengths to show that a strong linear relationship exists between them. This analysis varied detector layouts to develop this relationship. At low approach volumes, there was no effect of mean and 85th percentile speeds on delays; at higher approach volumes, 85th percentile speeds resulted in higher delays.

2.6 ADVANCE WARNING SIGNS

References *(20)*, *(21)*, and *(22)* document research on the effects of advance warning treatments on crashes and crash potential at high-speed signalized intersections. Pant and Huang *(20)* in a report entitled "Active Advance Warning Signs at High-Speed Signalized Intersections: Results of a Study in Ohio," stated that the crash potential at high-speed intersections is high in the dilemma zone. The dilemma zone is an area close to the intersection where drivers must decide whether to stop or to attempt to clear the intersection during the yellow clearance interval. The authors examined four types of advance warning signs at high-speed signalized intersections in Ohio. The advance warning signs examined were:

- PREPARE TO STOP WHEN FLASHING,
- Flashing Symbolic SIGNAL AHEAD,
- Continuously Flashing Symbolic SIGNAL AHEAD, and
- Passive Symbolic SIGNAL AHEAD.

A background review found that the most commonly used advance warning sign is the PREPARE TO STOP WHEN FLASHING sign. This sign as well as the Flashing Symbolic SIGNAL AHEAD have yellow flashers at the top and bottom of the sign that are activated near the end of the green interval and remain flashing until the end of the red interval.

The study performed by the authors collected and analyzed field data at selected high-speed intersections in Ohio. The authors found that, in some instances, active advance warning signs such as the PREPARE TO STOP WHEN FLASHING and the Flashing Symbolic SIGNAL

AHEAD encouraged higher speeds. Increased speeds occurred when the flasher was inactive and the signal indication was either green or yellow. The authors also noted that some motorists did not equate the PREPARE TO STOP WHEN FLASHING sign with the traffic signal at the intersection. Pant and Huang recommended that further research be conducted regarding the Flashing Symbolic SIGNAL AHEAD sign. They also noted that the Continuously Flashing Symbolic SIGNAL AHEAD sign appeared to be preferable to the PREPARE TO STOP WHEN FLASHING sign for reducing the speed of vehicles that were approaching the intersection.

Huang and Pant *(21)* published findings in a Transportation Research Record article entitled, "Simulation-Neural Network Model for Evaluating Dilemma Zone Problems at High-Speed Signalized Intersections." Their findings indicated that at a high-speed signalized intersection the difficulty of the dilemma zone decision is increased because drivers may not be able to stop using a reasonable deceleration rate or clear the intersection before the light turns red. Choices made by drivers in this situation may lead to increased crash risk. Various traffic control devices including advance warning signs, detectors, flashers, and signal timing have been used in an attempt to assist drivers in the dilemma zone.

Huang and Pant present a simulation-neural network model for evaluating the dilemma zone. By using a neural network in the simulation of the vehicle movements, the quality of the simulation was improved. The simulation model can also represent the effects of various traffic control devices that are used to reduce crash risk. It can, therefore, be used as a non-crash based method for evaluation of high-speed signalized intersections.

Pant, Xie, and Huang *(22)* studied the effects of two dynamic signs and a static sign at tangent and curved approaches to rural, high-speed signalized intersections. Their final report is entitled, "Evaluation of Detection and Signing Systems for High-Speed Signalized Intersections." As in reference *(20)*, they tested dynamic signs that begin to flash just prior to the onset of the yellow interval of the traffic signal, and static signs that flash continuously. The study revealed that the PREPARE TO STOP WHEN FLASHING sign and the flashing symbolic SIGNAL AHEAD sign generally have similar effects on driver behavior. However, the authors concluded that the PREPARE TO STOP WHEN FLASHING sign should not be used at a tangent approach to a high-speed intersection. They found that the continuously flashing symbolic SIGNAL AHEAD sign should be considered first.

2.7 LEFT TURNS AT HIGH-SPEED SIGNALIZED INTERSECTIONS

In his thesis entitled *Left-turn Treatment and Safety at High-Speed Signalized Intersections, (8)* Sankar addresses issues related with the safety of left-turn treatments at signalized intersections that have approach speeds of 56 km/h (35 mph) or greater. Sankar found that crashes involving left-turns were over-represented by a factor of three in the total crash population. The purpose of this study was to develop statistical models to assist engineers in design alternatives that take left-turn treatments and crashes into consideration. Sankar developed Linear and Poisson models for left-turn volumes in the interval from 500 to 1,000 vehicles per day. He interpreted the results to indicate that there are relationships between left-turn crash rates, traffic characteristics, and left-turn treatments.

2.8 SURVEY OF EXISTING PRACTICES

A telephone survey was conducted to determine polices in use for high-speed loop detection at signalized intersections. The 14 states contacted were selected based upon Internet chat information, knowledge of states with speed limits greater than 97 km/h (60 mph), states that have long sections of open highway where high-speed approaches are not uncommon, and personal contacts of the researchers.

Contacts included state traffic engineers and engineers responsible for traffic signal systems. Respondents were asked a series of questions related to loop detector placement for high-speed approaches. A copy of the survey questions is included in Appendix A.

As indicated in Table 2-2, nine of the 14 states contacted have a policy or procedure for loop detector placement at high-speed approaches. However, several of the policies do not include ILD spacings for speeds greater than 89 or 97 km/h (55 or 60 mph), and many of the states indicated that they try to avoid signalized approaches at 89 km/h (55 mph) or greater. For example, Wyoming drops the speed limit to 72 km/h (45 mph) at intersections when a traffic signal is installed. Alternately, the state of Washington has isolated signals posted at speeds of 89 km/h (55 mph) or greater and has a policy with a variable number of loops based upon 90th percentile speed, perception reaction times, and deceleration rates. Policies and procedures vary from state to state, and summaries of these policies are included in Appendix B.

Table 2-2. Summary of Telephone Survey.

STATE	POLICY / PROCEDURE FOR HIGH-SPEED DETECTION	COMMENTS
Arizona	Yes	Not used for high speeds; most approaches at 72 km/h (45 mph) or less. Placement based on February 1974 issue of *Traffic Engineering* article "Small Area Detection at Intersection Approaches."
California	Yes	Loop placement based on deceleration rate for dry condition.
Kentucky	Yes	Green Extension System for isolated signals or first in a series with 85th percentile speed ≥ 72 km/h (45 mph). Set of 2 loops with distances based on approach grades.
Maryland	Yes	No arterial or surface streets > 97 km/h (60 mph). Use a dilemma zone chart.
Minnesota	Yes	Based on design and operational requirements. Currently no intersections signed > 89 km/h (55 mph). Detector placement based on dilemma zone chart.
Missouri	Yes	85th percentile speed≥ 72 km/h (45 mph); new guideline in trial period. Two pulse detectors at 8 seconds and 5 seconds back from stop bar.
Montana	No	Although the daytime speed limit is 'reasonable and prudent' with no limit for passenger cars, they try to avoid high-speed approaches.
Nebraska	Yes	Approach speed up to 97 km/h (60 mph); 3 detectors based on 2-second extension.
New Jersey	No	No speed limits > 89 km/h (55 mph).
Ohio	Yes	For approach speeds from 64 - 97 km/h (40 - 60 mph), use 2 loops with placement based on approach speed.
Oklahoma	No	N / A
Tennessee	No	Do not have a written policy but use a standard loop placement procedure for speeds > 72 km/h (45 mph).
Washington	Yes	Procedure based on 90th percentile speeds for upstream dilemma zones and 10th percentile speeds for downstream dilemma zones, perception-reaction times, and deceleration rates. Number of loops varies.
Wyoming	No	Currently no signals greater than 72 km/h (45 mph), although they do have guides for 80 and 89 km/h (50 and 55 mph).

3.0 FIELD DATA COLLECTION AND EVALUATION

3.1 INTRODUCTION

The researchers' primary objective was to validate the new TxDOT procedures for loop detector placement on high-speed approaches to signalized intersections. The goal of the new procedures was to increase the safety at high-speed approach intersections above that of existing procedures. The focus of the data collection activity was to determine whether the new procedures provide adequate dilemma zone protection and how the protection from the new procedures compares with existing procedures.

Data collection for accomplishing this evaluation focused on driver behavior and vehicle crashes using a before/after study scenario. Given the short duration of the study plus the fact that speed limits were only recently increased, availability of crash data during both the "before" and "after" periods was limited. The "after" data were also limited by the typical delay involved in accident record keeping. The study team, nonetheless, attempted to collect, evaluate, and apply statistical analyses to crash data as appropriate.

The field data collection portion of this research involved two steps. The first step was to evaluate the performance of the detector system as vehicles approached the intersection. The second step included an evaluation of how well the overall signal system (including detectors) performed in terms of dilemma zone protection. The following sections describe the data collection, data reduction, and data analysis techniques.

3.2 DATA COLLECTION

This section of the report includes a description of the equipment used to collect the data during this project. It also describes the five sites analyzed and chronicles the experiences of the TTI researchers during the data collection effort.

3.2.1 Equipment

3.2.1.1 TTI Video Trailer

The research team used a wide array of data collection equipment to collect the data necessary for this research. One of TTI's data collection trailers provided mounting support for a Cohu charged couple display (CCD) camera raised via a telescoping pole to a height of 9.1 m (30 ft). The camera's focal length varied from 6 mm to 60 mm, and a field technician utilized its pan/tilt/zoom control from ground level to establish optimum settings. The trailer's location at each field site was approximately 300 m (1000 ft) from the intersection under analysis. From this perspective, the camera provided a large area view around each intersection. It allowed monitoring of the signal indications and actions by drivers over a distance of approximately

Figure 3-1. Camera Mounted on Telescoping Pole.

244 m (800 ft) of the approach to the intersection. Figure 3-1 is a photograph of the TTI video trailer.

3.2.1.2 Vehicle Identification and Classification System

Inductive loops detectors (ILDs) provided vehicle detection for the signalized intersection at each of the sites selected to be analyzed. Four of the five sites had existing ILDs configured for approach speeds of 89 km/h (55 mph) using the old TxDOT procedure for loop detector placement. TxDOT had installed its new detector layout at the Odessa site prior to TTI's data collection. Therefore, to conduct the before/after study, TTI's field team installed the old detector layout using temporary ILDs, representing the 89 km/h (55 mph) detector spacing. These temporary ILDs used three turns of 14 gauge wire and a road tape material called Polyguard. Leads connecting ILDs with the cabinet were also 14 gauge wire. Table 3-1 summarizes the distances from the stop bar for existing loops (89 km/h (55 mph)) and new loops (113 km/h (70 mph)).

The field data collection plan also included two classifiers from International Road Dynamics (IRD) for monitoring and recording speeds of each vehicle at 107 m (350 ft) from the stop bar and at 183 m (600 ft) from the stop bar. Detection for each of the classifiers required two piezoelectric sensors placed 3.0 m (10 ft) apart and one temporary ILD in each lane. The sequence was piezo-loop-piezo as shown by Figure 3-2. Figure 3-3 shows the layout of the equipment for a typical intersection. All distances are referenced to the stop line. It should be noted that both old and new procedures also required a presence loop at the stop bar.

Table 3-1. Placement of ILDs for Old and New Procedure (Distance from Stop Bar).

Old Procedure (89 km/h (55 mph))	New Procedure (113 km/h (70 mph))
24 m (80 ft)	107 m (350 ft)
43 m (140 ft)	145 m (475 ft)
67 m (220 ft)	183 m (600 ft)
98 m (320 ft)	N/A

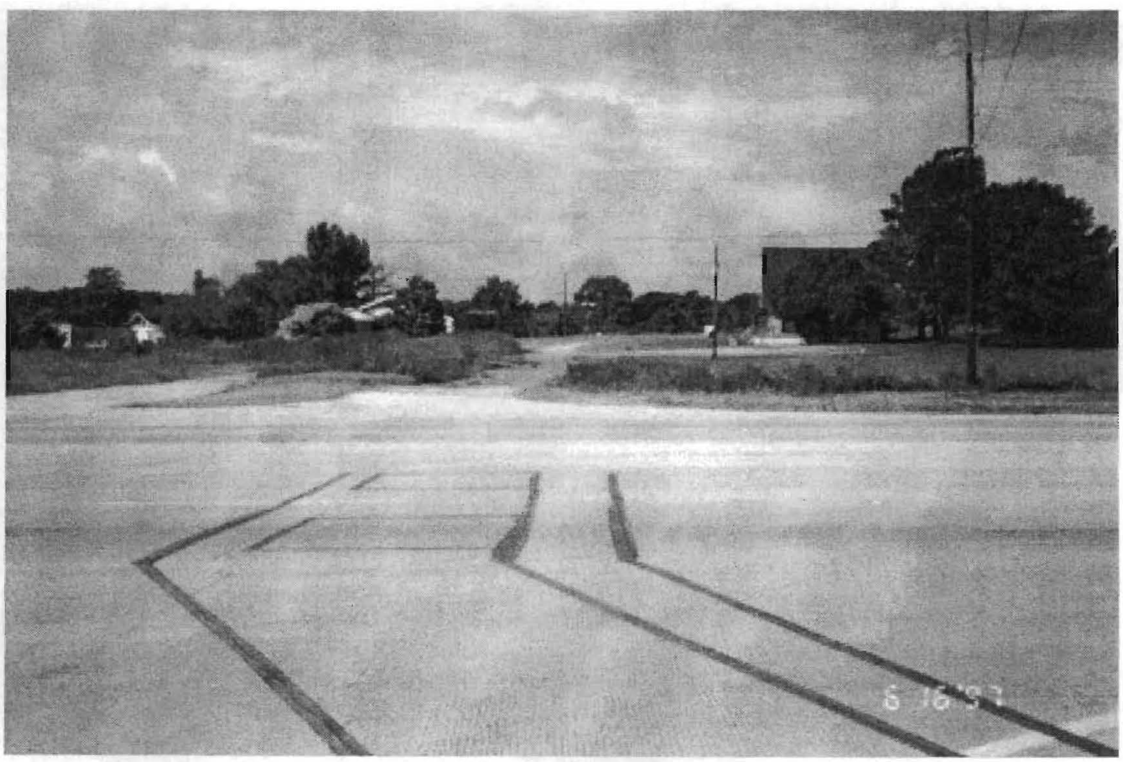

Figure 3-2. Temporary Inductive Loop and Piezoelectric Sensors.

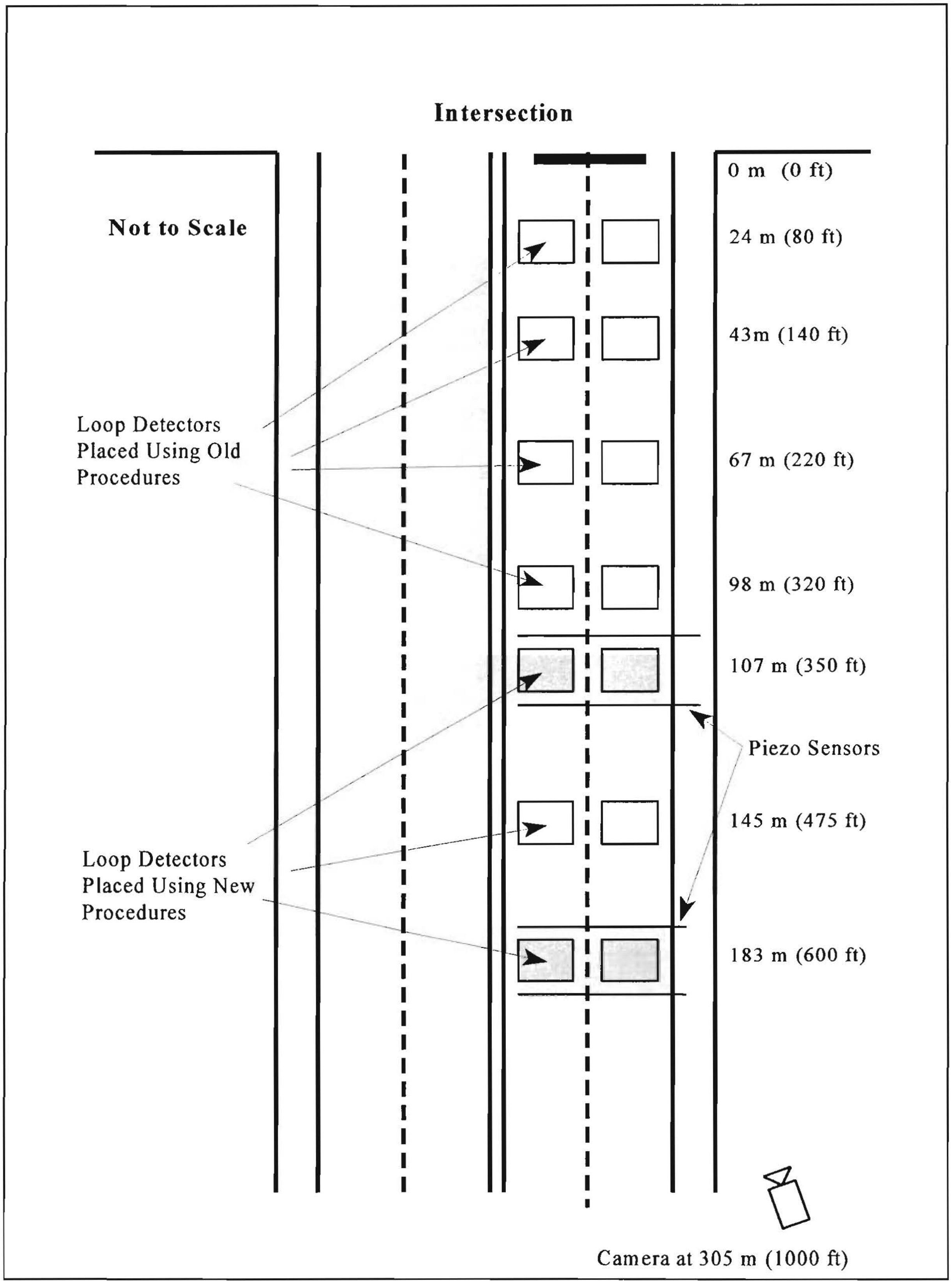

Figure 3-3. Location of Testing Equipment at Typical Signalized Intersection.

3.2.1.3 Autoscope 2004

TTI recorded the traffic stream at each intersection using a VHS video recorder. The data analysis phase used this recorded video to determine driver reactions to the yellow indication. The original data analysis plan also included the use of an Autoscope 2004 video image detection system to record signal phases and expedite the video replay process. However, there was a problem coordinating the Autoscope clock with other recording devices, so its use was ultimately discontinued.

During initial attempts, the Autoscope 2004 was placed inside the signal controller cabinet and connected to the phase indicator terminals for the phase of the approach under examination. These terminals, located on the cabinet's back panel, receive a signal from the signal controller upon initiation of each phase. This connection allowed the Autoscope to detect and record changes in the signal display for each portion (green, yellow, and red) of the relevant phase. Figure 3-4 pictures a signal control cabinet with the Autoscope installed inside. The Autoscope unit could also have been used for detection of vehicles as they traveled toward the intersection. The goal of using Autoscope was to reduce the amount of data analysis required. Recording the time at which each yellow phase occurred, and whether or not a vehicle was in the dilemma zone for that yellow phase, reduced the amount of video tape reviewed by a technician.

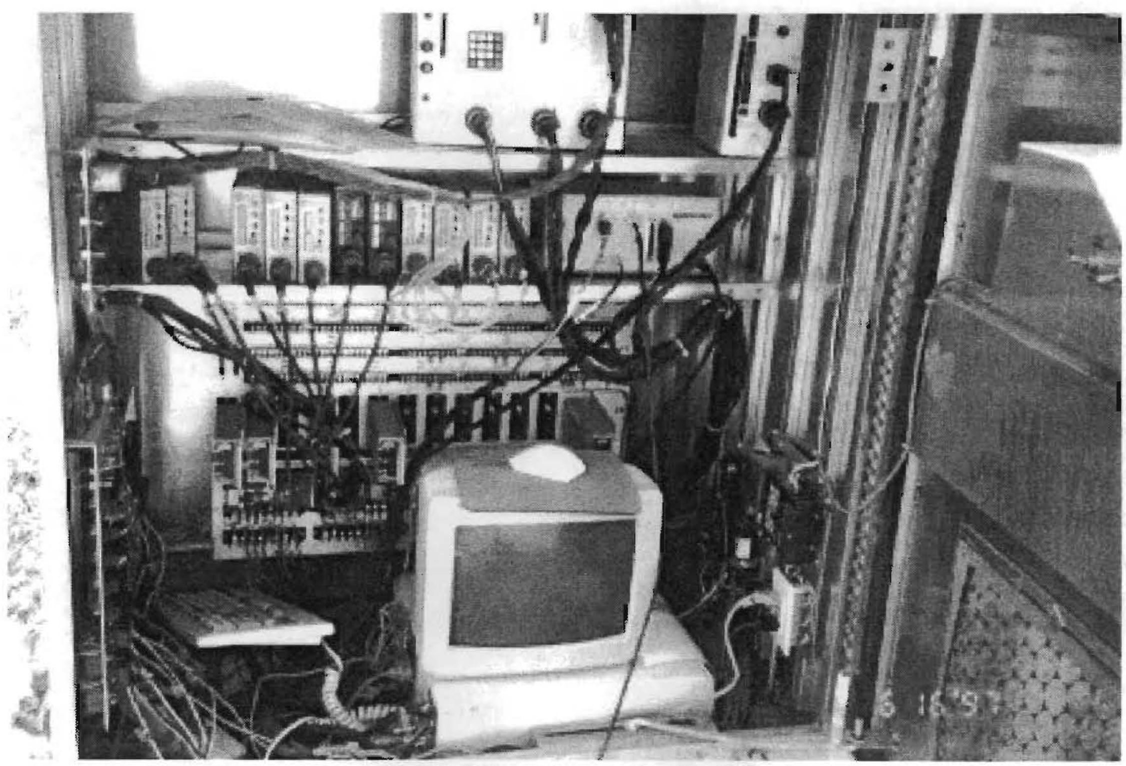

Figure 3-4. Signal Control Cabinet.

3.2.2 Data Collection Sites

A total of five sites were selected for analysis. Four of these sites were used for data collection, while one site was used to evaluate the test procedures used during the data collection. The sites used for this research were:

- FM 158 at FM 30/Elmo Weedon Road, Bryan, Texas,
- US 290 at Mason Road, Houston, Texas,
- SH 105 at Walden Road, Conroe, Texas,
- SH 105 at April Sound, Conroe, Texas, and
- Business IH-20 at County Road 1290, Odessa, Texas.

The new TxDOT procedures for ILD placement on high-speed approaches require a change in the green extension time. Table 3-2 presents the signal timings for the approach under analysis for the four data collection sites. The new green extension time of 1.2 seconds generally allows vehicles traveling greater than 97 km/h (60 mph) to continue past each successive detector and reach the end of the dilemma zone before the signal changes to yellow.

Table 3-2. Signal Timing Information for Approach Under Analysis.

Location	Min. Green (sec)	Max. Green (sec)	Existing Green Extension (sec)	New TxDOT Green Extension for 113 km/h (70 mph) (sec)
US 290 at Mason Road	25	80	2.0	1.2
SH 105 at Walden Road	20	60	1.0	1.2
SH 105 at April Sound	20	60	1.0	1.2
Business IH-20 at County Road 1290	25	60	1.0	1.2

Each of the sites was selected due to the high-approach speeds on at least one of the approaches to the intersections. The site used to test the procedures had a posted speed limit of 89 km/h (55 mph). The other four data collection sites each had a posted speed limit of 113 km/h (70 mph) on the approach that was studied during this research effort. The following sections describe each of the five sites.

3.2.2.1 FM 158 at FM 30/Elmo Weedon Road

The intersection of FM 158 at FM 30/Elmo Weedon Road in Bryan, Texas, was selected to test the methodology and procedures used for this research effort. The northbound approach of FM 158 was selected for analysis. While this intersection only had a posted speed limit of 89 km/h (55 mph), it was well suited for use as a pre-test site. Researchers placed the temporary inductive loop detectors on the pavement and connected the leads of these loops to the signal control cabinet. Piezo sensors were placed at 107 m (350 ft) and 183 m (600 ft) from the stop bar, and classifiers were connected to the piezo sensors to record the speed of each vehicle that passed over the sensors. Orange traffic cones were placed along the roadway to estimate the location of vehicles from the video recordings. Once the necessary equipment was placed on the roadway, the video camera was bolted to the telescoping pole and raised to a height of 9.1 m (30 ft). Figure 3-5 shows the view from this camera.

Researchers collected data for four days, alternating each day between the existing and the temporary loops. After the four days of data collection, the data were brought back to the office and examined in order to determine whether the correct types of information were being gathered. This pre-test helped the researchers refine the methodologies used to collect the data needed for the research effort. This site also gave TTI researchers the experience of connecting the Autoscope video detection system into a signal control cabinet (although the Autoscope's use was ultimately discontinued). After reviewing the data collected from this pre-test site and making minor adjustments to the testing procedures, preparations were made to collect data at the first of the four test sites.

3.2.2.2 US 290 at Mason Road

The intersection of US 290 and Mason Road is located on the north side of Houston at the entrance to the Fairfield community. This intersection is the only signalized intersection on US 290 on the north side of Houston. US 290, at this location, is a four-lane divided highway. TTI researchers selected the northbound approach to the intersection for analysis. This approach has two through lanes and a right-turn lane. Several traffic control devices warn motorists traveling toward this intersection that there is a signal ahead. Flashing beacons and signal ahead signs have been placed on each approach of US 290. The speed limit is posted at 113 km/h (70 mph). Figure 3-6 presents a picture of this intersection captured from COHU video camera, mounted at a height of 9.1 m (30 ft) on the TTI video trailer.

The existing loop configuration at this site was placed according to the old TxDOT procedures. TTI personnel placed temporary loops at the distances required by the new TxDOT

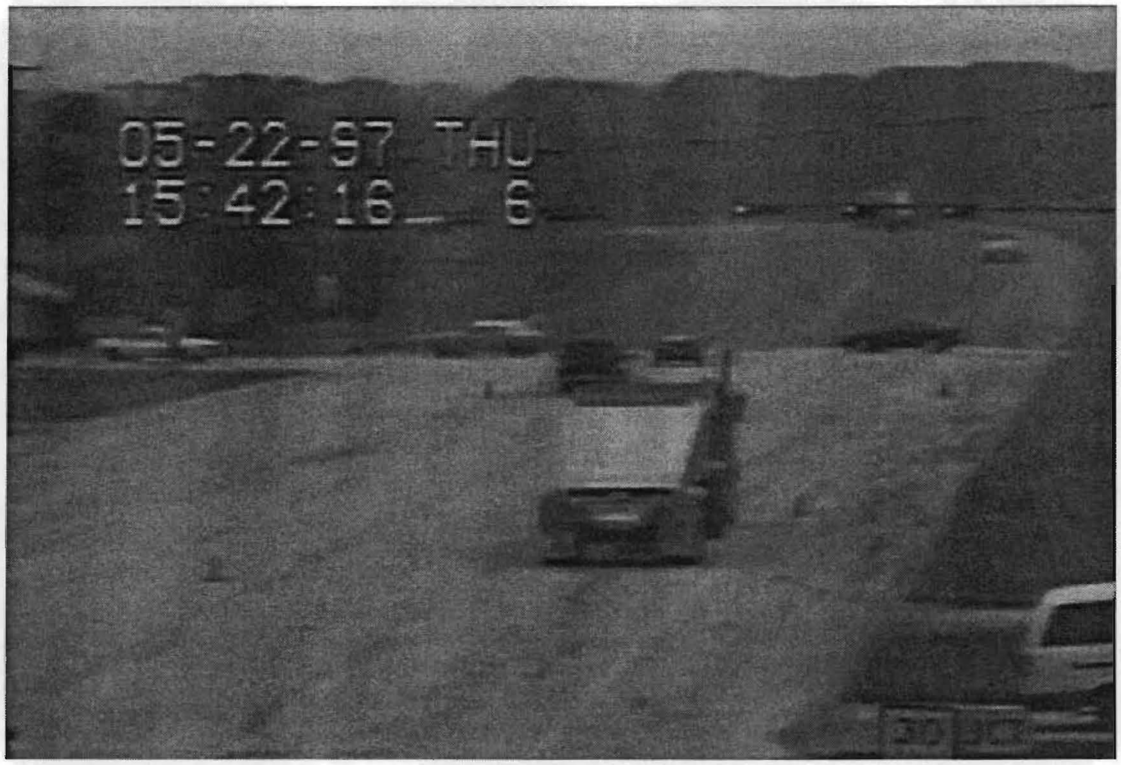

Figure 3-5. Northbound Approach of FM 158 at FM 30/Elmo Weedon Road.

procedures being analyzed in this research project. The classifiers were connected to the piezo sensors and the video equipment was put into its location. Orange traffic cones were placed every 31 m (100 ft) from the stop bar. The signals from the loop detectors and the video equipment were run over ground to the signal cabinet. The temporary loop detectors were connected to the loop input panel and tested. Problems occurred with the loop detectors due to the gauge of the wire used to connect the leads of the loops to the signal control cabinet. Once TTI researchers determined the problem, a larger gauge wire (14 gauge) was used to replace the smaller gauge wire (22 gauge). When the new wire was connected, the loop detectors functioned properly. The signal from the video camera was connected to the VCR and to the Autoscope unit. Data collection covered the hours between 7:00 a.m. and 12 a.m. each day. The original plan had the researchers switching back and forth between the existing and temporary loops on alternating days, but due to the difficulties with the lead wire on the temporary loops, this was not possible. Therefore, two days of data with the existing loops were recorded followed by two days with the temporary loops.

Figure 3-6. Westbound Approach of US 290 at Mason Road.

3.2.2.3 SH 105 at Walden Road

The intersection of SH 105 and Walden Road is located approximately 26 km (16 miles) west of Conroe, Texas. The westbound approach of SH 105 consists of two through lanes and a right-turn lane. Signal ahead signs warn motorists of the approaching signalized intersection. The posted speed limit at this site is 113 km/h (70 mph). Figure 3-7 illustrates this intersection.

Temporary ILDs were placed on the surface of the roadway according to the new loop configuration. Setup of the equipment at this intersection was similar to the previous test site with minor exceptions. Setup of the temporary loop detectors was delayed two days due to rain and a scheduling mix-up with traffic control personnel. The other notable difference in the setup was the presence of three driveways and an intersection that had to be crossed with the video cable. Only one of the driveways had to be crossed with the leads of the loop detectors. Leaving the video cable exposed resulted in another problem when a large dump truck broke the video cable. A total of four days of data were recorded at this site. The first two days recorded data

Figure 3-7. Westbound Approach of SH 105 at Walden Road.

using the existing ILDs, and the second two days recorded data using the new temporary ILDs. A failure of the classifiers at this location caused the speed data to be lost.

4.2.2.4 SH 105 at April Sound

The intersection of SH 105 at April Sound is approximately 8 km (5 miles) east of the previous test site. Again, the westbound approach of SH 105 was selected for analysis. This intersection consists of three through lanes and a left-turn lane. The posted speed limit is also 113 km/h (70 mph). Signal ahead signs are the only warning motorists receive before reaching the intersection. Figure 3-8 presents a picture of the intersection taken by the video camera.

The temporary ILDs were placed in the configuration needed for the new TxDOT procedures. The site required nine temporary inductive loops due to the additional through lane. (At the other sites, only six loops were needed.) This site required laying video cable across two separate intersections and inductive loop leads over one intersection. Breakage of the cables or wires did not occur, allowing four full days of data to be collected. The only problem that

Figure 3-8. Westbound Approach of SH 105 at April Sound.

occurred at this site was breakdown of the computer controlling the Autoscope; however, this did not cause a loss of data. A total of four days of data were recorded at this site; two days of data using the existing loops followed by two days using the temporary loops.

4.2.2.5 Business I-20 at County Road 1290

The final test site was located approximately 16 km (10 miles) east of Odessa, Texas, at the intersection of Business IH-20 and County Road 1290. The posted speed limit to the eastbound approach of Business IH-20 is 113 km/h (70 mph). This approach consists of two through lanes and a paved shoulder wide enough to be used as a right-turn lane. Figure 3-9 shows a picture of this intersection taken with the video camera.

At this site TxDOT had recently installed permanent inductive loops at the distances required by the new TxDOT procedures. Therefore, TTI researchers placed the temporary loops according to the old procedures. The temporary loops were connected to the signal control cabinet, the piezo sensors were connected to the classifiers, and the video camera was set up. Four days of data were collected at this site, alternating each day between the existing and the temporary inductive loops.

Figure 3-9. Eastbound Approach of Business IH-20 at County Road 1290.

3.3 DATA REDUCTION

Technicians marked various points along the approach to the intersection with orange traffic cones while collecting data in the field. Each point marked a particular distance from the stop bar at the intersection. These points were used to estimate a vehicle's distance from the intersection at the onset of the yellow phase. Based on a study by Bonneson, et al., (*3*), the dilemma zone for a 113 km/h (70 mph) approach speed ranges from 76 to 183 m (250 to 600 ft) from the stop bar at the intersection; therefore, the following distances were marked: 76, 91, 122, 152, and 183 m (250, 300, 400, 500, and 600 ft).

Video tape reduction efforts began by locating the points that were marked with orange traffic cones. Technicians marked each distance location on a clear sheet of plastic that covered the video monitor. Additional reference points, such as signs or poles, were also marked so that technicians could determine whether the camera had moved during filming efforts.

During each yellow phase, technicians recorded the following: time that the yellow phase began, green time preceding yellow phase, approximate location of each vehicle in the dilemma zone at the onset of yellow, and the action made by each driver in the dilemma zone. Table 3-3 lists the categories used to describe the actions of drivers in the dilemma zone. Vehicles beyond the dilemma zone that ran a red light were also recorded. The data reduced were separated into passenger cars and trucks. Trucks were vehicles with three or more axles.

Table 3-3. Driver Actions During Yellow Phase.

Category	Driver Action
1	Stop
2	Run Yellow Light
3	Run Red Light
4	Brake Before Passing Through Intersection
5	Swerve To Avoid Collision
6	Abrupt Stop

Of the six driver actions listed in Table 3-3, Actions 1 and 2 (stop and run yellow light) are the most desirable and result in the least number of crashes. Actions 3 and 6 (run red light and abrupt stop) are viewed as being most hazardous. Action 5 (swerve to avoid collision) is typically a result of another driver stopping abruptly. Action 4 (brake before passing through intersection) is a sign that the driver was located in the dilemma zone at the onset of yellow.

Data were typically collected over a four-day period at each site (two days with the old loop configuration and two days with the new loop configuration). For each site, the goal was to reduce six hours of data for each loop configuration. In most situations, this was accomplished by reducing three hours of data for each day that data were collected. The three hours included one hour of data for each of the following three conditions: off-peak, peak, and night. For some days, however, it was not possible to obtain data for each of the three conditions because of various problems (such as video that was difficult to view or roadway maintenance that was performed by TxDOT during data collection). In these situations, the data were either collected during another day (if possible) or were not obtained. Table 3-4 provides a summary of the data that were reduced at each of the field sites.

Table 3-4. Summary of Data Reduction.

Site	City	Location	Date	Time	Loop Configuration	Condition
1	Houston	US 290@Mason	6/3/97	2:00-3:00 pm	Old	Off-Peak
				5:00-6:00 pm	Old	Peak
				9:00-10:00 pm	Old	Night
			6/4/97	5:00-6:00 pm	Old	Peak
				8:00-9:00 pm	Old	Night
			6/5/97	2:00-3:00 pm	New	Off-Peak
				4:00-5:00 pm	New	Peak
				8:00-9:00 pm	New	Night
			6/6/97	2:00-3:00 pm	New	Off-Peak
				5:00-6:00 pm	New	Peak
				8:00-9:00 pm	New	Night
2	Conroe	SH 105@Walden	6/10/97	7:05-8:05 am	Old	Peak
				2:00-3:00 pm	Old	Off-Peak
				8:30-9:30 pm	Old	Night
			6/11/97	7:00-8:00 am	Old	Peak
				10:00-11:00 am	Old	Off-Peak
				8:30-9:30 pm	Old	Night
			6/12/97	7:00-8:00 am	New	Peak
				1:00-2:00 pm	New	Off-Peak
				8:30-9:30 pm	New	Night
				9:30-10:30pm	New	Night
			6/13/97	7:00-8:00 am	New	Peak
				1:00-2:00 pm	New	Off-Peak
3	Conroe	SH 105@April Sound	6/16/97	8:30-9:30 pm	Old	Night
			6/17/97	7:05-8:05 am	Old	Peak
				10:00-11:00 am	Old	Off-Peak
				4:00-5:00 pm	Old	Peak
				8:30-9:30 pm	Old	Night
			6/18/97	1:15-2:15 pm	New	Off-Peak
				8:30-9:30 pm	New	Night
			6/19/97	7:00-8:00 am	New	Peak
				10:00-11:00 am	New	Off-Peak
				8:30-9:30 pm	New	Night
4	Odessa	Business IH-20 @ CR 1290	6/23/97	2:00-3:00 pm	Old	Off-Peak
				5:00-6:00 pm	Old	Peak
				9:00-10:00 pm	Old	Night
			6/24/97	7:35-8:35 am	New	Peak
				10:00-11:00am	New	Off-Peak
				9:00-10:00 pm	New	Night
			6/25/97	7:00-8:00 am	Old	Peak
				10:00-11:00am	Old	Off-Peak
				9:00-10:00 pm	Old	Night
			6/26/97	7:35-8:35 am	New	Peak
				10:00-11:00am	New	Off-Peak
				9:00-10:00 pm	New	Night

3.4 DATA ANALYSIS

The goal of the data analysis was to compare the two types of loop configurations (old and new) for various traffic conditions. The data analysis was divided into the following three areas: approach speed, driver action, and vehicle location. The speed data collected in the field were used to investigate the approach speeds of vehicles at each site. Driver action and vehicle location at the onset of yellow were derived from the video reduction efforts. Separate analyses were performed for passenger cars and trucks. Below are descriptions of the methodologies used for each study.

3.4.1 Approach Speed

Technicians collected speed data using IRD classifiers that were capable of measuring individual vehicle speeds for each lane. Large samples of speed data measured at a location 183 m (600 ft) prior to the stop bar (at the beginning of the dilemma zone) were used to estimate the mean and 85th percentile approach speeds. The samples of data included speeds during the peak, off-peak, and nighttime conditions. To estimate free-flow speeds approaching the intersection and remove the effects of the signal on traffic speed, all speeds of 72 km/h (45 mph) or less were removed from the sample. Because the speed limits at all study sites were 113 km/h (70 mph), the researchers assumed that all vehicles traveling at 72 km/h (45 mph) or less were either turning at the intersection or stopping for the red light.

3.4.2 Driver Action

As discussed in the *Data Reduction* section, the actions of drivers were recorded for each vehicle caught in the dilemma zone at the onset of yellow (see Table 3-3). After the data were reduced, researchers discovered that very few drivers performed actions 4 (brake before passing through intersection), 5 (swerve to avoid a collision), or 6 (abrupt stop). Therefore, actions 4 through 6 were removed from the database and classified as either a 1 (stop), 2 (run yellow light), or 3 (run red light). This modification resulted in a more robust sample size for the statistical analysis.

Before a statistical analysis was performed, the data reduced from the video for each site were combined to generate daytime and nighttime data sets for both the new loop and old loop configurations. The daytime data set included both peak and off-peak conditions. A total of four data sets were generated for each site (see Table 3-5). In addition, passenger cars were analyzed separately from trucks.

Table 3-5. Data Sets Generated for Each Site.

Data Set	Condition	Loop Configuration
1	Day	Old
2	Day	New
3	Night	Old
4	Night	New

Researchers conducted statistical analyses on the data sets to determine how each loop configuration performed under different circumstances. Separate analyses were performed for day and night conditions. Analyses included comparing the percentage of vehicles in the dilemma zone and the action of drivers in the dilemma zone at the onset of yellow for each loop configuration. All statistical analyses were performed using a 90 percent confidence level.

3.4.3 Vehicle Location

While reducing the data, technicians approximated the location of each vehicle in the dilemma zone at the onset of yellow. This information was used to compare the locations of vehicles at the onset of yellow for the old and new loop configurations. In addition, the mean vehicle locations for the various driver actions (i.e., stop, run yellow, and run red) were computed and compared for the two loop configurations.

4.0 RESULTS

4.1 INTRODUCTION

The data analysis included investigating approach speeds to the intersection, driver actions in response to a yellow indication, and vehicle location at the onset of yellow. Following are discussions of the results from the data analysis. For the results on driver action and vehicle location, separate discussions are provided for passenger cars and trucks. A discussion is also provided on the crash analysis.

4.2 APPROACH SPEED

The project team collected speed data for all sites except Site 2, with the exception being due to equipment problems. The equipment used to measure speeds in the field was capable of distinguishing between passenger cars and trucks. Table 4-1 shows the calculated mean and 85th percentile speeds for both passenger cars and trucks at each site at a distance of 183 m (600 ft) from the stop line. Appendix C contains frequency plots and cumulative frequency plots of approach speeds for all vehicles at Sites 1, 3, and 4.

Table 4-1. Mean and 85th Percentile Speeds for Study Sites.

Site	Passenger Cars		Trucks	
	Mean Speed, km/h (mph)	85th %-ile Speed, km/h (mph)	Mean Speed, km/h (mph)	85th %-ile Speed, km/h (mph)
1	90.4 (56.2)	103 (64)	88.5 (55.0)	98 (61)
3	92.2 (57.3)	105 (65)	91.0 (56.5)	101 (63)
4	92.5 (57.5)	105 (65)	87.8 (54.6)	100 (62)

As shown in Table 4-1, the mean and 85th percentile speeds for passenger cars and trucks were similar for all sites. Although the speed limit at each site was 113 km/h (70 mph), the 85th percentile speeds for all sites were around 105 km/h (65 mph). It is also worth noting that although the new detector configuration for a 113 km/h (70 mph) approach speed was designed to allow vehicles traveling faster than 97 km/h (60 mph) to exit the dilemma zone before the onset of yellow, the mean speeds at each site were below 97 km/h (60 mph).

Cumulative frequency plots of speed (see Appendix C) were used to determine the percentage of vehicles at each site traveling faster than 97 km/h (60 mph). Results revealed that the percentages of vehicles traveling faster than 97 km/h (60 mph) ranged from 27 to 35 percent.

Therefore, 65 to 73 percent of vehicles were traveling at speeds slower than 97 km/h (60 mph), possibly resulting in being caught in the dilemma zone. These percentages could be reduced by using a loop configuration designed for a lower approach speed (for example, the 85th percentile speed).

4.3 DRIVER ACTION

4.3.1 Passenger Cars

As discussed in the *Data Analysis* section, data reduced from the video for each site were combined to generate data sets for day and night conditions. Data reduced for day conditions included both peak and off-peak periods, and each data set for day and night conditions utilized two to four hours of data. Table 4-2 shows a summary of driver action data.

The first statistical analysis performed on these data compared percentages of passenger cars in the dilemma zone for old and new loop configurations. This percentage was computed by dividing the total number of passenger cars in the dilemma zone by the total traffic volume. Table 4-3 shows the mean computed percentages of passenger cars in the dilemma zone, reduced in one-hour increments from raw data (see Appendix D). The analysis included running statistical t-tests on the raw data to determine differences between the old and new loop configurations. Separate analyses were performed for both day and night conditions for each site. Table 4-3 includes variances and calculated p-values, with shaded cells indicating data that were significantly different at the 90 percent confidence level (i.e., p-value ≤ 0.10).

As shown in Table 4-3, percentages of passenger cars in the dilemma zone for old and new loop configurations were significantly different for four cases (Cases 3, 5, 6, and 7). In each of these four cases, there was a higher percentage of passenger cars in the dilemma zone with the old loop configuration. Also, for Cases 2 and 4, the percentages of passenger cars in the dilemma zone were higher for the old loop configuration even though differences were not statistically significant at the 90 percent confidence level. Reasons for the data not being significant include high variances and small sample sizes.

Researchers performed another statistical test to determine if there were differences in actions taken by drivers at the onset of yellow for the different loop configurations. The percentage of drivers performing a certain action was computed by dividing the number of drivers performing that action by the total number of passenger cars in the dilemma zone. A Chi-Square test compared results for the old and new loop configurations. Tables 4-4 through 4-6 summarize the results for drivers stopping at the intersection, running the yellow light, and running the red light, respectively. The shaded cells in the tables highlight the data that were significantly different at the 90 percent confidence level (i.e., Chi-Square ≥ 2.706).

Table 4-2. Summary of Driver Action Data for Passenger Cars.

Site	Case	Condition	Loop Configuration	Hours of Data	Total Traffic Volume	Total Number of Drivers Performing Action[a]		
						1	2	3
1	1	Day	Old	3	3479	38	41	8
			New	4	4815	75	59	7
	2	Night	Old	2	683	19	13	0
			New	2	1326	10	18	1
2	3	Day	Old	4	1271	61	15	6
			New	4	1300	36	16	6
	4	Night	Old	2	397	21	8	3
			New	2	418	13	5	0
3	5	Day	Old	3	1553	47	44	10
			New	3	2198	23	35	2
	6	Night	Old	2	605	17	15	4
			New	2	728	11	14	4
4	7	Day	Old	4	1890	35	45	9
			New	4	1894	15	23	2
	8	Night	Old	2	330	5	7	1
			New	2	375	4	10	1

[a] Driver Action: 1 = stop at intersection; 2 = run yellow light; 3 = run red light.

Table 4-3. Percentage of Passenger Cars in Dilemma Zone.

Site	Case	Condition	Loop Configuration	Percent Passenger Cars in Dilemma Zone (Mean)	Variance	p-value[a]
1	1	Day	Old	2.4	0.279	0.17
			New	**2.9**	0.489	
	2	Night	Old	**5.7**	24.48	0.25
			New	2.2	0.00	
2	3	Day	Old	**6.8**	1.49	0.02
			New	4.4	1.73	
	4	Night	Old	**7.5**	43.96	0.30
			New	4.3	15.56	
3	5	Day	Old	**7.1**	3.69	0.03
			New	3.4	1.40	
	6	Night	Old	**6.0**	1.00	0.08
			New	4.0	0.666	
4	7	Day	Old	**4.7**	0.861	0.01
			New	2.2	0.780	
	8	Night	Old	3.7	10.62	0.47
			New	**3.9**	4.91	

[a] Significant difference if p-value ≤ 0.10.

Table 4-4. Percentage of Passenger Cars in Dilemma Zone Stopping at Intersection.

Site	Case	Condition	Loop Configuration	Percent Passenger Cars Stopping at Intersection (Mean)	Chi-Square[a]
1	1	Day	Old	42	1.948
			New	**50**	
	2	Night	Old	**68**	3.780
			New	36	
2	3	Day	Old	**76**	2.423
			New	62	
	4	Night	Old	63	0.230
			New	**70**	
3	5	Day	Old	**44**	1.030
			New	33	
	6	Night	Old	**49**	0.565
			New	39	
4	7	Day	Old	**42**	0.039
			New	38	
	8	Night	Old	23	0.444
			New	**26**	

[a] Significant difference if Chi-Square ≥ 2.706.

Table 4-5. Percentage of Passenger Cars in Dilemma Zone Running Yellow Light.

Site	Case	Condition	Loop Configuration	Percent Passenger Cars Running Yellow Light (Mean)	Chi-Square[a]
1	1	Day	Old	46	0.610
			New	46	
	2	Night	Old	32	2.799
			New	**60**	
2	3	Day	Old	17	1.702
			New	**27**	
	4	Night	Old	23	0.046
			New	**30**	
3	5	Day	Old	47	3.285
			New	**65**	
	6	Night	Old	40	0.284
			New	**47**	
4	7	Day	Old	48	0.533
			New	**57**	
	8	Night	Old	52	0.480
			New	**69**	

[a] Significant difference if Chi-Square ≥ 2.706.

Table 4-6. Percentage of Passenger Cars in Dilemma Zone Running Red Light.

Site	Case	Condition	Loop Configuration	Percent Passenger Cars Running Red Light (Mean)	Chi-Square[a,b]
1	1	Day	Old	**12**	1.567
			New	5	
	2	Night	Old	0	N/A
			New	**5**	
2	3	Day	Old	7	0.397
			New	**11**	
	4	Night	Old	**14**	N/A
			New	0	
3	5	Day	Old	**9**	2.354
			New	2	
	6	Night	Old	11	0.107
			New	**13**	
4	7	Day	Old	**10**	0.825
			New	5	
	8	Night	Old	**25**	0.011
			New	5	

[a] Significant difference if Chi-Square ≥ 2.706.

[b] N/A - data contained a value of zero.

As shown by Table 4-4, lower percentages of passenger cars stopped for the new loop configuration for five of the eight cases; however, differences were not statistically significant except for Case 2. Table 4-5 indicates that there were higher percentages of passenger cars running the yellow light for the new loop configuration for seven out of the eight cases. These differences were not statistically significant except in two cases (Cases 2 and 5). Table 4-6 shows that the new loop configuration resulted in lower percentages of passenger cars running the red light in five out of eight cases even though none were statistically significant at the 90 percent confidence level. In summary, the statistical analysis indicates little change in driver behavior for most situations when going from the old to the new loop configuration. However, in some cases, the new loop configuration resulted in fewer passenger cars stopping at the intersection and more passenger cars running the yellow light. The new loop configuration resulted in a lower percentage of passenger cars running the red light for the majority of the sites; however, differences were not statistically significant.

To further investigate the number of passenger cars running the red light, researchers also calculated the percentage of passenger cars running the red light based on total traffic volume by dividing the total number of passenger cars running the red light by the total traffic volume. They compared these percentages for the old and new loop configurations using a Chi-Square test. Table 4-7 indicates that the percentages of passenger cars running the red light were lower for the new loop configuration for seven out of the eight cases, but these differences were statistically significant for only two cases (Cases 5 and 7).

During data reduction efforts, technicians also recorded passenger cars that were located upstream of the dilemma zone (i.e, beyond 183 m [600 ft] from the stop line) at the onset of yellow and ran the red light. Table 4-8 shows the total number and percentage of passenger cars upstream of the dilemma zone that ran the red light. The percentage of passenger cars upstream of the dilemma zone running the red light was calculated by dividing the total number of passenger cars upstream of the dilemma zone running the red light by the total traffic volume. A *t*-test showed that there was no significant difference between the percentage of passenger cars running the red light for day versus night conditions (see Table 4-9). As Table 4-10 indicates, loop configuration had a significant effect on the number of passenger cars running a red light for Site 4 with the new loop configuration being better. However, because passenger cars running the red light were located upstream of the detection zone at the onset of yellow, detector configuration should have had no effect.

Table 4-7. Percentage of Total Passenger Cars Running Red Light.

Site	Case	Condition	Loop Configuration	Percent Passenger Cars Running Red Light (Mean)	Chi-Square[a, b]
1	1	Day	Old	**0.25**	0.800
			New	0.13	
	2	Night	Old	0.00	N/A
			New	**0.09**	
2	3	Day	Old	**0.48**	0.002
			New	0.45	
	4	Night	Old	**0.74**	N/A
			New	0.00	
3	5	Day	Old	**0.66**	8.724
			New	0.06	
	6	Night	Old	**0.66**	0.069
			New	0.56	
4	7	Day	Old	**0.47**	4.482
			New	0.11	
	8	Night	Old	**0.34**	0.008
			New	0.25	

[a] Significant difference if Chi-Square ≥ 2.706.

[b] N/A - data contained a value of zero.

Table 4-8. Passenger Cars Located Upstream of Dilemma Zone Running Red Light.

Site	Case	Condition	Loop Configuration	Number of Passenger Cars Running Red Light	Percent Passenger Cars Running Red Light
1	1	Day	Old	3	0.06
			New	7	0.20
	2	Night	Old	1	0.08
			New	4	0.59
2	3	Day	Old	1	0.08
			New	0	0.00
	4	Night	Old	0	0.00
			New	0	0.00
3	5	Day	Old	1	0.05
			New	2	0.13
	6	Night	Old	0	0.00
			New	0	0.00
4	7	Day	Old	2	0.11
			New	12	0.63
	8	Night	Old	0	0.00
			New	2	0.60

[a] Significant difference if p-value ≤ 0.10.

Table 4-9. Percentage of Passenger Cars Upstream of Dilemma Zone Running Red Light for Day/Night Conditions.

Site	Day		Night		p-value[a]
	Mean	Variance	Mean	Variance	
1	0.13	0.025	0.29	0.127	0.22
2	0.04	0.015	0.00	0.000	0.18
3	0.10	0.037	0.00	0.000	0.12
4	0.37	0.116	0.25	0.245	0.34

[a] Significant difference if p-value ≤ 0.10.

Table 4-10. Percentage of Passenger Cars Upstream of Dilemma Zone Running Red Light for Old/New Loop Configurations.

Site	Old Loop Configuration		New Loop Configuration		p-value[a]
	Mean	Variance	Mean	Variance	
1	0.22	0.090	0.15	0.034	0.33
2	0.00	0.000	0.06	0.020	0.18
3	0.10	0.052	0.03	0.005	0.27
4	0.58	0.145	0.07	0.012	0.01

[a] Significant difference if p-value ≤ 0.10.

4.3.2 Trucks

Truck volumes at Sites 2, 3, and 4 were relatively low, so the analysis of trucks included only Site 1. For Site 1, truck percentages ranged from five to ten percent. The analysis used 16 hours of data for the old configuration and 17 hours for the new configuration as indicated in Table 4-11. The analysis then used a statistical test to determine if there were differences in actions taken by drivers of trucks at the onset of yellow for the different loop configurations. The percentage of drivers performing a particular action was computed by dividing the total number of truck drivers performing that action by the total number of trucks in the dilemma zone.

Table 4-12 shows results of a Chi-Square test comparing results for the old and new loop configurations. The differences between old and new loop configurations for each driver action were statistically significant for each case (although one has to question the practical significance

Table 4-11. Summary of Driver Action Data for Trucks.

Loop Configuration	Hours of Data	Total Number of Drivers Performing Action[a]		
		1	2	3
Old	16	75	210	16
New	17	96	190	14

[a] Driver Action: 1 = stop at intersection; 2 = run yellow light; 3 = run red light.

Table 4-12. Percentage of Truck Drivers Performing Certain Actions in Dilemma Zone.

Action[a]	Loop Config	Mean	Chi-Square[b]
1	Old	25	3.703
	New	**32**	
2	Old	**71**	18.69
	New	63	
3	Old	**5**	4.779
	New	4	

[a] Driver Action: 1 = stop at intersection; 2 = run yellow light; 3 = run red light.

[b] Significant difference if Chi-Square ≥ 2.706.

of four percent versus five percent for Action 3.) These findings indicate that the new loop configuration resulted in more trucks stopping at the intersection, fewer trucks running the yellow light, and fewer trucks running the red light.

4.4 VEHICLE DISTANCES FROM STOP LINE

4.4.1 Passenger Cars

While tracking passenger cars through the dilemma zone to determine driver action, technicians also estimated the distance from the stop line of each passenger car in the dilemma zone at the onset of yellow. Researchers used these data to develop a graphical comparison between old and new loop configurations (see Appendix E). The majority of passenger cars at each site using the new loop configuration were located between 91 and 152 m (300 and 500 ft) from the intersection at the onset of yellow. This finding confirmed expectations because the new configuration used loops at 107, 145, and 183 m (350, 475, and 600 ft) from the intersection. With a green extension of 1.2 seconds, this configuration will allow passenger cars traveling faster than 97 km/h (60 mph) to pass through the dilemma zone before the onset of yellow. As discussed earlier in the section on *Approach Speeds*, many passenger cars were traveling less than 97 km/h (60 mph). Therefore, a passenger car detected at 183 m (600 ft) and traveling less than 97 km/h (60 mph) would travel to a point near 151 m (500 ft) from the intersection before the onset of yellow. Similarly, a passenger car detected at 145 m (475 ft) would travel to a point near 115 m (375 ft) before the onset of yellow. For these reasons, most passenger cars were located between 91 and 152 m (300 and 500 ft) before the onset of yellow for the new loop configuration.

Over 90 percent of the passenger cars were less than 152 m (500 ft) from the intersection at the onset of yellow when the new loop configuration was used. For the old loop configuration, however, higher percentages of passenger cars were located farther away from the intersection. This was also expected because the old loops were located 24, 43, 67, and 98 m (80, 140, 220, and 320 ft) from the intersection; therefore, passenger cars were not detected as far upstream with the old loop configuration.

For this study, the dilemma zone was delineated by a length that begins at a point on the approach where the probability of stopping is 90 percent and extends to a point where the probability is 10 percent. For 113 km/h (70 mph) approach speeds, the dilemma zone is expected to be between 76 and 183 m (250 and 600 ft) from the intersection. To study the effects of passenger car location on driver action, researchers matched passenger car locations with driver actions. This would determine the locations where drivers stopped at the onset of yellow and locations where drivers passed through the intersection. To accomplish this goal, driver actions were divided into the following two categories: *stop* (vehicles that stop at the intersection) and *run* (vehicles that either run the yellow light or run the red light).

Figure 4-1 shows the percentages of *stop* and *run* vehicles based on distances from the stop line. The *stop* and *run* trend lines intersect at approximately 118 m (390 ft). Below 118 m (390 ft), the majority of vehicles pass through the intersection at the onset of yellow, and above 118 m (390 ft), the majority of vehicles stop at the intersection. The figure also shows that 90 percent of all vehicles stop at the onset of yellow at a location approximately 175 m (575 ft) from the intersection. In addition, 90 percent of all vehicles pass through the intersection at the onset of yellow at a location approximately 80 m (260 ft) from the intersection. These distances correspond closely with the assumed locations of 183 m and 75 m (600 ft and 250 ft) used to establish the boundaries of the dilemma zone.

4.4.2 Trucks

As with passenger car data, technicians estimated the location of each truck in the dilemma zone at the onset of yellow to compare the location of trucks for the old and new loop configurations. Figure 4-2 shows a graphical representation of the results from Site 1. For the new loop configuration, the percentage of trucks at a particular location at the onset of yellow

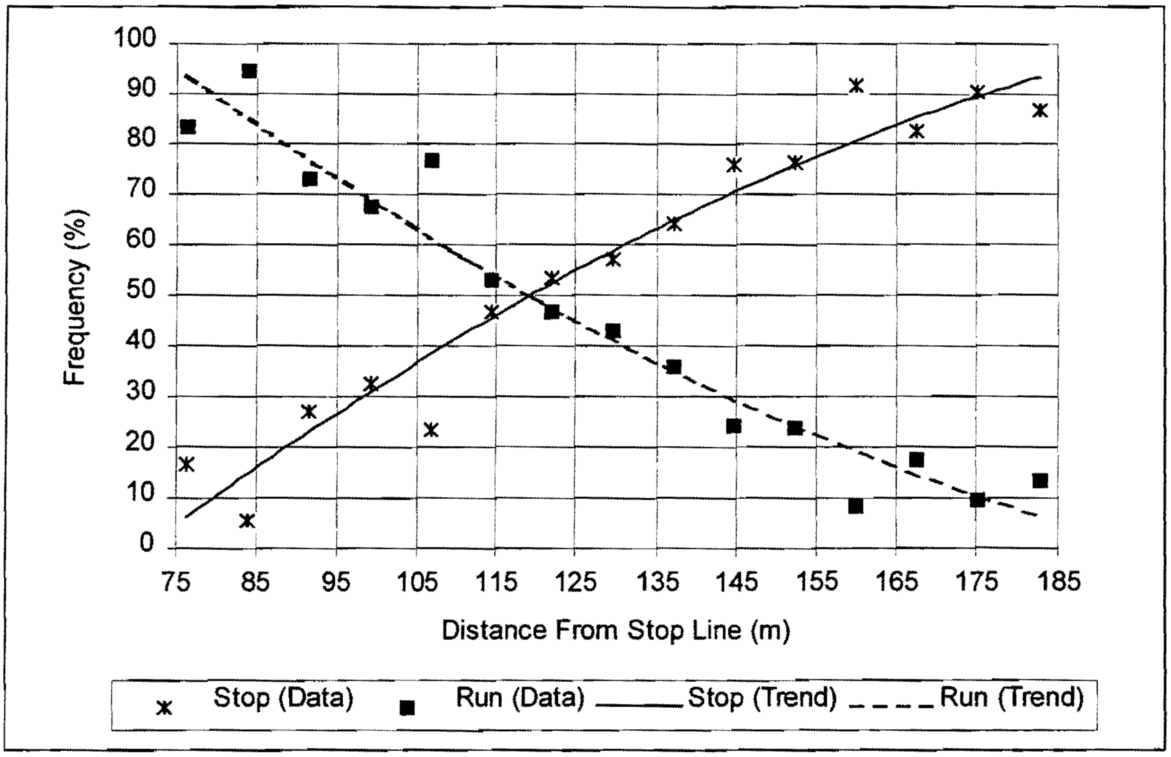

Figure 4-1. Frequency of Passenger Car Distances Based on Driver Actions.

increased as the distance from the intersection decreased. Approximately 40 percent of the trucks were located less than 91 m (300 ft) from the intersection at the onset of yellow, and almost 70 percent of the trucks were less than 122 m (400 ft) from the intersection. For the old loop configuration, a higher percentage of trucks was farther away from the intersection at the onset of yellow (i.e., approximately 40 percent of the passenger cars were located at a distance greater than 122 m [400 ft] from the intersection for the old loop configuration, compared to approximately 30 percent for the new loop configuration).

Figure 4-3 shows the percentages of trucks that stopped or passed through the intersection during yellow based on the distance from the stop line. The *stop* and *run* trend lines intersect at approximately 128 m (420 ft). This indicates that below 128 m (420 ft), the majority of trucks pass through the intersection at the onset of yellow, and above this value, the majority of vehicles stop at the intersection. The figure also shows that 90 percent of trucks stop at the onset of yellow at a location approximately 170 m (560 ft) from the intersection. Ninety percent of trucks pass through the intersection at the onset of yellow at a location approximately 75 m (250 ft) from the intersection. These distances are similar to passenger car results, even though it was expected that a higher percentage of trucks than cars would run the light instead of stop.

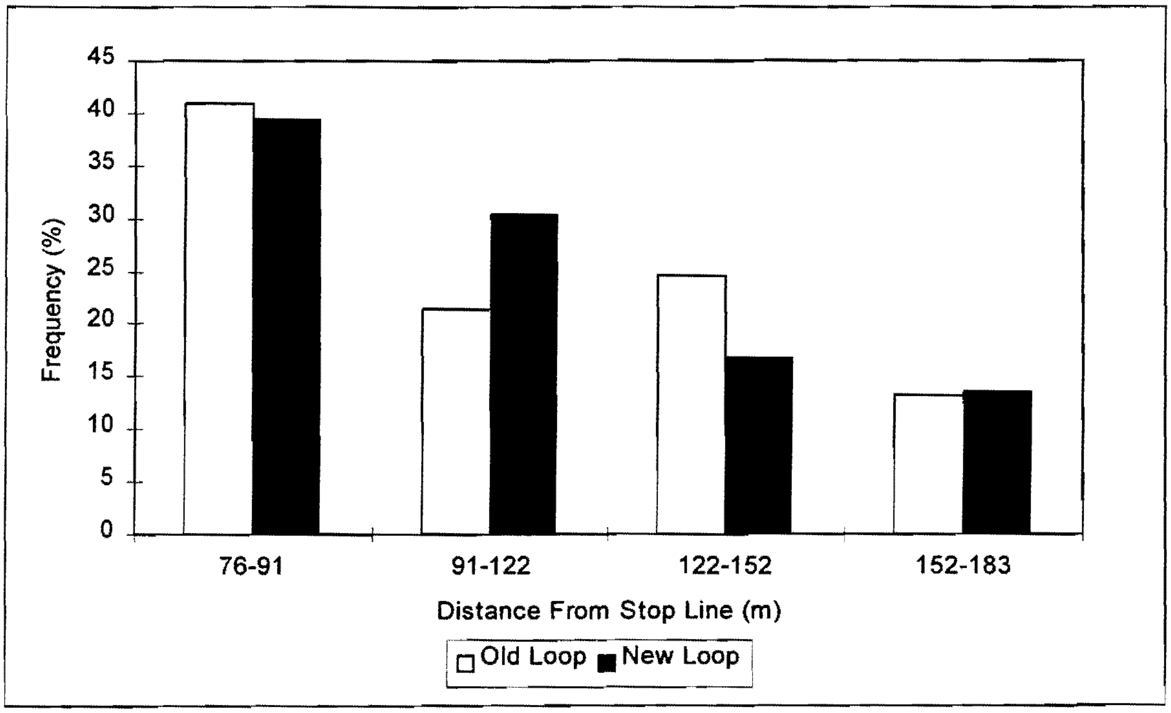

Figure 4-2. Percentage of Trucks at Selected Distances from Stop Line.

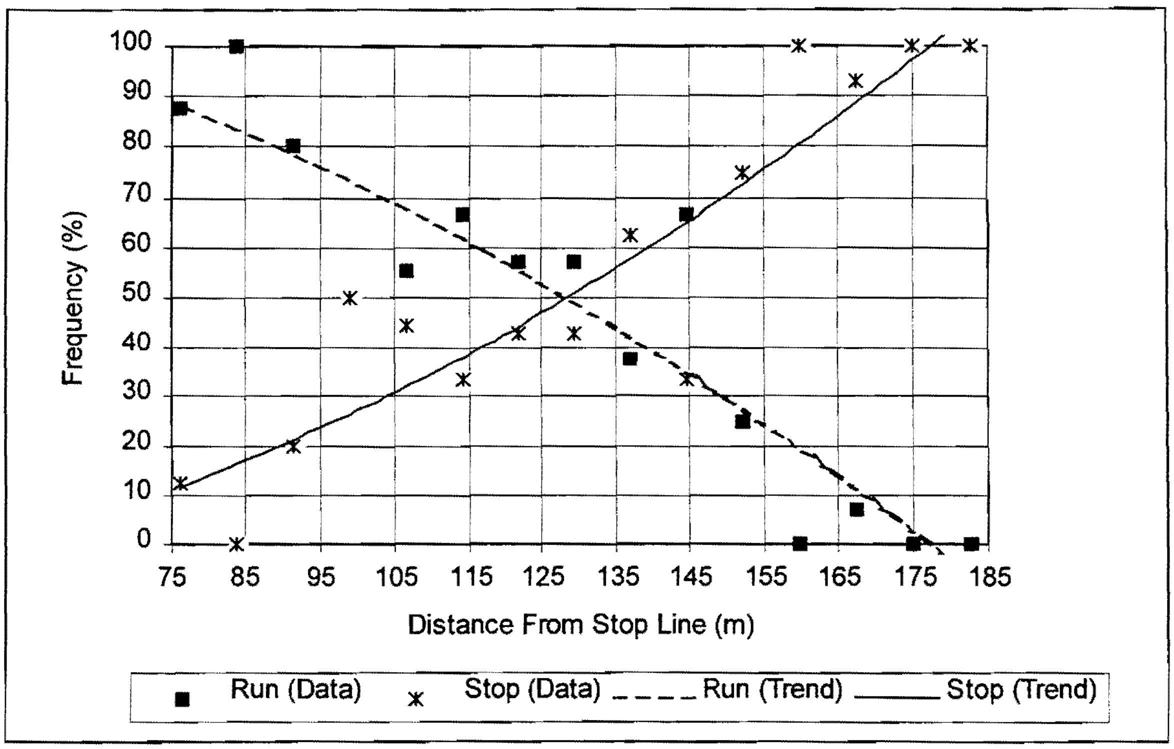

Figure 4-3. Frequency of Truck Distances Based on Driver Actions.

4.5 CRASH ANALYSIS

The objective of the crash analysis was to compare crash data before and after the new loop configuration had been installed. To obtain crash data, the researchers interviewed DPS personnel and used TxDOT's accident database, LANSER. Because the new loop configuration was only temporarily installed as part of this project at Sites 1, 2, and 3, a before/after crash study could not be performed for these sites.

The researchers attempted to obtain before and after crash data for Site 4 (in Odessa at the intersection of Business IH 20 and County Road 1290). The new loop configuration at this site had been installed in May 1997; therefore, the researchers were hopeful that before and after crash data would be available. After an attempt was made to retrieve the crash data for this intersection, however, it was discovered that the most current crash data were only available through February 1997.

In a further attempt to collect crash data at an existing field site, an additional site was chosen that was not part of the field study. This site was located in Brownwood at the intersection of US 377 and Crockett. The speed limit at this intersection was 80 km/h (50 mph). After an investigation of this site, however, researchers discovered that the new loop configuration was part of a new signal installation. Therefore, no before data were available at this site.

4.6 LOOP CONFIGURATION EVALUATION

One indicator of loop configuration performance is the maximum allowable headway (MAH). The MAH represents the maximum time headway that can occur between successive vehicle actuations before the phase in service gaps out. There is no set MAH that is best for all loop configurations. In general, shorter MAH's reduce the frequency of max-out and delay to waiting traffic; however, MAH's that are too short result in premature gap-outs. Bonneson and McCoy state that MAH's that are found to be effective range from 3 to 6 seconds*(18)*.

The *Manual of Traffic Detector Design (18)* provides a procedure for determining the MAH for a particular loop configuration and signal timing. It also provides methods for estimating the probability for max-out and average delay to vehicles on the cross street. The procedures in this manual were used to evaluate the old and new loop configurations for a 113 km/h (70 mph) approach speed.

To estimate the MAH for an intersection approach with advanced loops and with the stop detection zone inactive during the green phase, the manual provides the following equation:

$$MAH = PT + CE + \frac{D_1 - D_n + L_d + L_v}{V_a}$$

where,

PT = passage time setting, sec;
CE = call-extension setting for the stop line detector, sec;
D_1 = distance to leading edge of the advance detector furthest from the stop line, ft;
D_n = distance to the leading edge of the advance detector nearest to the stop line, ft;
L_d = length of an advance loop detector, ft;
L_v = detected length of vehicle, ft.

The steps presented in the manual for estimating the max-out probability and average delay to vehicles on the cross street approach are summarized in Appendix F.

To evaluate the two loop configurations, information about controller settings were needed. Most of this information was acquired by using typical values found at the field sites. Other information was assumed. The following conditions and values were used for the evaluation:

- Two lanes on the main street approach;
- Two-phase, fully actuated operations;
- NEMA controller;
- Average approach speed of 92 km/h (57 mph);
- Minimum green of 20 seconds;
- Maximum green of 60 seconds;
- Combined yellow and all-red of 5 seconds;
- Saturation flow rate of 1800 vphgpl;
- Green extension of 1.2 seconds for the new loop configuration;
- Green extension of 1.0 seconds for the old loop configuration.

The calculated MAH's for the old loop and new loop configurations were 5.6 seconds and 6.0 seconds, respectively. These fall in the range of 3 to 6 seconds found to be effective by Bonneson and McCoy (*18*). The estimated max-out probability and average delay to vehicles for a range of flow rates are shown in Figures 4-4 and 4-5.

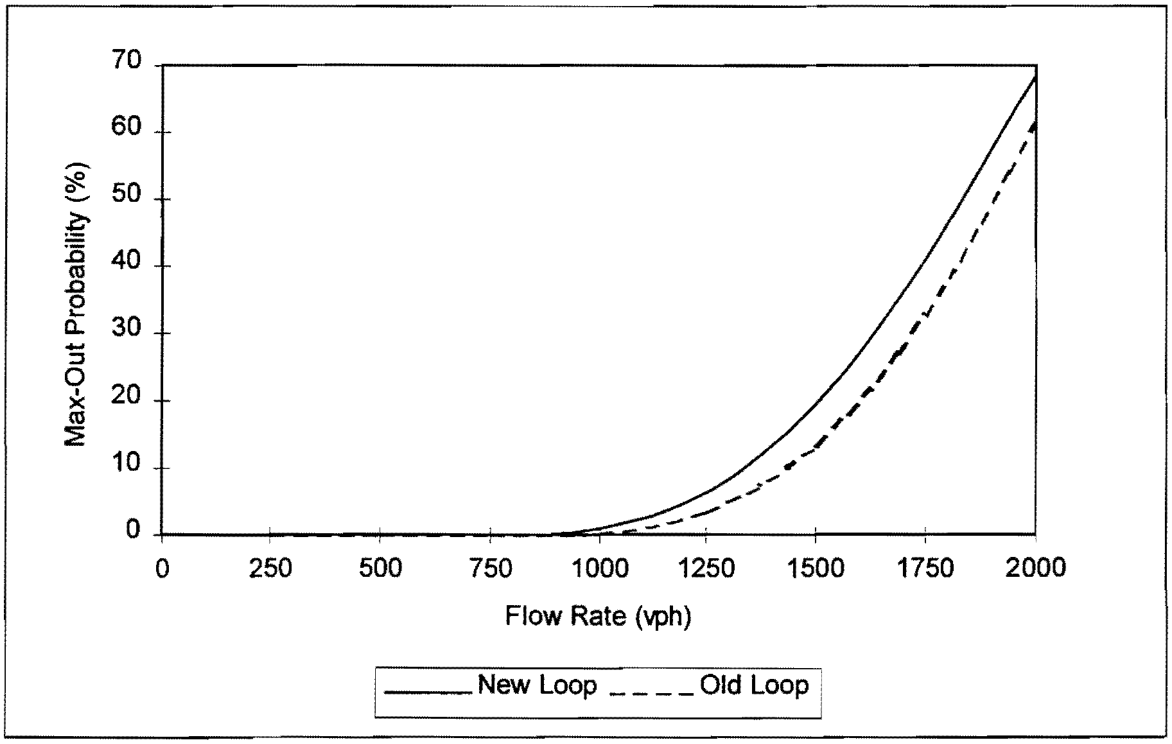

Figure 4-4. Probability of Max-Out.

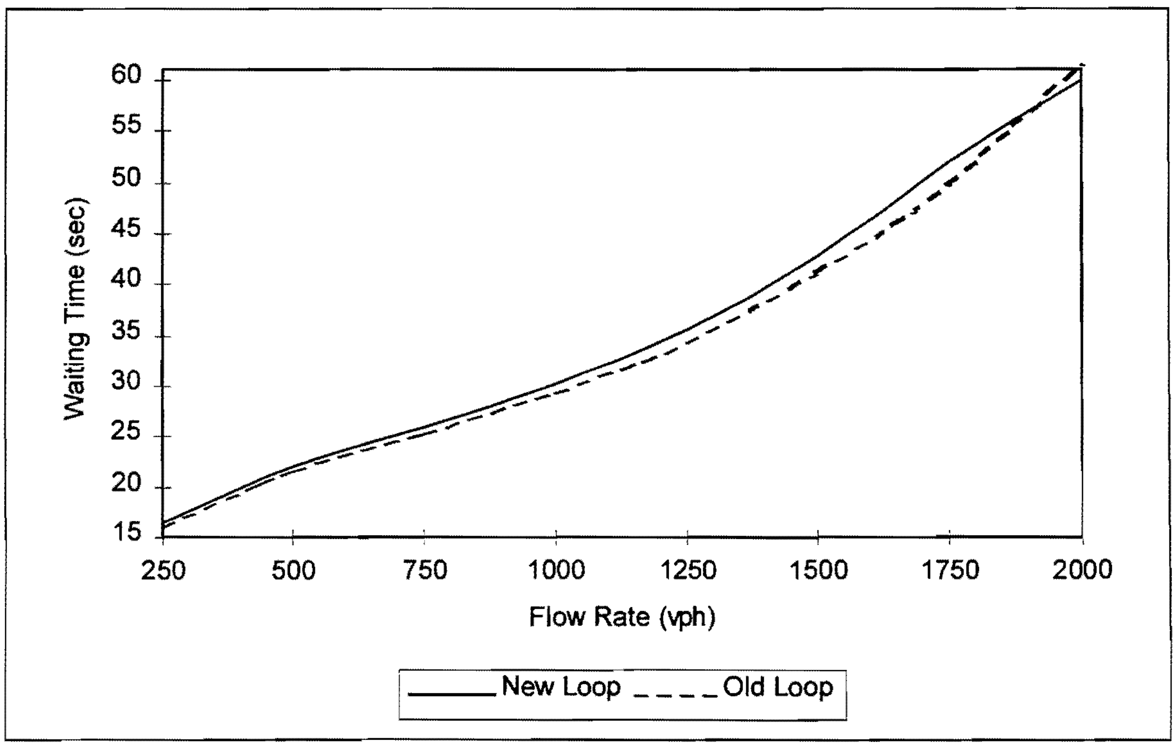

Figure 4-5. Average Delay to Cross Street Traffic.

As shown in Figure 4-4, the max-out probability for both loop configurations was zero for flow rates below 1000 vph. For flow rates above 1000 vph, the max-out probability begins to significantly increase for both loop configurations. Even though the new loop configuration results in a higher probability of max-out when compared to the old loop configuration, the difference is relatively small, even at higher traffic volumes.

Figure 4-5 illustrates that the average delay to cross street traffic increases in an almost linear fashion with increasing traffic flow. This figure also shows that the cross street delay is similar for both the old loop and new loop configurations. This similarity was expected because the calculations showed that the average green time on the study approach for both loop configurations were very similar. Results from the field sites also showed that the average green time did not change significantly for different loop configurations. Because the average green time on the study approach did not change significantly, the delay to cross street traffic remained relatively constant.

5.0 FINDINGS AND CONCLUSIONS

5.1 INTRODUCTION

The primary objective of this research was to validate the new TxDOT procedures for loop detector placement on high-speed approaches to signalized intersections. The goal of the new procedures is to increase the safety at high-speed approach intersections above that of existing procedures. The study approach involved conducting a before/after study at selected field sites to compare the proposed new loop configuration to the existing configuration.

5.2 SUMMARY OF FINDINGS

Four field sites located in the state of Texas were selected for this study. Each of the field sites had posted speed limits of 113 km/h (70 mph) with either two or three through lanes. The analysis included investigating approach speeds to the intersection, driver actions in response to a yellow indication, and vehicle location at the onset of yellow. In addition, the two loop configurations were evaluated by calculating the max-out probability and delay to cross street traffic for varying approach volumes. Following is a summary of the findings in this study.

5.2.1 Approach Speed

- The 85th percentile speeds for the field sites were below the 113 km/h (70 mph) posted speeds, ranging from 103 to 105 km/h (64 to 65 mph).
- Although the new detector configuration for a 113 km/h (70 mph) approach speed was designed to allow vehicles traveling faster than 97 km/h (60 mph) to exit the dilemma zone before the onset of yellow, 65 to 73 percent of vehicles at the study sites were traveling at speeds slower than 97 km/h (60 mph), possibly resulting in being caught in the dilemma zone.

5.2.2 Driver Action

5.2.2.1 Passenger Cars

- The old loop configuration typically resulted in a higher percentage of passenger cars in the dilemma zone when compared to the new loop configuration.
- In a majority of the cases, the new loop configuration resulted in fewer passenger cars stopping at the intersection, more passenger cars running the yellow light, and fewer passenger cars running the red light when compared to the old loop configuration.
- For both old and new loop configurations, a small percentage of drivers (less than one percent of the total volume) that were located upstream of the dilemma zone (greater than 183 m [600 ft] from the intersection) at the onset of yellow ran the red light.

5.2.2.2 Trucks

- The new loop configuration resulted in more trucks stopping at the intersection, fewer trucks running the yellow light, and fewer trucks running the red light when compared to the old loop configuration.

5.2.3 Vehicle Distances From Stop Line

5.2.3.1 Passenger Cars

- The majority of passenger cars for the new loop configuration were located between 91 and 152 m (300 and 500 ft) from the stop line at the onset of yellow.
- For the old loop configuration, a higher percentage of passenger cars in the dilemma zone were not detected, resulting in a higher percentage of passenger cars being located further from the intersection at the onset of yellow when compared to the new loop configuration.
- For distances less than 118 m (390 ft) from the stop line, the majority of vehicles passed through the intersection at the onset of yellow; above this distance, the majority of vehicles stopped at the intersection.
- Ninety percent of all vehicles in the dilemma zone stopped at the intersection when presented with a yellow indication at a location approximately 175 m (575 ft) from the stop line. In addition, 90 percent of all vehicles pass through the intersection at the onset of yellow at a location approximately 80 m (260 ft) from the stop line.

5.2.3.2 Trucks

- Similar to the results for passenger cars, a higher percentage of trucks were located farther upstream of the stop line at the onset of yellow for the old loop configuration when compared to the new loop configuration.
- For distances less than 128 m (420 ft) from the stop line, the majority of vehicles passed through the intersection at the onset of yellow; above this distance, the majority of vehicles stopped at the intersection.
- Ninety percent of all vehicles in the dilemma zone stopped at the intersection when presented with a yellow indication at a location approximately 170 m (560 ft) from the stopline. In addition, 90 percent of all vehicles pass through the intersection at the onset of yellow at a location approximately 75 m (250 ft) from the stop line.

5.2.4 Loop Configuration Evaluation

- The max-out probability for both old and new loop configurations was zero for flow rates below 1000 vph. For flow rates above 1000 vph, the max-out probability began to significantly increase for both loop configurations.
- The difference in max-out probability between the old and new loop configurations was relatively small.

- The average delay to cross street traffic was similar for both the old and new loop configurations

5.3 CONCLUSIONS

The results from the field study revealed that the new loop configuration is as good as, and in some cases better than, the old loop configuration. Because the new loop configuration can detect vehicles farther upstream from the intersection (at the beginning of the dilemma zone), it results in fewer vehicles being caught in the dilemma zone at the onset of yellow. Also, because the new loop configuration typically resulted in more vehicles running the yellow light instead of stopping, fewer rear-end crashes may result. In addition, the new loop configuration resulted in fewer vehicles running the red light, also a major cause of crashes.

6.0 IMPLEMENTATION

6.1 INTRODUCTION

TxDOT's new procedures for detector placement address all high-speed approaches from 70 km/h (45 mph) to 110 km/h (70 mph). The proposed detector spacing and green extension values for these speeds are as shown in Figure 6-1. The results of this research indicate that the new TxDOT detector placement performs as expected in detection of vehicles at much greater distances from the intersection. This provides more distance (or time) to make the appropriate decision with the onset of yellow. The new detector placement plan has already been implemented in a few districts where 110 km/h (70 mph) approaches exist. With the successful outcome of this project, this detector scheme should be implemented elsewhere at intersections which are otherwise safe for these speeds. Based on the findings from this study, it is recommended that the detector layout be based on the 85th percentile approach speed to the intersection (as opposed to the posted speed limit). Deliverables for this project include a standard sheet for implementation of the new detector layout on future construction projects.

6.2 FUTURE RESEARCH

Even though the new TxDOT procedures provide detection substantially farther away from the stop line than the old procedures, there are still uncertainties regarding effectiveness of vehicle detection systems at high-speed signalized intersections. Two of the areas that need further research include detection by lane and by vehicle type. In many cases, for example, ILDs on multilane approaches do not distinguish detections in lane one from lane two. Vehicle type is important from the standpoint of different operating characteristics between cars and other smaller vehicles, and trucks. An enhanced system that has the capability of detecting vehicle types and speeds could utilize inductive loops and the new series of Advanced Traffic Controllers that are already becoming available.

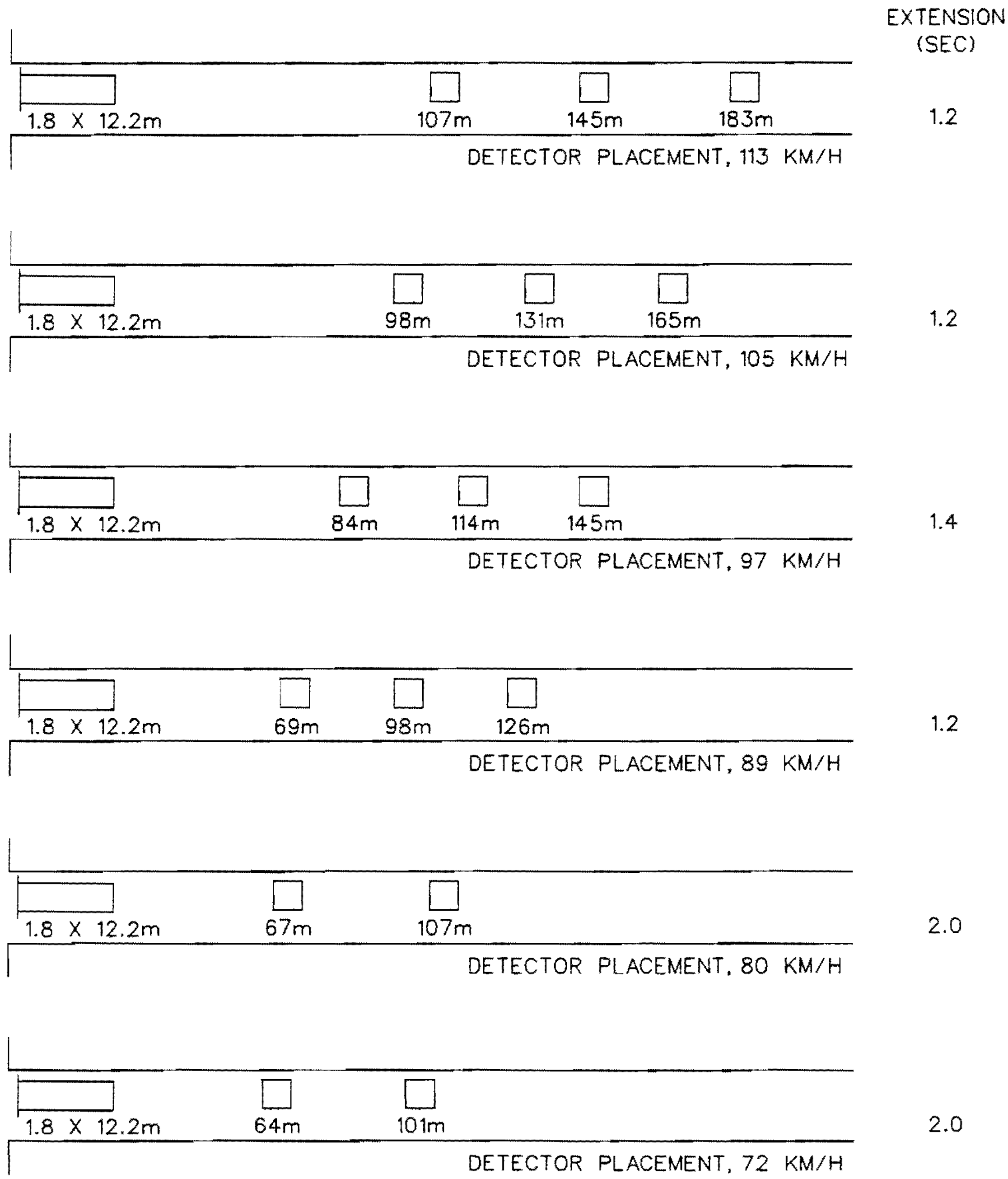

Figure 6-1. Proposed Detector Placement

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APPENDIX A

SURVEY OF EXISTING PRACTICES

Project 3977
TxDOT Detector Placement for High-Speed Approaches to Signalized Intersections

1a. Does ________________ have an established procedure for placement of loop detectors for high-speed approaches at actuated signalized intersections? [High speed is over 90 km/h (55 mph)] YES NO

1b. If yes, describe the type of procedure ____
____
____

2a. Is the procedure still being implemented? YES NO

2b. Where or to what extent has the procedure(s) been implemented? ________________
____

3. Was the procedure implemented due to the federal deregulation of maximum speed limits? YES NO

4a. What is the speed limit at the site(s) that the procedure has been implemented? _____

4b. Have any speed studies been conducted for the site(s)? YES NO

5a. What is the detector layout (if any) for the site(s)? ____________________________
____

5b. What are the controller settings for the site(s)? ______________________________
____

6. What type of advanced warning signs are used for the approaches to the intersection?
____

7a. Are there any data or reports available regarding the effectiveness, success, and/or problems involved with implementing the procedure? YES NO

7b. How may we obtain the data or reports? ______________________________________

8. What are the name, address, and telephone number of the most appropriate person to contact regarding the operational effectiveness of the procedure?
Name ______________________________ Position ________________
Address ____
Phone ________________ Fax ______________ E-mail ________________

APPENDIX B

EXISTING GUIDELINES ON LOOP PLACEMENT

CALIFORNIA

The California Department of Transportation has procedures for the placement of detectors at signalized intersections. The procedures involve placing one set of detectors a certain distance upstream of the intersection. Recommended placements are based on deceleration rates for dry conditions to allow a vehicle to come to a safe stop. Recommended placements are shown in the following table.

Speed, km/h (mph)	Deceleration Time (sec)	Deceleration Distance, m (ft)	Total Time (sec)	Total Distance, m (ft)	Recommended Placement, m (ft)	Yellow Interval (sec)
40 (25)	3.70	20.53 (67.4)	4.70	31.63 (103.8)	30 (98)	3.1
50 (31)	4.63	32.15 (105.5)	5.63	46.10 (151.2)	45 (148)	3.3
60 (37)	5.56	46.37 (152.1)	6.56	63.18 (207.3)	60 (197)	3.7
70 (43)	6.46	62.59 (205.3)	7.46	82.12 (269.4)	80 (262)	4.2
80 (50)	7.40	82.15 (269.5)	8.40	104.34 (342.3)	105 (345)	4.7
90 (56)	8.33	107.08 (351.3)	9.33	129.16 (423.8)	130 (427)	5.1
100 (62)	9.36	128.62 (422.0)	10.36	156.60 (513.8)	155 (509)	5.5
110 (68)	10.19	155.61 (510.5)	11.19	186.16 (610.8)	185 (607)	6.0

Deceleration Distance = $\frac{1}{2} dt^2$ or $\frac{1}{2} Vt$ or $V^2/2d$

Deceleration Time = V/d

Detector Setback = Deceleration Distance + Reaction Distance = $V^2/2d + Vr$

V = speed (m/s)

t = deceleration time (s)

d = deceleration rate = 3.6 m/s^2

r = reaction time = 1.00 s

KENTUCKY

Kentucky uses a Green Extension System to place detectors on high-speed approaches to signalized intersections. The following information was taken from Kentucky's traffic design manual.

A Green Extension System (GES) is a unique signal system design that has the ability to detect the presence of a vehicle before it travels into the Dilemma Zone and then ensure that this vehicle will continue to have a green indication as it passes through the intersection...

GES Systems should normally be considered only at isolated intersections or where the signal is the first signal in a series of signals where the 85th percentile speed is [72 km/h] 45 mph or greater. Multilane vehicle detector loops shall be placed in advance of the signal on each mainline approach. The initial interval for these approaches should be from 15 to 20 seconds. The maximum green time should normally be set at 90 seconds.

The following table shows signal timing and the distances from the stop bar that loops on the mainline approaches should be installed for various approach grades. The timing shall be placed on the controller and all mainline loop detectors shall be placed in the pulse mode. When the grades on each mainline approach are different, the grade that is the most critical will be used to determine loop distances on both approaches. Using the table, the critical grade will always be the one that shows the farthest loop distance from the stop bar.

Approach Grade (%)	Near Loop Distance From Stop Bar, [m] ft	Far Loop Distance From Stop Bar, [m] ft	Controller Setting Vehicle Interval (sec)
-8	[85] 278	[154] 506	3.5
-7	[81] 266	[147] 482	3.3
-6	[79] 259	[142] 467	3.2
-5	[77] 251	[138] 452	3.1
-4	[74] 244	[134] 439	3.0
-3	[72] 235	[128] 419	2.8
-2	[69] 228	[134] 406	2.7
-1	[68] 222	[120] 394	2.6
0	[66] 217	[117] 384	2.5
1	[64] 210	[112] 369	2.4
2	[62] 205	[110] 360	2.4
3	[61] 201	[107] 352	2.3
4	[60] 197	[105] 344	2.2
5	[59] 193	[102] 336	2.2
6	[58] 189	[100] 328	2.1
7	[56] 185	[98] 320	2.0
8	[55] 182	[96] 314	2.0

EXAMPLE:

Assume an intersection has an approach grade of - 6% and another of - 4%. The chart shows the loop spacing for the - 6% grade to be farther from the stop bar than the - 4% grade; therefore, the - 6% grade is considered the critical grade and the loop spacing used for both approaches will be [79 m] 259 feet for the near loop and [142 m] 467 feet for the far loop. A setting of 3.2 seconds should be set on the controller.

Generally, when a GES is the first traffic signal in a series of traffic signals, the approach coming from the adjacent traffic signals does not require GES capabilities and one mainline loop per lane is sufficient. The loop should be [1.8 m by 1.8 m] 6 ft by 6 ft installed in each through lane in advance of the stop bar at a distance that will allow a vehicle traveling [72 km/h] 45 mph to travel within [15 m]50 feet of the stop bar before receiving a yellow indication. The distance for this loop shall be calculated by using the vehicle interval that has been set on the controller for the opposing GES approach.

MARYLAND

The Maryland Department of Transportation uses a procedure for detector placement based on dilemma zone protection. The procedure calls for two sets of detectors to be placed, one set at the beginning of the dilemma zone and one set at the end of the dilemma zone. The dilemma zone includes the area where 10 percent to 90 percent of all vehicles will stop. The following table shows the recommended detector placements for speeds up to [97 km/h] 60 mph.

Approach Speed [km/h] mph	Dilemma Zone of Detector Setbacks	
	Distance from Intersection for Probability of Stopping of	
	10 %	90 %
[48] 30	[27 m] 90 ft	[53 m] 175 ft
[64] 40	[34] 110	[76] 250
[72] 45	[50] 165	[91] 300
[80] 50	[67] 220	[107] 350
[97] 60	[79] 260	[137] 450

MINNESOTA

The Minnesota Department of Transportation includes procedures for detector placement at signalized intersections based on design requirements (distance from intersection) and operational requirements (length of yellow and all red). The recommendations are given in the form of a dilemma zone chart. A summary of the values in the chart is given in the following table.

Speed, km/h (mph)	Distance From Stop Line, m (ft)	
	Dilemma Zone	Detector Placement
64 (40)	10 - 24 (32 - 78)	24 (78)
72 (45)	15 - 27 (50 - 90)	27 (90)
80 (50)	20 - 32 (67 - 105)	37 (122)
89 (55)	22 - 37 (72 - 122)	44 (145)
97 (60)	24 - 42 (79 - 138)	51 (168)
105 (65)	26 - 47 (85 - 153)	59 (192)

MISSOURI

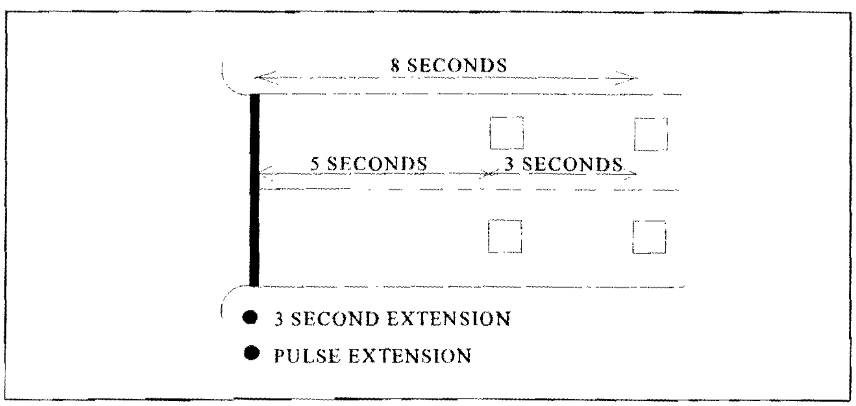
A new method for detector placement has been proposed and is being tested by the Missouri Department of Transportation. Following is an excerpt from the proposed methodology that would be included in *MoDOT Traffic Manual.*

With signals installed on high-speed roadways (85th percentile speed of [72 km/h] 45 mph or more), a single back detector may not be able to be placed in the proper location to keep vehicles from the "dilemma zone" conflict... If a back detector is placed too close to the intersection, it may not detect fast vehicles in time to control them with gap timing. If placed too far back with high gap times, the mainline will be needlessly favored with long green times.

A solution is the placement of two pulse detectors per lane per approach spaced far enough apart to take a high speed vehicle through the intersection. One detector is placed equal to the distance a vehicle takes to travel at the 85th percentile speed in 8 seconds back from the stop bar, and the second detector is placed in a similar fashion 5 seconds back from the stop bar. Minimum gap is set to 3 seconds using gap reduction control.

This allows for a vehicle approaching the intersection at the 85th percentile speed to hit the first detector and extend their call for at least 3 seconds once gap reduction is finished. At the 85th percentile speed, they will hit the next detector 5 seconds away from the stop bar, and extend their call another 3 seconds. After this seconds extension, the vehicle is 2 seconds away from the stop bar. This is close enough to allow it to clear during the yellow and red intervals.

Any vehicles moving faster than the 85th percentile speed will hit the 5-second detector before the gap times out, and vehicles traveling slower will gap out before reaching the 5-second detector. But this allows for a comfortable stopping distance at speeds below the 85th percentile speed. The "dilemma zone" is avoided with this strategy, except when the max green timer reaches zero. The green will be terminated regardless of the location of vehicles.



NEBRASKA

Nebraska uses a procedure for loop placement based on approach speed and stopping sight distance. The procedure calls for multiple loops placed at specific distances from the stop bar. The procedure is based on a 2 second perception/reaction time, a 10 ft/sec^2 deceleration rate, and a green extension of 2 seconds for each loop. The recommended spacings are shown in the following table.

Speed, km/h (mph)	Detector Location, m (ft)		
	First	Second	Third
64 (40)	88 (290)	60 (198)	37 (120)
72 (45)	107 (350)	74 (244)	50 (163)
80 (50)	127 (416)	91 (297)	62 (203)
89 (55)	148 (487)	108 (354)	76 (248)
97 (60)	172 (563)	127 (417)	91 (297)

OHIO

The Ohio Department of Transportation has a loop placement procedure for signalized intersections with approach speeds from 64 to 97 km/h (40 to 60 mph). The procedure calls for two sets of 1.8 m by 1.8 m (6ft by 6 ft) loops to be placed at specific locations from the intersection (based on approach speed). These detectors are used to extend the green time to reduce the potential for high-speed dilemma zone conflicts. Recommended loop placements are shown in the following table.

Speed, km/h (mph)	Detector Placement, m (ft)	
	First	Second
64 (40)	76 (250)	61 (200)
72 (45)	91 (300)	61 (200)
80 (50)	107 (350)	61 (200)
89 (55)	122 (400)	61 (200)
97 (60)	137 (450)	61 (200)

WASHINGTON

Washington uses procedures for loop placement based on dilemma zone protection. The following excerpt was taken from Washington's *Signal Design Guide.*

The concept is to locate advance detection at the beginning of the dilemma zone. The detector upon detecting a vehicle entering the dilemma zone will extend the green time to help the vehicle pass this zone. If the controller gaps out, the following vehicles will still be outside of the dilemma zone and still have adequate braking distance at a comfortable deceleration rate of 8-ft/sec. The last vehicle that extends the green time is already more than 2 seconds into the zone and will have adequate time to pass the intersection.

A driver behavior statistic is used to establish the beginning and end of the dilemma zone. It has been found that more than 90 percent of the drivers are willing to accept a deceleration rate of [2.4 m/sec] 8-ft/sec. The percentage drops down (to 10 percent) as the deceleration rate increases (to [6.1 m/sec] 20-ft/sec). Using the 90 percent and 10 percent stopping rates as reference points, the dilemma zone is defined as the difference between the two decelerating distances for the two extreme deceleration rates. The formula determining the upstream (UDZ) and downstream (DDZ) of the dilemma zone is derived from the basic uniformly accelerate motion equation.

The deceleration distance D for a vehicle traveling at an initial speed V_i and decelerating at a rate "a" to a final speed V_o is:

$$D=\frac{V_i^2-V_o^2}{2a}$$

The UDZ is defined as a necessary distance to bring a vehicle to a stop behind the stop bar at a comfortable deceleration rate of 8-ft/sec which would be accepted by more than 90 percent of the drivers.

$$UDZ=\frac{V_i^2-V_o^2}{2(8\text{-}ft/s^2)}+V_i(t)$$

The DDZ is defined similarly to the UDZ and uses the same equation; however, the deceleration rate is set at [6.1 m/sec] 20-ft/sec which would be accepted by less than 10 percent of the drivers.

$$DDZ=\frac{V_I^2-V_O^2}{S(20\text{-}ft/s^2)}+V_i(t)$$

Note that:

V_o = 0 (final speed at stop), mph

$V_i(t)$ = Distance covered by the vehicle traveling at V_i during the PIEV time t. We use 1 second for this PIEV time for urban areas, and we use 2.5 seconds for areas where a driver would not expect to encounter a signal.

V_i = Initial speed when the driver recognizes the yellow indication and starts decelerating. Some agencies use the 85th percentile as the Vi to establish the dilemma zone. WSDOT uses the 90th percentile speed to set the UDZ and the 10th percentile speed to set the DDZ. Using the 85th percentile speed alone limits the protection to a small range of speed, while using both 90th and 10th percentile speeds will certainly provide protection for a wider range of approach speed.

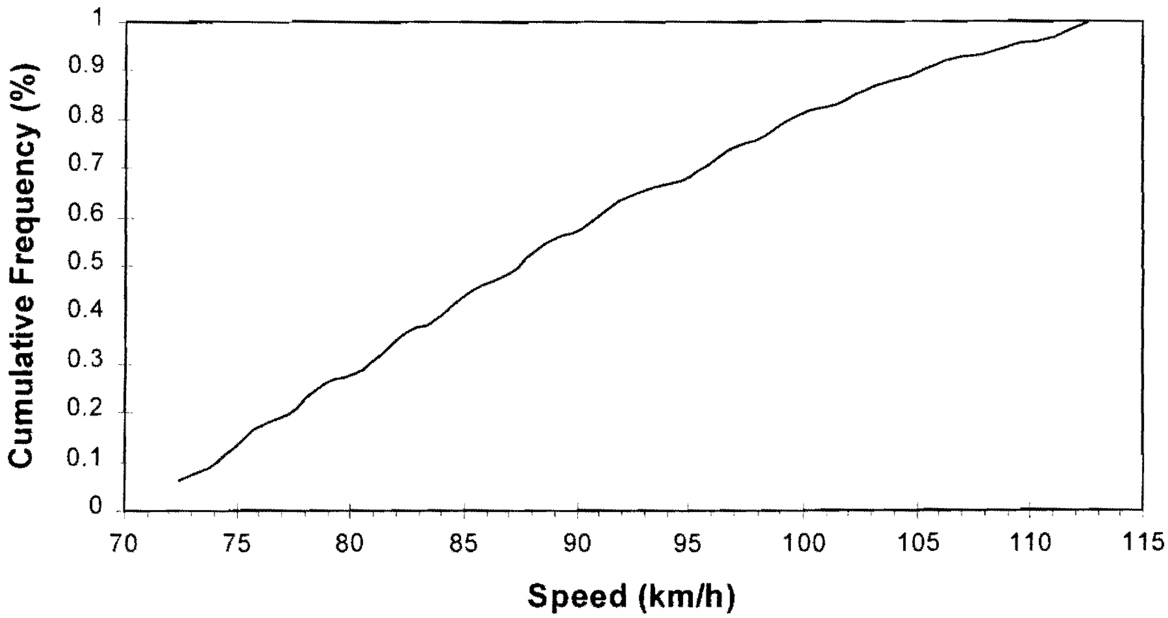
See the attached loop calculation sheet.

PLACEMENT OF ADVANCE LOOPS

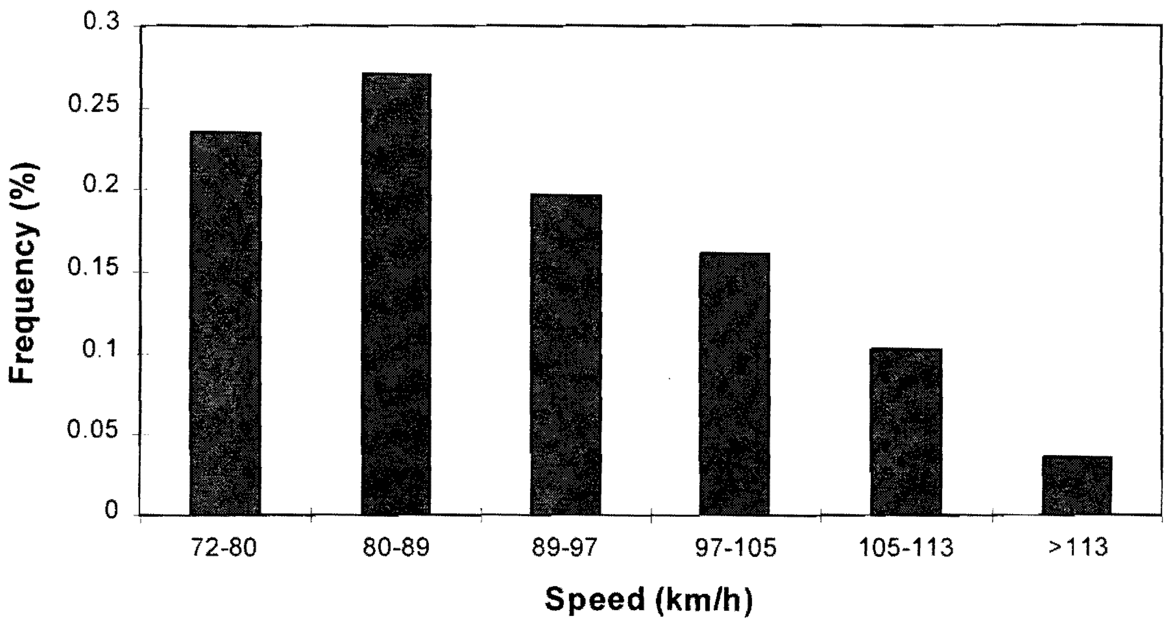
STEP	VARIABLE	EQUATION	VALUE	COMMENT(S)
1	UDZ_{90}	$\frac{(V_{90})^2}{16} + V_{90}$		Loop #1 location: Upstream end of dilemma zone for 90% speed (V_{90}).
2	DDZ_{10}	$\frac{(V_{10})^2}{40} + V_{10}$		Downstream end of dilemma zone for 10% speed (V_{10}).
3	LC1	$\frac{UDZ_{90} - DDZ_1}{V_{10}}$		V_{10} travel time from Loop #1 to downstream $DZ_{10.}$
4	Loop Criteria #1	LCI $\leq$ 3.0 sec ?		Does V_{10} clear in 3.0 sec? If yes, use Loop #1 only. Stop. If no, need 2nd loop. Proceed.
5	P_{MID}	$\frac{UDZ_{90} + DDZ_1}{2}$		Location for Loop #2.
6	LC2	$\frac{UDZ_{90} - P_{MID}}{V_{10}}$		V_{10} travel time from Loop #1 to Loop #2.
7	Loop Criteria #2	LC2 $\leq$ 3.0 sec ?		Does V_{10} clear in 3.0 sec? If yes, set Loop #2 at P_{MID}. If no, discuss with Signal Ops.

APPENDIX C

SPEED DATA

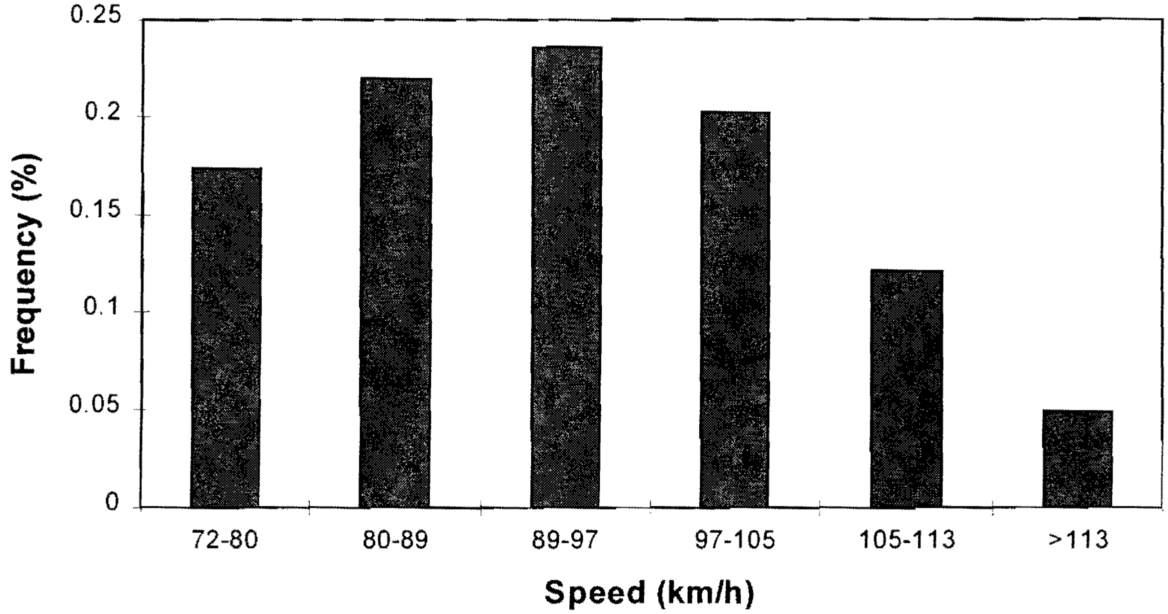


(b) Cumulative Frequency Plot

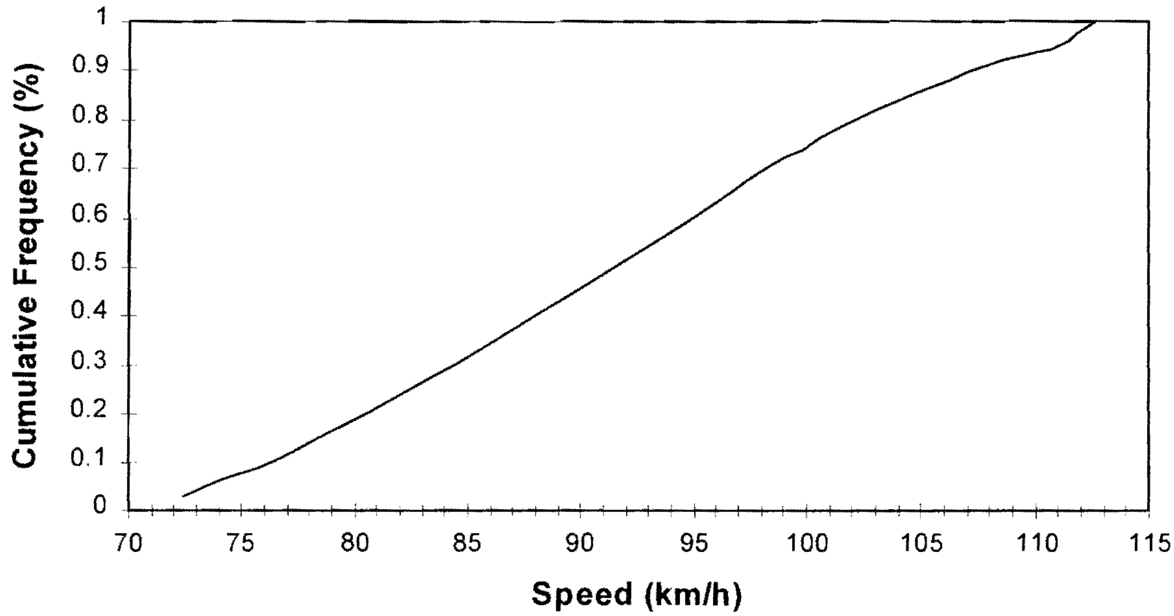


(a) Frequency Plot

Figure C-1. Approach Speeds for All Vehicles at Site 1.

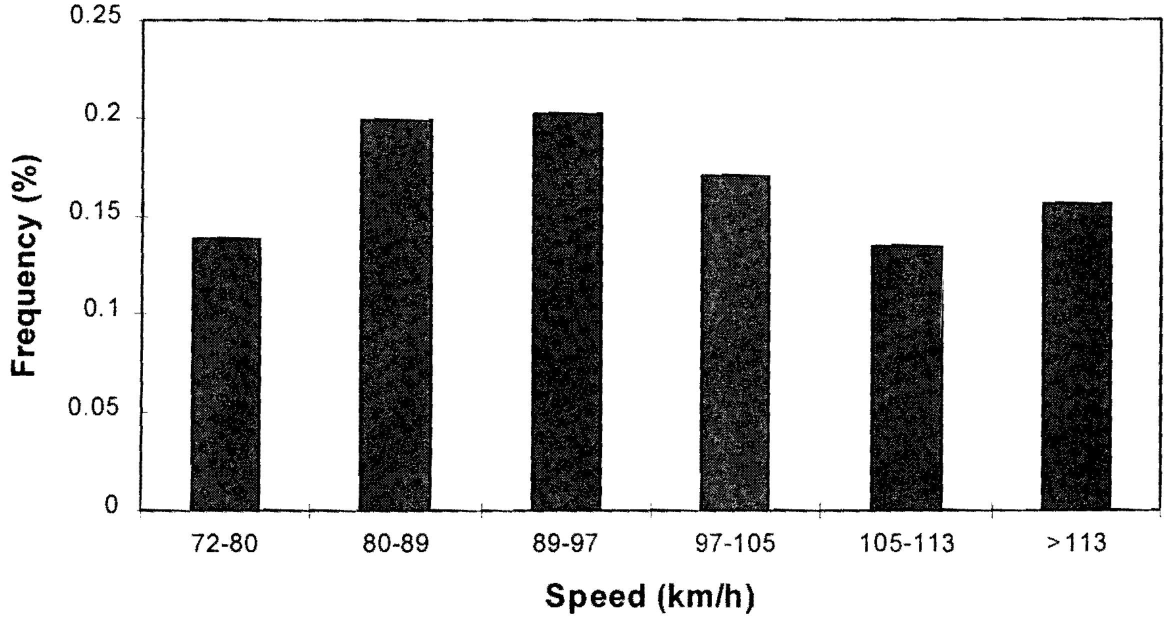


(a) Frequency Plot

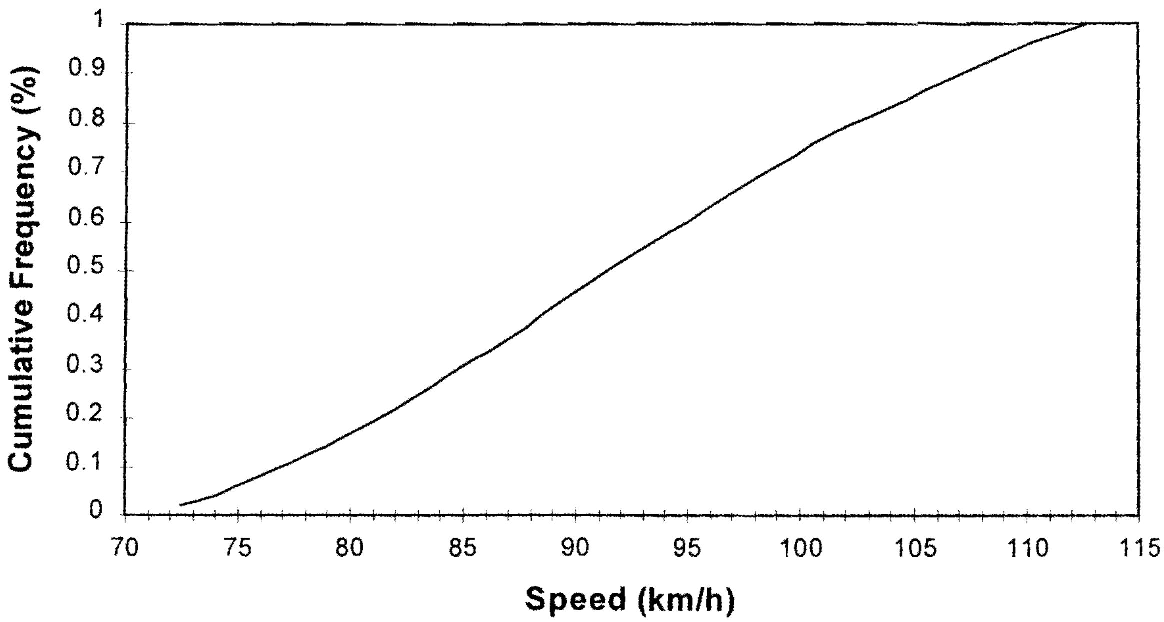


(b) Cumulative Frequency Plot

Figure C-2. Approach Speeds for All Vehicles at Site 3.



(a) Frequency Plot



(b) Cumulative Frequency Plot

Figure C-3. Approach Speeds for All Vehicles at Site 4.

APPENDIX D

DRIVER ACTIONS DATABASE FOR PASSENGER CARS

Site	City	Location	Date	Time	Loop Configuration	Condition	Traffic Volume	Percent Trucks	Driver Action[a]		
									1	2	3
1	Houston	US 290 @Mason	6/3/97	2:00-3:00 pm	Old	Off-Peak	781	9.9	5	6	3
				5:00-6:00 pm	Old	Peak	1380	4.5	10	22	5
				9:00-10:00 pm	Old	Night	445	8.1	9	1	0
			6/4/97	5:00-6:00 pm	Old	Peak	1318	3.9	23	13	0
				8:00-9:00 pm	Old	Night	238	11.3	10	12	0
			6/5/97	2:00-3:00 pm	New	Off-Peak	764	12.3	7	11	0
				4:00-5:00 pm	New	Peak	1196	6.1	12	15	3
				8:00-9:00 pm	New	Night	547	9.3	5	6	1
			6/6/97	2:00-3:00 pm	New	Off-Peak	1152	8.0	26	18	1
				5:00-6:00 pm	New	Peak	1703	3.5	30	15	3
				8:00-9:00 pm	New	Night	779	4.7	5	12	0
2	Conroe	SH 105 @Walden	6/10/97	7:05-8:05 am	Old	Peak	307	4.8	14	0	2
				2:00-3:00 pm	Old	Off-Peak	288	2.1	13	6	0
				8:30-9:30 pm	Old	Night	221	4.1	18	7	2
			6/11/97	7:00-8:00 am	Old	Peak	283	5.3	17	2	1
				10:00-11:00 am	Old	Off-Peak	331	5.1	17	7	3
				8:30-9:30 pm	Old	Night	176	1.7	3	1	1
			6/12/97	7:00-8:00 am	New	Peak	286	4.9	4	2	1
				1:00-2:00 pm	New	Off-Peak	337	5.3	8	8	2
				8:30-9:30 pm	New	Night	213	2.8	11	4	0
				9:30-10:30 pm	New	Night	205	2.9	2	1	0
			6/13/97	7:00-8:00 am	New	Peak	284	4.2	10	2	1
				1:00-2:00 pm	New	Off-Peak	393	3.6	14	4	2

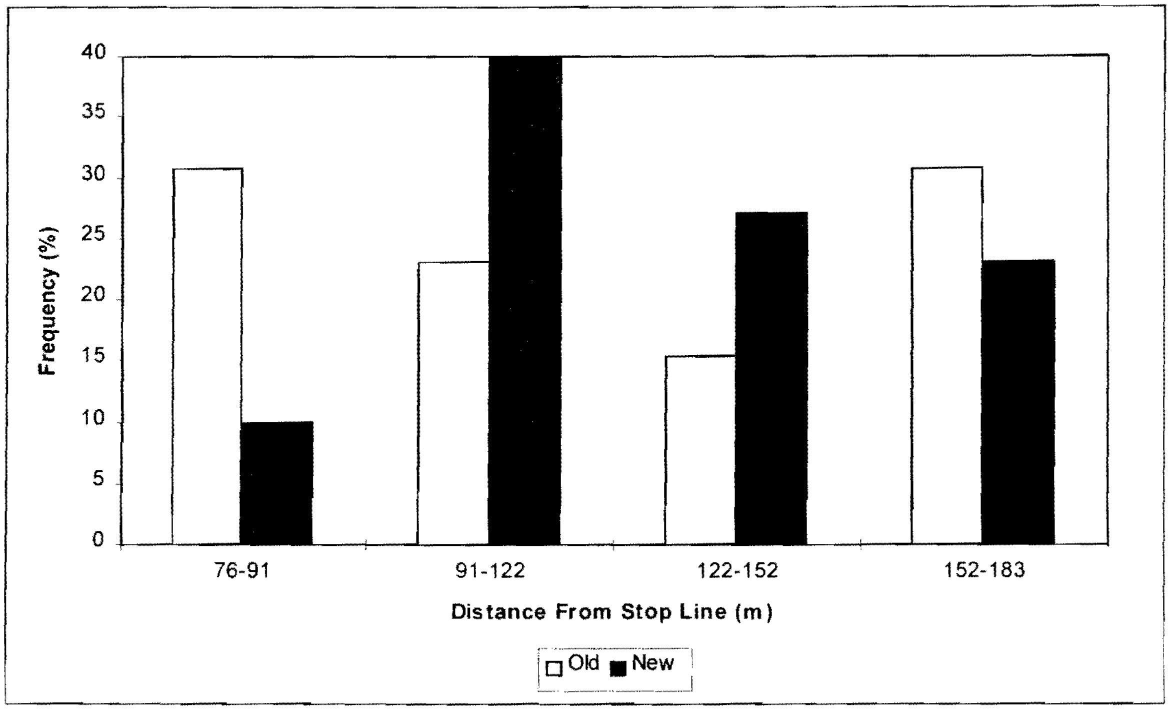
[a] Driver Action: 1 = stop at intersection; 2 = run yellow light; 3 = run red light.

Site	City	Location	Date	Time	Loop Configuration	Condition	Traffic Volume	Percent Trucks	Driver Action[a]		
									1	2	3
3	Conroe	SH 105 @April Sound	6/16/97	8:30-9:30 pm	Old	Night	305	1.0	11	4	1
			6/17/97	7:05-8:05 am	Old	Peak	410	6.6	6	12	2
				10:00-11:00 am	Old	Off-Peak	382	5.2	16	14	2
				4:00-5:00 pm	Old	Peak	612	1.6	25	18	6
				8:30-9:30 pm	Old	Night	300	0.7	6	11	3
			6/18/97	1:15-2:15 pm	New	Off-Peak	533	3.2	6	15	1
				8:30-9:30 pm	New	Night	349	1.1	4	9	3
			6/19/97	7:00-8:00 am	New	Peak	391	5.9	1	7	0
				10:00-11:00 am	New	Off-Peak	443	2.3	11	7	0
				8:30-9:30 pm	New	Night	379	0.8	7	5	1
4	Odessa	Business IH 20 @ CR 1290	6/23/97	2:00-3:00 pm	Old	Off-Peak	405	4.2	9	5	1
				5:00-6:00 pm	Old	Peak	596	2.5	5	20	3
				9:00-10:00 pm	Old	Night	146	1.4	0	1	1
			6/24/97	7:35-8:35 am	New	Peak	571	1.9	5	10	1
				10:00-11:00am	New	Off-Peak	340	2.9	5	5	0
				9:00-10:00 pm	New	Night	173	2.3	1	3	0
			6/25/97	7:00-8:00 am	Old	Peak	520	0.8	12	9	3
				10:00-11:00am	Old	Off-Peak	369	2.4	9	11	2
				9:00-10:00 pm	Old	Night	184	1.6	5	6	0
			6/26/97	7:35-8:35 am	New	Peak	590	1.9	2	4	0
				10:00-11:00am	New	Off-Peak	393	3.1	3	4	1
				9:00-10:00 pm	New	Night	202	1.0	3	7	1

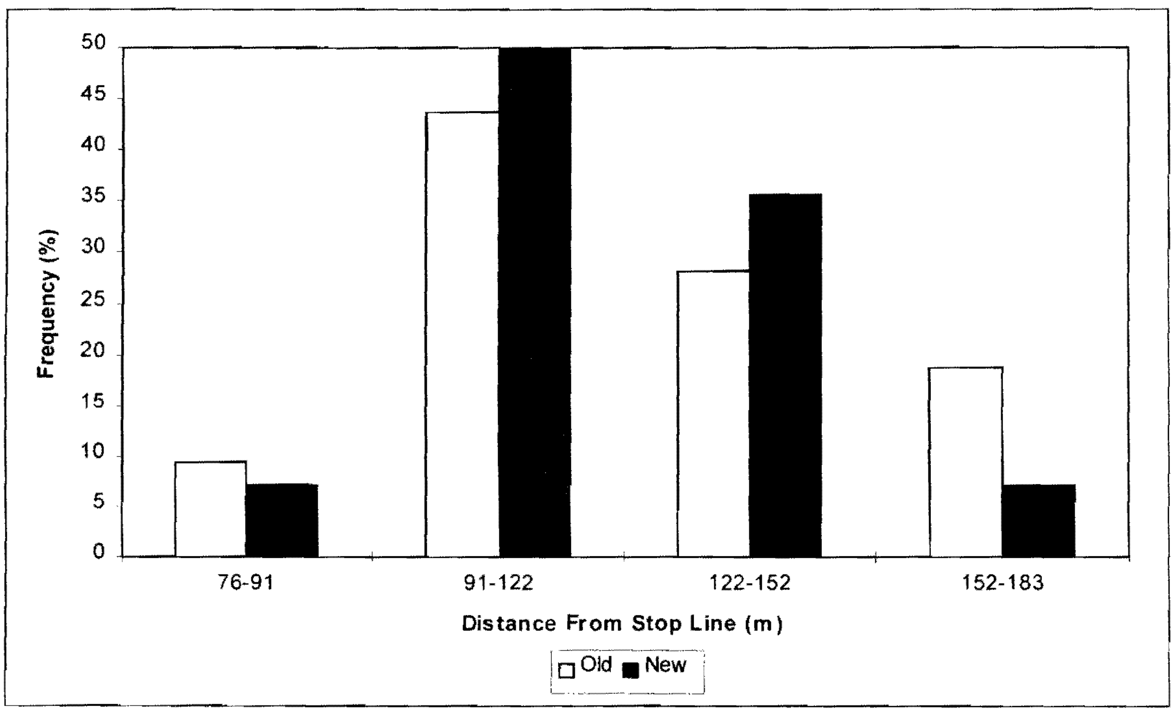
[a] Driver Action: 1 = stop at intersection; 2 = run yellow light; 3 = run red light.

APPENDIX E

DISTANCE FROM STOP LINE PLOTS FOR PASSENGER CARS

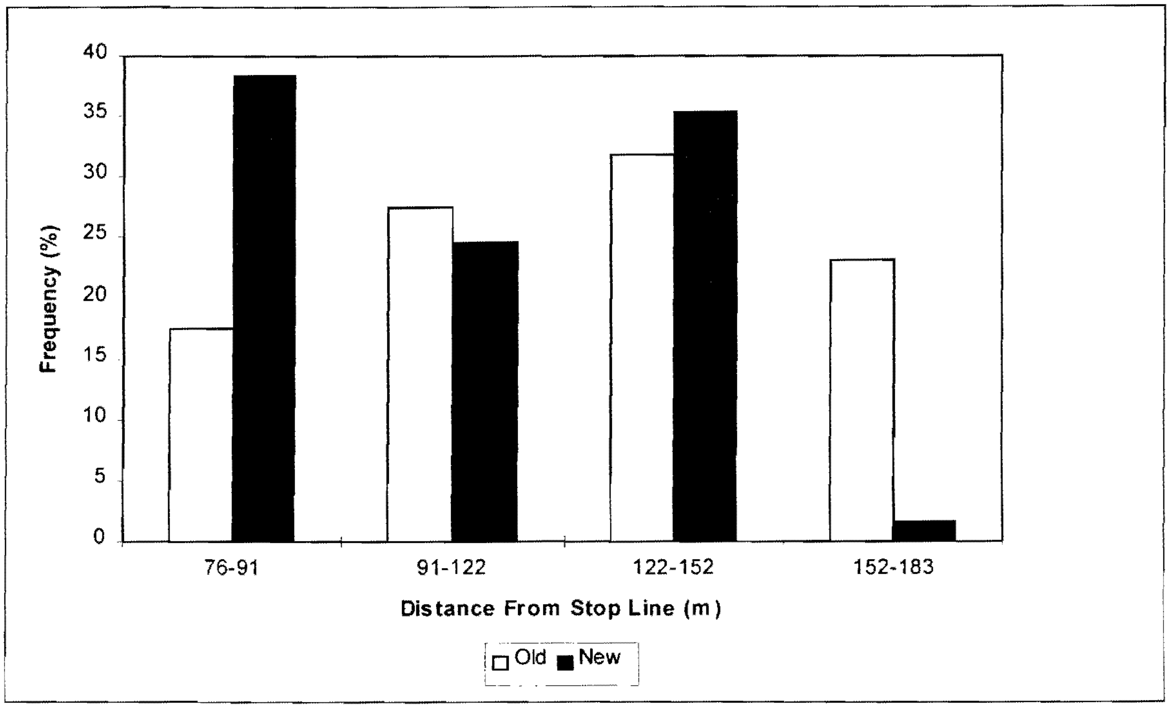


(a) Day Conditions

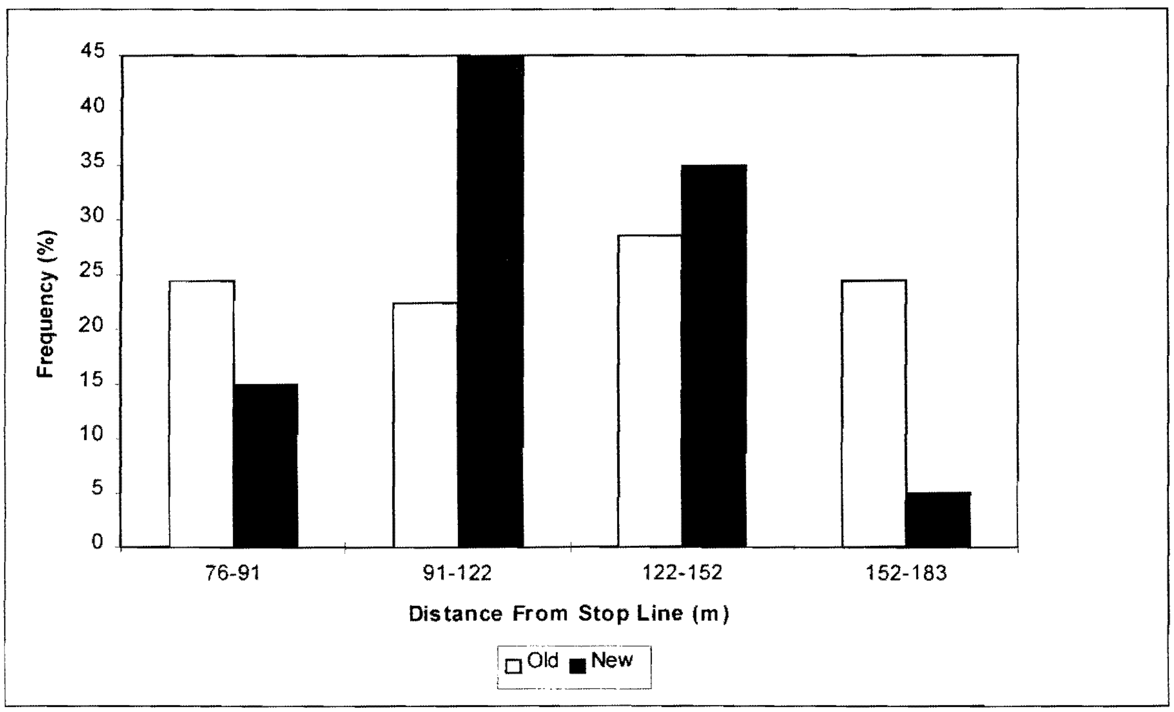


(b) Night Conditions

Figure E-1. Location of Passenger Cars for Site 1.

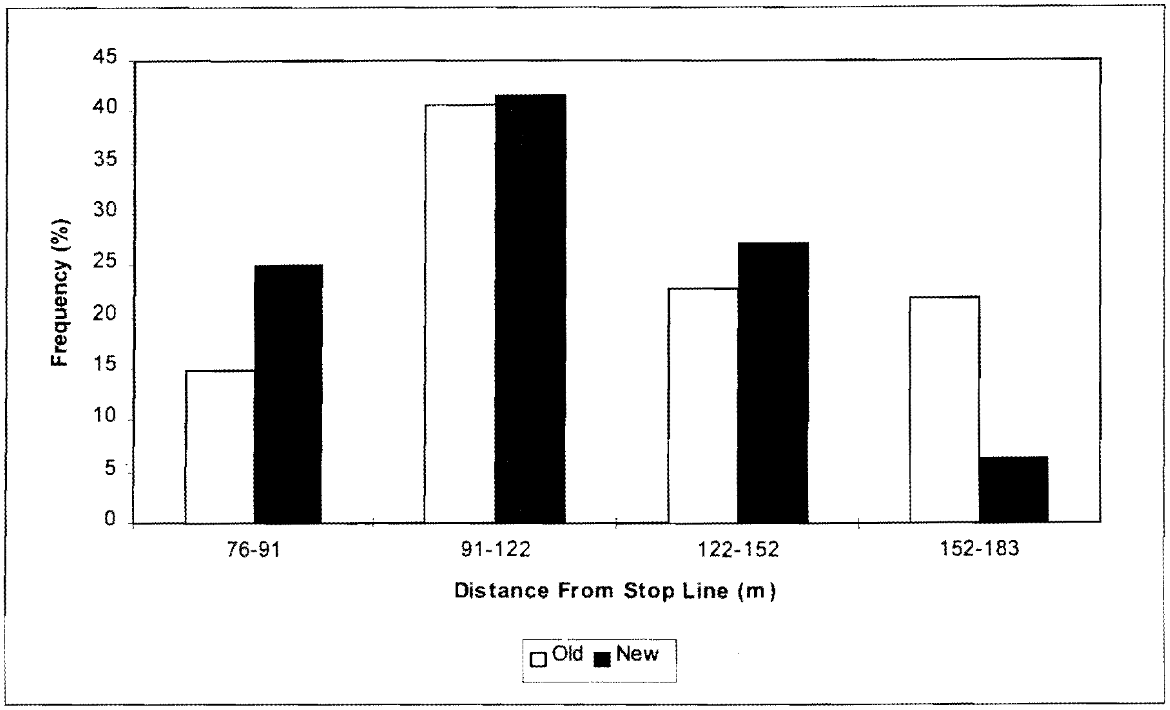


(a) Day Conditions

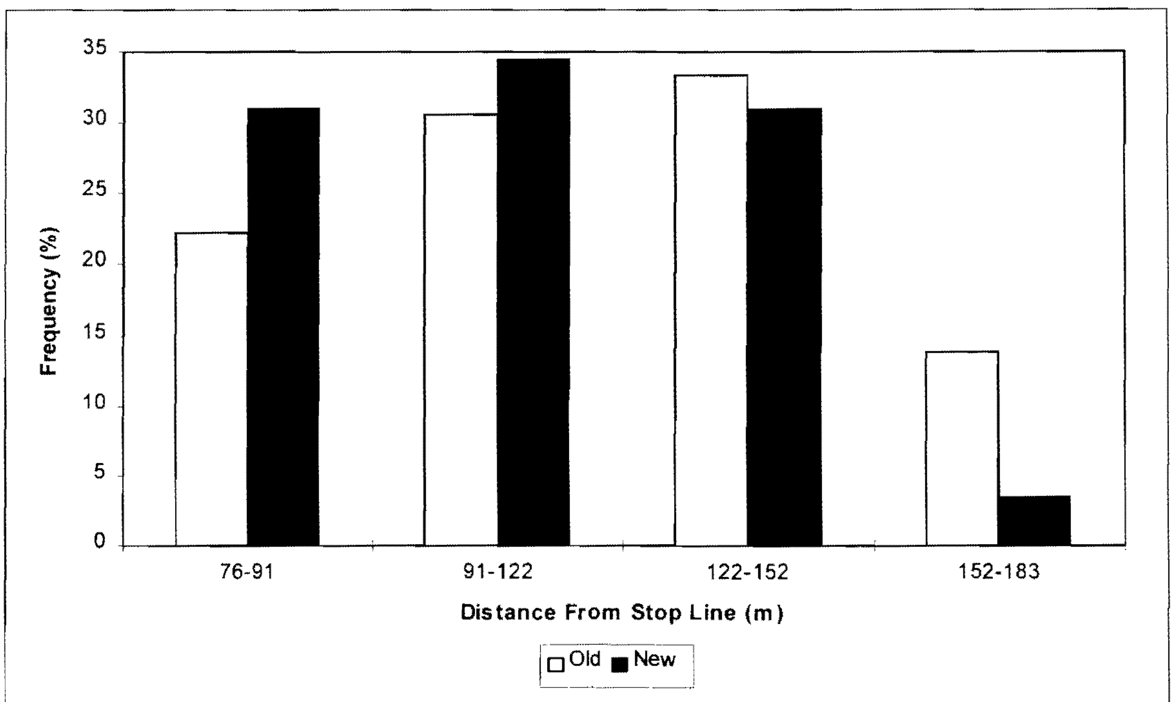


(b) Night Conditions

Figure E-2. Location of Passenger Cars for Site 2.

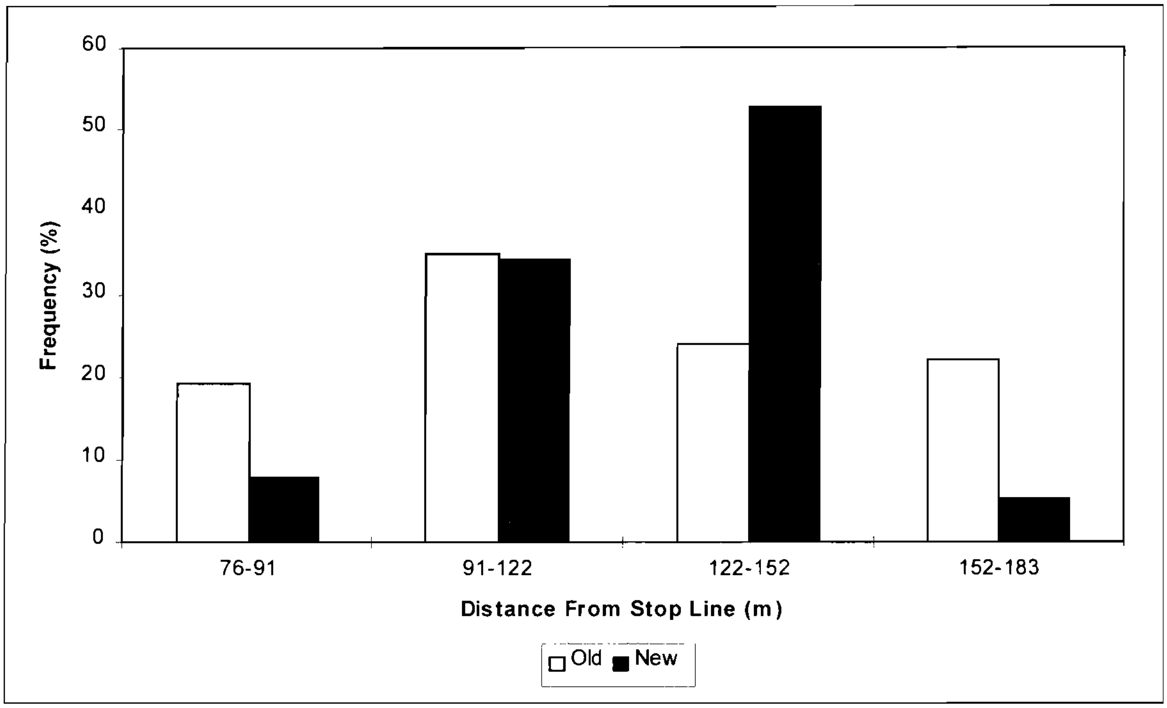


(a) Day Conditions

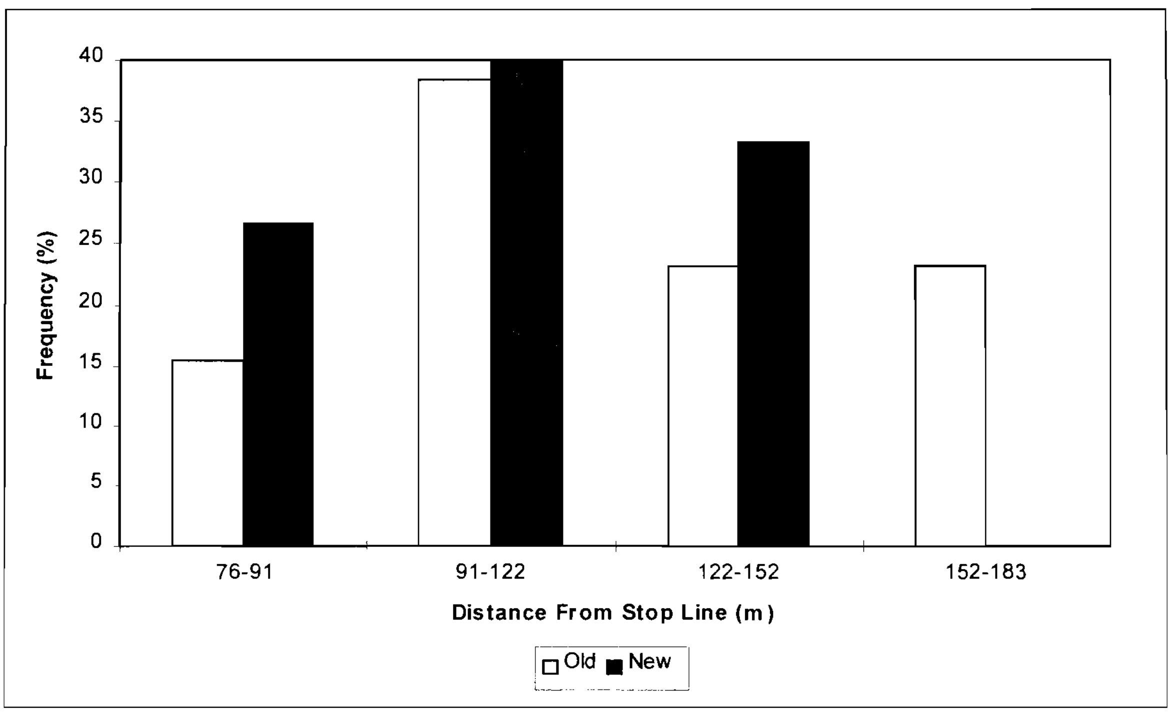


(b) Night Conditions

Figure E-3. Location of Passenger Cars for Site 3.



(a) Day Conditions



(b) Night Conditions

Figure E-4. Location of Passenger Cars for Site 4.

APPENDIX F

LOOP CONFIGURATION EVALUATION PROCEDURES

The following procedure was used in this study to evaluate the old and new loop configurations and their effects on max-out probability and delay to cross street traffic. The procedures were taken from the following reference:

Bonneson, J.A., and P.T. McCoy. *Manual of Traffic Detector Design*, First Edition. Washington, D.C.; Institute of Transportation Engineers, June 1994.

LOOP CONFIGURATION EVALUATION PROCEDURES

This procedure was developed for analyzing isolated, two-phase, fully actuated intersections. It is sensitive to a wide range of detection design and layouts as well as to two types of traffic signal controllers. It is assumed that the cycle length is not fixed in length and that vehicle arrivals are random. The methodology can be extended to multi-phase and fixed-cycle operation with some modification.

1. Calculate the equivalent maximum allowable headway, MAH

The procedure described above for maximum allowable headway should be used to obtain an estimate of the maximum allowable headway for each lane group in each signal phase, MAH_i. The flow rate in each lane group, q_i, should also be calculated. These quantities can then be used to calculate the equivalent MAH and total flow rate, q, using the equations below:

$$MAH = \frac{q_1 MAH_1 + q_2 MAH_2 + \ldots + q_m MAH_m}{q} \quad \textbf{(1)}$$

with

$$q = q_1 + q_2 + \ldots + q_m \quad \textbf{(2)}$$

where,

MAH = equivalent maximum allowable headway for the subject phase, sec;
q = total flow rate in the subject phase, vps;
m = number of lane groups served during the phase;
q_i = flow rate in lane group *i* (i = 1, 2, ..., m), vps; and
MAH_i = maximum allowable headway for lane group *i*, sec

2. Estimate the phase duration and cycle length

Subsequent models require an initial estimate of the phase durations and cycle length. A good initial estimate would be:

$$G = \frac{G_m + G_{max}}{2} \quad (3)$$

where,

$G =$ average duration of phase in service (green indication), sec;
$G_{max} =$ maximum green setting for the subject phase, sec;
$G_m =$ G_{min} for Type-170 controllers or 0.0 for NEMA controllers; and
$G_{min} =$ minimum green setting for subject phase, sec.

Once the phase durations have been estimated, the cycle length can be estimated as:

$$C = G_A + AR_A + G_B + AR_B \quad (4)$$

where,
$C =$ average cycle length, sec;
$G_{A,B} =$ average phase duration (green indication) for phase A or B, sec; and
$AR_{A,B} =$ yellow plus all-red interval for phase A or B, sec.

These estimates will be improved after a couple of iterations of Steps 3 through 11.

3. *Calculate the queue clearance time* G_q

$$G_q = \frac{q^*(C-G)}{n^* s^* - q^*} + l^* \leq G_m + G_{max} \quad (5)$$

where
$q^* =$ critical lane group flow rate for the subject phase, vps;
$n^* =$ number of lanes serving the critical lane group;
$s^* =$ saturation flow rate in the critical lane group, vpsgpl;
$l^* =$ start-up lost time for the critical lane group, sec; and
$G_q =$ average queue clearance time, sec.

It is recommended that the procedure described in Chapter 9 of the *Highway Capacity Manual* be used to estimate the saturation flow rate. This procedure will require reasonable estimates of phase duration and cycle length. Those estimates obtained from Step 2 can be used. However, if the phase durations and cycle length values change significantly after the completion of Step 11, the saturation flow rate may need to be recalculated.

4. *Calculate the average green extension, E*

$$E = \frac{1 - e^{-q(0.77MAH)}}{qe^{-q(0.77MAH)}} \tag{6}$$

where,

E=	average green extension time by arriving time, sec;
q =	total flow rate in the subject phase, vps; and
MAH =	equivalent maximum allowable headway of the subject phase, sec

5. *Calculate the time required to serve traffic,* G_{req}

$$G_{req} = G_{\min} + (G_q + E)e^{\frac{-G_{\min}}{G_q + E}} \tag{7}$$

6. *Calculate the duration of Event 1,* G_1

$$G_1 = \text{smaller of: } \begin{matrix} G_{req} \\ G_{\max} + G_m \end{matrix} \tag{8}$$

7. *Calculate the average headway between* G_m *and* G_{req} *in the conflicting phase,* $hc_{/(Bm<h<Greq)}$

Use Equation 9 with $Q = q_c$; $A = G_m$; $B = G_{req}$

$$H_{/A<h<B} = \frac{1}{Q} + \frac{A - Be^{-Q(B-A)}}{1 - e^{-Q(B-A)}} \tag{9}$$

8. *Calculate the duration of Event 2,* G_2

$$G_2 = \text{smaller of: } \begin{matrix} G_{req} \\ G_{\max} + hc_{/G_m<h<G_{req}} \end{matrix} \tag{10}$$

9. *Calculate the average phase duration, G*

$$G = G_1 + (G_2 - G_1)e^{-q_c(G_m + AR_c + MAH_c)} + (G_{req} + \frac{1}{q_c} - G_2)e^{-q_c(G_{req} + AR_c + MAH_c)} \tag{11}$$

where,

q_c = total flow rate in the conflicting phase, vps;
AR_c = yellow plus all-red interval of the conflicting phase, sec; and
MAH_c = equivalent maximum allowable headway of the conflicting phase, sec.

10. Repeat Steps 3 through 9 to estimate the average phase duration for the other signal phase.

11. Calculate the average cycle length, C

$$C = G_A + AR_A + G_B + AR_B \tag{12}$$

where,

$G_{A,B}$ = average phase duration (green indication) for phase A or B, sec; and
$AR_{A,B}$= yellow plus all-red interval for phase A or B, sec.

12. Repeat Steps 3 through 11 until the initial and final cycle length are equivalent.

After completing Step 12, the average signal timing for the intersection should be established. Proceed with the evaluation using Steps 13 through 20 (once for each phase).

13. Determine A and B from Table 1.

Table 1. Variable Values for Calculating R.

Controller Type	G_q	Flow Rate, Q	Upper Limit, B	Lower Limit, A
170	$G_q \geq Gmin$	q_c	$G_q+AR_c+MAH_c$	$G_{min}+AR_c+MAH_c$
	$Gq<Gmin$	q_c	AR_c+MAH_c	AR_c+MAH_c
NEMA	any	q_c	$G_q+AR_c+MAH_c$	AR_c+MAH_c

Controller type determines when the first conflicting-phase call can be placed. For Type-170 controllers, the time the first call is placed depends on the minimum green setting of the subject phase, G_{min}. If the first call comes in before G_{min} is timed out, the call for service is acted on by the controller at the end of G_{min}. If the call comes in after G_{min}, the call is acted on at the time of its arrival to the controller. For NEMA controllers, the time the first call is placed equals the time that it is acted on by the controller, regardless of the G_{min} setting.

14. Calculate the average headway between A and B in the conflicting phase, $hc_{/A<h<B}$

Use Equation 9 with the values of Q, A, and B from Table 1. Note that $hc_{/A<h<B} = A = B$ for the Type-170 controller when $G_q<G_{min}$.

15. Calculate the time between the first call and queue clearance, R

$$R = (B - hc_{/A<h<B})(e^{-QA} - e^{-QB}) + (B - A)(1 - e^{-QA}) \tag{13}$$

Note that R equals 0.0 for the Type-170 controller when $G_q < G_{min}$.

16. Calculate the average headway between A and B in the subject phase, $h_{/h<MAH}$

Use Equation 9 with Q = q, A = β, and B = MAH, where β = 2.0 seconds for phases serving one lane or β = 0.0 seconds for phases serving two or more lanes.

17. Calculate the number of arrivals needed to max-out, n

$$n = 1.18\frac{G_{max} - MAH - R}{h_{/h<MAH}} \geq 0.0 \tag{14}$$

18. Calculate the probability of a headway less than the MAH, p

$$p = 1 - (1 - \beta q)\, e^{-q(MAH-\beta)} \tag{15}$$

19. Calculate the probability of a max-out, P(max-out)

$$P(\max\text{-}out) = p^{n} \tag{16}$$

20. Calculate the average waiting time, W

$$W = G - \frac{1}{q_c} e^{-q_c(AR_c+MAH_c)} \tag{17}$$