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16. Abstract

Red-light-running is a significant problem throughout the United States and Texas. It is associated with frequent and severe crashes. Engineering countermeasures represent a useful means of combating the red-light-running problem because they are passively applied (in contrast to enforcement countermeasures which are considered to be overt and punitive) and are in the direct control of the agency responsible for the signal. The objective of this research project was to describe how engineering countermeasures can be used to minimize the frequency of red-light-running and associated crashes at intersections.

This report documents the work performed, findings, and conclusions reached as a result of a two-year research project. During the first-year, engineering countermeasures were identified and implemented at 10 intersections in five Texas cities. Before-after studies of red-light-running frequency were then conducted at each intersection. Also, the three-year crash history for each intersection was compared to its observed frequency of red-light-running. The findings from these studies indicate that the frequency of red-light-running decreases in a predictable way with decreasing approach flow rate, longer clearance path lengths, longer headways, and longer yellow interval durations. The crash data analyses indicate that right-angle crashes increase exponentially with an increasing frequency of red-light-running. Models for computing an intersection approach's red-light-running frequency and related crash rate are described. Guidelines for selecting appropriate engineering countermeasures and evaluating their performance are provided.

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ENGINEERING COUNTERMEASURES TO REDUCE RED-LIGHT-RUNNING

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DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data published herein. The contents do not necessarily reflect the official view or policies of the Federal Highway Administration (FHWA) and/or the Texas Department of Transportation. This report does not constitute a standard, specification, or regulation. It is not intended for construction, bidding, or permit purposes. The engineer in charge of the project was James Bonneson, P.E. #67178.

NOTICE

The United States Government and the State of Texas do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

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CHAPTER 1. INTRODUCTION

OVERVIEW

Statistics indicate that red-light-running has become a significant safety problem throughout the United States. Retting et al. (*1*) report that about one million collisions occur at signalized intersections in the U.S. each year. Of these collisions, Mohamedshah et al. (*2*) estimate that at least 16 to 20 percent can be attributed directly to red-light-running. Retting et al. also report that motorists involved in red-light-running-related crashes are more likely to be injured than those in other crashes. In fact, they found that 45 percent of red-light-running-related crashes involve injury whereas only 30 percent of other crashes involve injury.

A 1998 survey of Texas drivers, conducted by the Federal Highway Administration (FHWA) (*3*), found that two of three Texans witness red-light-running every day. About 89 percent of these drivers believe that red-light-running has worsened over the past few years. The largest percentage (66 percent) perceive the reason for red-light-running is that the red runner is "in a hurry." An examination of nationwide fatal crash statistics by the Insurance Institute for Highway Safety found that Texas had the fourth highest number of red-light-running-related deaths per 100,000 population between 1992 and 1998 (*4*).

There is a wide range of potential countermeasures to the red-light-running problem. These solutions are generally divided into two broad categories: engineering countermeasures and enforcement countermeasures. Enforcement countermeasures are intended to encourage drivers to adhere to the traffic laws through the threat of citation and possible fine. In contrast, engineering countermeasures (which include any modification, extension, or adjustment to an existing traffic control device) are intended to reduce the chances of a driver being in a position where he or she must decide whether or not to run the red indication. Studies by Retting et al. (*1*) have shown that countermeasures in both categories are effective in reducing the frequency of red-light-running. However, most of the research conducted to date has focused on the effectiveness of enforcement; little is known about the effectiveness of many engineering countermeasures.

In summary, red-light-running is a significant problem throughout the United States and Texas. It appears to be a growing problem that leads to frequent and severe crashes. Engineering countermeasures represent an attractive means of combating the red-light-running problem because they are passively applied (in contrast to enforcement countermeasures which are considered to be overt and punitive) and are in the direct control of the agency responsible for the signal.

This report describes the factors that are associated with red-light-running as well as several countermeasures that have been used to reduce its frequency. Initially, there is an examination of the red-light-running process in terms of the events necessary to precipitate a red-light-running event. Then, various engineering countermeasures are identified. Next, a before-after study is described. This study is intended to facilitate the evaluation of selected countermeasures and to calibrate a model for predicting the frequency of red-light-running. The data are then analyzed and the findings used to develop guidelines for selecting and evaluating engineering countermeasures.

RESEARCH OBJECTIVE

The objective of this research project was to describe how engineering countermeasures could be used to minimize the frequency of red-light-running and associated crashes at intersections. Satisfying of the following goals helped achieve this objective:

- Quantify the effect of various traffic characteristics and traffic control factors on the frequency of red-light-running.
- Quantify the relationship between red-light-running and crash frequency.
- Identify promising engineering countermeasures and quantify their effects.
- Facilitate implementation of engineering countermeasures through development of a guide.

RESEARCH SCOPE

A red-light-running event can be characterized by traffic movement type, entry time of the red-light-running vehicle, and the motivation underlying the driver's decision to run the red indication. "Traffic movement type" reflects the different expectations and experiences of the left-turn versus the through driver. Relative to the through driver, the left-turn driver is forced (by geometry) to travel through the intersection at a slow rate of speed and is also more likely to experience lengthy delays.

"Entry time of the red-light-running driver" relates to the time that the driver enters the intersection after the onset of the red indication. When a driver enters late into the red, it may be an indication of deficiencies in signal visibility or driver sight-distance along the intersection approach. It may also be an indicator of driver indifference to the traffic laws regarding the red indication. Intuitively, crash potential is higher when a red-light-runner enters several seconds after the red onset. Fortunately, about 85 percent of all red-light-runners enter the intersection within the first 1.5 s of red (*5*) so the frequency of related crashes due to late entries is relatively low.

"Driver Decision Type" describes the basis for the driver's decision to run the red indication. An "avoidable" red-running event is committed by a driver who believes that it is possible to safely stop but decides it is in his or her best interest to run the red indication. In contrast, an "unavoidable" event is committed by a driver who either (1) believes that he or she is unable to safely stop and consciously decides to run the red, or (2) is unaware of the need to stop.

This research focuses on the unavoidable red-light-running by through drivers that takes place during the first few seconds after the onset of red. Red-light-running events having these characteristics occur frequently and are most treatable by engineering countermeasures. Moreover, efforts to reduce this type of red-light-running are likely to have the greatest return in terms of a reduced number of crashes.

RESEARCH APPROACH

The research approach is based on a two-year program of development and evaluation that was directed at producing information engineers could use to reduce red-light-running. During the first year of the research, the project team identified causes of red-light-running and a range of engineering countermeasures. Researchers developed a before-after study plan to evaluate the effectiveness of alternative countermeasures at 10 signalized intersections in Texas. In the second year, several countermeasures were implemented and evaluated through the direct measurement of red-light-running frequency. The project compared the crash history of the study sites to the observed frequency of red-light-running.

One product of this research is a guideline document. This document provides technical guidance for engineers interested in using engineering countermeasures to reduce red-light-running at problem intersections. It also provides tools for evaluating the effectiveness of selected engineering countermeasures.

CHAPTER 2. RED-LIGHT-RUNNING PROCESS AND COUNTERMEASURES

OVERVIEW

This chapter describes the red-light-running process and the countermeasures described in the literature as having some effect on the frequency of red-light-running. Initially, the red-light-running process is described in terms of the events that lead to red-light-running and the factors that have some influence on a driver's propensity to run the red indication. The chapter concludes with a discussion of red-light-running countermeasures, with a focus on "engineering" countermeasures.

Red-Light-Running Process

Several events must occur together to result in a driver running the red indication. Additional events must then follow for a red-light-running-related crash to occur. Table 2-1 lists these events in roughly the same sequence that they must occur to produce a red-run event and subsequent crash.

Table 2-1. Events Leading to Red-Light-Running and Related Crashes.

Type	Event	RLR Freq.	Rt. Angle Crash	Rear-end Crash
Exposure Events	1. Vehicle *i* is *x* sec. travel time from the intersection ($x < 6.0$ s).	✔	✔	✔
	2. Phase terminates (yellow presentation).	✔	✔	✔
	3. Phase termination is by phase max-out (or controller is pretimed).	✔	✔	✔
Contrib-utory Events	4. Vehicle *i* does not stop.	✔	✔	✔
	5. Vehicle *i*'s entry time occurs after yellow ends.	✔	✔	
	6. Vehicle *i*'s clearance time occurs after all-red ends.		✔	
	7. Conflicting vehicle *k* enters intersection *y* sec. after all-red ends.		✔	
	8. Vehicle *j* stops (and it is in front of vehicle *i*).			✔

Note: RLR = red-light-running

The first three events listed in Table 2-1 represent exposure events because they "set the stage" for the contributory events that follow. Thus, exposure to red-light-running requires: (1) sufficient traffic volume to result in one or more vehicles on the intersection approach; (2) a phase termination; and (3) pretimed control or, if the control is actuated and advance detection is used, the termination is by "max-out" (i.e., maximum green limit is reached). Consideration of the first two events suggests that exposure to red-light-running increases with flow rate on the subject approach and the number of signal cycles.

The contributory events that lead to red-light-running include: (1) the vehicle does not stop, and (2) the vehicle's time of entry into the intersection occurs after the indication changes from yellow to red. Consideration of these two events suggests that the frequency of red-light-running will increase whenever drivers are less likely to stop and when the yellow interval is reduced.

The "vehicle does not stop" event is the most complex event of those listed in Table 2-1. The probability of this event is discussed herein in terms of its inverse, the probability of stopping. It reflects the uncertainty (or indecision) exhibited by the population of drivers on an intersection approach at the onset of the yellow indication. The event is complex because many factors can affect the probability of stopping (e.g., travel time to intersection, speed, etc.).

The last two columns of Table 2-1 relate to the two types of crashes most commonly found at signalized intersections. Both types require the same exposure events. The right-angle crash also requires: (1) the red-light-running vehicle to be present in the intersection when the all-red interval ends, and (2) a conflicting vehicle to enter the intersection while it is occupied by the red-light-running vehicle. Consideration of these two events suggests that the frequency of right-angle crashes increases with a decrease in the all-red interval and an increase in the conflicting movement flow rate.

In summary, the following factors influence the frequency of red-light-running and related crash frequency:

- flow rate on the subject approach (exposure factor),
- number of signal cycles (exposure factor),
- phase termination by max-out (exposure factor),
- probability of stopping (contributory factor),
- yellow interval duration (contributory factor),
- all-red interval duration (contributory factor),
- entry time of the conflicting driver (contributory factor), and
- flow rate on the conflicting approach (exposure factor).

Each of these factors is described more fully in a later section of this chapter.

Review of Texas Law

To provide some perspective on the problem of red-light-running in Texas, it is important to be familiar with the applicable laws, codes, and ordinances. Chapter 544 of the Texas Transportation Code (*6*) deals with traffic signs, signals, and markings; section 544.007 specifically addresses traffic-control signals.

Section 544.007 is somewhat ambiguous concerning the specific problem of red-light-running, in that the definition of a driver's responsibility when encountering a yellow signal is not fully specified. According to Subsection (b), a driver waiting at an intersection when his or her signal turns green must wait until all other legally entering vehicles have cleared the intersection before proceeding. Therefore, Texas law implies that a vehicle that enters an intersection legally (i.e., during yellow) may still be in the intersection after a conflicting movement receives a green signal.

This law is sometimes referred to as the "permissive yellow" law in comparison to more restrictive laws that require drivers to have exited the intersection before the end of the yellow interval. Parsonson et al. (*6*) indicate that at least half of the states in the United States follow the permissive rule. The advantage of the permissive rule is that it enables most drivers to be lawful in their responses to the yellow indication.

The disadvantage of the permissive law is that it creates a situation where the cross-street driver receives a green indication but must yield the right-of-way (to a crossing vehicle) before entering the intersection. Parsonson et al. (*6*) indicate that 60 percent of drivers are unaware that they have to yield the right-of-way when presented the green indication. Moreover, when asked the question, "What would you think if traffic engineers decided to time yellow lights so that there might be a vehicle going through the intersection when you get your green," 69 percent of drivers said that they disapproved of this practice because it was dangerous. The solution advocated by Parsonson et al. (*6*) is to provide an all-red interval (following the yellow interval) of sufficient duration to permit drivers to clear the intersection before a conflicting phase is presented with a green indication.

EXPOSURE FACTORS

This section summarizes the literature as it relates to events that expose drivers to conditions that may precipitate red-light-running. These events were previously discussed with regard to Table 2-1. The factors that underlie these events include flow rate, number of signal cycles, and phase termination by max-out.

Flow Rate on the Subject Approach

Flow rate on the subject approach is important to the discussion of red-light-running. Each vehicle on the intersection approach at the onset of yellow is exposed to the potential for red-light-running. The number of drivers running the red each signal cycle will likely increase as the flow rate increases.

Three studies have reported sufficient data to examine the effect of flow rate on red-light-running frequency or related crashes. Kamyab et. al (*7*) observed 1242 hours of operation at 12 urban intersections in Iowa. For each intersection approach, they reported the average daily traffic volume, the observed number of red-light-runners, and the duration of the study. The relationship between the computed hourly approach flow rate and red-light-running frequency is shown in Figure 2-1 (with square data points). This trend line indicates that red-light-running increases at a rate of about 3.0 red-light-runners per 1000 vehicles.

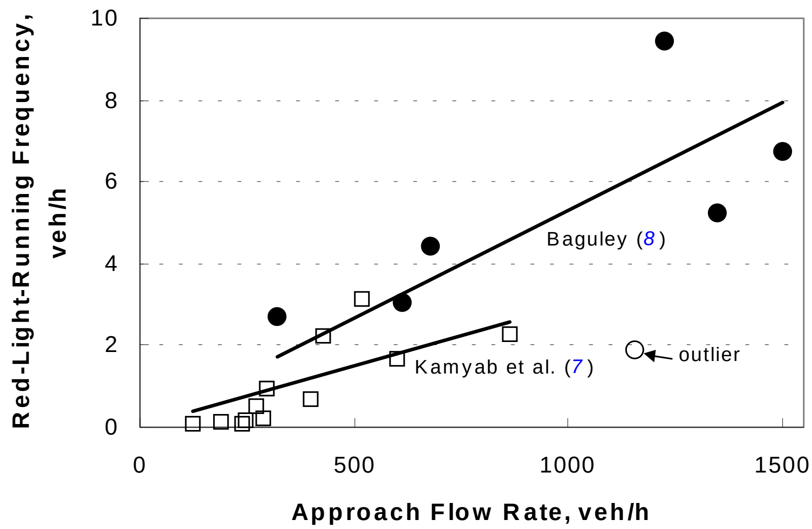

Figure 2-1. Effect of Flow Rate on the Frequency of Red-Light-Running.

Baguley (*8*) examined the frequency of red-light-running at seven rural intersections in England. He found that red-light-running frequency was positively correlated with approach flow rate. He also noted that there was a slight positive correlation with approach speed and inverse correlation with daily cross-street volume.

The relationship between flow rate and red-light-running frequency using Baguley's (*8*) data is shown in Figure 2-1 (using circular data points). Six intersections (shown with solid circles) had daily cross-street volumes of less than 7500 vehicles. The seventh intersection (shown with an open circle and labeled "outlier") had an exceptionally high daily cross-street volume of 17,000 vehicles that Baguley speculated may explain its very low frequency of red-light-running. The trend line indicates that red-light-running increases at a rate of about 5.3 red-light-runners per 1000 vehicles.

Mohamedshah et al. (*2*) examined the effect of flow rate (and other variables) on red-light-running-related crashes. They obtained crash data for 1756 urban intersections in California. The data were screened to include only those crashes attributable to a red-light-running event. They found that crash frequency increased with flow rate on the subject approach. Their findings indicate that approach crash frequency increases from 0.25 crash/yr at a two-way volume of 8000 veh/day to 0.5 crash/yr at 50,000 veh/day.

Number of Signal Cycles

As noted previously, some researchers recognize that the frequency of red-light-running and related crashes is largely affected by the frequency with which the yellow indication is presented (*9*).

A cycle length change from 60 to 120 s reduces the number of times that the yellow is presented by 50 percent. In theory, a similar reduction in red-light-running frequency should also be observed. Recognition of this relationship is often exhibited by the researchers reporting red-light-running statistics normalized by cycle frequency. For example, Van der Horst and Wilmink (*9*) discuss the use of "percent of cycles with at least one red-light-runner."

Phase Termination by Max-Out

Green-extension detection systems use one or more detectors located in advance of the intersection to hold the phase in green as long as the approach is occupied. In this manner, drivers are not exposed to the yellow indication, and red-light-running is reduced. However, if the green is held to its maximum limit, the phase "maxes-out" and is forced to end, regardless of whether a vehicle is approaching the intersection. An actuated phase that maxes-out has the potential to expose drivers to a red-light-running situation. Similarly, a pretimed signal phase always ends independently of vehicle presence on the approach and has the potential to expose drivers to a red-light-running situation.

Zegeer and Deen (*10*) evaluated the effect of green-extension systems on the frequency of red-light-running. Their evaluation focused on two rural intersections. It revealed a 65 percent reduction in red-running frequency due to the use of a green-extension system.

As noted previously, the benefits of a green-extension system can be negated if the phase maxes-out. The probability of max-out is dependent on flow rate in the subject phase and the "maximum allowable headway," as dictated by the detector design. The maximum allowable headway (MAH) is the largest headway in the traffic stream that can occur and still sustain a continuous extension of the green interval. The relationship between max-out probability, MAH, maximum green, and flow rate is illustrated in Figure 2-2.

Bonneson and McCoy (*11*) indicate that the MAH values shown in Figure 2-2 (i.e., 4.0 and 7.0 s) represent the range of values for most detection designs. To illustrate the implications of alternative detection designs, consider the following example. If a phase has a flow rate of 1200 veh/hr, a maximum green duration of 30 s, and no advance detection (i.e., only a stop-line detector), then its probability of max-out will be about 0.05 (1 out of 20 cycles). However, if a green-extension system is used, the resulting max-out probability will increase to 0.7 (7 out of 10 cycles). One option available to reduce this probability is to increase the maximum green setting; however, this increase may also increase the delay to waiting vehicles.

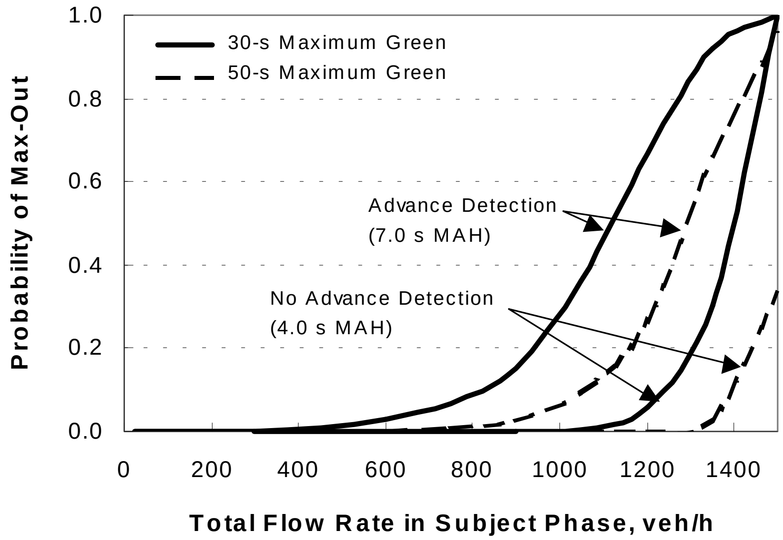

Figure 2-2. Effect of Flow Rate and Detection Design on Max-Out Probability.

CONTRIBUTORY FACTORS

Two contributory factors underlie the events that lead to red-light-running. These factors include the "probability of stopping" and the "yellow interval duration." The former factor represents the complex decision-making process that drivers exhibit at the onset of the yellow indication. A review of the literature indicates that this decision is affected by the driver's assessment of the prevailing traffic and roadway conditions. It is also affected by the driver's estimate of the consequences of stopping (or not stopping).

The yellow interval duration contributes to red-light-running in a more fundamental manner. The start of this interval defines the instant when the decision-making process should begin. During this interval, a decision is made and acted upon. The end of this interval defines the instant when the red indication is presented (whereupon entry to the intersection represents a red-light-running event). Both factors, and their relationship to the frequency of red-light-running, are described in this section.

Probability of Stopping

Many researchers have studied the decision to stop in response to the yellow indication. Van der Horst and Wilmink (*9*) studied this decision process and found that a driver's propensity to stop is based on three components. These components and the factors that influence them are listed in Table 2-2. Each component is discussed in the following subsections.

Table 2-2. Factors Affecting Driver Decision at Onset of Yellow Indication.

Components of the Decision Process	Factor
Driver behavior	Travel time, Speed, Actuated control, Headway, Coordination, Approach grade, Yellow interval
Estimated consequences of not stopping	Threat of right-angle crash Threat of citation
Estimated consequences of stopping	Threat of rear-end crash Expected delay

Driver Behavior

In the case of an "unavoidable" red-light-running event, the driver's response to the yellow indication is affected by his or her perceived ability to stop and his or her awareness of the need to stop. An "unavoidable" event is committed by a driver who either (1) believes that he or she is unable to safely stop and consciously decides to run the red indication, or (2) is unaware of the need to stop. This ability and awareness is influenced by the seven factors listed in Table 2-2. Each of these factors is discussed in the following paragraphs.

Travel Time. Studies indicate that a driver's decision to stop at yellow onset is based partly on his or her estimate of speed and distance to the stop line (*12, 13, 14, 15, 16, 17*). Through these estimates, the driver assesses his or her ability to stop and the degree of comfort associated with the stop. Several researchers have measured driver response to the yellow indication in terms of the travel time to the intersection at the onset of yellow (*9, 12, 13, 14, 15, 16*). The relationship between travel time and probability of stopping reported by each researcher is shown in Figure 2-3.

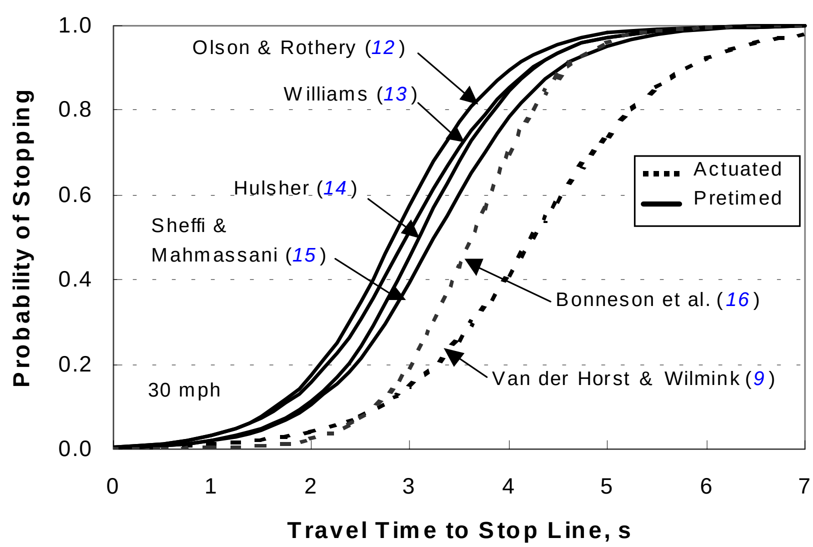

Figure 2-3. Probability of Stopping as a Function of Travel Time and Control Type.

The trends in Figure 2-3 indicate that there is a range, between about 2- and 5-s travel time from the intersection stop line, where drivers are collectively indecisive about the decision to stop. The solid and dashed lines suggest that there is a difference in driver behavior at pretimed and at actuated intersections. This trend is discussed in a subsequent section titled Actuated Control and Coordination.

Speed. A driver's decision to stop may be skewed by his or her limited ability to estimate travel time to the intersection at higher speeds. Allsop et al. (*17*) found that drivers tend to underestimate actual travel time by about 30 percent. Related to this observation is the reported finding that high-speed drivers tend to be less likely to stop than low-speed drivers when at the same travel time from the stop line at the onset of the yellow indication (*15, 16*). The trend reported by Bonneson et al. (*16*) is shown in Figure 2-4.

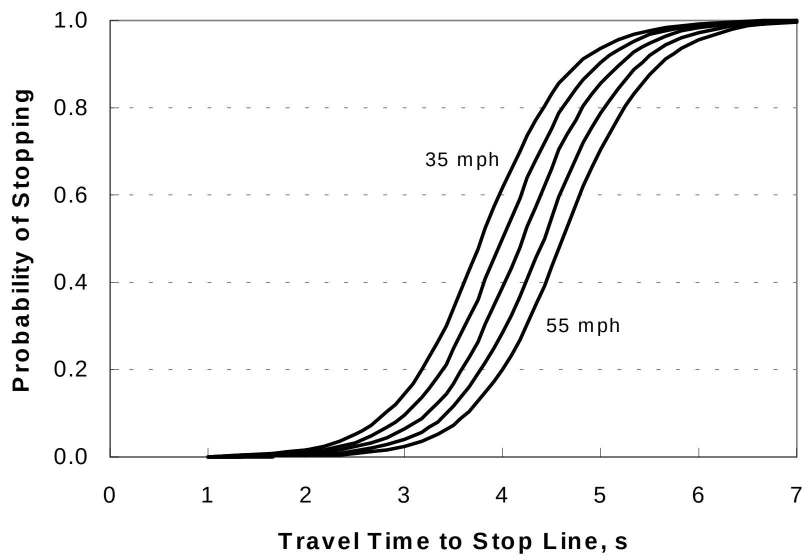

Figure 2-4. Probability of Stopping as a Function of Travel Time and Speed.

The trends shown in Figure 2-4 suggest that the time interval within which drivers are indecisive varies slightly with approach speed. Drivers that are 4.0 s from the stop line have a 0.6 probability of stopping if they are traveling at 35 mph; however, they have only a 0.2 probability if they are traveling at 55 mph. This behavior suggests that the degree to which a driver underestimates his or her travel time increases with speed.

Actuated Control. Evidence of the effect of intersection control type on the probability of stopping has been reported by Van der Horst and Wilmink (*9*). They found evidence that drivers approaching an actuated intersection are less likely to stop than if they are approaching a pretimed intersection. This finding suggests that drivers learn which signals are actuated and then develop an expectation of service as they travel through the detection zone. This effect of control type on the probability-of-stopping is shown in Figure 2-3.

Coordination. Van der Horst and Wilmink (*9*) extrapolated the aforementioned driver expectancy associated with actuated control to drivers traveling within platoons through a series of interconnected signals. Drivers in a platoon are believed to develop an *ad hoc* expectancy as they travel without interruption through successive signals. Their expectancy is that each signal they approach will remain green until after they (and the rest of the platoon) pass through the intersection. Their desire to stay within the platoon makes them less willing to stop at the onset of the yellow indication.

Approach Grade. Chang et al. (*18*) examined the effect of "approach grade" on the probability of stopping. They found that drivers on downgrades were less likely to stop (at a given travel time from the stop line) than drivers on level or upgrade approaches. The effect of grade is shown in Figure 2-5 for an approach speed of 30 mph. The trends in this figure suggest that only about 38 percent of drivers will stop on a 5 percent downgrade when they are 4 s travel time from the stop line. In contrast, 66 percent will stop if they are on a 5 percent upgrade.

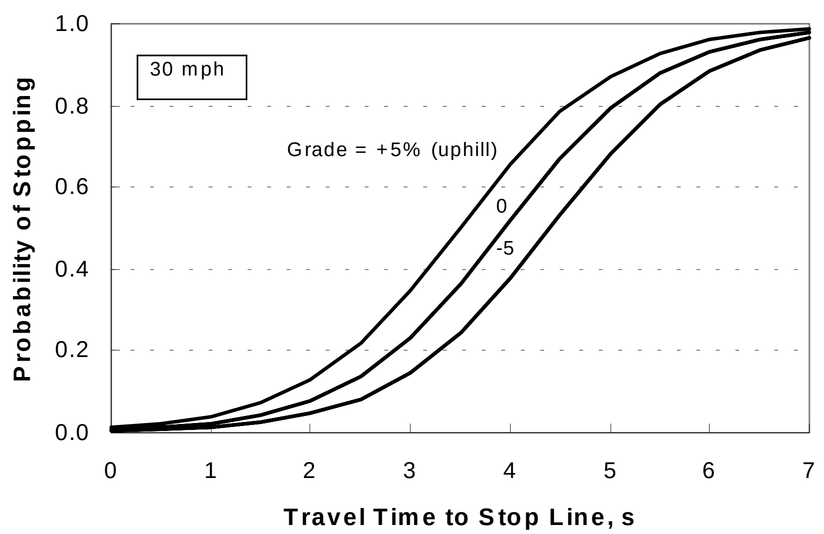

Figure 2-5. Probability of Stopping as a Function of Travel Time and Approach Grade.

Yellow Interval Duration. Van der Horst and Wilmink (*9*) have noted that long yellow intervals can lead to bad behavior because the last-to-stop drivers are not "rewarded" with a red indication as they arrive at the stop line. Instead, the yellow remains lit as they roll up to the stop line. These drivers will be more inclined not to stop the next time they approach the intersection. Several researchers have found that a driver adjusts his or her stopping behavior to offset the effect of longer change intervals (*8, 9, 19*). This behavior is illustrated in Figure 2-6 and is based on the data reported by Van der Horst and Wilmink (*9*). This figure indicates that drivers that are 4.0 s from the stop line have a probability of 0.5 of stopping if the yellow is 3 s in duration; however, they have only a 0.34 probability if the yellow is 5 s long.

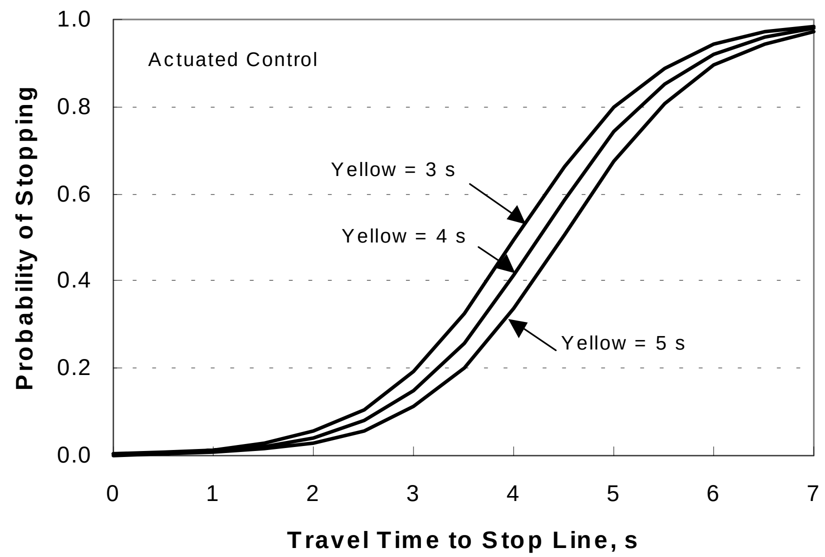

Figure 2-6. Probability of Stopping as a Function of Travel Time and Yellow Duration.

Finally, a study by Mahalel and Prashker (*19*) indicates that a lengthy "end-of-phase" warning interval can lead to an increased indecision zone. Specifically, they found that when a 3-s yellow was preceded by a 3-s flashing green, the indecision zone ranged from 2 to 8 s. This range is larger than that for signals without a flashing green interval (i.e., 2 to 5 s). They cite evidence that an increased indecision zone increases the frequency of rear-end crashes.

Headway. Drivers traveling through an intersection may be more cognizant of vehicles immediately ahead of them or just behind them than they are of the signal indication. Thus, they are likely to be drawn through the intersection by a preceding driver, even though the yellow (or red) indication is presented. In fact, Allsop et al. (*17*) found that drivers that are "closely following" (i.e., 2 s or less headway to the vehicle ahead) are more likely to run the red indication than are drivers that are neither closely following nor being closely followed (i.e., freely flowing drivers).

An analysis of the data reported by Allsop et al. (*17*) is shown in Figure 2-7. The trends in this figure indicate that about 50 percent of drivers (at 3 s travel time from the stop line) are likely to stop if flowing freely on the approach. However, only about 42 percent of drivers will stop if they are within 2 s of the vehicle ahead. If these drivers are being closely followed, this percentage drops even further. This latter behavior is discussed in the section titled Consequences of Stopping.

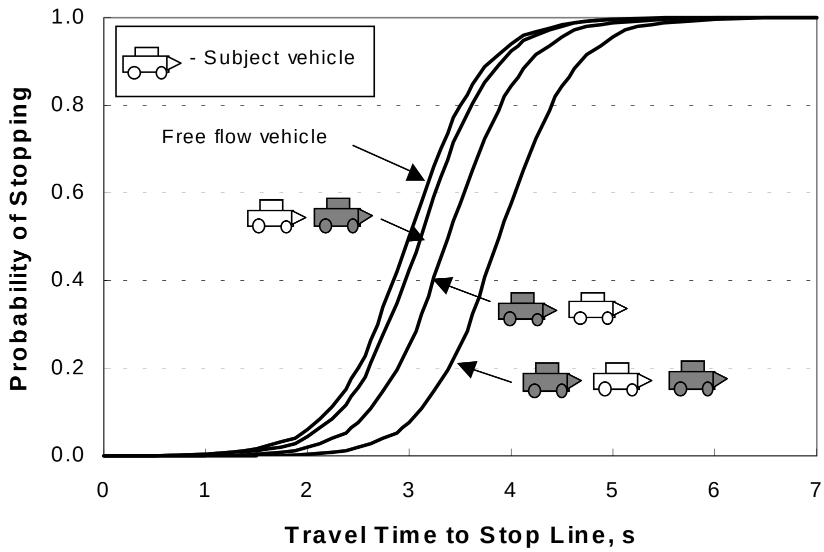

Figure 2-7. Probability of Stopping as a Function of Travel Time and Proximity of Other Vehicles.

Consequences of Not Stopping

The driver's response to the yellow indication is also affected by his or her consideration of the consequences of not stopping and the consequences of stopping. The former consideration includes an estimate of the potential for a right-angle crash and the potential for receiving a citation. The latter consideration is discussed in a subsequent section titled Consequences of Stopping.

Threat of Right-Angle Crash. A driver contemplating running the red indication may assess the threat of a right-angle crash by estimating the number of vehicles in the conflicting traffic stream. This number is estimated by scanning the intersection ahead and by recalling prior experience at this intersection. In this regard, a study by Baguley (*8*) found an inverse correlation between the frequency of red-light-running and the daily cross-street volume. His data indicate that drivers are six times more likely to run the red indication when the cross street has a daily traffic volume of 2000 veh/day compared to when it has 17,000 veh/day.

Threat of Citation. Van der Horst and Wilmink (*9*) noted that drivers consider the potential for being cited when deciding whether to run the red indication. The findings from a survey of drivers, conducted by Retting and Williams (*20*), support this claim. They found that 46 percent of drivers (in cities without automated enforcement) believe that someone who runs the red indication is likely to receive a citation. This percentage increases to 61 percent in cities with automated enforcement.

Consequences of Stopping

A driver's concern about the threat of a rear-end crash or lengthy delay is also factored into the decision to stop when presented with a yellow indication.

Threat of Rear-End Crash. Drivers that are being closely followed when the light turns from green to yellow may be more reluctant to stop because of the greater likelihood of a rear-end crash. In a laboratory setting, Allsop et al. (*17*) observed that drivers being closely followed (i.e., when the following vehicle's headway was less than 2 s) at the onset of yellow were more likely to run the red indication.

Figure 2-7 shows the effect close following on the probability-of-stopping. The trends in this figure indicate that about 50 percent of drivers (at 3 s travel time from the stop line) are likely to stop if flowing freely on the approach. However, only about 25 percent of drivers will stop if they are being closely followed. This percentage drops to 8 percent when the driver is *both* closely followed and closely following another vehicle.

Expected Delay. A survey conducted by the FHWA (*3*) indicated that 66 percent of Texas drivers believe red-light-running is due to drivers who are in a hurry. Obviously, the delay associated with stopping is contrary to most driver's desire to reach his or her destination quickly.

A review of the literature did not uncover any research conducted on the effect of the drivers' expected delay on the decision to stop at the onset of yellow. However, some evidence of this influence can be found in an examination of the data reported by Zegeer and Deen (*10*). These data include conflicts and flow rates observed throughout the day at two intersections, both before and after installation of a green-extension system (via multiple advance detectors). About two-thirds of the conflicts observed were red-light-runs. The relationship between conflict rate (in units of "conflicts per 1000 vehicles") and time-of-day is shown in Figure 2-8.

The trends in Figure 2-8 indicate that drivers traveling during the noon and evening peak traffic hours are more likely to run a red light than during other hours of the day. This trend was exhibited in both the "before" and "after" periods. As delays tend to be highest during the peak hours, the trends suggest that drivers may be more inclined to run the red indication as the expected delay increases.

Yellow Interval Duration

The yellow interval duration is generally recognized as a key factor that affects the frequency of red-light-running. This recognition has led several researchers to recommend setting the yellow interval duration based on the probability of stopping (*9, 12, 18*). These researchers suggest that the yellow interval should be based on the 85th (or 90th) percentile driver's travel time to the stop line. This approach is illustrated in Figure 2-9 where the trends shown suggest that a yellow interval of 4.2 s is sufficient for 85 percent of drivers. Only 15 percent of drivers would choose to run the red indication if they are more than 4.2-s travel time from the stop line at the onset of yellow and are in the "first-to-stop-position."

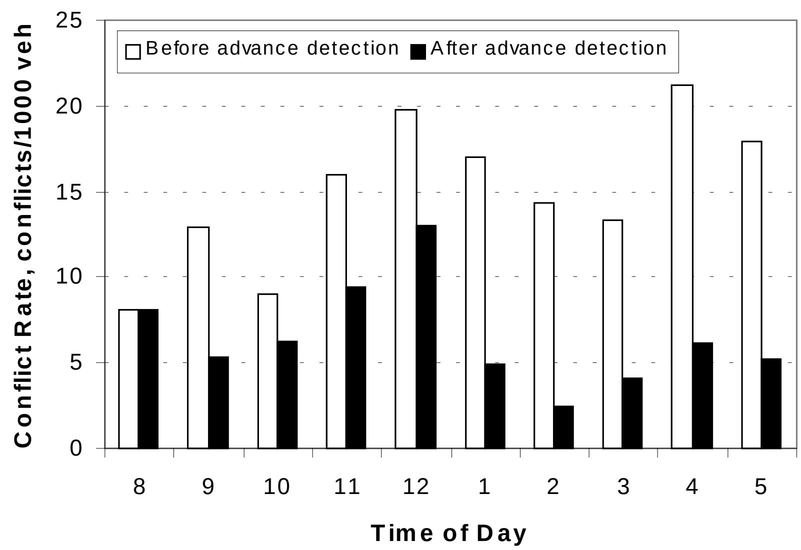

Figure 2-8. Variation of Red-Light-Running and Other Conflicts by Time-of-Day.

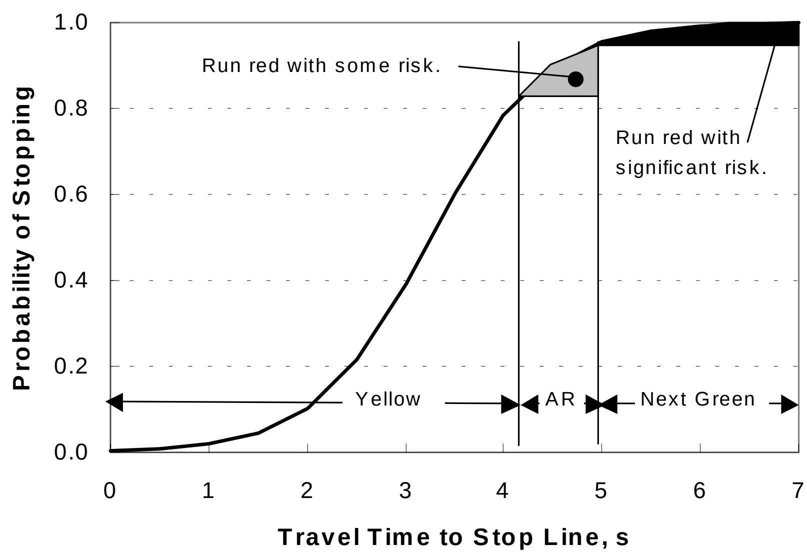

Figure 2-9. Relationship between Probability of Stopping and Yellow Interval Duration.

FACTORS LEADING TO CONFLICT

Once the driver has been presented the yellow indication and has chosen not to stop, there is a threat of conflict with other vehicles. This conflict can lead to a crash if one or both drivers are unable to effect an evasive maneuver. The frequency of a rear-end conflict (that occurs when the lead driver decides to stop and the following driver decides not to stop) is dependent on: (1) the probability of a red-light-running event, (2) the probability that two vehicles are present on the subject approach, and (3) the probability that the driver of the lead vehicle chooses to stop. The second probability is based on the flow rate on the subject approach as a contributing factor. The first and third probabilities were the subject of the preceding section.

The frequency of a right-angle conflict is dependent on: (1) the probability of a red-light-running event, (2) the probability that a vehicle is present on the conflicting approach, and (3) the probability that it enters the intersection before the red-light-running vehicle clears it. The second and third probabilities are based on two contributing factors and one exposure factor. The contributing factors include the duration of the all-red clearance interval and the entry time of the conflicting driver. The exposure factor is the flow rate on the conflicting approach. These factors are discussed further in this section.

All-Red Interval Duration

The *Manual on Uniform Traffic Control Devices* (*21*) states that the yellow change interval may be followed by an all-red clearance interval to provide additional time before conflicting traffic movements are released. However, according to Parsonson et al. (*6*), there is no consensus at this time on whether this means that the clearance interval should be sufficiently long to completely clear the intersection or the degree to which the concept should be applied systemwide. This lack of a guidance has led to an inconsistency in the use of the all-red interval among agencies and may contribute to an increase in crashes due to driver confusion or a lack of driver respect for the signal.

A benefit of the all-red clearance interval is to provide a degree of protection against a right-angle conflict should a vehicle run the red indication. This benefit is realized if the all-red interval equals or exceeds the time required by the clearing vehicle to cross the intersection. Figure 2-9 illustrates the benefit of an all-red (AR) interval in terms of its ability to protect about two-thirds of the red-light-running vehicles from conflict (i.e., 10 of the 15 percent of all drivers that run the red). The trends in this figure suggest that if a 0.8-s all-red interval is used, then only 5 percent of drivers would be at significant risk for a right-angle conflict.

Entry Time of the Conflicting Driver

The lead driver in a conflicting traffic stream could be in one of four states after receiving the green indication. These states are: (1) the driver is stopped at the stop line and pauses to verify that the intersection is clear before proceeding; (2) the driver is stopped at the stop line and tries to anticipate the onset of green by rolling forward during the all-red interval; (3) the driver is approaching the intersection but is slowing to stop for the red interval; or (4) the driver is approaching the intersection but is anticipating the onset of green and maintains a nominal speed. The risk of conflict increases from State 1 to State 4. Any of the four states can occur; however, States 1 and 2 are most likely to occur at intersections when the flow rates are moderate to high.

Researchers (*14, 18*) have examined the times associated with States 1 and 2 and found that almost all stopped, first-in-queue drivers require more than 1.0 s to reach the path of the clearing vehicle. This finding suggests that the red indication would have to be run *and* the clearing vehicle would have to be in the intersection 1.0 s or more after the conflicting movement receives the green for a conflict to occur. Hence, when flow rates are moderate to high, conflicting streams are separated in time even when an all-red interval is not provided. However, this protection may not extend to low-volume evening hours when States 3 or 4 are likely to occur.

Flow Rate on the Conflicting Approach

By definition, a conflict requires two or more vehicles to interact where one or more of these vehicles have to take an evasive action to avoid a collision. Thus, the frequency of conflict is a function of the flow rate of the conflicting traffic movements. As evidence of this effect, Mohamedshah et al. (*2*), in a study of red-light-running crash frequency, found that right-angle crashes on the major street increased with an increase in the flow rate on the minor street.

RED-LIGHT-RUNNING COUNTERMEASURES

There is a wide range of potential countermeasures to the red-light-running problem. These solutions are generally divided into two broad categories: engineering countermeasures and enforcement countermeasures. Enforcement countermeasures are intended to encourage drivers to adhere to the traffic laws through the threat of citation and possible fine. In contrast, engineering countermeasures are intended to reduce the frequency that drivers are put in a position where they must decide whether or not to run the red indication. The relationship between countermeasure category and the type of decision made by the driver (when running the red) is shown in Table 2-3.

Table 2-3. Relationship between Countermeasure Category and Driver Decision Type.

Driver Decision Type	**Possible Scenario**	**Countermeasure Category**	
		Engineering	**Enforcement**
"Avoidable"	Congested, Cycle overflow	Less	**Most Effective**
"Unavoidable"	Incapable of stop, Inattentive	**Most Effective**	Less

The information in Table 2-3 suggests that there are two basic types of decisions associated with a red-light-running event. An "avoidable" red-running event is committed by a driver who believes that it is possible to safely stop but decides it is in his or her best interest to run the red indication. In contrast, an "unavoidable" event is committed by a driver who either (1) believes that he or she is unable to safely stop and consciously decides to run the red indication, or (2) is unaware of the need to stop. Hence, the avoidable event consists only of drivers who intentionally run the red while the unavoidable event consists of drivers who intentionally or unintentionally run the red indication.

A nonscientific survey conducted by a news magazine of 4711 readers indicates that 51 percent have unintentionally run a red light (*22*). This finding suggests that the majority of red-light-running events are currently considered to be unavoidable by the drivers who commit these events.

An avoidable red-running event is likely committed by a driver who is frustrated by excessive delay or congested flow conditions. This driver may also be indifferent to traffic laws. Short of major resource investments to increase capacity, enforcement countermeasures are likely to be the most effective means of curbing this driver's inclination to run the red indication.

An unavoidable event is likely to be committed by a driver who is incapable of stopping (e.g., due to a poorly judged downgrade or relatively high speed) or just inattentive (i.e., does not see the change to yellow). This event may also be precipitated when the yellow interval is not sufficiently long as to give drivers time to reach the intersection when they are at a distance within which they can not comfortably stop. Engineering countermeasures, such as a longer yellow interval or a more visible signal indication, are likely to be the most effective means of helping these drivers avoid red-light-running.

Engineering Countermeasures

Engineering countermeasures can be placed into three categories, depending on their method of implementation. Signal Operation countermeasures are implemented through modification to the signal phasing, cycle length, or change interval. Motorist Information countermeasures are implemented through enhancements to the signal display or providing advance information to the driver about the existence of a signal ahead. The Physical Improvement category includes a group of more substantial modifications to the intersection that are intended to solve serious safety or operational problems. Table 2-4 lists these countermeasure categories and some common countermeasures. Each countermeasure is described in a subsequent section.

Also listed in Table 2-4 are the results of a survey conducted by the Institute of Transportation Engineers (ITE) (*23*). The survey was facilitated using an Internet web-site. Requests for participation in the survey were sent by e-mail to 1500 ITE members. A total of 90 responses were received with about 65 percent of the respondents employed by city, county, or state transportation agencies. About 77 percent of the respondents have implemented engineering countermeasures to reduce red-light-running.

Table 2-4. Engineering Countermeasures to Red-Light-Running.

Countermeasure Category	Specific Countermeasure	Respondents that have used Countermeasure [1]
Signal Operation (modify signal phasing, cycle length, or change interval)	Increase the yellow interval duration	39 %
	Provide green-extension (advance detection)	not asked
	Improve signal coordination	39 %
	Improve signal operation (phasing, cycle length)	59 %
Motorist Information (provide advance information or improved notification)	Provide pre-yellow signal indication	not asked
	Improve sight distance	20 %
	Improve visibility of signal (12" lens, LED [2], add heads)	47 %
	Increase conspicuity of signal (back plate, strobe)	not asked
	Add advance warning signs (with or without flashers)	41 %
Physical Improvement (implement safety or operational improvements)	Remove unneeded signals	6 %
	Add capacity with additional traffic lanes	0 %
	Flatten sharp vertical curves	2 %
	Soften sharp horizontal curves	not asked

Notes:
1 - Data obtained from survey conducted by the Institute of Transportation Engineers in the Fall of 2000 (*23*).
2 - LED: light emitting diode. Signal indication utilizes LEDs as the light source in lieu of an incandescent lamp.

The last column of Table 2-4 indicates the extent to which each countermeasure has been used by the responding agencies. The percentages shown indicate that improvements to signal phasing and cycle length have been used most frequently. Physical changes to the intersection have received the least consideration. This latter finding is likely a reflection of the significant cost associated with the implementation of physical improvements.

Increase the Yellow Interval Duration

Increasing the yellow interval duration has a direct effect on the frequency of red-light-running. Figure 2-4 suggests that the yellow interval duration should range from 4.5 to 5.5 s (depending on speed) to be consistent with a travel time within which 90 percent of drivers will stop. Retting and Greene (*24*) cite several studies that have shown that an increase in yellow duration results in significant reductions in red-light-running, right-angle crashes, or both. Van der Horst and Wilmink (*9*) documented the relationship between red-light-running frequency and yellow interval duration at 11 intersections. This relationship is shown in Figure 2-10. The trend shown suggests that yellow intervals in excess of 3.5 s are associated with minimal red-light-running.

Van der Horst and Wilmink (*9*) have also noted that the trend shown in Figure 2-10 does not stay at “0.0 percent of cycles with red-light-running” for yellow interval durations in excess of 5.0 s. Specifically, they note that there are “...changes in drivers’ behavior...” for overly long yellow warning intervals. Presumably, the change to which they are referring is an increase in the frequency of red-light-running with an increase in yellow duration beyond 5.0 s.

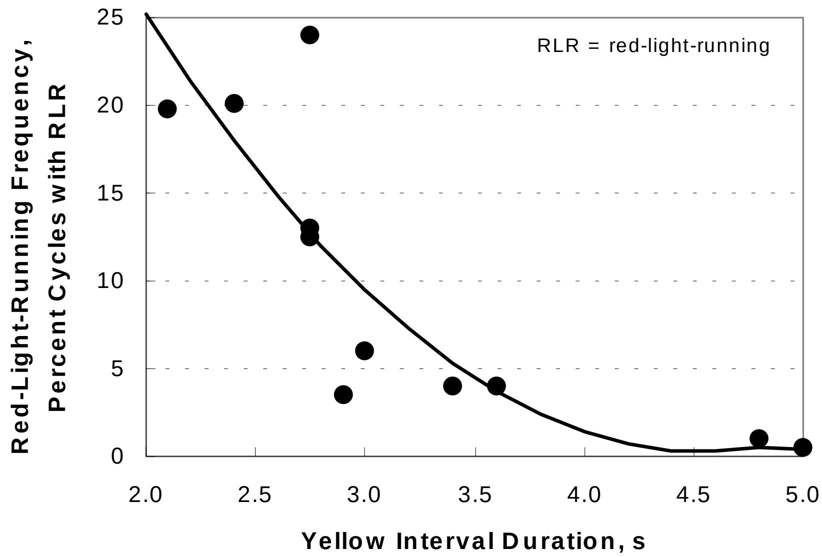

Figure 2-10. Relationship between Red-Light-Running Frequency and Yellow Duration.

Provide Green-Extension

Green-extension (i.e., advance detection) is a countermeasure used at intersections with actuated control. It employs advance detectors on the major-road approaches. The detector placement and controller settings are designed such that a lengthy gap in traffic is needed before the phase is allowed to terminate. This scheme ensures that the approach is effectively clear of vehicles when yellow is presented *unless* the phase is forced to end because it has reached its maximum duration (i.e., it maxes-out). Zegeer and Deen (*10*) have found that green-extension has the potential to reduce the frequency of red-light-running by 65 percent.

Improve Signal Coordination

Drivers approaching the intersection while the green is displayed and while traveling within a platoon are likely to expect that the indication will remain green at least until they pass through the intersection. This expectation was noted by Van der Horst and Wilmink (*9*). It suggests that the platoons formed by signal coordination can *increase* the frequency of red-light-running, especially if any portion of the platoon arrives near the end of the green interval. However, if the coordination plan is coupled with an increase in cycle length (which is typically the case), a reduction in delay, and the progression bands are such that the platoons do not arrive near the end of the green indication, then coordination may reduce the frequency of red-light-running.

Improve Signal Operation

This countermeasure includes changes in the signal phasing or cycle length that reduce red-light-running. In general, changes that reduce delay should reduce red-light-running. Data reported by Zegeer and Deen (*10*) support this observation. In fact, any improvements in the signalization that reduce delay are likely to reduce red-light-running.

As noted previously, an increase in cycle length reduces driver exposure to the yellow indication. For example, a cycle length change rom 60 to 120 s reduces the number of times that the yellow is presented by 50 percent. In theory, a similar reduction in red-light-running frequency should also be observed. However, an increase in cycle length should also be coupled with signalization improvements that reduce delay (e.g., signal coordination) to ensure a significant reduction in red-light-running.

Provide Pre-Yellow Signal Indication

"Pre-yellow" information has been provided in other countries (e.g., Mexico and Israel) by flashing the green indication several seconds before the onset of yellow. A study by Mahalel and Prashker (*19*) found that this technique can increase crash frequency. Specifically, they found that when a 3-s yellow was preceded by a 3-s flashing green, there was a corresponding increase in the "indecision zone." This zone was represented by a length of the intersection approach within which drivers are collectively indecisive about the choice of stopping or going at the onset of yellow. The indecision zone on the approach with flashing green started at 8-s travel time to the stop line and ended at 2-s travel time (compared to 5 s and 2 s when flashing green was not used). They cite evidence that an increased indecision zone increases the frequency of rear-end crashes.

Improve Sight Distance

Improvements to driver sight distance along the intersection approach intuitively will have a beneficial effect on intersection safety. If the sight restriction limits the driver's view of the signal indications, there is likely to be more frequent red-light-running. Sight distance restrictions of this type are often caused by sharp curvature (horizontal or vertical) or foliage from trees adjacent to the street. Vehicles parked adjacent to the traffic lanes can also obstruct the driver's view of the signal heads when these heads are pole-mounted on the far right- or left-side corners of the intersection.

Improve Visibility of Signal

Several changes can be made to improve the visibility of the yellow and red signal indications. Countermeasures include: increasing the lens size to 12 inches, adding supplemental signal heads, adding a second red indication to each head, and adding LED signal indications. Polanis (*25*) investigated the safety effects of the first three countermeasures listed. His investigation took place in the city of Winston-Salem, North Carolina. His data were obtained from "before" and "after" periods of approximately equal durations, varying from 36 to 48 months each. He found that right-angle crashes were reduced by all three countermeasures by an amount ranging from 33 to 47 percent.

Increase Conspicuity of Signal

Two countermeasures were identified that could be used to improve the conspicuity of the signal head. These countermeasures include: adding a strobe light in the signal indication and adding back plates to the signal heads.

The strobe light is a horizontal bar or halo-shaped bulb positioned across the middle of the red signal lens. It flashes a white light about 60 times per minute while the red indication is lit. Cottrell (*26*) studied the use of a horizontal strobe light in the red signal indication at six intersections in Virginia. He collected crash data for three years before and three years after installation of the strobe lights. In spite of finding an overall reduction in right-angle crashes, the variation of change among sites led him to conclude that there was "no evidence that strobe lights were consistently effective in reducing accidents" (*26*, p. 40).

Polanis (*25*) investigated the effect of signal head back plates on crash frequency. His study included six intersections in one city. He found that right-angle crashes were reduced 32 percent by the use of back plates.

Add Advance Warning Signs

The basic advance warning sign is intended to forewarn drivers that they are approaching a signalized intersection. This sign uses the "Signal Ahead" (symbolic) message. It is sometimes accompanied by flashing beacons to ensure that drivers will detect and interpret the sign's meaning. Polanis (*25*) evaluated the effectiveness of this sign at 11 intersections and found it reduced right-angle crashes by 44 percent.

Another type of advance warning sign is the "Be Prepared to Stop When Flashing" sign. This sign has the beacons flashing during the last few seconds of green. It is sometimes referred to as an "advance warning sign with active flashers." In this mode, the flashing is intended to indicate to the driver when the signal indication is about to change from green to yellow. This sign has been found to be particularly effective at reducing red-light-running and associated crashes.

Two studies have been conducted for the purpose of evaluating the effectiveness of the advance warning sign with active flashers. Agent and Pigman (*27*) compared the frequency of red-light-running found at 16 intersections with no advance warning signs with that found at two intersections equipped with signs and active flashers. One hundred signal cycles were observed at each intersection. They found that the intersections with the active flashers had 67 percent fewer red-light-runners than those with no signs. A more recent study by Farraher et al. (*5*) at one intersection found that the active flashers reduced red-light-running by 29 percent.

Remove Unneeded Signals

Retting et al. (*1*) have noted that red-light-running can be eliminated at low-volume intersections by removing the traffic signal. They evaluated the impact of signal removal at 199 intersections in Philadelphia and found that crashes were reduced by 24 percent. Signal removal is a viable countermeasure if the signal can be removed without degrading the operation or safety of the intersection. It is particularly attractive if the red-light-running occurs on the side street and is believed to be a result of excessive delay to side-street vehicles.

Add Capacity with Additional Traffic Lanes

Drivers have been observed to run the red indication when congestion is present and traffic queues are not fully served at the end of the phase. In these situations, drivers in the queue continue to enter the intersection for several seconds after the onset of the red indication. Logically, they are motivated to run the red indication out of a desire to avoid the delay associated with waiting for the next green indication. In some instances, changes to the signal phasing can improve this situation. However, additional capacity, in the form of additional traffic lanes at the intersection, is often the only viable solution.

Flatten Sharp Curves

Sharp curvature can also complicate the intersection environment and lead to higher levels of red-light-running. Sharp crest vertical curvature on the intersection approach can limit sight distance to the intersection ahead. In such instances, drivers may not have adequate time to detect and react to the signal indication. Sharp horizontal curvature through the intersection places excessive demands on the driver. The need to safely negotiate the curvature typically takes priority over the driver's monitoring of the signal indication and can precipitate frequent red-light-running.

Enforcement Countermeasures

Enforcement countermeasures require the use of police presence or some type of automated monitoring system. Police presence has been shown to have a significant short-term effect but is costly to sustain and any ensuing police chases may present a danger to bystanders. Automated enforcement typically uses a camera located on the intersection approach and connected electronically to the signal controller. A recent review of the effectiveness of such camera systems by Retting et al. (*1*) indicates that they have the potential to reduce right-angle crashes by 32 to 42 percent. One drawback of automated systems is that they cannot be used to identify the offending driver–just the vehicle. The legal implications of this characteristic have prevented some states from using automated systems. More importantly, survey results reported by Retting et al. (*1*) indicate that almost one-third of the U.S. drivers are strongly opposed to the use of automated systems.

CHAPTER 3. MODEL DEVELOPMENT AND COUNTERMEASURE SELECTION

OVERVIEW

This chapter describes the development of a model to predict the frequency of red-light-running. This model describes the factors that contribute to red-light-running and the nature of their contribution. The model is used to identify the most promising engineering countermeasures.

MEASURES OF EFFECTIVENESS

A review of the literature indicates that several measures are used to quantify red-light-running events. The more commonly used measures include: "percent of cycles with one or more red-light-runners," "hourly red-light-running rate," and "percent of vehicles that run the red." These measures are all based on the frequency of red-light-running, as normalized by exposure or location. Table 3-1 identifies the family of frequency-based measures related to red-light-running.

Table 3-1. Red-Light-Running-Related Measures of Effectiveness.

Incident	Frequency-Based Measure	Exposure [1, 2]	Location
Entry during yellow interval.	1. Vehicles entering during the yellow interval.	...per hour ...per cycle ...per vehicle	...per lane ...per approach ...per intersection
	2. Cycles with one or more entries on yellow.		
Entry during red interval (RLR).	3. Vehicles entering during the red interval.		
	4. Cycles with one or more entries on red.		
	5. Vehicles in intersection after end of all-red.		
	6. Vehicles entering in first "X" seconds of red.		
Conflict due to RLR.	7. Vehicle-vehicle conflict.		

Notes:
1 - "per vehicle" relates to the total number of vehicles counted for the subject location.
2 - If the numerator and denominator have common units (e.g., cycles with one or more entries per cycle), then the ratio is often multiplied by 100 and expressed as a percentage.

The second column in Table 3-1 lists the measures that can be used to quantify the red-light-running problem at an intersection. Each of these measures can be converted into a rate-based measure by dividing the frequency measure by an exposure factor. Three factors are listed in column 3 of Table 3-1. For example, Measure 3 can be reported as a rate in terms of "vehicles running the red per hour," "vehicles running the red per cycle," or "vehicles running the red per total vehicles." Frequencies or rates can be quantified for a given lane, approach, or for the overall intersection.

Measures 1 and 7 are included in Table 3-1 because they also provide some measure of driver behavior at the end of the phase. The former provides information about the driver's propensity to enter the intersection after the yellow is presented. Logically, large rates for this measure would correlate with large red-light-running rates. The conflict rate is also a useful measure as it combines the behavior of drivers on the subject approach with those on the conflicting approaches. Of those listed, this measure is likely to have the best correlation with the red-light-running-related crash rate.

MODEL DEVELOPMENT

This section describes the development of a model for predicting the frequency of red-light-running. The model is based on the probability of a driver stopping (following the onset of the red indication) when that driver is *t* seconds travel time from the stop line. This probability reflects the travel time of the last driver that decides to go and the first driver that decides to stop. It is represented mathematically as a probability distribution due to differences among drivers.

Different probability distributions have been used to represent the probability-of-stopping relationship. Sheffi and Mahmassani (*15*) used the normal distribution. Bonneson et al. (*16*) used the logistic distribution. The latter distribution is attractive because its cumulative form exists as a closed-form equation whereas that for the normal distribution requires a cumbersome integration. The logistic distribution is represented by the following equation:

$$p_{stop}(t) = \frac{1}{1 + e^{(\alpha - t)/\beta}} \qquad \textbf{(1)}$$

where,

$p_{stop}(t)$ = probability of stopping in response to the yellow indication when at a given travel time t;
t = travel time to the stop line at the onset of yellow, s;
α = shift parameter (equals the travel time at which the probability of stopping is 0.5), s; and
β = shape parameter, s^{-1}.

The complement to the probability of stopping is the "probability of going." This latter probability can be computed as:

$$p_{go}(t) = 1 - p_{stop}(t) \qquad \textbf{(2)}$$

where, $p_{go}(t)$ = probability of going in response to a yellow indication when at a given travel time t. The probability of going is illustrated in Figure 3-1 as it relates to travel time.

Shown at the bottom of Figure 3-1 is a schematic of an intersection approach with two vehicles. The travel time and travel distance axes are related by the approach speed. The figure shows each vehicle's location at the onset of the yellow indication. The probability curve in Figure 3-1 indicates that the driver nearest the stop line has a 75 percent chance of going. If this driver should choose to go, he or she will legally enter the intersection during the yellow indication. In contrast, the more distant driver has a 5 percent chance of going. If this more distant driver should go, he or she will run the red indication as the yellow interval is shorter than his or her travel time to the intersection.

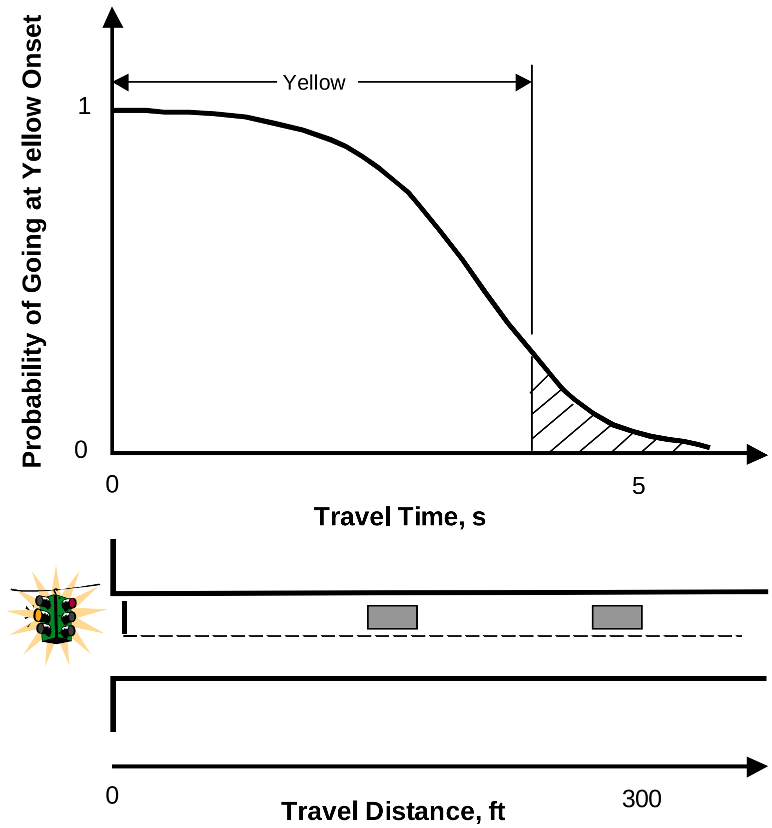

Figure 3-1. Probability of Going at Yellow Onset.

The probability-of-going distribution can be used to develop an equation for predicting the expected number of red-light-runners per cycle. It is developed by integrating the product of the traffic flow rate and the probability of going over all travel times in excess of the yellow duration. This formulation is based on the assumption that each driver, in a given lane, decides to go (or stop) independently of any preceding driver. This assumption is relatively strong when the headway between vehicles in the same lane exceeds 4 s. More complicated formulations would be required to address the joint probability of there being two or more drivers within, say, 4 s travel time of the intersection at the onset of yellow and their respective decisions to stop or go.

It was noted in the previous paragraph that the integration should be over all travel times in excess of the *yellow* duration. However, if advance detection is used on the subject intersection approach *and* the approach phase does not terminate by max-out, then the area monitored by the advance detectors is ensured to be free of vehicles at the onset of yellow. In this case, the interval of integration should include only those travel times in excess of the quantity D/v where D is the distance between stop line and the most distant upstream detector and v is vehicle speed.

The red-light-running model is based on the development of a mathematical relationship that combines the first five events listed in Table 2-1 and the concepts shown in Figure 3-1. The basic form of the red-light-running model is:

$$E[R] = m \int_{v=0}^{\infty} \int_{t=T}^{\infty} p(v)\ p_{go}(t)\ q(t)\ dt\ dv \qquad \textbf{(3)}$$

with,

$$T = p_x Y + (1 - p_x) \max\!:\left[Y, \frac{D}{v}\right] \qquad \textbf{(4)}$$

where,

$E[R]$ = expected red-light-running frequency, veh/h;
m = number of signal cycles per hour (= 3600/C), cycles/hr;
v = vehicle speed on the intersection approach, ft/s;
C = cycle length, s;
T = travel time before red-light-running can occur, s;
$p(v)$ = probability of speed v;
$q(t)$ = flow rate t seconds travel time from the stop line at the onset of yellow, veh/s;
p_x = probability of phase termination by max-out (= 1.0 if the movement is pretimed or if it is actuated but does not have advance detection);
Y = yellow interval duration, s;
max $[a,b]$ = the larger of variables a and b; and
D = distance between stop line and the most distant upstream detector, ft.

To simplify the mathematics of Equations 3 and 4, it can be assumed that the effect of speed variation is negligible and that the average speed V can be used to represent the entire traffic stream. This simplification eliminates the first integration and leaves the following model form:

$$E[R] = m \int_{t=T}^{\infty} p_{go}(t)\ q(t)\ dt \qquad \textbf{(5)}$$

with,

$$T = p_x Y + (1 - p_x) \max\!:\left[Y, \frac{D}{V}\right] \qquad \textbf{(6)}$$

where, V is the average running speed; ft/s. If the subject intersection approach is pretimed, then p_x = 1.0 and the integral in Equation 5 is bound to the interval from $t = Y$ to $t = \infty$. If the subject approach is actuated and Y is larger than the travel time D/V, then the integral in Equation 5 is also bound to the interval from $t = Y$ to $t = \infty$. Values of D/V for stop-line-only detection are typically less than 2.0 s; values for approaches with advance detection are typically between 5.5 and 6.5 s.

The integral in Equation 5 computes the expected number of vehicles running the red at the end of a signal phase for a given intersection approach. The product "$q(t)\ dt$" in the integral represents the count of vehicles during a given time dt. The term $p_{go}(t)$ represents the probability of these vehicles "going" if they are t seconds from the stop line. The integral sums all such possible events for the approach (i.e., for all lanes). For the typical range of yellow interval durations, it is reasonable to assume that $q(t)$ is equal to the average approach flow rate q (i.e., $q(t) = q$).

Equations 1, 2, and 5 can be combined and integrated to yield the following generalized form of the red-light-running model:

$$E[R] = \frac{Q}{C}\frac{1}{b_1}\ln\left[1 + e^{(b_0 - b_1 T + b_2 x_2 + \ldots + b_n x_n)}\right] \qquad \textbf{(7)}$$

where,

$E[R]$ = expected red-light-running frequency, veh/h;

Q = approach flow rate (= $q \times 3600$), veh/h;

$\ln[x]$ = natural log of x;

x_i = variables describing selected traffic and geometric characteristics; and

b_i = regression coefficients, i = 0, 1, 2, ..., n.

Regression constants have been substituted in Equation 7 for the shift and shape parameters from Equation 1 (i.e., $\alpha = (b_0 + b_2 x_2 + \ldots + b_n x_n) \times \beta$ and $\beta = 1/b_1$).

In the literature, two of the more commonly used measures of effectiveness are "percent of vehicles running the red" and "percent of cycles with one or more red-light-runners." Both of these measures can be computed (and related to) the expected red-light-running frequency $E[R]$. For example, the "percent of vehicles running the red" $P_{V,RLR}$ on a given intersection approach can be computed as:

$$P_{V,\,RLR} = 100\ \frac{E[R]}{Q} \qquad \textbf{(8)}$$

If it is assumed that, for a given lane, only one vehicle runs the red per cycle when there is a red-light-runner, then the "percent of cycles with one or more red-light-runners" $P_{C,RLR}$ on an approach can be computed as:

$$P_{C,\,RLR} = 100\left[1 - \left(1 - \frac{E[R]}{n\ m}\right)^{n}\right] \qquad \textbf{(9)}$$

where, n = the number of approach lanes.

COUNTERMEASURES TO BE EVALUATED

Based on a review of the countermeasures listed in Table 2-4 and discussions with engineers in Texas, it was determined that five countermeasures would be most appropriate for further study. These countermeasures are:

- increase the yellow interval duration,
- improve signal coordination,
- improve signal operation,
- improve visibility of signal through use of back plates, and
- improve conspicuity of signal through use of LED indications.

Several countermeasures listed in Table 2-4 were not selected. For example, the literature review indicated that pre-yellow information (e.g., flashing green indication for last few seconds of green) led to an increase in rear-end crashes, so this measure was ruled out. A similar conclusion was reached regarding the use of strobe lights.

Providing green-extension through advance detection was ruled out because this detection mode was more suitable for rural intersections (which was beyond the scope of the research project). "Improving sight distance" is a viable countermeasure but its application was determined to require an excessive amount of effort to find urban intersections with sight restrictions along the approach. Removal of unneeded signals is also a viable countermeasure but represents a small subset of the intersections with a red-light-running problem. Finally, adding lanes and flattening curves was ruled out due to the significant cost associated with their implementation.

CHAPTER 4. STUDY SITE SELECTION AND DATA COLLECTION

OVERVIEW

This chapter describes the field study sites, the criteria used for their selection, and the field data collection plan. This plan describes the procedures used in the data collection process. Data were collected to evaluate the effectiveness of selected engineering countermeasures and to quantify the relationship between red-light-running and crash frequency. A study "site" is defined to be one signalized intersection approach.

The data collection plan represents a hybrid design that combined both a cross-section study and a before-after study. The objective of the cross-section study was to quantify the effect of various factors (e.g., area population, speed, grade, yellow duration) on the frequency of red-light-running. The objective of the before-after study was to quantify the effect of selected engineering countermeasures. Both studies used the same study sites and database. Thus, the focus of site selection was to identify sites suitable for the before-after study and to determine which sites collectively offered a range of factors for the cross-section study.

SITE SELECTION

Site Selection Criteria

This section describes the criteria used to select the field study sites. Preliminary analysis indicated that a minimum of 20 study sites would be needed to provide the necessary data. The criteria used for site selection included the following items:

- Collectively, the study sites should reflect a range of yellow and all-red interval durations.
- Collectively, the study sites should represent small, medium, and large Texas cities.
- Collectively, the study sites should represent approach grades from −5 to +1 percent.
- Collectively, the study sites should represent speed limits from 30 to 50 mph.
- Approaching drivers should have a clear view of the signal heads for 7-s travel time.
- Intersections should be in an urban or suburban area.
- Crash history for the previous three years should be available.
- Intersection skew angle should be less than 5 degrees.
- There should be a minimum approach flow rate of 400 veh/hr/lane during the peak hour.

Satisfaction of these criteria was considered a desirable goal rather than a requirement because of the difficulty of finding sites that could satisfy all criteria within the time schedule of the project. The selection process was guided by a search for "typical" intersections that were not previously identified as having a problem with red-light-running.

Study Site Characteristics

The field study sites were identified through a series of activities. Initially, the research team solicited the names of potential study sites from the members of the project monitoring committee. This solicitation was followed by telephone contacts with traffic engineers in several Texas cities. Finally, the research team members added the names of several potential study sites from their own experiences that satisfied the selection criteria.

The research team visited each potential site and made measurements of its size and traffic control characteristics. For each study site, a traffic engineer with the agency responsible for the site was contacted to solicit his or her interest in participating in the project. Finally, the attributes of the candidate study sites were reviewed and 20 sites were selected. These sites are found at 10 intersections in five Texas cities. The characteristics of each intersection are listed in Table 4-1.

Table 4-1. Intersection Characteristics.

City	Intersection[1]	Characteristic			
		Study Sites[2] (Approach)	Cycle Length[3], s	Advance Detection	Enforcement Lights?
Mexia	Bailey St. (F.M. 1365) & Milam St. (U.S. 84)	EB, WB	75	No	No
	S.H. 14 & Tehuacana Hwy. (S.H. 171)	EB, WB	37-66	No	No
College Station	Texas Ave. (S.H. 6) & G. Bush Dr. (F.M. 2347)	NB, SB	89-131	No	No
	College Main & University Dr. (F.M. 60)	EB, WB	110	No	No
Richardson	Plano Road & Belt Line Road	SB, EB	75-108	No	Yes
	Greenville Ave. & Main Street	SB, EB	69-111	No	Yes
Corpus Christi	F.M. 2292 & S.H. 44	EB, WB	57-156	Yes	No
	U.S. 77 & F.M. 665 (City of Driscoll)	NB, SB	42-86	Yes	No
Laredo	Loop 20 & Los Presidentes	NB, SB	90	No	No
	U.S. 83 & Prada Machin	NB, SB	53-90	Yes	No

Notes:
1 - North-south street is listed first.
2 - A "site" is defined as one intersection approach. NB: northbound; SB: southbound; EB: eastbound; WB: westbound.
3 - Cycle length range represents the 15th and 85th percentile values observed at the site on one day.

The five cities included in the study collectively represent a wide range in population. This characteristic facilitated the examination of "small town" versus "big city" driver behavior. The city of Driscoll (near Corpus Christi) has a population of 811 persons, the city of Mexia has a population of 7000, and the combined cities of Bryan/College Station have a population of 107,000. The city of Laredo has a population of 181,000 and the city of Corpus Christi has a population of 276,000. Finally, the city of Richardson is in the Dallas/Fort Worth metropolitan area which has a population of about three million persons.

The City of Richardson has an active red-light-running enforcement program that uses enforcement lights on the signal poles to help police officers determine the status of the red indication from a strategic position downstream from the intersection. It should be noted that this program has been in place for more than three years and its "novelty" effect was considered to be negligible. As such, it was not believed to have an effect on the proposed study findings (enforcement activities were not in progress during any study).

At each intersection listed in Table 4-1, two intersection approaches were selected for field study and crash history analysis. The study site (i.e., intersection approach) characteristics are listed in Table 4-2. The data in this table indicate that the study sites collectively offer a reasonable range of speeds, grades, all-red interval durations, and signal supports.

Table 4-2. Study Site Characteristics.

City	Study Site	Characteristic					
		Speed Limit, mph	Approach Lanes	Grade, [1] %	Clearance Length, ft [2]	All-Red Interval, s	Signal Support
Mexia	EB Milam St.	35	2	-2.8	70	1.0	Mast arm
	WB Milam St.	35	2	+2.8	70	1.0	Mast arm
	EB S.H. 171	30	1	-0.5	93	1.0	Span wire
	WB S.H. 171	30	1	0.0	93	1.0	Span wire
College Station	NB Texas Ave.	40	3	0.0	95	1.0	Mast arm
	SB Texas Ave.	40	3	-0.5	102	2.0	Mast arm
	EB University Dr.	35	3	+0.5	67	1.0	Mast arm
	WB University Dr.	35	3	+0.2	63	1.0	Mast arm
Richardson	SB Plano Road	40	3	+0.5	102	2.0	Mast arm
	EB Belt Line Road	35	3	0.0	145	2.5	Mast arm
	SB Greenville Ave.	30	3	+0.5	94	2.0	Mast arm
	EB Main Street	30	2	0.0	98	2.0	Mast arm
Corpus Christi	EB S.H. 44	50	2	0.0	95	2.0	Span wire
	WB S.H. 44	50	2	0.0	95	2.0	Span wire
	NB U.S. 77	40	2	+0.3	90	2.1	Span wire
	SB U.S. 77	40	2	0.0	90	2.1	Span wire
Laredo	NB Loop 20	40	2	-1.8	89	1.0	Mast arm
	SB Loop 20	40	2	+0.9	89	1.0	Mast arm
	NB U.S. 83	55	2	+1.5	98	2.0	Mast arm
	SB U.S. 83	55	2	-1.3	98	2.0	Mast arm

Notes:
1 - Grade: plus (+) grades are upgrades in a travel direction toward the intersection.
2 - Length of the clearance path measured from the near-side stop line to the far-side stop line.

Each study site was reviewed to determine the most appropriate type of red-light-running countermeasure for that location. This determination was made based on discussions with the traffic engineer responsible for the intersection, operating conditions at the intersection, and the visibility of the signal indications. Also, one site in each city was unchanged so it could serve as a control (or comparison) site. This control site ensured that any observed countermeasure effect was "adjusted" to reflect only the changes in red-light-running that occurred as a result of the countermeasure. The countermeasure for each site is identified in Table 4-3.

Table 4-3. Countermeasure Implemented at Each Study Site.

City	Study Site	Countermeasure
Mexia	EB Milam St.	Added LED lighting to all signal indications
	WB Milam St.	Added LED lighting to all signal indications
	EB S.H. 171	Control approach (no changes)
	WB S.H. 171	Control approach (no changes)
College Station	NB Texas Ave.	Increased yellow interval duration by 1.5 s
	SB Texas Ave.	Control approach (no changes)
	EB University Dr.	Increased yellow interval duration by 0.8 s
	WB University Dr.	Increased yellow interval duration by 0.9 s
Richardson	SB Plano Road	Increased yellow interval duration by 0.6 s
	EB Belt Line Road	Increased yellow interval duration by 0.6 s
	SB Greenville Ave.	Increased yellow interval duration by 0.6 s
	EB Main Street	Control approach (no changes)
Corpus Christi	EB S.H. 44	Increased yellow interval duration by 0.5 s [1]
	WB S.H. 44	Added back plates & increased yellow interval duration by 0.5 s
	NB U.S. 77	Increased cycle length by 20 s (via longer minimum green)
	SB U.S. 77	Increased cycle length by 20 s (via longer minimum green)
Laredo	NB Loop 20	Increased cycle length by 10 s and improved coordination [2]
	SB Loop 20	Increased cycle length by 10 s and improved coordination [2]
	NB U.S. 83	Added back plates & yellow LED lighting to all signal heads
	SB U.S. 83	Control Approach (no changes)

Notes:
1 - This approach was planned to serve as a control approach; however, the yellow interval was increased by local engineers. This change was not a requested countermeasure.
2 - Loop 20 at Los Presidentes. Northbound and southbound yellow intervals were reduced by 0.7 and 0.5 s, respectively, by local engineers. These reductions were not requested countermeasures.

There were two unintended deviations from the planned set of countermeasures. One deviation was that the control site was eliminated in Corpus Christi. Specifically, the eastbound approach on S.H. 44 was planned to be the control site; however, the yellow interval duration was found to have been changed sometime prior to the "after" study. The second deviation was the reduction in yellow duration on both Loop 20 approaches in Laredo. The consequence of this change is that the benefits of the countermeasures implemented at this intersection are confounded by the combination of changes that occurred at this intersection.

Table 4-4 lists several additional characteristics of the study sites. The information in this table is categorized by "before" and "after" study period because several of the characteristics were changed between periods, as identified in Table 4-3. The characteristics in Table 4-4 include information about the signal indications and the signal head supports used at each intersection. These data were included in the database as it was thought that they might have an effect on signal visibility or conspicuity.

The data listed in Tables 4-2 and 4-4 indicate that the study sites collectively offer good representation of typical yellow interval durations, all-red interval durations, speeds, signal displays, and approach traffic lanes. The range of values for grade was not as broad as desired. However, this lack of breadth was not seen as a limitation because steep downgrades are not commonly found in Texas.

DATA COLLECTION PLAN

The data collection plan consisted of two activities. The first activity relates to the field study of the sites described in the preceding section. The second activity relates to the assembly of crash records for the three most recent years at each study site. The types of data collected and the methods used to collect these data are described in the remainder of this section.

The field study consisted of a before-after study of two sites (i.e., approaches) at each of 10 intersections. One or more of the five countermeasures identified in the previous chapter were implemented at most of the sites. Those sites for which a countermeasure was not implemented were used as control sites. The "after" study was scheduled to take place no sooner than two (and preferably six) months after implementation of the countermeasure. This approach was intended to minimize the countermeasure's novelty effect on driver behavior.

Field Data Collection

The field study of each site included the collection of a wide range of geometric, traffic flow, traffic control, and operational characteristics. These data were collected using a variety of methods including videotape recorders, laser speed guns, and site surveys. The data collected during each field study and the methods of collection are listed in Table 4-5.

Table 4-4. Study Site Characteristics by Study Period.

City	Study Site	Study Period	Characteristic[1]					
			Yellow Interval, s	Yellow LED	Red LED	Back Plate	Span Wire	Ave. Cycle Length, s
Mexia	EB Milam St.	Before	4.0	no	YES	YES	no	75
		After	4.0	**YES**	YES	YES	no	56
	WB Milam St.	Before	4.0	no	YES	YES	no	74
		After	4.0	**YES**	YES	YES	no	56
	EB S.H. 171	Before	4.0	no	no	YES	YES	55
		After	4.0	no	no	YES	YES	52
	WB S.H. 171	Before	4.0	no	no	YES	YES	53
		After	4.0	no	no	YES	YES	52
College Station	NB Texas Ave.	Before	3.5	no	no	YES	no	110
		After	**5.0**	no	no	YES	no	117
	SB Texas Ave.	Before	3.5	no	no	YES	no	110
		After	3.5	no	no	YES	no	117
	EB University Dr.	Before	3.2	no	no	no	no	110
		After	**4.0**	no	no	no	no	111
	WB University Dr.	Before	3.2	no	no	no	no	110
		After	**4.1**	no	no	no	no	111
Richardson	SB Plano Road	Before	4.4	no	no	YES	no	93
		After	**5.0**	no	no	YES	no	102
	EB Belt Line Road	Before	4.0	no	no	YES	no	92
		After	**4.6**	no	no	YES	no	101
	SB Greenville Ave.	Before	3.6	no	no	YES	no	89
		After	**4.2**	no	no	YES	no	85
	EB Main Street	Before	3.7	no	no	YES	no	88
		After	3.7	no	no	YES	no	84
Corpus Christi	EB S.H. 44	Before	4.2	no	no	no	YES	107
		After	**4.7**	no	no	no	YES	127
	WB S.H. 44	Before	4.2	no	no	no	YES	108
		After	**4.7**	no	no	**YES**	YES	121
	NB U.S. 77	Before	4.0	no	YES	no	YES	64
		After	4.0	no	YES	no	YES	**82**
	SB U.S. 77	Before	4.0	no	YES	no	YES	62
		After	4.0	no	YES	no	YES	**83**
Laredo	NB Loop 20	Before	4.5	no	no	YES	no	90
		After	3.8	no	no	YES	no	**100**
	SB Loop 20	Before	4.5	no	no	YES	no	90
		After	4.0	no	no	YES	no	**100**
	NB U.S. 83	Before	5.1	no	no	no	no	72
		After	5.0	**YES**	no	**YES**	no	76
	SB U.S. 83	Before	5.1	no	no	no	no	80
		After	4.9	no	no	no	no	80

Note:
1 - Underlined responses indicate one of the countermeasures identified in Table 4-3.

Table 4-5. Database Elements and Data Collection Method.

Category	Data Type	Data Collection Method		
		Reduced from Videotape	Site Survey	Agency Files
Geometric Characteristics	Number and width of intersection traffic lanes		✔	
	Distance to adjacent signalized intersections		✔	
	Approach grade		✔	
	Photo log		✔	
Traffic Flow Characteristics	Traffic movement counts	✔		
	Flow rate at the end of the phase	✔		
	Heavy-vehicle percentage	✔		
Traffic Control Characteristics	Speed limit		✔	
	Phase sequence		✔	
	Yellow interval duration		✔	
	All-red clearance interval duration		✔	
Operational Characteristics	Cycle length	✔		
	Average running speed		✔	
	Count of yellow-light-runners	✔		
	Count of red-light-runners	✔		
Safety Characteristics	Overall crash frequency during past 3 years			✔
	Average daily traffic volume by leg			✔

During the study of each site, one videotape recorder was positioned upstream of the intersection such that its field-of-view included the appropriate signal heads and all lanes of the subject approach. Typically, the recorder was located about 150 ft upstream of the subject stop line. The data were extracted from the videotape during its replay in the office. A sample of the vehicle speeds on the intersection approach in the subject lanes was also taken while the approach was being videotaped. Speeds were measured only for those vehicles unaffected by signal-related queues. This speed was intended to represent that of vehicles on the approach at the onset of the yellow indication. Data were collected for the through traffic movements and any turn movements that shared a through traffic lane.

Observations during the field studies indicated that flow rates varied considerably during the signal cycles, often due to upstream signalization. At some intersections, the platoons of traffic created by these upstream signals would often arrive near the end of the phase. When this occurred, the propensity for red-light-running appeared to be higher than at intersections of similar volume but with random arrivals or with platoons arriving nearer to the start of green. To facilitate an examination of the effect of platoon arrival on red-light-running, the flow rate at the end of the phase was included in the database. This flow rate was estimated by counting the vehicles arriving to the intersection during the last 8.0 s of the phase’s green indication.

Crash Data Collection

The crash data collection activity consisted of the acquisition of historical crash records for each intersection included in the field studies. To facilitate the analysis, computerized databases were requested from the Texas Department of Public Safety and the appropriate city agencies. The request was for the most recent 36 months for which complete information was available. These data were requested for all four approaches to each intersection. They were used to quantify the relationship between red-light-running and crash frequency.

A traffic engineer at each agency responsible for a study intersection was contacted to gather additional information needed for the crash data analysis. Specifically, inquiry was made regarding the history of the study site for the purpose of ruling out crash data during months (or years) during which the geometry or control mode was not consistent with that of the existing intersection. Also, the average daily traffic volume for each intersection leg was obtained from this contact.

CHAPTER 5. DATA ANALYSIS

OVERVIEW

This chapter summarizes an analysis of the causes and effects of red-light-running. Specifically, the analysis examined three issues: (1) the factors associated with red-light-running, (2) the effectiveness of selected countermeasures, and (3) the effect of red-light-running on crash frequency. The first issue is addressed through the process of identifying useful variables for inclusion in the red-light-running model, as described in Chapter 3. The second issue is addressed through an analysis of before-after data. The third issue is addressed through an analysis of crash data, and the correlation between these data and red-light-running frequency. The first two analyses are based on field data and are included in the next section. The third analysis is based on crash data and is described in the last section of this chapter.

ANALYSIS OF FIELD DATA

This section describes the findings from an investigation of the factors associated with red-light-running and the effectiveness of selected countermeasures. The findings presented are the result of a statistical analysis of a red-light-running database assembled for this research. Initially, the database content is summarized and reviewed for the existence of basic cause-and-effect relationships. Then, the red-light-running model is calibrated and used to examine the sensitivity of red-light-running to various traffic, signalization, and geometric characteristics. Finally, the effectiveness of selected engineering countermeasures are evaluated.

Database Summary

The database assembled for this research includes the traffic, geometric, and control characteristics for 10 intersections in Texas. Two approaches were studied at each intersection. Traffic data recorded at each intersection included: vehicle count and classification for each signal cycle, cycle length, number of red-light-running vehicles per cycle, average running speed, and flow rate at the end of the phase. Details of the geometric and control characteristics recorded for each site are provided in Chapter 4.

Descriptive Statistics

Summary statistics describing the variables in the database are provided in Tables 5-1, 5-2, and 5-3. The data in these tables reflect six hours of data collection at each intersection approach. The data in Table 5-1 indicate that more than 10,018 signal cycles were observed at 20 intersection approaches. During these cycles, 586 vehicles entered the intersection (as defined by the stop line) after the change in signal indication from yellow to red.

Table 5-1. Database Summary - Total Observations.

City	Study Site	Study Period	Cycles	Heavy Vehicles [1]		All Vehicles [1]	
				Observations	RLR	Observations	RLR
Mexia	EB Milam St.	Before	285	198	4	2526	13
		After	384	334	3	2477	9
	WB Milam St.	Before	288	206	2	2728	9
		After	377	277	4	2276	14
	EB S.H. 171	Before	281	26	0	509	0
		After	405	22	0	572	3
	WB S.H. 171	Before	386	55	0	1073	1
		After	407	36	0	1132	4
College Station	NB Texas Ave.	Before	195	128	1	6801	24
		After	179	181	0	7530	7
	SB Texas Ave.	Before	191	120	0	6372	10
		After	179	201	0	7808	35
	EB University Dr.	Before	194	91	1	5732	33
		After	189	186	0	7070	12
	WB University Dr.	Before	195	220	4	5546	60
		After	190	321	6	6630	35
Richardson	SB Plano Road	Before	231	208	0	4673	4
		After	208	182	0	5341	4
	EB Belt Line Road	Before	232	73	0	3382	6
		After	210	87	0	4540	1
	SB Greenville Ave.	Before	234	72	0	1764	3
		After	250	66	0	1478	2
	EB Main Street	Before	243	134	0	5179	25
		After	250	144	2	4711	15
Corpus Christi	EB S.H. 44	Before	193	289	3	3683	16
		After	162	501	1	3359	7
	WB S.H. 44	Before	192	261	1	3768	11
		After	148	364	0	3899	10
	NB U.S. 77	Before	326	523	5	2484	15
		After	255	807	12	2603	21
	SB U.S. 77	Before	339	522	2	2314	13
		After	250	638	4	2353	15
Laredo	NB Loop 20	Before	234	252	5	2625	14
		After	205	316	4	2787	22
	SB Loop 20	Before	235	188	2	2946	9
		After	197	270	1	2748	29
	NB U.S. 83	Before	299	236	3	2209	20
		After	275	431	9	2461	23
	SB U.S. 83	Before	266	210	3	2135	9
		After	259	336	2	2578	23
		Total:	10,018	9712	84	142,802	586

Note:
1 - "All Vehicles" include both passenger cars and heavy vehicles. A "heavy vehicle" is defined as any vehicle with more than four tires on the pavement, with the exception of a 1-ton pickup truck with dual tires on the rear axle (this truck was considered to be a "passenger car").

Table 5-2. Database Summary - Speed Statistics and Yellow Intervals.

City	Study Site	Study Period	Running Speed			Yellow Interval, s [1]		
			Obser-vations	Average, mph	85^{th} %, mph	Observed	Computed	Difference (Y_{obs}-Y_{comp})
Mexia	EB Milam St.	Before	100	35.8	39.0	4.0	4.1	-0.1
		After	100	35.1	39.0	4.0	4.1	-0.1
	WB Milam St.	Before	100	33.6	37.0	4.0	3.5	0.5
		After	100	30.8	35.2	4.0	3.4	0.6
	EB S.H. 171	Before	20	31.9	37.0	4.0	3.8	0.2
		After	32	32.7	38.4	4.0	3.9	0.1
	WB S.H. 171	Before	36	31.4	35.8	4.0	3.6	0.4
		After	34	32.3	36.0	4.0	3.6	0.4
College Station	NB Texas Ave.	Before	100	41.2	45.2	3.5	4.3	-0.8
		After	100	37.6	42.0	5.0	4.1	0.9
	SB Texas Ave.	Before	100	39.0	43.0	3.5	4.2	-0.7
		After	99	42.2	47.0	3.5	4.5	-1.0
	EB University Dr.	Before	100	32.6	37.0	3.2	3.7	-0.5
		After	100	32.6	37.0	4.0	3.7	0.3
	WB University Dr.	Before	100	34.3	39.0	3.2	3.8	-0.6
		After	100	35.3	40.0	4.1	3.9	0.2
Richardson	SB Plano Road	Before	81	41.0	45.0	4.4	4.3	0.2
		After	100	37.7	42.0	5.0	4.0	1.0
	EB Belt Line Road	Before	93	37.9	42.0	4.0	4.1	-0.1
		After	100	34.7	38.2	4.6	3.8	0.8
	SB Greenville Ave.	Before	41	31.1	36.0	3.6	3.6	0.0
		After	100	34.9	39.0	4.2	3.8	0.4
	EB Main Street	Before	100	28.5	32.0	3.7	3.4	0.4
		After	100	27.9	32.0	3.7	3.4	0.4
Corpus Christi	EB S.H. 44	Before	100	48.8	53.0	4.2	4.9	-0.7
		After	100	48.2	53.0	4.7	4.9	-0.2
	WB S.H. 44	Before	100	49.9	56.0	4.2	5.1	-0.9
		After	100	47.4	55.0	4.7	5.0	-0.3
	NB U.S. 77	Before	100	36.7	42.0	4.0	4.1	-0.1
		After	100	34.9	41.0	4.0	4.0	0.0
	SB U.S. 77	Before	100	39.1	42.0	4.0	4.1	-0.1
		After	100	39.4	44.0	4.0	4.2	-0.2
Laredo	NB Loop 20	Before	100	45.5	50.2	4.5	4.9	-0.4
		After	100	45.7	52.0	3.8	5.1	-1.3
	SB Loop 20	Before	100	43.7	50.0	4.5	4.6	-0.1
		After	100	46.1	51.2	4.0	4.7	-0.7
	NB U.S. 83	Before	100	51.9	60.2	5.1	5.2	-0.1
		After	100	50.6	58.2	5.0	5.1	-0.1
	SB U.S. 83	Before	100	50.8	57.0	5.1	5.4	-0.3
		After	100	51.0	58.0	4.9	5.4	-0.5

Note:

1 - "Computed": Based on the following equation, as described by Kell and Fullerton (28): $Y = t + V/(2\,a + 64\,G)$ where, Y = yellow interval, t = driver perception-reaction time (= 1.0 s), a = deceleration rate (10 ft/s^2), and G = approach grade (ft/ft).

Table 5-3. Database Summary - Statistics for Selected Variables.

Variable	Statistic [1]			
	Average	Std. Deviation	Minimum	Maximum
Approach Flow Rate, veh/h	612	351	75	1551
Cycle Length, s	89	23	47	161
Yellow Interval Duration, s	4.2	0.57	3.2	5.1
Running Speed, mph	39	6.9	28	52
Clearance Path Length, ft	92	17	63	145
Platoon Ratio	1.65	0.82	0.48	5.4

Note:
1 - With two exceptions, statistics are based on six observations of the subject variable for each study site–one observation for each of the six study hours. Running speed statistics are based on the observation of 3636 individual vehicle speeds. Clearance path length is based on one measurement at each of the 20 intersection approaches.

Of the 586 vehicles observed to run the red indication, 84 were heavy vehicles and 502 were passenger cars. Overall, 0.86 percent (= 84 /9712 × 100) of heavy vehicles violated a red indication and 0.38 percent (= 502/133,090 ×100) of passenger cars violated the red indication. A paired test of these two proportions indicates that their difference is significantly different from zero (p = 0.0001). From this test, it can be concluded that heavy vehicle operators are perhaps twice as likely to run the red indication as passenger car drivers. Zegeer and Deen (*10*) also found that heavy vehicles were more than twice as likely to run the red indication during a study of two high-speed rural intersections.

Table 5-2 illustrates the range of speeds found at the study sites. These speeds represent "running speeds" because they were collected after the queue had cleared following the start of each phase. They were obtained during a spot speed survey conducted simultaneously with the collection of the red-light-running data. An analysis of speed variance indicated that 100 speed observations would yield an estimate of the mean running speed with a precision of ±1 mph or less with a 95 percent confidence level. At a few locations, there was insufficient traffic volume to obtain 100 observations of running speed during the 6-hour study.

Table 5-2 also compares the yellow interval observed at each site with that computed using an equation offered by Kell and Fullerton (*28*). Differences between the observed and computed yellow intervals ranged from −1.3 s to 1.0 s (= $Y_{observed} - Y_{computed}$). Prior to the implementation of countermeasures, 75 percent of the sites had yellow intervals that were within ±0.5 s of those recommended by the equation. After the yellow intervals were changed, there were only 65 percent of the sites within ±0.5 s of the equation.

Table 5-3 summarizes the statistics associated with selected traffic characteristics included in the database. In general, these statistics indicate that there is a wide range of flow rates, speeds, yellow-interval durations, and cycle lengths represented in the database.

The "platoon ratio" listed in the last row of Table 5-3 represents the ratio of the flow rate at the end of the phase to the average flow rate. Observations during the field studies indicated that red-light-running appeared to be more frequent at intersections with platoons arriving near the end of the green indication.

Measures of Effectiveness

Red-Light-Running Rates. As a first step in the analysis of the data, red-light-running rates were computed for each intersection approach. Two rates were computed. The first rate is expressed in terms of red-light-running events per 1000 vehicles. The second rate represents the number of red-light-running events per 10,000 vehicle-cycles where, "cycles" represents the *average* number of cycles per hour during the period for which vehicles are counted. The use of "vehicle-cycles" is supported by the ratio *Q/C* that appears in Equation 7. Both rates are listed in Table 5-4.

The overall average rates are 4.1 red-light-runners per 1000 vehicles and 1.0 red-light-runners per 10,000 vehicle-cycles. The former rate is similar to that found in the literature. Specifically, data reported by Kamyab et al. (*7*) indicate an average rate of 3.0 red-light-runners per 1000 vehicles. Data reported by Baguley (*8*) indicate an average rate of 5.3 red-light-runners per 1000 vehicles.

The red-light-running rates listed in Table 5-4 provide some indication of the extent of red-light-running at the intersections studied. The vehicle-based rates listed in column 7 indicate that one approach in College Station and four in Laredo greatly exceed the average rate of 4.1 red-light-runners per 1000 vehicles. The rates listed in column 8 also indicate that four approaches in Laredo and one in College Station exceed the average rate of 1.0 red-light-runners per 10,000 veh-cycles. However, there is a small discrepancy as to which approaches are the most problematic. This discrepancy illustrates the importance of considering both volume and number-of-cycles when computing the red-light-running rate. The vehicle-cycle-based rate represents a more reliable measure of the propensity for red-light-running than the vehicle-based rate.

Entry Time of the Red-Light-Running Driver. The time after the end of the yellow indication at which the red-light-runner enters the intersection is logically correlated with the potential for a right-angle collision. As this "time into red" increases, crash frequency is also likely to increase. For example, drivers who run only the first 0.5 s of red are not likely to be involved in a crash as the conflicting vehicles typically have not yet begun to move into the intersection (especially if there is an all-red interval or 0.5 s or more). On the other hand, drivers entering the intersection 2.0 s after the end of the all-red interval are very likely to collide with a crossing vehicle.

The extent to which a red-light-runner enters after the end of the yellow interval is shown in Figure 5-1. Also shown is the extent to which a red-light-runner enters after the end of all-red. In the few instances where more than one vehicle ran the red for a given signal phase, only the "time of entry" for the *last* vehicle to run the red indication was recorded in the database. Hence, the trends in Figure 5-1 are based on the 541 phases where at least one vehicle ran the red. In 60 of these 541 phases, the last red-light-runner to enter did so after the end of the all-red interval.

Table 5-4. Red-Light-Running Rates at Each Study Site.

City	Study Site	Study Period	Total Observations			Red-Light-Running Rate [1]	
			Vehicles	Cycles	RLR	RLR per 1000 vehicles	RLR per 10,000 veh-cyc
Mexia	EB Milam St.	Before	2526	285	13	5.1	1.1
		After	2477	384	9	3.6	0.6
	WB Milam St.	Before	2728	288	9	3.3	0.7
		After	2276	377	14	6.2	1.0
	EB S.H. 171	Before	509	281	0	0.0	0.0
		After	572	405	3	5.2	0.8
	WB S.H. 171	Before	1073	386	1	0.9	0.1
		After	1132	407	4	3.5	0.5
College Station	NB Texas Ave.	Before	6801	195	24	3.5	1.1
		After	7530	179	7	0.9	0.3
	SB Texas Ave.	Before	6372	191	10	1.6	0.5
		After	7808	179	35	4.5	1.5
	EB University Dr.	Before	5732	194	33	5.8	1.8
		After	7070	189	12	1.7	0.5
	WB University Dr.	Before	5546	195	60	**10.8**	**3.3**
		After	6630	190	35	5.3	1.7
Richardson	SB Plano Road	Before	4673	231	4	0.9	0.2
		After	5341	208	4	0.7	0.2
	EB Belt Line Road	Before	3382	232	6	1.8	0.5
		After	4540	210	1	0.2	0.1
	SB Greenville Ave.	Before	1764	234	3	1.7	0.4
		After	1478	250	2	1.4	0.3
	EB Main Street	Before	5179	243	25	4.8	1.2
		After	4711	250	15	3.2	0.8
Corpus Christi	EB S.H. 44	Before	3683	193	16	4.3	1.4
		After	3359	162	7	2.1	0.8
	WB S.H. 44	Before	3768	192	11	2.9	0.9
		After	3899	148	10	2.6	1.0
	NB U.S. 77	Before	2484	326	15	6.0	1.1
		After	2603	255	21	8.1	1.9
	SB U.S. 77	Before	2314	339	13	5.6	1.0
		After	2353	250	15	6.4	1.5
Laredo	NB Loop 20	Before	2625	234	14	5.3	1.4
		After	2787	205	22	7.9	**2.3**
	SB Loop 20	Before	2946	235	9	3.1	0.8
		After	2748	197	29	**10.6**	**3.2**
	NB U.S. 83	Before	2209	299	20	**9.1**	1.8
		After	2461	275	23	**9.3**	**2.0**
	SB U.S. 83	Before	2135	266	9	4.2	1.0
		After	2578	259	23	**8.9**	**2.1**
Total Observations & Average Rates:			142,802	10,018	586	4.1	1.0

Note:
1 - Values in **bold** font exceed the average rate by a factor of 2.0 or more.

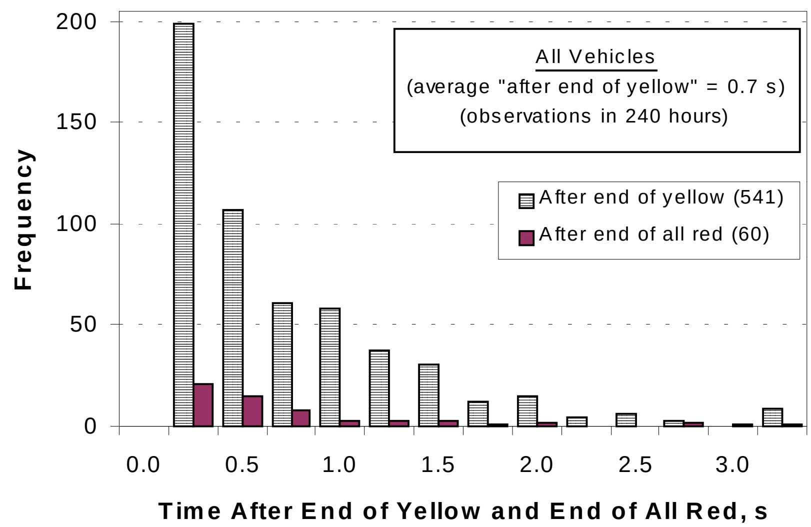

Figure 5-1. Frequency of Red-Light-Running as a Function of Time into Red.

The trends in Figure 5-1 indicate that more than half of the red-light-running occurred in the first 0.5 s of red. The average red-running driver entered about 0.7 s after the end of the yellow interval. About 80 percent of the drivers entered within 1.0 s after the end of the yellow, which is consistent with the trend reported by Farraher et al. (*5*). Also shown in this figure is the frequency of red-light-running based on the time after the end of the all-red interval. The most flagrant driver entered 14 s after the all-red interval ended. About 80 percent of these drivers entered within 1.0 s after the end of the all-red interval.

The frequency of red-light-running for heavy vehicles was also examined. The trends were similar to those shown in Figure 5-1. The one exception is that about 80 percent of the heavy vehicle drivers entered within 1.7 s after the end of the yellow interval (as compared to 1.0 s for all vehicles combined). The latter statistic suggests that some heavy vehicle drivers are less likely (or able) to stop at the onset of yellow than are passenger car drivers.

There is speculation among engineers that lengthy yellow intervals may be abused by some drivers (*9*). Specifically, it is believed that drivers adapt to an increase in the yellow duration and continue to run the red indication with the same frequency as before the increase. This issue was evaluated by increasing the yellow duration at seven of the 20 study sites (increases ranged from 0.5 to 1.5 s with an average of 0.8 s). The results of this examination are shown in Figure 5-2.

The trends in Figure 5-2 indicate that the increase in yellow duration had the desired effect of *decreasing* the frequency of red-light-running (from 113 red-runners during the 42 hours of "before" data to 58 red-runners during the 42 hours of "after" data). The average time of entry into the red is the same (0.6 s) for both study periods.

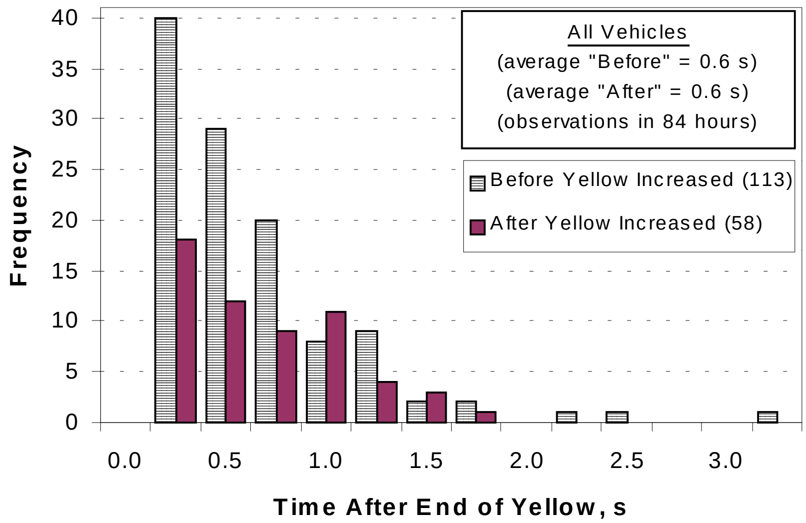

Figure 5-2. Effect of an Increase in Yellow Interval Duration on the Frequency of Red-Light-Running.

The data in Figure 5-2 were examined more closely to explore the effect of an increase in yellow duration on driver behavior. An examination of the "before" data indicated that the increase in yellow at the seven sites should have reduced red-light-running from 113 vehicles to 18 vehicles (i.e., an 84 percent reduction). Yet, after extrapolation of the 113 to an equivalent "after" count (where the number of vehicle-cycles of exposure was increased by 7.2 percent), the actual reduction due to the increased yellow interval was only 52 percent (= 100 − 58/113 × 100/1.072). This finding indicates that drivers do adapt to an increase in yellow duration. However, yellow increases in the range of 0.5 to 1.5 s, that do not yield yellow durations in excess of 5.5 s, are still likely to reduce red-light-running by about 50 percent.

Model Calibration and Sensitivity Analysis

Analysis of Factor Effects

This section describes an analysis of the relationship between red-light-running frequency and selected variables in the database. The analysis considered a wide range of variables. They include: approach flow rate, cycle length, yellow interval duration, heavy-vehicle percentage, running speed, clearance path length, platoon ratio, approach grade, number of approach lanes, LED signal indications, use of signal head back plates, use of advance detection, and signal head mounting. The findings described in this section focus on those variables found to have a visible correlation with red-light-running frequency.

Each variable included in the database represents the total events observed during a one hour period. Given that there were six hours of data for each of 20 approaches and two study periods, there are a total of 240 observations in the database.

The effect of approach flow rate on red-light-running frequency is illustrated in Figure 5-3. This figure indicates that red-light-running frequency increases with increasing flow rate. The pattern in the data indicates that the relationship is linear with negligible red-light-running at zero flow rate.

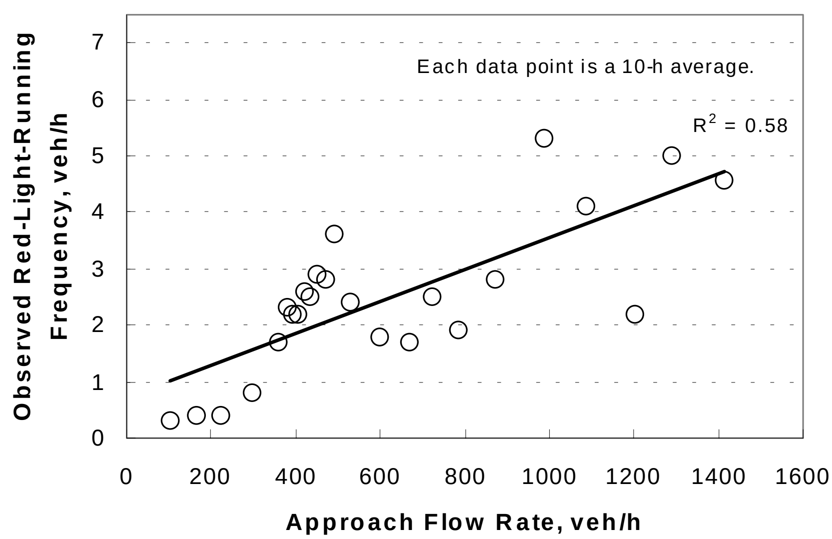

Figure 5-3. Red-Light-Running Frequency as a Function of Approach Flow Rate.

There are only 24 data points shown in Figure 5-3. In fact, each data point in this figure (and in subsequent figures in this section) represents an average over 10 hours. This aggregation was needed because plots with 240 data points tended to obscure the portrayal of trends in the data. To overcome this problem, the hourly data were sorted by the independent variable (e.g., approach flow rate), placed in sequential groups of 10, and averaged over the group for both the independent and dependent variables. This procedure was only used for graphical presentation; the 240 hour-based data points were used for all statistical analyses.

Figure 5-4 presents a comparison of the relationship between approach-flow-rate-to-cycle-length ratio and the frequency of red-light-running. Like that found for approach flow rate, red-light-running frequency increases with increasing flow-rate-to-cycle-length ratio. However, the degree of correlation (i.e., R^2) associated with this ratio is larger than that found for approach flow rate alone indicating that both flow rate and cycle length (or its inverse, number of cycles) are correlated with the frequency of red-light-running.

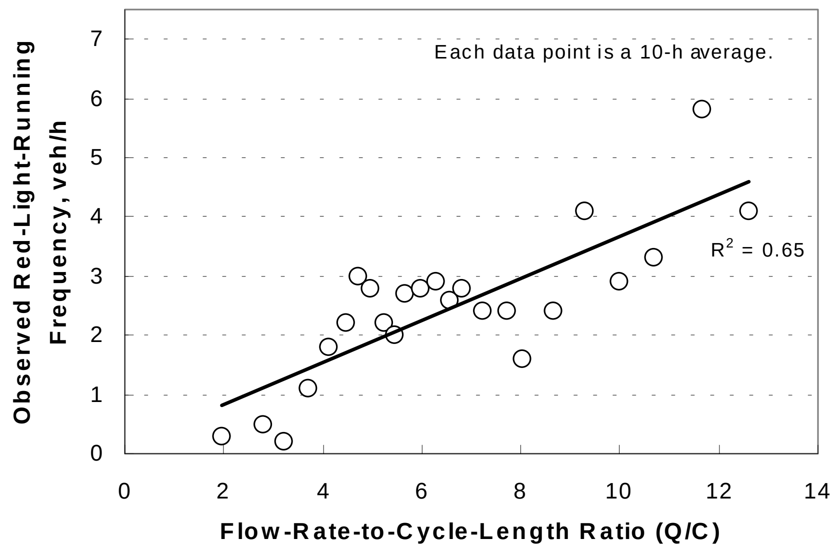

Figure 5-4. Red-Light-Running Frequency as a Function of Flow-Rate-to-Cycle-Length Ratio.

Figure 5-5 illustrates the relationship between yellow interval duration and red-light-running frequency. The best-fit trend line shown suggests a curved relationship where the frequency of red-light-running is relatively low for yellow intervals between 3.8 and 5.0 s. Red-light-running increases significantly for yellow intervals less than 3.5 s. This trend is consistent with that reported by Van der Horst and Wilmink (*9*). The trend line shown also suggests that the frequency of red-light-running increases slightly for yellow interval durations longer than 4.5 s. However, further investigation of the data indicates that this increase is associated with other factor influences.

The relationship between approach running speed and red-light-running frequency is shown in Figure 5-6. The trend suggests that more red-light-running will occur at higher speeds. The scatter in the data suggests that this relationship is not as strong as it is for approach flow rate or yellow interval duration.

The relationship between the length of the clearance path through the intersection and red-light-running frequency is illustrated in Figure 5-7. The trend line shown indicates that red-light-running is less frequent at wider intersections. This trend is logical and suggests that drivers are more reluctant to run the red indication if they believe their "time of exposure" to a right-angle crash is lengthy. This time of exposure directly relates to the time needed by the driver to cross the intersection.

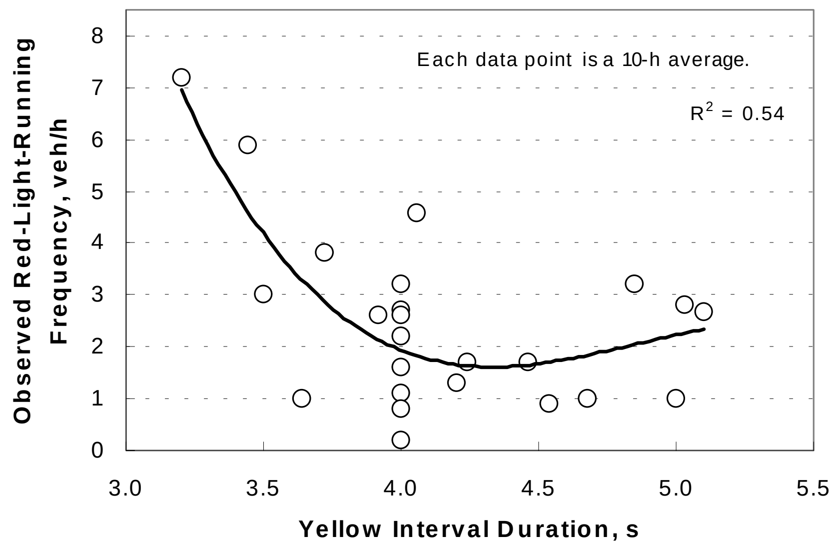

Figure 5-5. Red-Light-Running Frequency as a Function of Yellow Interval Duration.

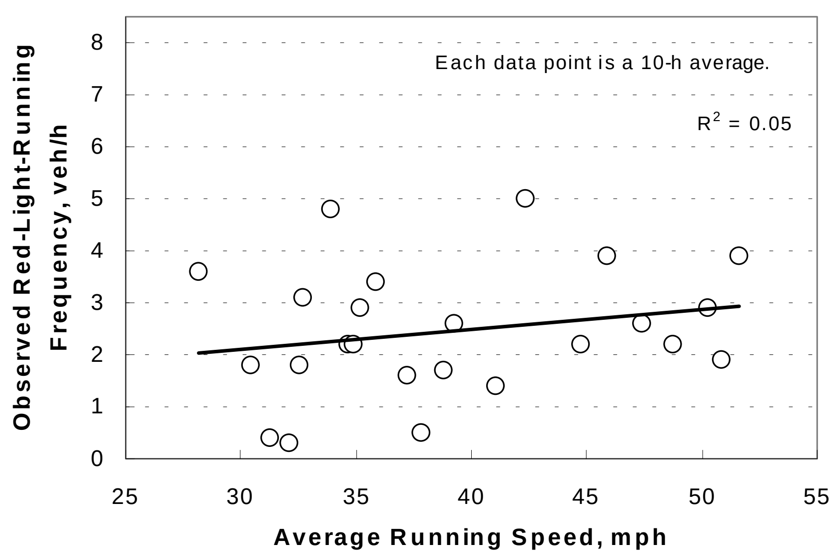

Figure 5-6. Red-Light-Running Frequency as a Function of Speed.

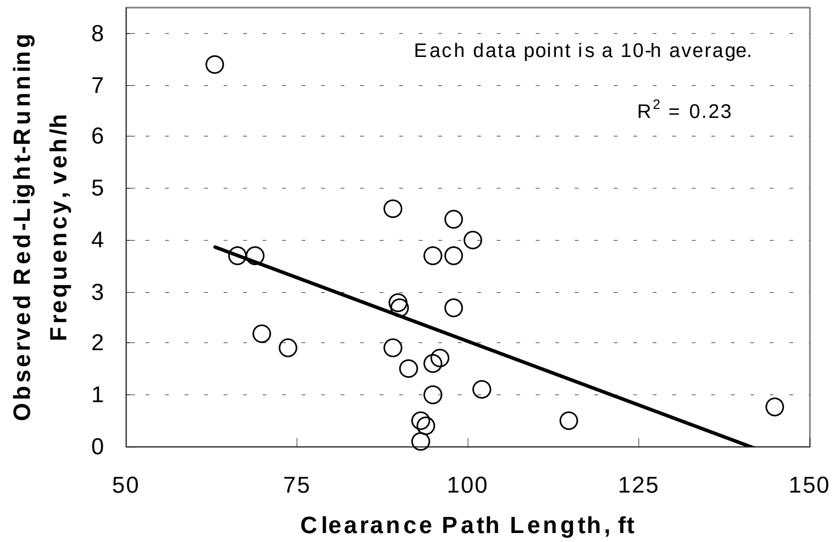

Figure 5-7. Red-Light-Running Frequency as a Function of Clearance Path Length.

The relationship between platoon ratio and the frequency of red-light-running is shown in Figure 5-8. The trend line indicates that red-light-running is more frequent with larger platoon ratios (i.e., more dense platoons). This finding suggests that signal timing plans that concentrate platoon arrivals, especially toward the end of the phase, may increase the frequency of red-light-running.

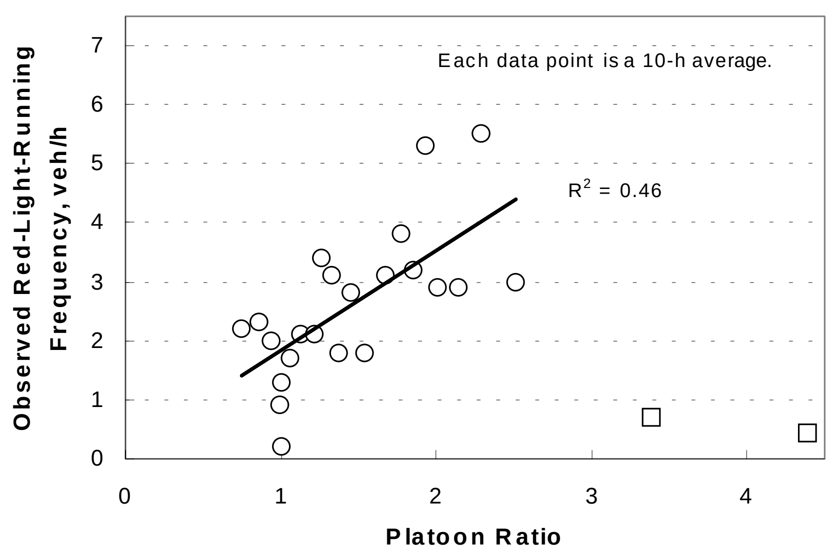

Figure 5-8. Red-Light-Running Frequency as a Function of Platoon Ratio.

Two of the data points in Figure 5-8 (identified using square data points) do not conform to the linear trend shown. These data points are associated with two sites in one city. They also are the only data points to represent exceptionally high platoon ratios. These commonalities leave open the possibility that these two data points represent unusual conditions and should be considered "outliers." It is also possible that, for platoon ratios in excess of 2.5, red-light-running frequency decreases with an increase in platoon ratio. Additional data from sites with platoon ratios in excess of 2.5 would be needed to clarify this issue.

The effect of signal head back plates on the frequency of red-light-running is shown in Figure 5-9. For this examination, the database was aggregated into 40 data points representing the average red-light-running frequency at each of the 20 sites and for each study (i.e., "before" and "after"). The data were then segregated into those site-and-study combinations that had back plates and those combinations that did not have back plates.

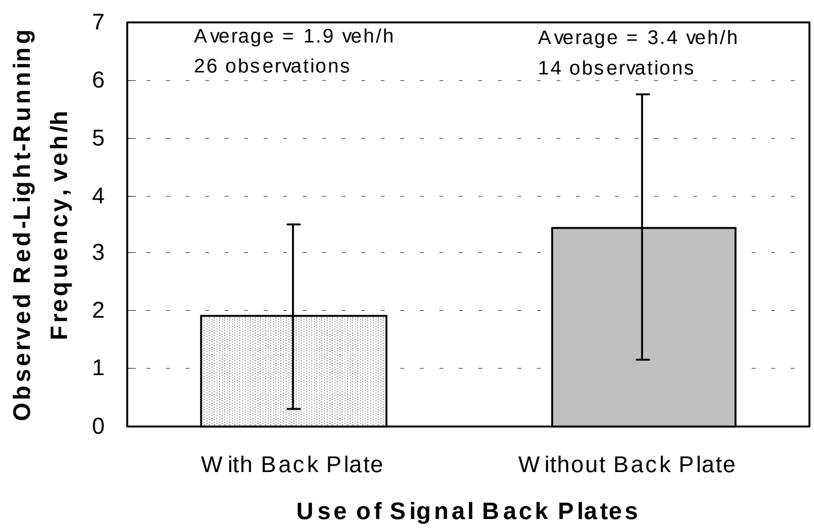

Figure 5-9. Red-Light-Running Frequency as a Function of Back Plate Use.

The information in Figure 5-9 indicates that those sites with back plates have a lower frequency of red-light-running. The average red-light-running frequency for those sites with back plates was 1.9 veh/h compared to 3.4 veh/h for those sites without back plates (i.e., a 44 percent reduction at sites with back plates). The standard deviation among sites is shown by the vertical lines centered on each bar. Although the standard deviation is relatively large, a statistical analysis of the difference between the two averages indicates that it is statistically significant ($p = 0.03$).

Statistical Analysis Method

A preliminary examination of the data indicated that it is neither normally distributed nor of constant variance, as is assumed when using traditional least-squares regression. Under these conditions, the generalized linear modeling technique, described by McCullagh and Nelder (*29*), is appropriate because it accommodates the explicit specification of an error distribution using maximum-likelihood methods for coefficient estimation.

The distribution of red-light-running frequency can be described by the family of compound Poisson distributions. In this context, there are two different sources of variability underlying the distribution. One source of variability stems from the differences in the mean red-running frequency m among the otherwise "similar" intersection approaches. The other source stems from the randomness in red-light-running frequency at any given site, which likely follows the Poisson distribution.

Abbess et al. (*30*) have shown that if event occurrence at a particular location is Poisson distributed then the distribution of events of a group of locations can be described by the negative binomial distribution. The variance of this distribution is:

$$V(x) = E(m) + \frac{E(m)^2}{k} \tag{10}$$

where, x is the observed red-light-running frequency for a given approach having an expected frequency of $E(m)$ and dispersion parameter k.

The Nonlinear Regression procedure (NLIN) in the SAS software (*31*) was used to estimate the red-light-running model coefficients. It was necessary to use this procedure because of the nonlinear form of the model. The "loss" function associated with NLIN was specified to equal the scaled deviance for the negative binomial distribution. The procedure was set up to estimate model coefficients based on maximum-likelihood methods.

The disadvantage of using NLIN is that k must be specified, yet its value has to be computed from the variability in the distribution of residuals. This requirement was overcome through an iterative procedure. Initially, NLIN was used, with k set to equal 1.0, to tentatively calibrate the model and compute a predicted red-light-running frequency for each observation. Then, the Generalized Modeling (GENMOD) procedure in SAS was used to obtain a better estimate of k. Specifically, GENMOD was used to regress the relationship between the predicted and observed red-light-running frequencies (the natural log of the predicted values was specified as an offset variable and the "log" link function was used). GENMOD automates the k-estimation process using maximum-likelihood methods. This new estimate of k from GENMOD was then used in a second application of NLIN and the process repeated until convergence was achieved between the k value used in NLIN and that obtained from GENMOD. Convergence was typically achieved in two iterations.

Model Calibration

The regression analysis revealed that mathematic relationships existed between red-light-running frequency and yellow interval duration, use of signal head back plates, speed, clearance path length, and platoon ratio. The regression coefficient associated with each factor was found to be significant at a level of confidence that exceeded 95 percent. As a result of this analysis, the linear regression terms in Equation 7 were specified as:

$$E[R] = \frac{Q}{C} \frac{1}{b_1} \ln\left[1 + e^{(b_0 - b_1 T + b_2 Bp + b_3 V + b_4 L_p + b_5 R_p)}\right] e^{(b_6 I_C + b_7 I_L)} \qquad \textbf{(11)}$$

with,

$$T = p_x Y + (1 - p_x) \max\!: \left[Y, \frac{D}{V}\right] \qquad \textbf{(12)}$$

The platoon ratio R_p used in Equation 11 represents the ratio of the flow rate at the end of the phase to the approach flow rate. This ratio was computed as:

$$R_p = \frac{Q_e}{Q} \qquad \textbf{(13)}$$

where,

Bp = presence of back plates on the signal heads, (1 if present, 0 if not present);
V = average running speed, mph;
L_p = clearance path length, ft;
R_p = platoon ratio;
I_C = indicator variable for city of Corpus Christi, (1 if data apply to this city, 0 otherwise);
I_L = indicator variable for city of Laredo, (1 if data apply to this city, 0 otherwise); and
Q_e = phase-end flow rate, veh/h.

The regression analysis indicated that the calibrated model could not account for all of the red-light-running observed in two of the five cities included in the study. This finding is likely due to different levels of enforcement and to variation in driver behavior among cities. Two indicator variables were added to the model to account for these differences.

The statistics related to the calibrated red-light-running model are shown in Table 5-5. The calibrated coefficient values can be used with Equations 11, 12, and 13 to predict the hourly red-light-running frequency for a given intersection approach. A dispersion parameter k of 9.0 was found to yield a scaled Pearson χ^2 of 1.04. The Pearson χ^2 statistic for the model is 239 and the degrees of freedom are 231 (= n-p-1 = 239-7-1). As this statistic is less than $\chi^2_{0.05,\,231}$ (= 267), the hypothesis that the model fits the data cannot be rejected. The R^2 for the model is 0.49. An alternative measure of model fit that is better-suited to negative binomial error distributions is R_K^2, as developed by Miaou (*32*). R_K^2 for the calibrated model is 0.83.

Table 5-5. Calibrated Red-Light-Running Model Statistical Description.

Model Statistics		Value		
	R^2 (R_K^2):	0.49 (0.83)		
	Scaled Pearson χ^2:	1.04		
	Pearson χ^2:	239 ($\chi^2_{0.05,\,231}$ = 267)		
	Dispersion Parameter k:	9.0		
	Observations:	239 hours		
	Standard Error:	±1.8 veh/h		
Range of Model Variables				
Variable	**Variable Name**	**Units**	**Minimum**	**Maximum**
Q	Approach flow rate	veh/h	75	1551
C	Cycle length	s	47	161
Y	Yellow interval duration	s	3.2	5.1
V	Average running speed	mph	28	52
L_p	Clearance path length	ft	63	145
R_p	Platoon ratio	--	0.5	5.4
Calibrated Coefficient Values				
Variable	**Definition**	**Value**	**Std. Dev.**	**t-statistic**
b_0	Intercept	2.30	0.74	3.1
b_1	Effect of travel time	0.927	0.155	6.0
b_2	Effect of back plates on signal heads	-0.334	0.150	-2.2
b_3	Effect of running speed	0.0435	0.0155	2.8
b_4	Effect of clearance path length	-0.0180	0.0048	-3.8
b_5	Effect of platoon ratio	0.220	0.100	2.2
b_6	Added effect of City of Corpus Christi	0.745	0.193	3.9
b_7	Added effect of City of Laredo	0.996	0.192	5.2

The regression coefficients for the model are listed in the last rows of Table 5-5. The t-statistic shown indicates that all coefficients are significant at a 95 percent level of confidence or higher. A negative coefficient indicates that red-light-running decreases with an increase in the associated variable value. Thus, approaches with higher speeds are likely to have a higher frequency of red-light-running. Red-light-running is also more frequent on approaches with platoons concentrated near the end of the phase. In contrast, red-light-running is less frequent at intersections with wider cross streets or at those with back plates on the signal heads. Red-light-running is 2.1 and 2.7 (= $e^{0.745}$ and = $e^{0.996}$) times more frequent in the cities of Corpus Christi and Laredo than it is in the other three cities. This trend may be due to a lower level of enforcement of red-light-running in the cities of Corpus Christi and Laredo.

The fit of the model was assessed using the prediction ratios plotted against the predicted red-light-running frequency. The prediction ratio PR_i for intersection approach i represents its residual error standardized (i.e., divided) by the square root of its predicted variance. This ratio is computed as:

$$PR_i = \frac{y_{o,i} - y_{p,i}}{\sqrt{V(x)}} \tag{14}$$

with,

$$V(x)_i = y_{p,i} + \frac{y_{p,i}^2}{k} \tag{15}$$

where,

$V(x)_i$ = variance of red-light-running for sites similar to site i;
$y_{o,i}$ = observed red-light-running frequency for site i; and
$y_{p,i}$ = predicted red-light-running frequency for site i.

The plot of prediction ratios for the study sites is provided in Figure 5-10. The trends shown in this figure indicate the model provides a good fit to the data. In general, the prediction ratios exhibit the desired feature of being centered around zero (i.e., the average *PR* is 0.0). However, it is noted that there is a slight tendency to underestimate the higher observed red-light-running rates (i.e., 10 of the 239 observations exceed a ratio of +2.0). Regardless, the pattern in the data indicates that the ratios are distributed normally about zero and within the range of ±2.0. This trend is the desired result; it is a consequence of the specification of the negative binomial error distribution in the regression model.

A second means of assessing the model's fit is through the graphical comparison of the observed and predicted red-light-running frequencies. This comparison is provided in Figure 5-11. The trend line in this figure does *not* represent the line of best fit; rather, it is a "$y = x$" line. The data would fall on this line if the model predictions exactly equaled the observed data. The trends shown in this figure indicate that the model is able to predict the red-light-running without bias. The scatter in the data suggests that there is still some unexplained variability in the data. In general, the model is able to predict the red-light-running frequency at a given site with a standard error of ±1.8 veh/h.

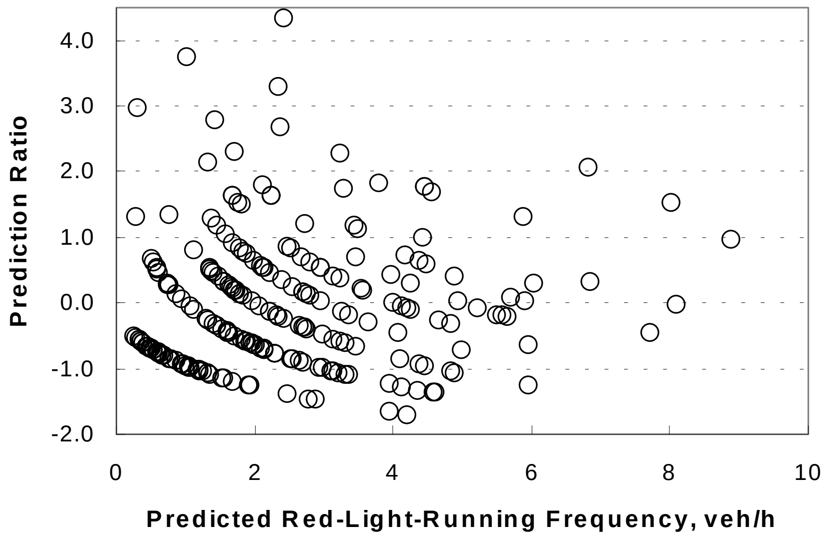

Figure 5-10. Prediction Ratio versus Predicted Red-Light-Running Frequency.

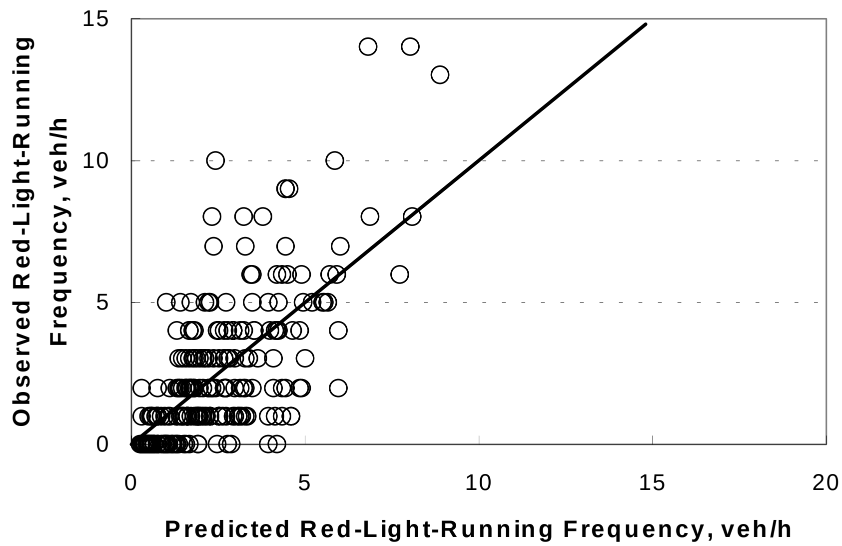

Figure 5-11. Comparison of Observed and Predicted Red-Light-Running Frequency.

The calibrated model can be rewritten to yield the following form:

$$E[R] = \frac{Q}{C}\,\frac{1}{0.927}\,\ln\left[1 + e^{(2.30 - 0.927T - 0.334Bp + 0.0435V - 0.0180L_p + 0.220R_p)}\right] \qquad \textbf{(16)}$$

with,

$$T = p_x Y + (1 - p_x)\max\!:\left[Y, \frac{D}{V}\right] \qquad \textbf{(17)}$$

Computation of the platoon ratio requires knowledge of the phase-end flow rate Q_e. This flow rate can be directly measured by counting the vehicles crossing the stop line in the last 5 to 10 s before the onset of the yellow indication. Alternatively, the platoon ratio can be approximated using guidance in Chapters 10 and 16 of the *Highway Capacity Manual* (*33*) (i.e., Exhibits 10-18 and 16-11, respectively). The information in these tables indicates that R_p has values ranging from 0.33 to 2.0 for very poor to exceptionally good progression, respectively. A value of 1.0 is recommended for isolated intersection approaches.

Given that enforcement levels and driver behavior may vary among some cities, it may be necessary to calibrate Equation 16 to local conditions. This calibration would be accomplished by observing the hourly count of red-light-running x on a few intersection approaches. These approaches would be selected because they have a "typical" level of red-light-running. The model would then be used to estimate $E[R]$ for each approach. The calibration factor C_f would then be computed as the ratio of the total count of red-light-runners to the total expected red-light-running frequency (i.e., $C_f = \Sigma x / \Sigma E[R]$).

Sensitivity Analysis

This section describes a sensitivity analysis of the calibrated red-light-running model. For this analysis, the relative effect of each variable was evaluated in terms of the increase or decrease in red-light-running caused by a change in the variable's value. Each model variable was evaluated separately from the other variables. Thus, the relative effect of a variable was evaluated with the values of the other model variables held fixed. For this analysis, it was assumed that the conditions in Equation 17 were such that the variable T was equal to the yellow interval duration Y (i.e., pretimed or no advance detection).

Analysis Methodology. For a given base variable, the relative effect of a small change (or deviation) from the base value was computed using Equation 16 twice, once using the "new" value and once using the "base" value. The ratio of the expected red-light-running frequencies was then computed as:

$$MF = \frac{E[R]_{new}}{E[R]_{base}} \qquad \textbf{(18)}$$

where, *MF* represents a "modification factor" indicating the extent of the change in red-light-running due to a change in the base value. For example, if Equation 16 is evaluated once for a proposed yellow duration of 4.0 s and again for a base yellow duration of 3.0 s, the resulting *MF* from Equation 18 is about 0.475 (the exact value ranges from 0.450 to 0.500, depending on the value of the other model variables). Thus, the 1.0-s increase in yellow translates into a 52.5 percent (= 100 − 0.475 × 100) reduction in red-light-running.

Two trends emerged during the development of the modification factors. First, the value of the modification factor is *not* dependent on the "base" value. Rather, it is only dependent on the magnitude of the change in values. Thus, a 1.0-s increase in yellow duration yields a *MF* of about 0.47 regardless of whether the change is from 3.0 to 4.0 s, 4.0 to 5.0 s, or any two other values that reflect an increase of 1.0 s.

The second trend that emerged is that the *MF* is largely insensitive to changes in the other variable values. In general, *MF* values vary less than ±5.0 percent for the range of typical values for the other variables. For example, a 1.0-s increase in yellow on a 45-mph approach yields a *MF* of 0.475. If the speed on the approach is 55 mph, then the *MF* is 0.498 (a largely insignificant 5 percent increase relative to the *MF* for 45 mph).

Approach Flow Rate. Equation 16 indicates that the change in red-light-running frequency is directly related to the change in approach flow rate. If the new flow rate increases 50 percent, then so does the frequency of red-light-running. If the new flow rate is one-half of the base rate, then the frequency of red-light-running is also one-half of the base value. The modification factor for flow rate is computed as the ratio of the new and base flow rates (i.e., $MF = Q_{new} / Q_{base}$).

Cycle Length. Equation 16 indicates an inverse relationship between a change in cycle length and the frequency of red-light-running. That is, an increase in cycle length corresponds to a decrease in the frequency of red-light-running. The effect of a change in cycle length is illustrated in Figure 5-12. Trend lines for three "base" cycle lengths are illustrated. A 20-s increase in cycle length from 90 to 110 s corresponds to an *MF* of 0.82 which corresponds to a reduction of 18 percent (= 100 − 0.82 × 100).

Yellow Interval Duration. The effect of a change in yellow interval duration on the frequency of red-light-running is shown in Figure 5-13. The trend in this figure indicates that an increase in yellow interval duration decreases red-light-running. For example, an increase in yellow duration of 1.0 s is associated with an *MF* of 0.47 which corresponds to a 53 percent reduction. This magnitude of reduction was noted previously with regard to the discussion of Figure 5-2.

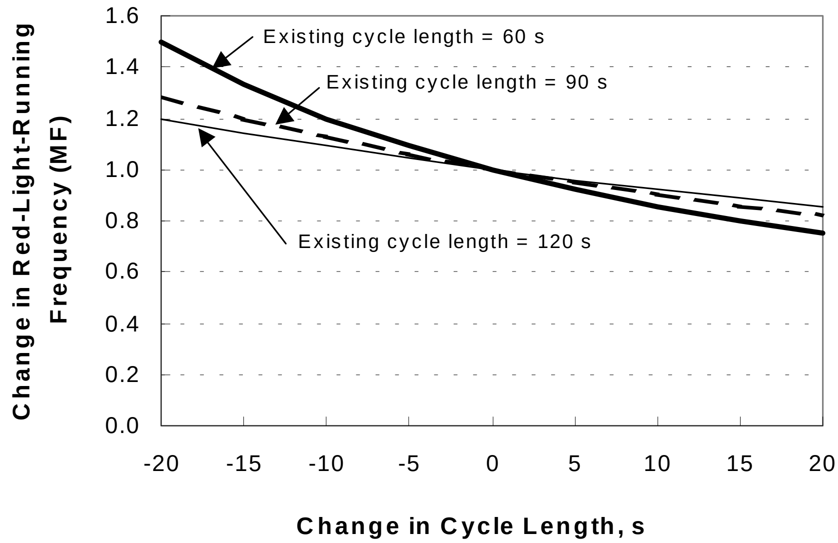

Figure 5-12. Effect of a Change in Cycle Length on Red-Light-Running.

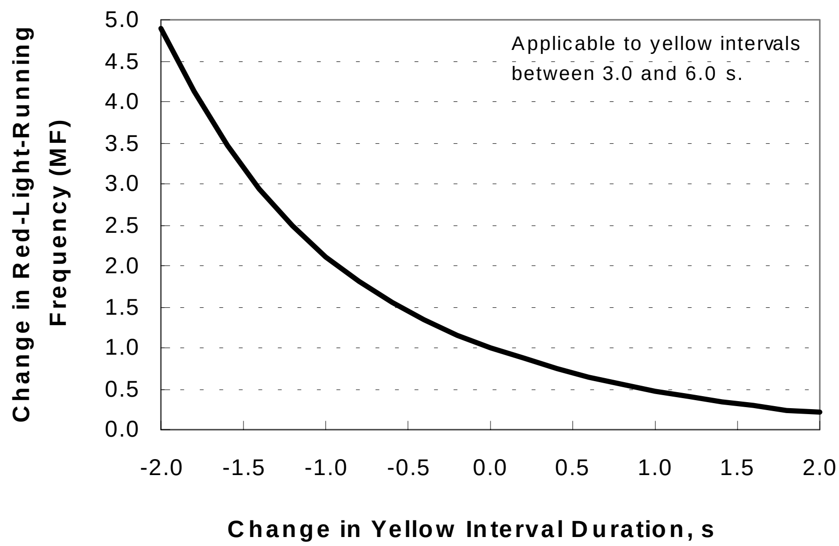

Figure 5-13. Effect of a Change in Yellow Interval Duration on Red-Light-Running.

Average Running Speed. The effect of a change in average running speed on the frequency of red-light-running is shown in Figure 5-14. The trend in this figure indicates that an increase in running speed is associated with an increase in red-light-running. For example, an increase in speed of 10 mph is associated with an *MF* of 1.45 which corresponds to a 45 percent increase in red-light-running.

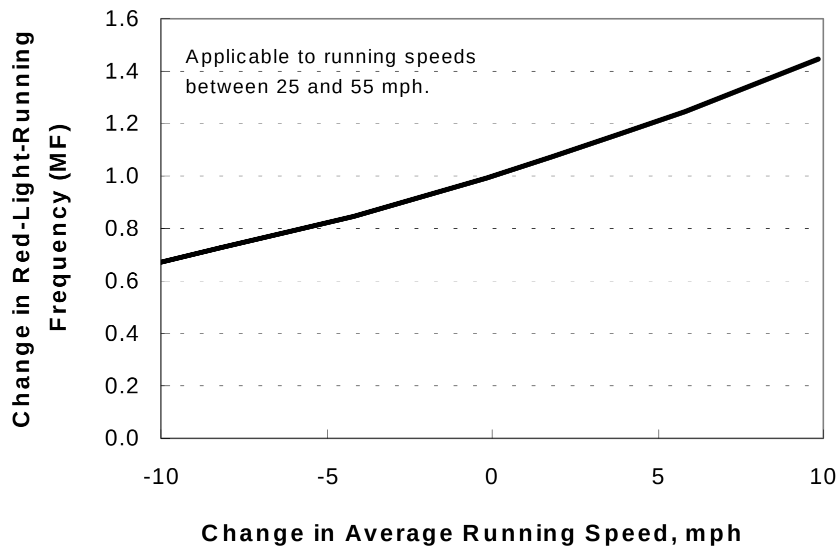

Figure 5-14. Effect of a Change in Average Running Speed on Red-Light-Running.

Length of Clearance Path. The effect of a change in clearance path length on red-light-running is shown in Figure 5-15. The trend in this figure indicates that an increase in path length is associated with a decrease in red-light-running. For example, if approach "A" has a clearance path that is 40 ft longer than approach "B," then its *MF* is 0.55. This value indicates that approach "A" has 45 percent less red-light-running than approach "B" (all other factors being the same).

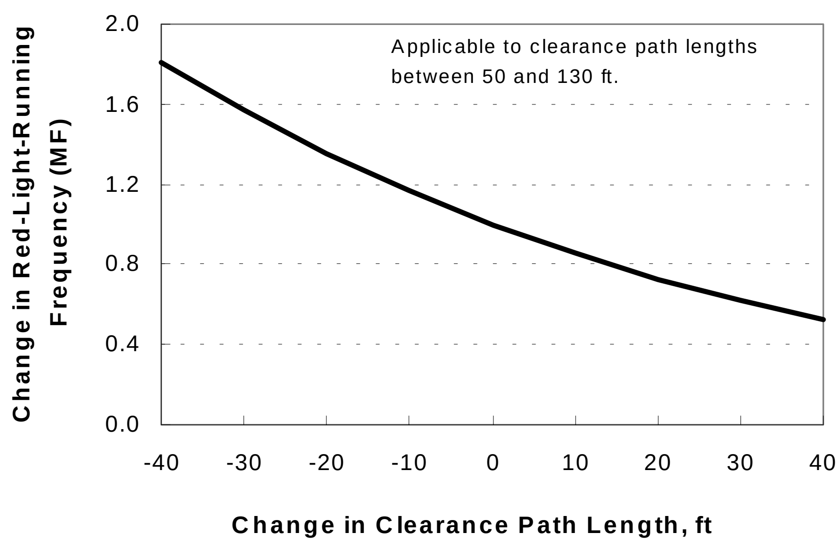

Figure 5-15. Effect of a Change in Clearance Path Length on Red-Light-Running.

Platoon Ratio. The effect of a change in platoon ratio on the frequency of red-light-running is shown in Figure 5-16. The trend in this figure indicates that an increase in platoon ratio is associated with an increase in red-light-running. For example, if efforts to improve signal coordination yield a platoon ratio increase of 1.0, then the *MF* is 1.21 which corresponds to a 21 percent increase in red-light-running.

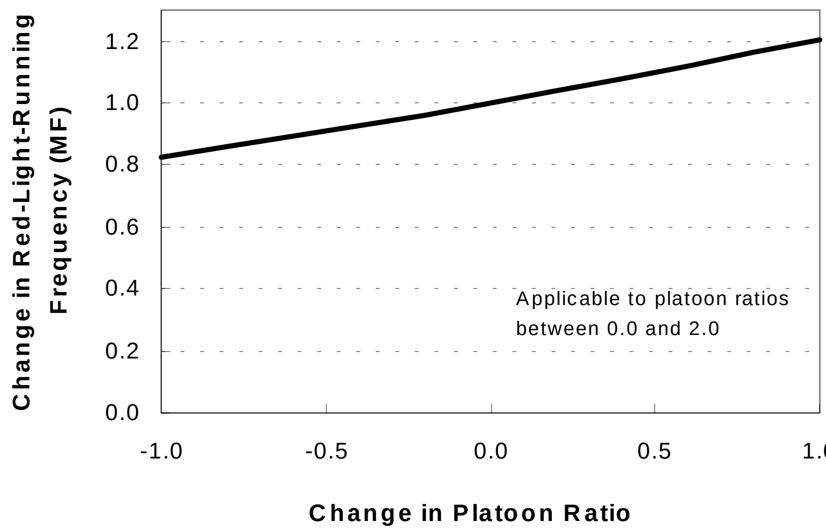

Figure 5-16. Effect of a Change in Platoon Ratio on Red-Light-Running.

Use of Back Plates. An analysis of Equation 16, relative to the use of back plates, suggests that back plates are associated with a lower frequency of red-light-running. The modification factor for adding back plates is 0.75, which corresponds to a reduction of 25 percent. This reduction is slightly lower than the 44 percent value found in the examination of the red-light-running database (as noted in the discussion of Figure 5-9). The modification factor for removing back plates is 1.33 which corresponds to an increase of 33 percent.

Examination of a Common Yellow Interval Equation

This section examines the relationship between red-light-running and a "computed" yellow interval duration. The computed yellow interval is obtained from an equation that is commonly used by engineers (*28*). It is based on 1.0-s reaction time and the time needed to decelerate from the 85th percentile speed at a rate of 10 ft/s^2. The results of this examination are shown in Figure 5-17. The data shown are listed in Table 5-2.

The data in Figure 5-17 indicate that there is a trend toward more red-light-running when the observed yellow duration is shorter than the computed duration. A regression analysis of the relationship between yellow interval difference and red-light-running frequency indicated that the relationship is statistically significant (i.e., $p = 0.001$). A similar finding was previously reported by Retting and Greene (*24*) in an examination of red-light-running at several intersections.

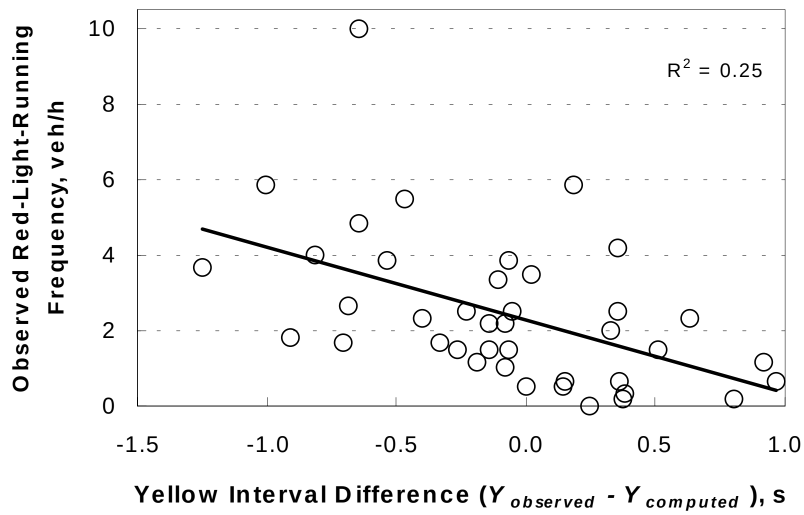

Figure 5-17. Red-Light-Running Frequency as a Function of Yellow Interval Difference.

Figure 5-18 illustrates the effect of yellow interval duration and 85th percentile speed on the frequency of red-light-running. The trend lines are based on the red-light-running model described previously (i.e., Equation 16). The 85th percentile speed was estimated as being 12 percent larger than the average running speed (as needed by the model). This percentage was obtained from a linear regression analysis of the average and 85th percentile speeds listed in Table 5-2 ($R^2 = 0.98$).

The trends in Figure 5-18 indicate that the frequency of red-light-running decreases with an increase in yellow interval duration. They also indicate that, for the same yellow duration, the number of red-light-running events is higher on higher-speed approaches.

The black dots in Figure 5-18 indicate the value of the computed yellow interval duration for the corresponding speed. The equation used to compute this duration is also shown in the figure. The location of the dots suggests that use of this equation yields about 2.0 red-light-runners per hour (or 0.8 red-light-runners per 10,000 vehicles) for the "typical" conditions represented in the figure.

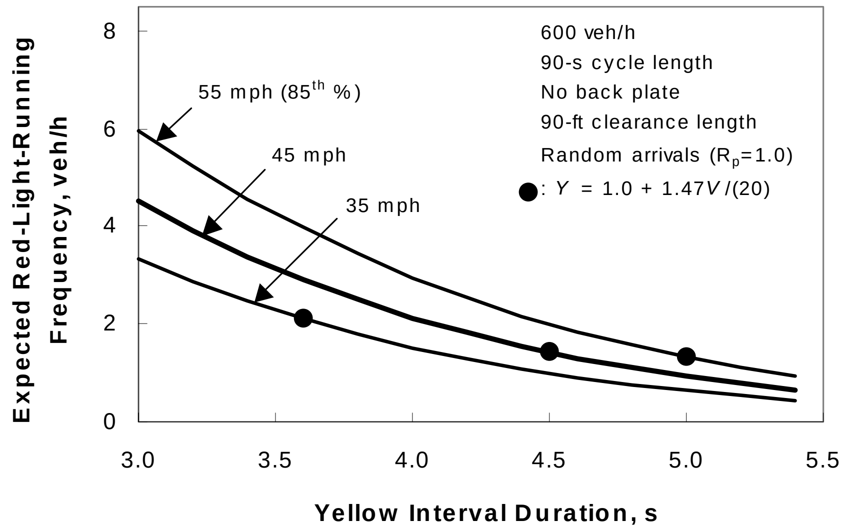

Figure 5-18. Predicted Effect of Yellow Duration and Speed on Red-Light-Running Frequency.

Evaluation of Countermeasures

This section describes the analysis and evaluation of the countermeasures implemented at each of the study sites. Initially, the statistical analysis methods are described. Then, the statistical test for comparing red-light-running frequencies during the "before" and "after" periods is described. Finally, the effects of the various countermeasures are quantified and the findings discussed.

Statistical Analysis Method

Hauer (*34*) and others have observed that intersections selected for safety improvements are often in a class of "high-crash" locations. As a consequence of this selection process, these intersections tend to exhibit significant crash reductions after specific improvements are implemented. While the observed reduction is factual, it is not typical of the benefit that could be derived from the improvement if it were applied to other locations. Hauer (*34*) advocates the use of the empirical Bayes method to more accurately quantify the true crash reduction potential of a specific improvement or countermeasure.

The empirical Bayes method is extended in this research to the analysis of red-light-running frequency. It should be noted that the site selection process was guided by a search for intersections that were not previously identified as having a problem with red-light-running. As such, the study sites were not expected to have abnormally high red-light-running frequencies prior to treatment. Hence, the empirical Bayes method is not required but it is still applied in this research to provide a more defensible position from which to make claims about countermeasure effectiveness.

The empirical Bayes method provides an unbiased estimate of the red-light-running frequency in the "before" period. This estimate is based on a weighted combination of the observed frequency of red-light-running during the "before" period x and the predicted red-light-running frequency $E[R]$, obtained from Equation 16. The estimate obtained in this manner $E[R|x]$ is a more accurate estimate of the expected red-light-running frequency at the subject approach than either of the individual values (i.e., $E[R]$ or x). The following equations were used to compute $E[R|x]$:

$$E[R|x] = E[R] \times weight + \frac{x}{H} \times (1 - weight) \qquad \textbf{(19)}$$

with,

$$weight = \left(1 + \frac{E[R]\, H}{k} \right)^{-1} \qquad \textbf{(20)}$$

where,

$E[R|x]$ = expected red-light-running frequency given that x were observed in H hours, veh/h;
x = observed red-light-running frequency in the "before" period, veh;
H = time interval during which x was observed, h; and
$weight$ = relative weight given to the prediction of expected red-light-running frequency.

Table 5-6 illustrates the application of Equations 19 and 20 to the "before" data. Column 3 lists the count of red-light-runners during the 6-hour study period at each site. Column 4 represents the expected red-light-running frequency $E[R]$ obtained from Equation 16. Specifically, this equation was used to estimate $E[R]$ for each of the six study hours at a given site. The value in column 4 represents an average of the six hourly estimates. The values in columns 5 and 6 are based on the direct application of Equations 20 and 19, respectively.

The estimate $E[R|x]$ is an unbiased estimate of the red-light-running frequency for the "before" study. However, it is still "biased" if compared directly to the observed count of red-light-runners during the "after" period. This bias stems from the fact that conditions (e.g., flow rate, speed, etc.) typically change during the interim between the two studies. In this case, two to six months lapsed between the two studies. To remove this bias, $E[R|x]$ was multiplied by a ratio (= $E[R]^*/E[R]$) that accounts for the change in conditions that have occurred since the "before" study. This computation is illustrated in the following equation:

$$RLR^* = E[R|x] \times \frac{E[R]^*}{E[R]} \times 6.0 \qquad \textbf{(21)}$$

Table 5-6. Expected Red-Light-Running during the "Before" and "After" Periods.

City	Study Site	RLR, veh/6 hours	*E[R]* veh/h	*weight*	*E[R|x]* veh/h	*E[R]** veh/h	RLR* veh/6 hours
Mexia	EB Milam St.	13	1.6	0.48	1.9	2.4	17.1
	WB Milam St.	9	1.6	0.49	1.5	1.9	11.0
	EB S.H. 171	0	0.4	0.80	0.3	0.3	1.5
	WB S.H. 171	1	0.6	0.73	0.5	0.6	2.9
College Station	NB Texas Ave.	24	4.0	0.27	4.0	3.8	22.7
	SB Texas Ave.	10	2.9	0.34	2.1	4.3	18.3
	EB University Dr.	33	6.1	0.20	5.6	8.1	45.1
	WB University Dr.	60	7.1	0.17	9.5	8.7	69.5
Richardson	SB Plano Road	4	1.6	0.49	1.1	1.4	5.8
	EB Belt Line Road	6	0.8	0.66	0.8	0.9	5.9
	SB Greenville Ave.	3	1.2	0.55	0.9	1.4	6.1
	EB Main Street	25	1.9	0.45	3.1	1.9	19.4
Corpus Christi	EB S.H. 44	16	2.2	0.40	2.5	1.7	11.4
	WB S.H. 44	11	2.5	0.38	2.1	2.4	11.9
	NB U.S. 77	15	4.1	0.27	2.9	3.9	16.7
	SB U.S. 77	13	2.0	0.43	2.1	1.9	12.1
Laredo	NB Loop 20	14	2.8	0.35	2.5	3.1	16.4
	SB Loop 20	9	2.8	0.35	1.9	2.9	12.2
	NB U.S. 83	20	2.7	0.36	3.1	2.8	19.7
	SB U.S. 83	9	2.4	0.39	1.8	2.8	12.9

The ratio used in Equation 21 follows the recommendation of Hauer (*34*). The terms include the expected red-light-running frequency in the "before" period *E[R]* and that in the "after" period had no countermeasure been implemented *E[R]**. This ratio is multiplied by the quantity "6.0" to facilitate the before-after comparison of red-light-running during a 6-hour time period. The value obtained from Equation 21 (*RLR**) represents the expected number of red-light-runners that would have occurred in the "after" period had the countermeasure not been applied. This value is listed in the last column of Table 5-6.

Each value of *E[R]** listed in Table 5-6 was derived by using Equation 16 six times–once for each of the six "after" study hours at a given site. With one exception, the flow rate, speed, and other conditions present during each hour of the "after" study were used in Equation 16. The only exception was the variable associated with the countermeasure applied at the site. This variable was unchanged from its condition in the "before" period. For example, at a site where the countermeasure was an "increased yellow duration," *E[R]** was estimated using the flow rate and speed observed during the "after" study and the yellow duration from the "before" study. The value of *E[R]** listed in column 7 represents an average of the six hourly estimates obtained for each site.

Evaluation

The effectiveness of the countermeasures listed in Table 4-3 were evaluated using a "log-odds ratio" test, as described by Griffin and Flowers (*35*). This test compares the ratio of the red-light-running frequency of the "after" study period to that of the "before" period. This ratio is computed for both the sites receiving a countermeasure (i.e., the treated sites) and those not receiving a countermeasure (i.e., the control sites). The ratio of these two ratios represents the "relative change" due to the countermeasure with respect to any change found at the control sites. To determine if this relative change is significant, it can be used to compute a "z-statistic" that follows the standard normal distribution. This statistic can be used to identify the probability of falsely rejecting the null hypothesis (i.e., that there is no change). The results of the analysis are listed in Table 5-7.

The information listed near the bottom of Table 5-7 indicates that, overall, the mix of countermeasures appear to have resulted in a 48 percent reduction in red-light-running. This reduction is statistically significant. It should also be noted that red-light-running *increased* overall by 45 percent at the control sites (= 1.45 × 100 − 100), as indicated by the statistic in column 6.

Several of the countermeasures reduced red-light-running. The use of yellow LEDs in Mexia is associated with a 49 percent reduction in red-light-running. The sample size is too small to be certain that this reduction is significant (i.e., there is a 31 percent chance that more observations would indicate that there is truly no reduction due to the use of LEDs). Nevertheless, it represents the "best estimate" of the effect of this countermeasure.

The increase in yellow duration at the College Station and Richardson sites is associated with a decrease in red-light-running. The last two rows of Table 5-7 combine the data from these two sites. Analysis of this data indicates that a change in yellow duration is associated with a 70 percent reduction in red-light-running. This reduction is statistically significant.

The findings from the study in Corpus Christi were difficult to interpret due to an unexpected deviation from the study plan. The deviation was that an unintended change was made to the yellow duration at the control site prior to the "after" study. The trend in the data associated with this city suggests that the combined use of back plates and increased yellow duration is associated with an 18 percent reduction in red-light-running. The increase in cycle length is associated with a 25 percent increase in red-light-running. This effect of cycle length is contrary to the findings presented in a previous section. However, these trends are likely biased by "regional" changes in red-light-running, as were found in the other cities. Hence, the findings from this city are difficult to properly interpret without data from a control site.

Table 5-7. Countermeasure Effectiveness.

City	Treatment	Observed Hours [1]	RLR Frequency [2]		After/Before Ratio (R) [3]	Relative Change [4] (RC)	z-statistic (z) [p-value] [5, 6, 7]
			"Before" Study	"After" Study			
Mexia	Yellow LEDs	24	28.2	23	0.82	-49	1.01 [0.31]
	Control	24	4.4	7	1.59		
College Station	Increase yellow	36	137.3	54	0.39	-79	**4.79 [0.00]**
	Control	12	18.3	35	1.91		
Richardson	Increase yellow	36	17.9	7	0.39	-49	1.21 [0.23]
	Control	12	19.4	15	0.77		
Corpus Christi	Back plates, inc. yellow	24	23.3	19	0.82	-18	0.66 [0.51]
	Control	--	--	--	--		
	Increase cycle	24	28.8	36	1.25	25	0.89 [0.37]
	Control	--	--	--	--		
Laredo	Improve coordination [8]	24	28.5	51	1.79	0	0.01 [0.99]
	Control	12	12.9	23	1.78		
	Yellow LED+back plates	12	19.7	23	1.17	-35	0.91 [0.36]
	Control	12	12.9	23	1.78		
Overall	Mix	180	283.7	213	0.75	-48	**3.36 [0.00]**
	Control	60	55.0	80	1.45		
College Sta. Richardson	Increase yellow	72	155.2	61	0.39	-70	**4.62 [0.00**
	Control	24	37.7	50	1.33		

Notes:

1 - Hours listed are evenly distributed among the "before" and the "after" studies.

2 - RLR Frequency: red-light-running events during a 6-hour period.

3 - $R = RLR_{after} / RLR_{before}$.

4 - $RC = (R_{treatment}/R_{control} - 1) \times 100$. Negative values of RC indicate a reduction in RLR frequency.

5 - $z = L \,/\, L_{se}$ where, $L = \ln(R_{treatment}/R_{control})$; "$\ln(x)$" = natural log of x; and

$L_{se} = (1/RLR_{after\,,\,treatment} + 1/RLR_{before\,,\,treatment} + 1/RLR_{after\,,\,control} + 1/RLR_{before\,,\,control})^{0.5}$.

6 - p-value: probability that the null hypothesis (i.e, no change, RC = 0.0) is erroneously rejected.

7 - Underlined values identify changes that are statistically significant (with less than 5 percent chance of error).

8 - Results are confounded by an unintended reduction in yellow interval duration between studies.

Two countermeasure combinations were evaluated in Laredo. At one site, both yellow LED indications and back plates were added to the signal heads. The data from this site indicated that this countermeasure combination is associated with a 35 percent reduction in red-light-running. However, the sample size is too small to be certain that this reduction is significant (i.e., there is a 36 percent chance that more observations would indicate that there is truly no reduction due to these countermeasures). If the 25 percent reduction due to the use of back plates (as described in a preceding section) is extracted from the 35 percent reduction found at this site, the result is a 13 percent reduction due to the use of yellow LED indications (= 100 − 100 [1 − 0.35]/[1 − 0.25]).

The potential red-light-running reduction associated with yellow LEDs at the Laredo site is consistent with that found at the Mexia sites. The data from both cities were combined but statistical significance was still not achieved. It should also be noted that the effect of yellow LEDs was not found to be statistically significant when calibrating the red-light-running model. Nevertheless, the evidence in Table 5-7 is that LEDs are likely to have a real, but modest, reduction potential that is of practical significance. Based on this analysis, it rationalized that yellow LEDs can reduce red-light-running by at least 13 percent.

The evaluation of "improved coordination" at the second site in Laredo was confounded by an unintended reduction in the yellow interval duration. The relative change in red-light-running (RC) associated with this treatment is 0.0 which suggests that any benefit of improved coordination was offset by the reduction in yellow duration.

Model Extensions - Identifying Problem Locations

The red-light-running model (i.e., Equation 16) can also be used to identify problem intersection approaches. The process is one of observing the count of red-light-runners x during a peak traffic hour, using Equation 19 to compute the expected red-light-running frequency *given* that x red-runners were observed $E[R|x]$ (with H = 1.0 hour), using Equation 16 to compute the expected red-light-running frequency $E[R]$ for a "typical" approach, and then computing the following index:

$$Index = \frac{E[R|x] - E[R]}{\sqrt{\frac{E[R]^2}{k} + \sigma_m^2}} \qquad \textbf{(22)}$$

with,

$$\sigma_m^2 = (1 - weight)\ E[R|x] \qquad \textbf{(23)}$$

where, σ_m^2 is the variance of the expected red-light-running frequency $E[R|x]$ (*34*).

The index value is an indicator of the extent of the red-light-running problem for a given intersection approach. In general, intersection approaches with a positive index value have more red-light-running than the "typical" approach. An approach with an index of 2.0 has a greater problem than an approach with an index of 1.0. If a group of approaches are evaluated, their indices should be ranked and those with the largest values given priority for treatment (using one or more countermeasures).

ANALYSIS OF CRASH DATA

This section describes the findings from an investigation of the relationship between crash rate and the rate of red-light-running on an intersection approach. The findings presented are the result of an analysis of a crash history database assembled for this research. Initially, the database content is summarized and reviewed for the existence of basic cause-and-effect relationships. Then the crash rate model is calibrated, and the quality of its fit to the data is examined.

Database Review and Analysis

Table 5-8 documents the three-year crash history for each of the approaches included in the study of red-light-running. With one exception, the crash records were obtained from the database maintained by the Texas Department of Public Safety (DPS). The records for the city of Richardson were obtained from the City of Richardson Traffic Engineering Department. Only those crashes that are described as right-angle or left-turn-related were extracted from the records as these crashes were most likely to be correlated with red-light-running.

Table 5-8. Crash Frequency at Each Study Site.

City	Intersection Approach	Average Daily Volume [1]		Approaches	Total Crashes, [2] c/3 yrs	Approach Crash Freq., [3] c/app/3 yrs
		Approach Volume, veh/d	Crossing Volume, veh/d			
Mexia	Milam St.	12,300	3400	2	5	2.5
	S.H. 171	7200	8800	2	0	0.0
College Station	Texas Ave.	49,200	19,800	2	5	2.5
	University Dr.	40,300	5500	2	9	4.5
Richardson	SB Plano Road	36,000	37,800	1	8	8.0
	EB Belt Line Road	37,800	36,000	1	7	7.0
	SB Greenville Ave.	17,500	37,600	1	7	7.0
	EB Main Street	37,600	17,500	1	4	4.0
Corpus Christi	S.H. 44 (3-leg)	18,700	1600	2	4	2.0
	U.S. 77	18,800	4900	2	11	5.5
Laredo	Loop 20 (3-leg)	15,900	2700	2	3	1.5
	U.S. 83	11,300	2000	2	2	1.0
Total or Average:		302,600	177,600	20	65	3.3

Notes:
1 - Daily volumes correspond to 1998 except those for Richardson, which correspond to 1999.
2 - "c/3 yrs:" right-angle and left-turn crashes for three years for all approaches studied. Crash data correspond to the years 1997, 1998, and 1999 except those for Richardson, which correspond to 1998, 1999, and 2000.
3 - "c/app/3 yrs:" average crashes per approach during a three-year period.

The level of detail in the DPS data was not sufficient to accurately assign crashes to a specific intersection approach. Therefore, crashes for both approaches studied at a given intersection were combined and used to compute an "average approach crash rate." The red-light-running data were similarly combined for these approaches to compute an "average approach red-light-running rate." The only exception to this treatment was with the data from the city of Richardson. These data were sufficiently detailed that individual crash rates and red-light-running rates could be computed for each approach.

Also shown in Table 5-8 are the average daily traffic volumes corresponding to the second (or middle) year of the three-year crash records. The second-year volume was reasoned to be most representative of the volumes present during the three-year crash history. This volume was estimated from data provided by the traffic engineering agencies responsible for the intersections studied. In most instances, interpolation between two or more of the volumes provided was needed to obtain the estimate for the desired second year. The "crossing" volume shown in the table represents the daily traffic volume on the street that intersects (or crosses) the subject approach.

The last column of Table 5-8 lists the three-year crash frequency averaged by the total number of approaches studied. The rate is represented in terms of the number of crashes per approach during a three-year period. The overall average rate is 3.3 crashes per approach per three years.

Model Calibration

Non-linear regression analysis (with a Poisson error distribution) was used to quantify the relationship between total crashes and red-light-running rate. The red-light-running rate was expressed as the number of red-light-runners per 1000 approach vehicles. These red-light-running rates were previously listed in Table 5-4. The model used for the regression analysis is:

$$C_3 = m_y \, e^{b_0} \, ADT_s^{b_1} \, ADT_c^{b_2} \, RLR_r^{b_3} \qquad \mathbf{(24)}$$

where,

C_3 = three-year count of right-angle and left-turn crashes, c/app;
m_y = number of years associated with the crash data (= 3.0), years;
ADT_s = average daily traffic volume on the subject approach, veh/d;
ADT_c = average daily traffic volume on the cross street, veh/d;
RLR_r = red-light-running rate on the subject approach, number of red-light-running events per 1000 vehicles; and
b_i = regression coefficients, i = 0, 1, 2, 3.

The results of the regression analysis are summarized in Table 5-9. The calibrated coefficient values can be used with Equation 24 to predict the annual crash count on a given intersection approach (i.e., $m_y = 1.0$). The Pearson χ^2 statistic for the model is 8.86 and the degrees of freedom is 9 (= $n - p - 1$ = 12 − 2 − 1). As this statistic is less than $\chi^2_{0.05,\,9}$ (= 16.9), the hypothesis that the model fits the data cannot be rejected. The coefficient of determination (R^2) of 0.57 is relatively large and suggests that the calibrated model accounts for most of the variability in the crash data.

Table 5-9. Calibrated Crash Model Statistical Description.

Model Statistics		Value		
	R^2:	0.57		
	Pearson χ^2:	8.86 ($\chi^2_{0.05,\,9}$ = 16.9)		
	Observations:	12 approaches or approach pairs		
	Standard Error:	±1.6 c/app per three years (0.95 c/app/yr)		
Range of Model Variables				
Variable	**Variable Name**	**Units**	**Minimum**	**Maximum**
ADT_c	Average daily volume on the cross street	veh/d	1600	37,800
RLR_r	Red-light-running rate	RLR/1000 veh	0.6	8.3
Calibrated Coefficient Values				
Variable	**Definition**	**Value**	**Std. Dev.**	**t-statistic**
b_0	Intercept term	-5.88	2.05	-2.9
b_2	Cross-street volume adjustment factor	0.614	0.194	3.2
b_3	Red-light-running rate adjustment factor	0.387	0.272	1.4

The regression analysis indicated that the coefficient b_1 was not significantly different from zero so the corresponding variable was excluded from the model. The coefficient associated with the red-light-running rate b_3 has a large standard deviation and, consequently, has a 20 percent chance (based on the *t* distribution with 9 degrees of freedom) of being equal to zero (i.e., having no effect on crash frequency). More data would be needed to determine if this coefficient is truly different from zero. However, it is left in the model at this time as there is a strong likelihood that it accurately reflects the effect of red-light-running on crash frequency.

The fit of the model to the data is illustrated in Figure 5-19. The figure compares the observed and predicted crash frequencies. The trend line in this figure does *not* represent the line of best fit; rather, it is a "$y = x$" line. The data would fall on this line if the model predictions exactly equaled the observed data. The trends shown in this figure indicate that the model is able to predict red-light-running-related crashes without bias. The scatter in the data suggests that there is still some unexplained variability in the data. In general, the model is able to predict crash frequency at a given site with a standard error of ±1.6 crashes per approach per three years.

The calibrated model for predicting the annual crash frequency C_1 (right-angle plus left-turn-related) for an intersection approach is represented by the following equation:

$$C_1 = 0.00278\ ADT_c^{0.614}\ RLR_r^{0.387} \qquad (25)$$

The relationship between observed crash frequency and red-light-running rate is illustrated in Figure 5-20. The calibrated model is illustrated by the trend lines shown in this figure. In general, crash frequency increases with increasing cross-street volume and red-light-running rate.

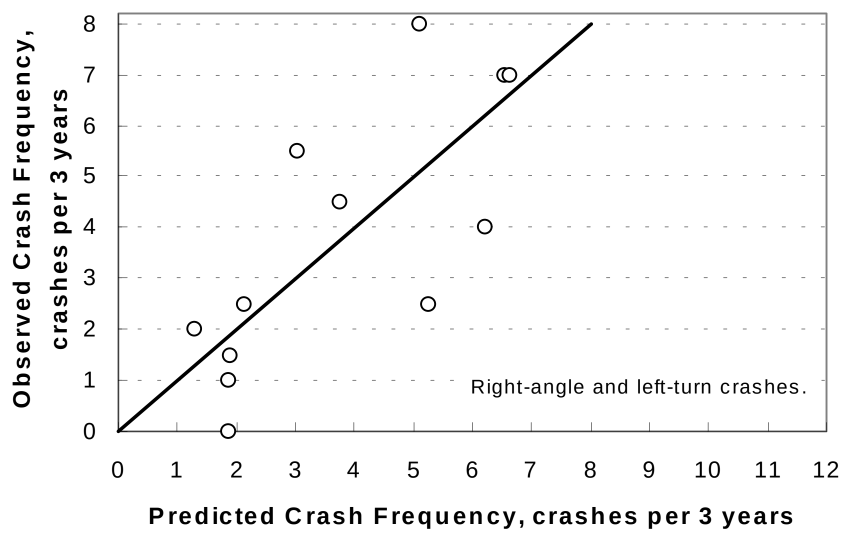

Figure 5-19. Comparison of Observed and Predicted Intersection Crashes.

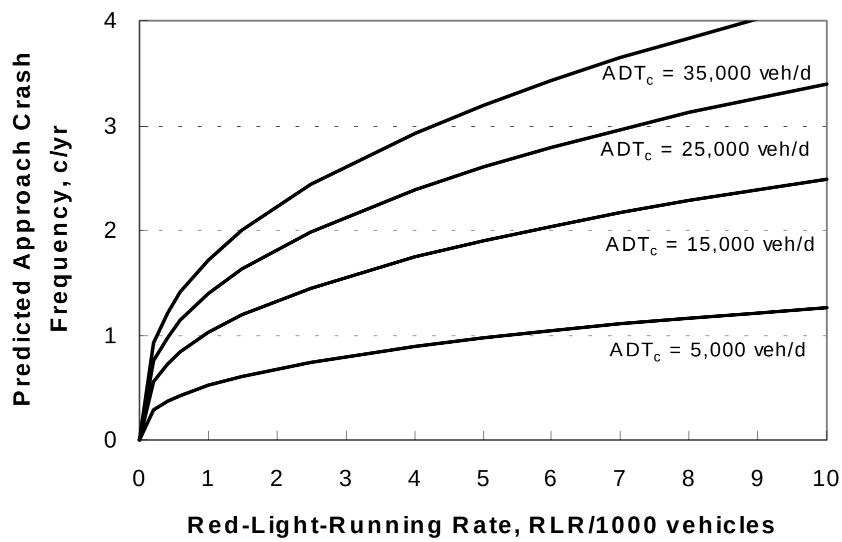

Figure 5-20. Predicted Effect of Red-Light-Running on Intersection Crash Frequency.

Sensitivity Analysis

This section describes a sensitivity analysis of the calibrated crash prediction model. For this analysis, the relative effect of a change in the red-light-running rate was evaluated in terms of the increase or decrease in red-light-running-related crashes. For this analysis, the daily traffic on the subject approach and the cross street is assumed to be constant.

Analysis Methodology

The relative effect of a small change (or deviation) from the base red-light-running rate was computed using Equation 25 twice, once using the "new" rate and once using the "base" rate. The ratio of the expected crash frequencies was then computed as:

$$CMF = \frac{C_{1,new}}{C_{1,base}} \qquad \textbf{(26)}$$

where, *CMF* represents a "crash modification factor" indicating the extent of the change in crashes due to a change in the base value. For example, consider a proposed countermeasure that is expected to yield 3.0 red-light-runners per 1000 vehicles, relative to a base situation with 6.0 red-light-runners per 1000 vehicles. If the cross-street volume is 15,000 vehicles per day, Equation 25 predicts 1.5 and 2.0 annual crashes per approach for the "proposed" and "base" conditions, respectively. The resulting *CMF* from Equation 26 is 0.75. Thus, the countermeasure should yield a 25 percent (= 100 − 0.75 × 100) reduction in red-light-running-related crashes.

Equations 25 and 26 can be combined to yield a more useful form for calculating the *CMF*. After simplification, the resulting equation is:

$$\begin{aligned} CMF &= \left(\frac{E[R]_{new}}{E[R]_{base}} \right)^{0.387} \\ &= MF^{\,0.387} \end{aligned} \qquad \textbf{(27)}$$

where, *MF* represents the "modification factor" associated with a countermeasure.

The effect of a change in red-light-running on the frequency of red-light-running-related crashes is shown in Figure 5-21. The trend in this figure indicates that a decrease in red-light-running is associated with a decrease in crash frequency. For example, a decrease in red-light-running frequency from 6.0 to 3.0 red-light-runners per hour yields an *MF* of 0.50 (= 3.0/6.0). The trend line in Figure 5-21 indicates that an *MF* of 0.50 equates to a CMF of 0.75 which corresponds to a 25 percent crash reduction. An increase in red-light-running from 4.0 to 8.0 events per hour yields an *MF* of 2.0 s which translates into a *CMF* of 1.3 and a 30 percent increase in related crashes.

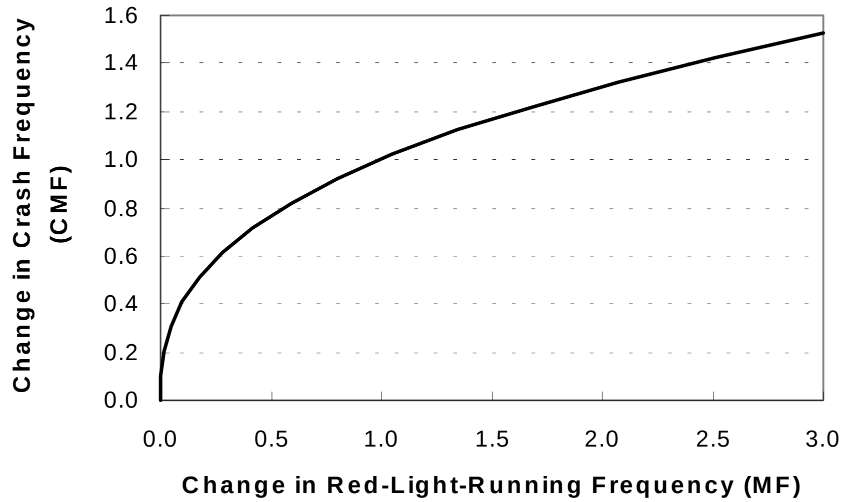

Figure 5-21. Effect of a Change in Red-Light-Running on Crash Frequency.

CHAPTER 6. CONCLUSIONS

OVERVIEW

Statistics indicate that red-light-running has become a significant safety problem throughout the United States. Mohamedshah et al. (*2*) estimate that at least 16 to 20 percent of intersection crashes can be attributed directly to red-light-running. Retting et al. (*1*) also report that motorists involved in red-light-running-related crashes are more likely to be injured than in other crashes. A 1998 survey of Texas drivers, conducted by the FHWA (*3*), found that two of three Texans witness red-light-running every day. About 89 percent of these drivers believe that red-light-running has worsened over the past few years.

There is a wide range of potential countermeasures to the red-light-running problem. These solutions are generally divided into two broad categories: engineering countermeasures and enforcement countermeasures. Enforcement countermeasures are intended to encourage drivers to adhere to the traffic laws through the threat of citation and possible fine. In contrast, engineering countermeasures are intended to reduce the chances of a driver being in a position where he or she must decide whether or not to run the red indication.

The objective of this research project was to describe how traffic engineering countermeasures could be used to minimize the frequency of red-light-running and associated crashes at urban intersections. This chapter documents the findings and conclusions from the research conducted for this project.

SUMMARY OF FINDINGS

Red-Light-Running Process

A review of the literature revealed that red-light-running is the consequence of several events occurring at the same time. They include: the light changing to yellow while a vehicle is on the intersection approach, the vehicle's driver not stopping the vehicle, and the vehicle actually entering the intersection after the yellow interval has ended. The following factors are related to the occurrence of these events and, thus, have some effect on the frequency of red-light-running:

- flow rate on the subject approach (exposure factor),
- number of signal cycles (exposure factor),
- phase termination by max-out (exposure factor),
- probability of stopping (contributory factor), and
- yellow interval duration (contributory factor).

Exposure Factors

Flow rate and the number of signal cycles (per hour) represent factors that expose drivers to conditions that may precipitate red-light-running. The number of drivers running the red each signal cycle is positively correlated with flow rate at the onset of the yellow indication. The frequency of red-light-running increases as more drivers are on the intersection approach at the onset of yellow. The frequency of red-light-running is also positively correlated with the frequency with which the yellow indication is presented. A cycle length change from 60 to 120 s reduces the number of times that the yellow is presented by 50 percent. In theory, a similar reduction in red-light-running frequency should also be observed.

Contributory Factors

Two contributory factors underlie the events that lead to red-light-running. These factors include the "probability of stopping" and the "yellow interval duration." The former factor represents the complex decision-making process that drivers exhibit at the onset of yellow. A review of the literature indicates that this decision is affected by the driver's assessment of the prevailing traffic and roadway conditions and by his or her estimate of the consequences of stopping (or not stopping).

The yellow interval duration contributes to red-light-running in a more fundamental manner. The start of this interval defines the instant when the decision-making process should begin. During this interval, a decision is made and acted upon. The end of this interval defines the instant when the red indication is presented (whereupon entry to the intersection represents a red-light-running event).

Probability of Stopping. Many researchers have studied the decision to stop in response to the yellow indication. Van der Horst and Wilmink (*9*) studied this decision process and found that a driver's propensity to stop is based on three components. These components include: driver behavior (e.g., travel time, speed, approach grade, headway), the consequences of not stopping (e.g., citation), and the consequences of stopping (e.g., rear-end crash).

A review of the literature indicates that drivers are less likely to stop when they have a short travel time to the intersection, are traveling at higher speeds, are traveling in platoons, are on steep downgrades, are faced with relatively long yellow indications, and are being closely followed. A driver is also likely to weigh the consequences of stopping and not stopping when making his or her decision. Research indicates that drivers are less likely to stop if they believe the crossed street has a low flow rate, there is little threat of enforcement, there is a threat of rear-end collision, and the expected delay is lengthy.

Yellow Interval Duration. The yellow interval duration is generally recognized as a key factor that affects the frequency of red-light-running. This recognition has led several researchers to recommend setting the yellow interval duration based on the probability of stopping (*9, 12, 18*).

These researchers suggest that the yellow interval should be based on the travel time of the 85th (or 90th) percentile driver. The corresponding yellow interval duration should range from 4.0 to 5.5 s (with larger values appropriate for higher-speed approaches). Data reported by Van der Horst and Wilmink (*9*) indicate that yellow intervals in excess of 3.5 s are associated with minimal red-light-running.

Analysis of Red-Light-Running Data

Database Summary

The database assembled for this research includes the traffic characteristics, control features, and geometric elements for 10 intersections in Texas. Two approaches were studied at each intersection. Traffic data recorded at each intersection included: vehicle count and classification, cycle length, number of red-light-running vehicles per cycle, average running speed, and flow rate at the end of the signal phase.

The database consists of six hours of data collection at each of 20 intersection approaches. More than 10,018 signal cycles were observed. During these cycles, 586 vehicles entered the intersection (as defined by the stop line) after the change in signal indication from yellow to red.

Of the 586 vehicles observed to run the red indication, 84 were heavy vehicles and 502 were passenger cars. Overall, 0.86 percent of heavy vehicle drivers violated the red indication, and 0.38 percent of passenger car drivers violated the red indication. This finding indicates that heavy vehicle operators are twice as likely to run the red indication as passenger car drivers.

Red-Light-Running Rates

A review of the literature indicates that several measures are used to quantify red-light-running events. The more commonly used measures include: "percent of cycles with one or more red-light-runners," "hourly red-light-running rate," and "percent of vehicles that run the red." These measures are all based on the frequency of red-light-running, as normalized by exposure (e.g., volume or cycle) or location (e.g., lane or approach).

Two red-light-running rates were evaluated for their ability to provide useful information about the extent of red-light-running at a given location. The first rate is expressed in terms of red-light-running events per 1000 vehicles. The second rate represents the number of red-light-running events per 10,000 vehicle-cycles. The typical intersection approach experiences from 3.0 to 5.0 red-light-runners per 1000 vehicles and about 1.0 red-light-runners per 10,000 vehicle-cycles. The former rate varies more among intersections than does the latter rate because it does not include the effect of cycle length on the frequency red-light-running.

Entry Time of the Red-Light-Running Driver

The time after the end of the yellow indication at which the red-light-runner enters the intersection is logically correlated with the potential for a right-angle collision. As this "time into red" increases, the crash frequency is also likely to increase. Analysis of driver entry time, indicates that about 80 percent of drivers entered the intersection within 1.0 s after the end of the yellow. In contrast, about 80 percent of the heavy vehicle drivers entered within 1.7 s after the end of the yellow interval (as compared to 1.0 s for all vehicles combined). The latter statistic suggests that many heavy vehicle drivers are less likely (or able) to stop at the onset of yellow than are passenger car drivers.

There is speculation among engineers that lengthy yellow intervals may be abused by some drivers. Specifically, it is believed that drivers adapt to an increase in the yellow duration and continue to run the red indication with the same frequency as before the increase. Analysis indicates that drivers do adapt to an increase in yellow duration, however; the frequency of red-light-running is still reduced. Specifically, a nominal increase of 0.5 to 1.5 s of yellow, such that the yellow duration does not exceed 5.5 s, is found to decrease the frequency of red-light-running by 50 percent at several intersections.

Analysis of Factors Affecting the Frequency of Red-Light-Running

The relationship between a range of factors and red-light-running frequency was examined. These factors represent the traffic characteristics, signal control features, and geometric elements at the intersections studied. The variables associated with these factors include: approach flow rate, cycle length, yellow interval duration, heavy-vehicle percentage, running speed, clearance path length, platoon ratio, approach grade, number of approach lanes, LED signal indications, use of signal head back plates, use of advance detection, and signal head mounting. Of these variables, seven were found to be correlated with red-light-running frequency. Their effect on red-light-running frequency is illustrated in Table 6-1 for specified changes in the variable value.

The information in Table 6-1 is intended to illustrate the individual effect of each variable on red-light-running. The magnitude of the effect is dependent on the change of the associated variable. Other magnitudes would be obtained for different changes in the associated variable. In general, a decrease in red-light-running is found to be associated with a decrease in flow rate, an increase in yellow duration, a decrease in speed, an increase in clearance path length (i.e., a wider intersection), a decrease in platoon density, and the addition of signal head back plates.

For the variable changes listed in Table 6-1, an increase in yellow duration is associated with the largest reduction in red-light-running. The addition of signal head back plates is associated with a more modest reduction. However, this countermeasure is attractive because it is relatively inexpensive to implement.

Table 6-1. Effect of Selected Variables on the Frequency of Red-Light-Running.

Variable	Effect of a Reduction in the Variable Value [1]		Effect of an Increase in the Variable Value [1]	
	Variable Change	RLR Frequency Change [2]	Variable Change	RLR Frequency Change [2]
Approach flow rate	-1.0 %	-1.0 %	1.0%	1.0 %
Cycle length	from 90 to 70 s	29 %	from 90 to 110 s	-18 %
Yellow interval duration	-1.0 s	110 %	1.0 s	-53 %
Running speed	-10 mph	-33 %	10 mph	45 %
Clearance path length	-40 ft	81 %	40 ft	-48 %
Platoon ratio	-1.0	-18 %	1.0	21 %
Use of back plates	remove back plates	33 %	add back plates	-25 %

Notes:
1 - Negative changes represent a reduction in the associated variable.
2 - RLR Frequency Change is computed as $100 \times (MF - 1)$ where MF is the modification factor associated with the variable.

Examination of a Common Yellow Interval Equation

This section examines the relationship between red-light-running and a "computed" yellow interval duration. The computed yellow interval is obtained from an equation that is commonly used by engineers (*28*). It is based on 1.0-s reaction time and the time needed to decelerate from the 85th percentile speed at a rate of 10 ft/s^2. The results of this examination indicate that there is a trend toward more red-light-running when the observed yellow duration is shorter than the computed duration. It was found that the use of this equation should yield about 0.8 red-light-runners per 10,000 veh-cycles for "typical" conditions (i.e., no back plates and a 90-ft clearance path).

Evaluation of Countermeasures

Several countermeasures were evaluated in the context of a before-after study at 10 intersections in Texas. One of the following countermeasures was implemented at one or more intersection approaches:

- Add LED lighting to the yellow indications (49 percent reduction).
- Increase the yellow interval duration (70 percent reduction).
- Add back plates and increase yellow interval duration (18 percent reduction).
- Increase cycle length and improve signal operation (uncertain effect).
- Improve progression and increase cycle length (uncertain effect).
- Add back plates and add LED lighting to the yellow indications (35 percent reduction).

Four of the six countermeasures were found to reduce red-light-running. The percent of red-light-running reduced is included in the preceding list of countermeasures. Of these reduction percentages, only that associated with an increase in the yellow interval duration was found to be statistically significant. This qualification cannot be extended to the other reduction percentages as the corresponding sample size is too small from a statistical standpoint; however, they still are strong indicators that the associated countermeasure is able to reduce red-light-running.

Conclusions could not be reached about two countermeasures because of unexpected deviations from the study plan. One deviation resulted in a loss of the control site, and the other deviation stemmed from an unintended reduction in the yellow duration at an intersection. These deviations confounded the analysis and made it impossible to make accurate statements about the effectiveness of the associated countermeasures.

Synthesis of Countermeasure Effectiveness

The findings from this research regarding the effectiveness of several engineering countermeasures are summarized in Table 6-2. The information presented reflects the findings from the before-after studies, a sensitivity analysis of the red-light-running model, and a review of the literature. In general, the reduction percentages associated with red-light-running frequency are likely to be more reliable than those for red-light-running-related crashes. This observation is based on a critical review of the studies that underlie the reported percentages. Nevertheless, while a reported crash reduction percentage should be considered "approximate," the fact that it is negative should be taken as strong evidence that the associated countermeasure will reduce the frequency of red-light-running.

Relationship between Red-Light-Running and Crash Frequency

This section describes the findings from an investigation of the relationship between crash rate and the rate of red-light-running on 20 intersection approaches. The findings presented are the result of a statistical analysis of a three-year crash history assembled for each of the intersections included in the field study of red-light-running. The calibrated model for predicting the annual crash frequency revealed that crash frequency increases with increasing cross-street volume and red-light-running rate (expressed as red-light-runners per 1000 vehicles).

An examination of the effect of a change in red-light-running on the frequency of red-light-running-related crashes indicated that a decrease in red-light-running is associated with a decrease in crash frequency. Specifically, a 50 percent decrease in red-light-running frequency yields a 25 percent crash reduction. In contrast, a 100 percent increase in red-light-running yields a 30 percent increase in related crashes.

CONCLUSIONS

Several conclusions were reached based on the findings of this research. These conclusions are summarized in this section.

Table 6-2. Engineering Countermeasures to Red-Light-Running.

Countermeasure Category	Specific Countermeasure	Reported RLR Effectiveness [1]	
		Frequency	Related Crashes
Signal Operation (modify signal phasing, cycle length, or change interval)	Increase the yellow interval duration	-50 to -70 %	--
	Provide green-extension (advance detection)	-45 to -65 %	--
	Improve signal coordination	Varies [2]	--
	Improve signal operation (increase cycle length 20 s)	-15 to -25 % [3]	--
Motorist Information (provide advance information or improved notification)	Improve sight distance	--	--
	Improve visibility of signal (12" lens, add heads)	--	-33 to -47 %
	Improve visibility of signal with yellow LEDs	-13 %	--
	Increase conspicuity of signal with back plates	-25 %	-32 %
	Add advance warning signs without flashers	--	-44 %
	Add advance warning signs with active flashers	-29 to -67 %	--
Physical Improvement (implement safety or operational improvements)	Remove unneeded signals	--	-24 %
	Add capacity with additional traffic lanes	--	--
	Flatten sharp curves	--	--

Note:
1 - Negative values indicate a reduction. "--": data not available.
2 - Red-light-running frequency is likely to increase with improved coordination; however, this increase may be offset by the larger cycle length typically required for good progression.
3 - Reductions associated with an increase in cycle length may not be realized if motorist delay increases significantly.

The primary conclusion reached is that there are a variety of countermeasures available to engineers to treat intersections with excessive red-light-running or red-light-running-related crashes. As is the case with most engineering improvements, proper treatment of the red-light-running problem requires careful diagnosis to ensure that a problem truly exists. This diagnosis requires a site survey to collect data that quantify the frequency of red-light-running and associated crashes. The survey should also identify conditions that may contribute to the occurrence of red-light-running. Thereafter, if the frequency is determined to be excessive, a series of engineering countermeasures should be evaluated for their applicability and effectiveness. The most promising countermeasures should then be implemented *and* a follow-up evaluation conducted to determine whether red-light-running and related crashes have been reduced. The guidelines provided in the appendix to this report describe this process and can be used to reduce red-light-running at most intersections.

The following specific conclusions are reached as a result of an examination of the data collected for this research:

- The typical intersection approach experiences from 3.0 to 5.0 red-light-runners per 1000 vehicles and 1.0 red-light-runners per 10,000 veh-cycles. An intersection with a red-light-running rate that is larger than that of the typical intersection should be the primary target of a treatment program.

- A heavy vehicle operator is twice as likely to run the red indication as is a passenger car driver. Intersection approaches with a large number of trucks are likely to derive greater benefit from countermeasure treatment (through a significant reduction in truck-related right-angle crashes).

- Red-light-running is more frequent at intersections with platoons arriving near the end of the green indication. Engineers developing signal coordination plans should be careful to avoid having platoons arrive near the end of the signal phase. If this situation cannot be avoided, then a longer cycle length should used with the coordination plan to minimize the frequency of red-light-running.

- Red-light-running rates should be reported as "events per 10,000 vehicle-cycles." This rate statistic is a reliable measure of the propensity for red-light-running and can be used to assess the extent of the red-light-running problem at an intersection or compare it to other intersections.

- About 80 percent of drivers that red-light-run enter the intersection within 1.0 s after the end of the yellow. Hence, engineering countermeasures focused on driver recognition of, and response to, the yellow indication are likely to be the most cost-effective.

- There is speculation among engineers that drivers will adapt to an increase in the yellow duration and continue to run the red indication with the same frequency as before the increase. Analysis confirms that drivers do adapt to the increase in yellow duration. However, a nominal increase of 0.5 to 1.5 s of yellow (such that the yellow duration does not exceed 5.5 s) will still decrease the frequency of red-light-running by at least 50 percent. This finding is evidence of the benefit of a properly timed yellow interval, where the interval duration is based on engineering analysis and consideration of traffic conditions, control device visibility, and intersection sight distance.

- In addition to an increase in yellow interval duration, several other engineering countermeasures were identified as having the potential to reduce red-light-running. Specifically, it was found that the use of back plates would reduce red-light-running by 25 percent; a 20-s increase in cycle length would reduce red-light-running by 18 percent (provided that delays do not substantially increase); and that the use of yellow LEDs may reduce red-light-running by 13 percent.

- Advance warning signs with active flashers and the use of multiple advance detector systems have been found to significantly reduce red-light-running; however, neither countermeasure was explicitly evaluated for this research.

- An analysis of the relationship between crash rate and the rate of red-light-running on an intersection approach indicates that crash frequency increases with increasing cross-street volume and red-light-running rate (expressed as red-light-runners per 1000 vehicles).

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APPENDIX

GUIDELINES FOR SELECTING AND EVALUATING ENGINEERING COUNTERMEASURES TO REDUCE RED-LIGHT-RUNNING

INTRODUCTION

There are a variety of countermeasures available to engineers to treat intersections with excessive red-light-running or red-light-running-related crashes. As is the case with most engineering improvements, proper treatment of the red-light-running problem requires careful diagnosis to ensure that a problem truly exists. This diagnosis should be based on an observational study and a site survey that identify conditions that may contribute to the occurrence of red-light-running. Data collected during the survey should then be used to quantify the frequency of red-light-running and associated crashes. Thereafter, if the frequency is determined to be excessive, a series of engineering countermeasures should be evaluated for applicability and effectiveness. The most promising countermeasures should then be implemented *and* a follow-up evaluation conducted to determine whether red-light-running and related crashes have been reduced. The guidelines provided herein describe this process.

The guidelines described in this document are intended to help the engineer select and evaluate engineering countermeasures to reduce red-light-running. These guidelines are presented in a simplified manner to facilitate their implementation. In this regard, the basis for the quantitative information provided is not specifically identified. Rather, the engineer is referred to the research report for the derivation or source of this guidance.

Objective

The objective of this guideline document is to describe a process for selecting and evaluating engineering countermeasures to reduce red-light-running at problem intersections. The guidelines are presented in a series of steps that should be followed when an intersection approach is believed to have a red-light-running problem. If, after the trial of one or more engineering countermeasures, the red-light-running problem still exists, the engineer may need to consider some type of enforcement activity. If the problem is deemed to be "area-wide," then enforcement coupled with a public awareness campaign may be appropriate.

Scope

The guidelines presented in this document are based on research directed at the more common types of red-light-running. To this extent, these guidelines are most applicable to the following conditions:

- Drivers traveling through the intersection (as opposed to turning at it).
- Urban or suburban intersections.
- Red-light-running that is perceived by the driver as being "unavoidable."

An "unavoidable" event is committed by a driver who either (1) believes that he or she is unable to safely stop and consciously decides to run the red indication, or (2) is unaware of the need to stop. In contrast, an "avoidable" red-running event is committed by a driver who believes that it is possible to safely stop but decides it is in his or her best interest to run the red indication.

In spite of the aforementioned limitations of scope, it is possible that the guidelines in this document can be extended, with some success, to the treatment of red-light-running by drivers turning left or right, drivers at rural intersections, or drivers whose decision can be characterized as "avoidable."

IDENTIFY PROBLEM AND CAUSE

This section describes the process for determining the nature of the red-light-running problem at an intersection and defining its potential causes. The tasks involved in making this determination are discussed in this section, they include:

a. Gather Information
b. Confirm Extent of Problem
c. Identify Possible Causes

In the first task, data are collected that describe the intersection's crash history and its present condition in terms of traffic volume, traffic control features, and geometric elements. Then, these data are used to confirm that a problem truly exists and to identify their cause.

Gather Information

Following notification of a possible red-light-running problem, the engineer should make a preliminary assessment of the problem to determine its extent and whether it requires a solution. This assessment involves gathering information about the intersection so that the problem can be confirmed and its cause identified. Information sources include: crash history, first-hand observation, and a site survey.

Crash History

The analyst should obtain crash data summaries for the most recent three years at the subject intersection. The focus of this examination should be to identify the number of red-light-running-related crashes (i.e., right-angle and left-turn-related) that have occurred on each approach. Summarizing this information on a collision diagram is a useful method of portraying the information and assessing which approaches are potential trouble spots.

Observational Study

An important component of the problem-cause identification process is the first-hand observation of traffic operations. This observation should be scheduled to coincide with the occurrence of the reported problems (e.g., peak traffic demand hour). Initially, the engineer should drive through the subject intersection and attempt to experience the problem. Then, the engineer should observe intersection operation from a curb-side vantage point.

With some patience, the engineer should observe several red-light-running events during this study period. Through these observations, the engineer should identify possible causes for the red-light-running, the traffic movements experiencing the red-light-running, a sense of how late into the red indication the red is run, and whether the red-light-running was unavoidable (from the driver's point of view).

Particular attention should be paid to signal head visibility along the intersection approach, to the percentage of heavy vehicles, and to the extent of motorist delay. Sharp horizontal or vertical curvature should be fully evaluated to determine if it restricts signal visibility *and* if it distracts the drivers' attention (from the signal indication) as they approach the intersection during the green indication.

At the conclusion of this study, the engineer should define the traffic movements that are the subject of further study. Specifically, a "subject" traffic movement can include any of the following:

- a left-turn movement in an exclusive lane,
- a right-turn movement in an exclusive lane and controlled by a signal, and
- any combination of movements that are served in a lane shared with the through movement.

Data for each of the subject traffic movements are collected during the site survey.

Site Survey

The engineer should have a survey conducted of the intersection's traffic characteristics, control features, and geometric elements. The survey of the intersection's control features and geometric elements is recorded on a condition diagram that includes a scale drawing of the intersection in plan-view. The following two distances need to be measured during this survey for each of the subject traffic movements:

P1. Clearance path length (measured from the stop line of the subject movement to the furthest edge of the last conflicting lane crossed) L_p, ft.

P2. Distance from the stop line to the upstream edge of the most distant detector D, ft. If no detectors are provided, then D is 0.0. If only stop-line detection is provided, then D equals the length of this detector. If advance detectors are provided, then D is measured as the distance from the stop line to the most distant detector, as defined.

Other conditions recorded on the condition diagram should include: street width, pavement markings, speed limits, and the location of all relevant traffic signs (including "signal-ahead" warning signs). An example condition diagram is shown in Figure A-1.

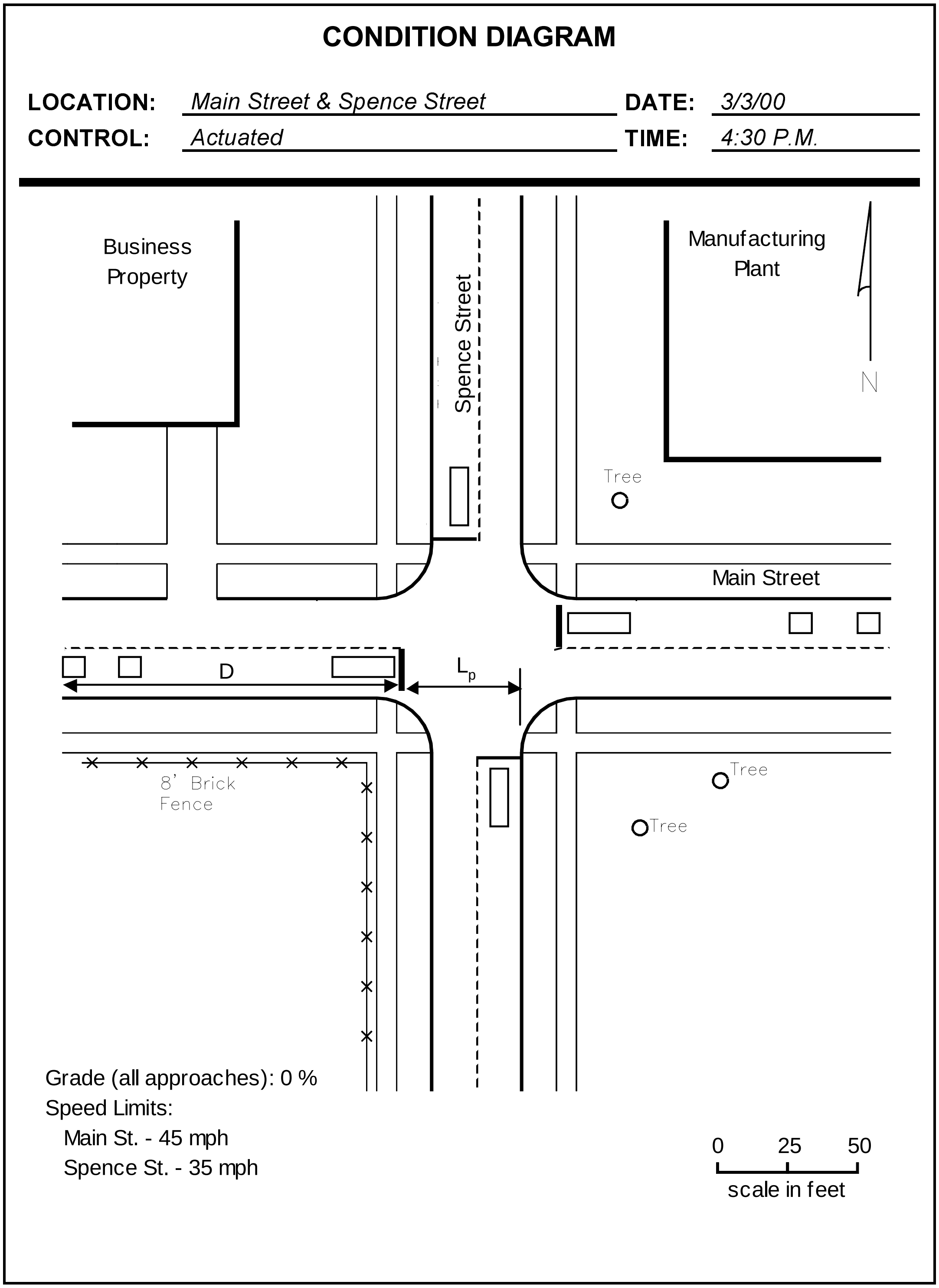

Figure A-1. Sample Condition Diagram.

The survey of traffic characteristics and signal conditions should take place during a one-hour period coincident with the time period when red-light-running is most notable. Typically, this one-hour period coincides with a peak traffic hour. The following traffic data are measured or estimated for each of the subject traffic movements:

T1. Count of vehicles crossing the stop line Q, veh/h;
T2. Duration of the yellow interval Y, s;
T3. Platoon arrival type based on the descriptors in Table A-1;
T4. Approach speed (measurement of 100 free flowing vehicles), mph; and
T5. Count of vehicles crossing the stop line *after* the end of the yellow interval x, veh/h.

The duration of the all-red interval should also be recorded if a right-angle crash problem is revealed in the crash history analysis. Data items T3 and T4 are optional and can be estimated (using techniques described in a subsequent section) if resources are not available to collect them.

Table A-1. Relationship between Platoon Ratio and Progression Quality.

Progression Quality [1]	Arrival Type
Very poor - Dense platoon containing more than 80 percent of the volume, arriving at the start of the red indication.	1
Unfavorable - Either (1) a moderately dense platoon arriving in the middle of the red indication, or (2) a dispersed platoon containing 40 to 80 percent of the volume arriving during the red indication.	2
Random arrivals - Either (1) random arrivals at an isolated intersection, or (2) coordinated operation where the main platoon contains less than 40 percent of the volume.	3
Favorable - Either (1) a moderately dense platoon arriving in the middle of the green indication, or (2) a dispersed platoon containing 40 to 80 percent of the volume arriving throughout the green indication.	4
Highly Favorable - Dense to moderately dense platoon containing over 80 percent of the lane group volume, arriving at the start of the green phase.	5
Exceptional - Very dense platoons progressing through a number of closely-spaced intersections with minimal side-street entries.	6

Note:
1 - Descriptions extracted from Exhibits 16-4 and 16-11 in the *Highway Capacity Manual* (*33*).

The following signal information is recorded for each of the subject traffic movements:

S1. Presence of back plates on the controlling signal heads;
S2. Use of 8-inch or 12-inch signal heads;
S3. Number of controlling signal heads;
S4. Use of dual-red indications per head;
S5. Use of LEDs in the yellow indications of the controlling signal heads; and

S6. Cycle length C, s. If the intersection is actuated, then compute an average signal cycle length based on an average of at least five cycles (each measured as the elapsed time from the end of one main-street green indication to the end of the next main-street green indication).

Item S6 is quantified once for the entire intersection.

Confirm Extent of Problem

During this task, the information gathered in the previous task is evaluated to determine if a red-light-running problem exists for the subject traffic movement. This task requires the computation of the expected red-light-running frequency for a "typical" movement $E[R]$ and a comparison between it and the observed frequency x. A similar comparison is made between the average crash rate and that observed on the approach serving the subject movement. The findings from this comparison are then used to confirm whether a red-light-running problem truly exists.

Compute Expected Frequency of Red-Light-Running

The computation of expected red-light-running frequency requires an initial computation of platoon ratio, 85th percentile speed, and effective yellow interval duration. These computations are described in the next subsections.

Identify Platoon Ratio. The platoon arrival type descriptor is used to estimate the platoon ratio R_p. The relationship between arrival type and platoon ratio is shown in Table A-2. If the arrival type was not determined during the site survey, platoon ratio can be estimated using the guidance provided in column 2 of Table A-2.

Table A-2. Conditions Associated with Various Platoon Ratios.

Arrival Type	Conditions Under Which Arrival Type is Likely To Occur [1]	Platoon Ratio (R_p)
1	Very poor - Occurs for coordinated progression on two-way streets where one travel direction does not receive good progression. Signals are spaced 1600 ft or less.	0.33
2	Unfavorable - A less extreme version of Arrival Type 1. Signals spaced at or more than 1600 ft but less than 3200 ft.	0.67
3	Random arrivals - Isolated signals spaced at or more than 3200 ft.	1.00
4	Favorable - Occurs for coordinated operation, often only in one direction on a two-way street. Signals are typically between 1600 ft and 3200 ft.	1.33
5	Highly Favorable - Occurs for coordinated operation. More likely to occur with signals less than 1600 ft.	1.67
6	Exceptional - Typical of one-way streets in dense networks and central business districts. Signal spacing is typically less than 800 ft.	2.00

Note:
1 - Descriptions extracted from Exhibits 10-18 and 16-4 in the *Highway Capacity Manual* (33).

Compute 85th Percentile Speed. The approach speeds collected during the site survey are used to compute the 85th percentile (highest) speed for the subject traffic movement. If resources are limited, the posted speed limit can be used as a direct estimate of the 85th percentile speed.

Determine Effective Yellow Interval Duration. If the intersection has an advance detection system, the "effective" yellow interval duration *Y** must be computed. This interval reflects the ability of advance detection to minimize red-light-running. The value of *Y** is computed as

$$Y^{*} = Larger\ of\!:\ [Y,\ T_D] \qquad \textbf{(A-1)}$$

where, T_D is obtained from Figure A-2, and *Y* is the yellow interval duration measured during the site survey. The dashed line in Figure A-2 illustrates the travel time associated with typical advance detection design practice in Texas.

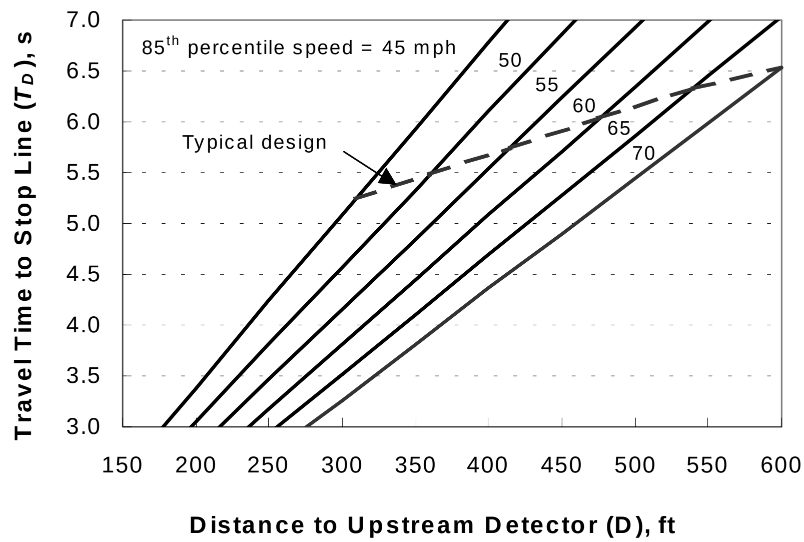

Figure A-2. Travel Time from Detector to Stop Line.

There are three exceptions to the use of Equation A-1. If an exception applies, then the effective yellow duration equals the actual yellow duration (i.e., *Y** = *Y*). The exceptions are:

- If the signal is pretimed, then *Y** = *Y*.
- If the signal is actuated but the subject traffic movement does not have advance detection, then *Y** = *Y*.
- If the signal is actuated and the subject movement has advance detection but traffic volumes are so high that the phase frequently maxes-out, then *Y** = *Y*.

Compute Frequency of Red-Light-Running. The following equation is used to compute the expected red-light-running frequency for each of the subject traffic movements:

$$E[R] = C_f \times \frac{Q}{C} \times P_r \times MF_L \times MF_R \qquad \textbf{(A-2)}$$

where,
$E[R]$ = expected red-light-running frequency, veh/h;
C_f = calibration factor;
Q = subject traffic movement flow rate, veh/h;
C = cycle length, s;
P_r = red-light-running propensity for 90-ft clearance path lengths (see Table A-3), s;
MF_L = modification factor for clearance path lengths other than 90 ft (see Figure A-3); and
MF_R = modification factor for platoon ratios other than 1.0 (see Figure A-4).

Table A-3. Propensity (P_r).

85th Percentile Speed, mph	Use of Back Plates	Effective Yellow Interval Duration, s						
		3.0	3.5	4.0	4.5	5.0	5.5	6.0
		Propensity (P_r), s						
30	No	0.43	0.29	0.19	0.12	0.08	0.05	0.03
	Yes	0.32	0.21	0.14	0.09	0.06	0.04	0.02
35	No	0.50	0.34	0.23	0.15	0.10	0.06	0.04
	Yes	0.38	0.26	0.17	0.11	0.07	0.04	0.03
40	No	0.58	0.40	0.27	0.18	0.12	0.07	0.05
	Yes	0.45	0.30	0.20	0.13	0.08	0.05	0.03
45	No	0.68	0.47	0.32	0.21	0.14	0.09	0.06
	Yes	0.52	0.36	0.24	0.16	0.10	0.06	0.04
50	No	0.78	0.55	0.38	0.25	0.17	0.11	0.07
	Yes	0.61	0.42	0.28	0.19	0.12	0.08	0.05
55	No	0.89	0.64	0.44	0.30	0.20	0.13	0.08
	Yes	0.70	0.49	0.34	0.22	0.15	0.09	0.06
60	No	1.02	0.74	0.52	0.35	0.24	0.15	0.10
	Yes	0.81	0.57	0.40	0.27	0.17	0.11	0.07
65	No	1.15	0.85	0.60	0.42	0.28	0.19	0.12
	Yes	0.93	0.67	0.46	0.31	0.21	0.14	0.09
70	No	1.29	0.97	0.70	0.49	0.33	0.22	0.14
	Yes	1.05	0.77	0.54	0.37	0.25	0.16	0.11

The calibration factor C_f is used to adapt Equation A-2 to local conditions. Prior to the first use of this equation, it should be calibrated by the transportation agency. The calibration factor will have a value greater than 1.0 if red-light-running is more frequent in the local agency jurisdiction than in the cities used to develop the model parameters. The factor will have a value smaller than 1.0 if red-light-running is less frequent than in the cities used to develop the model parameters. A procedure for calibrating the model is described in the last section of this guideline document.

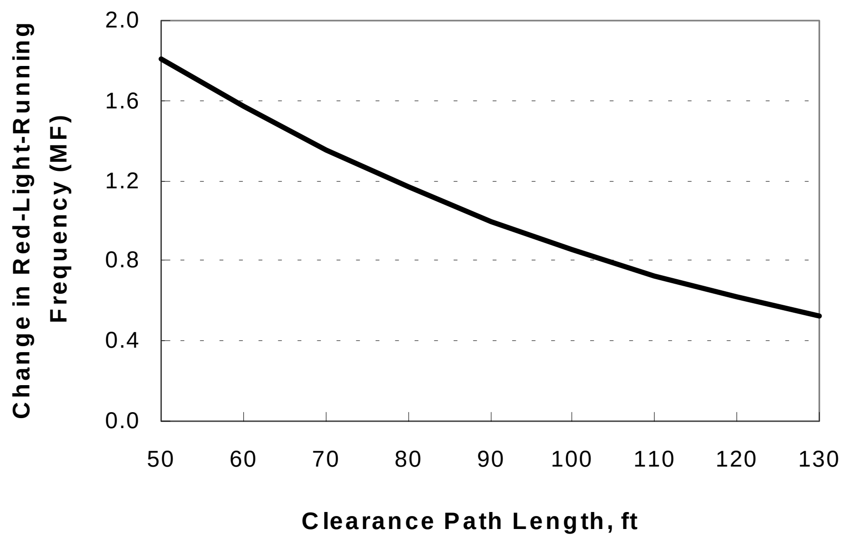

Figure A-3. Modification Factor for Clearance Path Length.

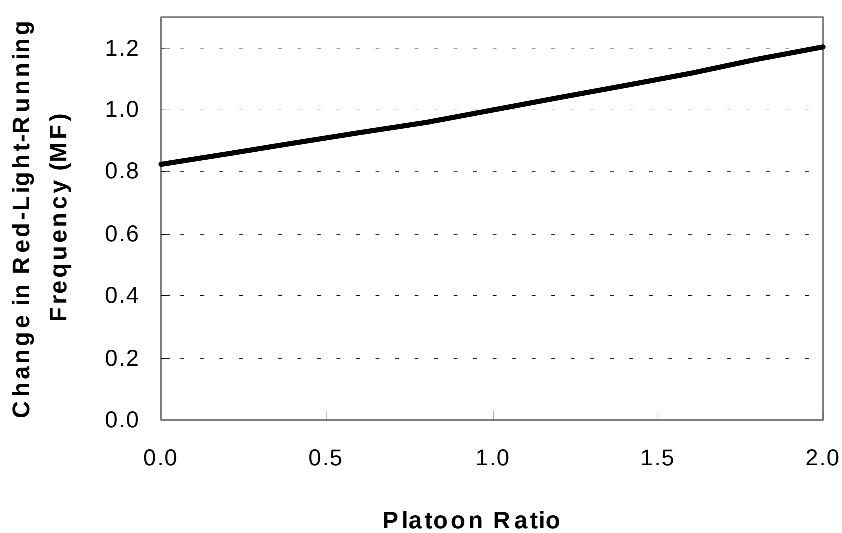

Figure A-4. Modification Factor for Platoon Ratio.

Example Application. The eastbound approach to the intersection of Main Street and Spence Street is reported to have red-light-running problem. A site survey yields the following data:

- Clearance path length L_p: 50 ft
- Distance to the most distant upstream detector D: 225 ft
- Flow rate Q: 500 veh/h (combined left, through, and right-turns in a shared lane).
- Yellow interval duration Y: 4.0 s
- Arrival type: 3 (random arrivals)
- Count of red-light-runners x: 6 veh/h
- Use of back plates: no
- Signal head diameter: 12-inch
- Number of controlling signal heads: 2
- Use of LED in the yellow indications: no
- Cycle length C: 90 s (average of 5 measurements)

Based on an arrival type of 3, the platoon ratio is identified as 1.0. The 85th percentile speed is estimated to equal the posted speed limit of 45 mph. Figure A-2 indicates that the travel time to the stop line T_D is 3.8 s. As this value is less than the yellow interval duration, the *effective* yellow interval duration is equal to 4.0 s.

To estimate the expected frequency of red-light-runners *E[R]*, the following variables are obtained from the figure or table indicated in parentheses:

- Calibration Factor C_f: 1.1 (based on a previous calibration effort)
- Propensity P_r: 0.32 (from Table A-3)
- Modification factor for clearance path length MF_L: 1.8 (from Figure A-3)
- Modification factor for platoon ratio MF_R: 1.0 (from Figure A-4)

These variables yield an expected red-light-running frequency of 3.5 veh/h. The calculation of this value is shown in Equation A-3.

$$E[R] = 1.1 \times \frac{500}{90} \times 0.32 \times 1.8 \times 1.0 = 3.5 \qquad \textbf{(A-3)}$$

Identify Red-Light-Running Index

Identify Index. During this step, the observed red-light-running frequency x is compared with that expected for the typical intersection *E[R]*. The comparison is facilitated by the use of an "index" value that compares the two frequencies in light of the uncertainty associated with their estimation. The index value is obtained from Figure A-5.

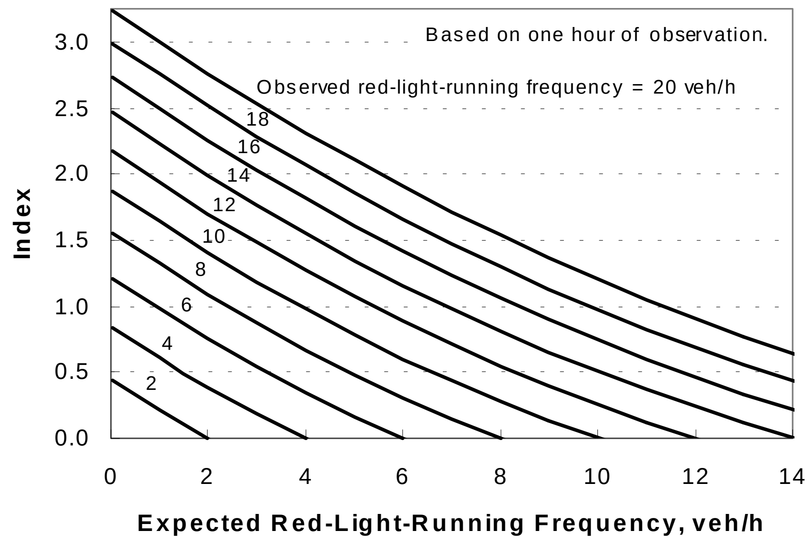

Figure A-5. Index Value Associated with the Expected and Observed Red-Light-Running Frequencies.

The extent of the red-light-running problem for the subject movement is indicated by the magnitude of its index. A movement with an index of 2.0 has a greater problem than a movement with an index of 1.0.

For a given intersection approach, a truly "problem" movement will have an index in excess of 1.0. If several movements are being evaluated on the same approach, the largest index of these movements is used to evaluate the need for treatment. If several movements are being evaluated at different intersections, their indices are ranked and those with the largest values given priority for treatment.

If the observed red-light-running frequency x is less than the expected red-light-running frequency $E[R]$ then the index is 0.0. If this situation occurs, the subject movement is not likely to have a red-light-running problem. However, the crash history should also be evaluated to confirm this finding, as described in the next section.

Example Application. A subject traffic movement is observed to have six red-light-running events during a peak traffic hour. It is also known that the "typical" movement with similar traffic and signal conditions has 3.5 red-light-running events per hour. Figure A-5 indicates that the index associated with this combination is about 0.45. This value is less than 1.0 which suggests that a modest red-light-running problem may exist; however, the extent is small and it is not known with sufficient certainty to justify treatment. The crash history at this intersection should be consulted to confirm that a problem does not exist at this location.

Assess Crash History

The crash history for the approach serving the subject movement can also be used to identify a problem location. For a given intersection approach, more than three crashes, of a type associated with red-light-running, recorded during the most recent three-year period are required to indicate a potential red-light-running problem. If this condition is satisfied and either (1) the expected red-light-running rate (= 2.8 $\times$ *E[R]* $\times$ *C/Q*) is greater than 1.0 red-light-runner per 10,000 veh-cycles or (2) the index is greater than 0.5, then a red-light-running problem is likely to exist on the subject approach. Crash types associated with red-light-running include right-angle and left-turn-related.

Identify Possible Causes

The successful treatment of an intersection approach with a red-light-running problem requires the thoughtful selection of effective countermeasures. The selection process should include consideration of the characteristics of red-light-runners at the problem intersection and the factors that may be causing drivers to run the red indication. Possible causes for red-light-running and the character of the red-light-running driver should have been obtained during the Observational Study.

The relationship between red-light-running causes and driver characteristics is illustrated in Table A-4. The engineer should attempt to identify the most likely combinations occurring for the subject traffic movement. This insight will be useful when selecting one or more candidate countermeasures, as described in the next section.

Many of the "causes" listed in Table A-4 are self-explanatory; however, a couple are worthy of added clarification. "Judged safe as driver <2-s ahead is RLR" means that the driver has judged it safe to run the red indication because they are closely following (i.e., have a headway less than 2.0 s with) another red-light-light runner. In this situation, a succession of vehicles passing through the intersection after red is quite visible to drivers in conflicting movements. Moreover, these conflicting drivers are legally required to yield the right-of-way to vehicles in the intersection before they enter it. "Expectation of green when in platoon" means that the driver was traveling along a street with a coordinated signal system. Drivers in a through movement platoon tend to develop an expectation of continued receipt of the green indication as long as they stay in the platoon. Such drivers are prone to run the red indication in order to stay within the platoon.

SELECT COUNTERMEASURES

This section describes the process for selecting the most viable countermeasures given that a red-light-running problem has been confirmed to exist. The tasks involved in making this determination are discussed in this section, they include:

a. Select Candidate Countermeasures
b. Evaluate Countermeasures

In the first task, the a set of candidate red-light-running countermeasures are selected. Then, they are evaluated and combined, as needed, to achieve a target level of acceptable red-light-running.

Table A-4. Red-Light-Running Driver Characterizations and Possible Causes.

Possible Cause of RLR [1]	Traffic Movement		Entry Time of RLR		RLR Decision Type	
	Left-Turn	Through	First Seconds of Red	Late Into Red	Unavoid-able	Avoid-able
Congestion or excessive delay	✔	✔	✔			✔
Disregard for red (low threat of citation)	✔	✔	✔	✔		✔
Judged safe due to low conflicting vol.	✔	✔	✔	✔		✔
Judged safe due to narrow cross street		✔	✔			✔
Judged safe as driver <2-s ahead is RLR	✔	✔		✔		✔
Expectation of green when in platoon		✔	✔		✔	✔
Downgrade steeper than expected		✔	✔		✔	
Speed higher than posted limit		✔	✔		✔	✔
Unable to stop (excessive deceleration)		✔	✔		✔	
Pressured by closely following vehicle		✔	✔		✔	
Tall vehicle ahead blocked view	✔	✔	✔		✔	
Unexpected, first signal encountered		✔	✔	✔	✔	
Not distracted, just did not see signal		✔	✔	✔	✔	
Distracted and did not see traffic signal		✔	✔	✔	✔	
Restricted view of signal		✔	✔		✔	
Confusing signal display	✔	✔	✔	✔	✔	

Note:
1 - RLR = red-light-runner.

Select Candidate Countermeasures

Once the more likely causes of red-light-running have been identified, the engineer can select the more effective countermeasures for formal evaluation. Table A-5 lists the countermeasure categories associated with each of the "causes" described in Table A-4.

The information in Table A-5 indicates the countermeasure categories that are most likely to have an effect on red-light-running. However, it should be noted that any countermeasure can be effective in a given situation. The list in Table A-5 is intended to guide the selection of candidate countermeasures for the more common situations. The specific countermeasures associated with each countermeasure category are listed in Table A-6.

Table A-5. RLR Problems Addressed by Engineering Countermeasure Categories.

Possible Cause of RLR	Engineering Countermeasure Category			Enforcement
	Signal Operation	Motorist Information	Physical Improvements	
Congestion or excessive delay	✔		✔	
Disregard for red (low threat of citation)				✔
Judged safe due to low conflicting vol.			✔	✔
Judged safe due to narrow cross street				✔
Judged safe as driver <2-s ahead is RLR				✔
Expectation of green when in platoon	✔			
Downgrade steeper than expected	✔			
Speed higher than posted limit	✔			
Unable to stop (excessive deceleration)	✔			
Pressured by closely following vehicle	✔			
Tall vehicle ahead blocked view		✔[1]		
Unexpected, first signal encountered		✔		
Not distracted, just did not see signal		✔		
Distracted and did not see traffic signal		✔		
Restricted view of signal		✔	✔	
Confusing signal display		✔		

Note:
1 - Motorist information for this "cause" would be provided by "advance warning signs with active flashers" or pole-mounted supplemental signal heads located on the far left- or right-side corner of the intersection.

The information presented in Table A-6 reflects the findings from several research projects. The reduction percentages listed are an indication of the relative effectiveness of each countermeasure. Data are provided only for those countermeasures for which quantitative information is available. However, all countermeasures listed are expected to have the potential to reduce red-light-running. The modification factors listed in column 4 of Table A-6 are discussed in the next section.

At the conclusion of this task, the engineer should have identified one or more of the countermeasures listed in Table A-6. Candidate countermeasures are those that are appropriate for the type of red-light-running that has been observed on the subject intersection approach.

Table A-6. Engineering Countermeasures to Red-Light-Running.

Countermeasure Category	Specific Countermeasure	RLR Effectiveness [1]	
		Reduction	Mod. Factor
Signal Operation (modify signal phasing, cycle length, or change interval)	Increase the yellow interval duration	-50 to -70 %	Figure A-7
	Provide green-extension (advance detection)	-45 to -65 %	Figure A-7 [2]
	Improve signal coordination	Varies [3]	Figure A-8 [3]
	Improve signal operation (increase cycle length 20 s)	-15 to -25 % [4]	Figure A-9 [4]
Motorist Information (provide advance information or improved notification)	Improve sight distance	--	--
	Improve visibility of signal (12" lens, add heads)	--	--
	Improve visibility of signal with yellow LEDs	-13 %	0.87
	Increase conspicuity of signal with back plates	-25 %	0.75
	Add advance warning signs without flashers	--	--
	Add advance warning signs with active flashers	-29 %	0.71
Physical Improvement (implement safety or operational improvements)	Remove unneeded signals	-100 %	0.0
	Add capacity with additional traffic lanes	--	--
	Flatten sharp curves	--	--

Note:
1 - "–": data not available.
2 - The modification factor for advance detection is obtained from Figure A-7 with the "change in yellow duration" equal to the difference Y^* - Y where Y^* is obtained from Equation A-1.
3 - Red-light-running frequency is likely to increase with improved coordination; however, this increase may be offset by the larger cycle length typically required for good progression.
4 - Reductions associated with an increase in cycle length may not be realized if motorist delay increases significantly.

Evaluate Countermeasures

During this task, a target red-light-running frequency is specified. Then, one or more viable countermeasures are evaluated to verify that the target frequency can be achieved. The combined effect of several countermeasures is represented by the joint modification factor *JMF*. This factor can be computed using the following equation:

$$JMF = MF_1 \times MF_2 \ldots. \times MF_n \qquad \textbf{(A-4)}$$

where, MF_i equals the modification factor associated with countermeasure *i*.

Identify Target Modification Factor

The "target" modification factor can be obtained from Figure A-6. If the joint modification factor from one or more countermeasures combine to yield a value equal to the target value, then the frequency of red-light-running for the subject movement should equal that of the "typical" movement. Thus, the target modification factor represents a reasonable criterion by which alternative countermeasures are evaluated or combined with other countermeasures.

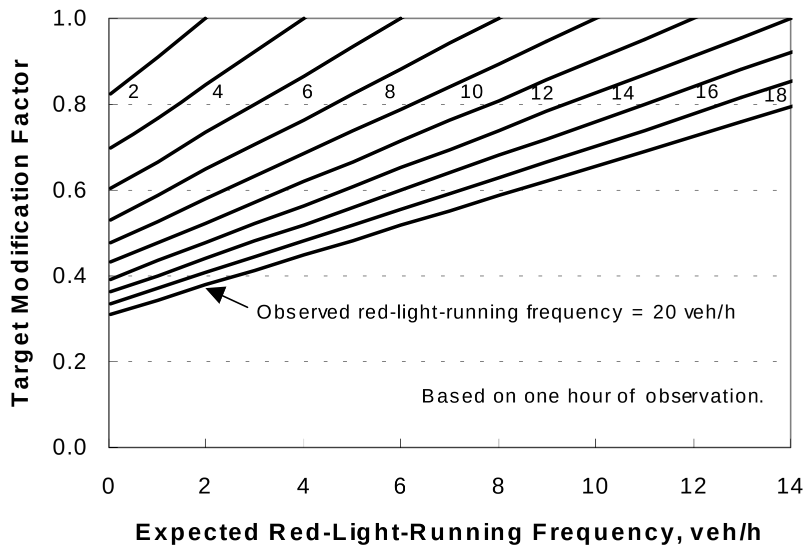

Figure A-6. Target Modification Factor.

Compute Joint Modification Factor

The effectiveness of the countermeasures is combined in this step using Equation A-4. Specifically, the modification factors for the various countermeasures are multiplied together to obtain the *JMF*. The objective is to apply a sufficient number of countermeasures to yield a *JMF* that is equal to, or less than, the target modification factor. The modification factors are listed in column 4 of Table A-6. This column references Figures A-7, A-8, and A-9 as the source for modification factors associated with yellow interval duration, platoon ratio, and cycle length, respectively.

Compute Crash Reduction Factor

The effect of the countermeasures on red-light-running-related crash frequency can be evaluated using the *JMF* associated with these countermeasures. Specifically, Figure A-10 can be used for this purpose. The *JMF* represents the proposed change in red-light-running frequency (i.e., the *x*-axis). The trend line shown can be used to relate this change to that in crash frequency.

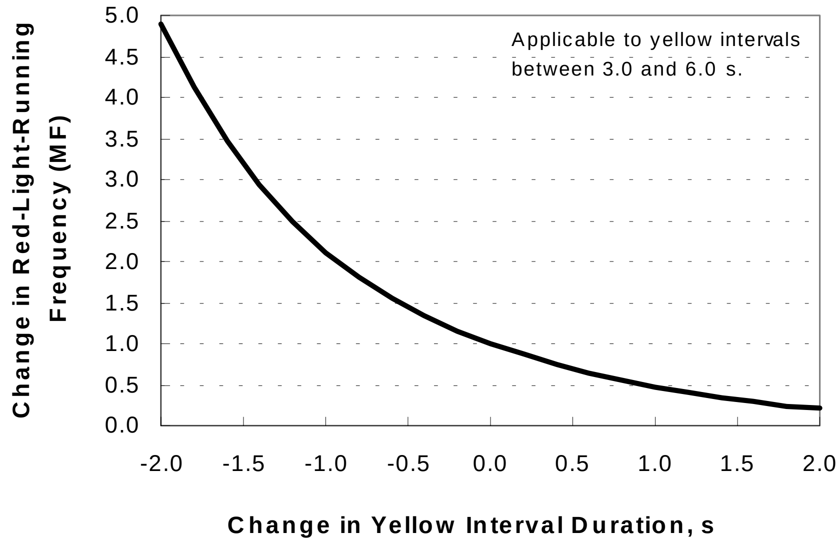

Figure A-7. Modification Factor for Yellow Duration.

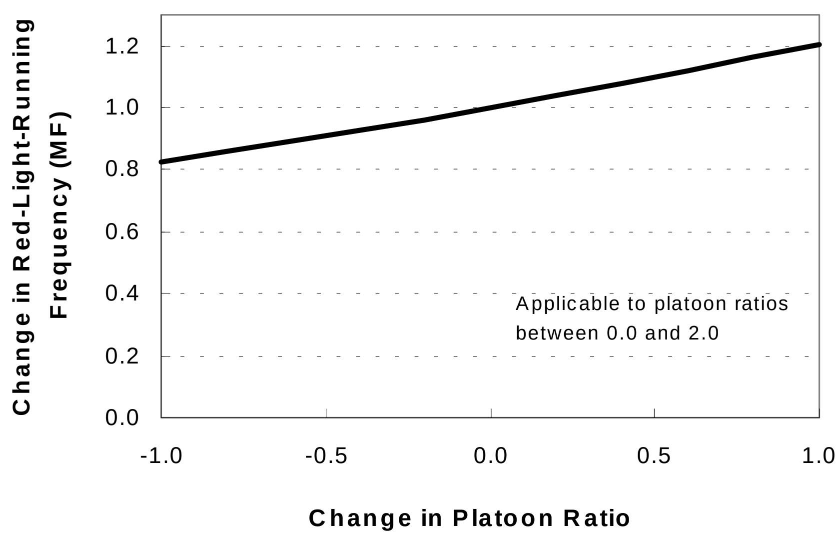

Figure A-8. Modification Factor for Platoon Ratio.

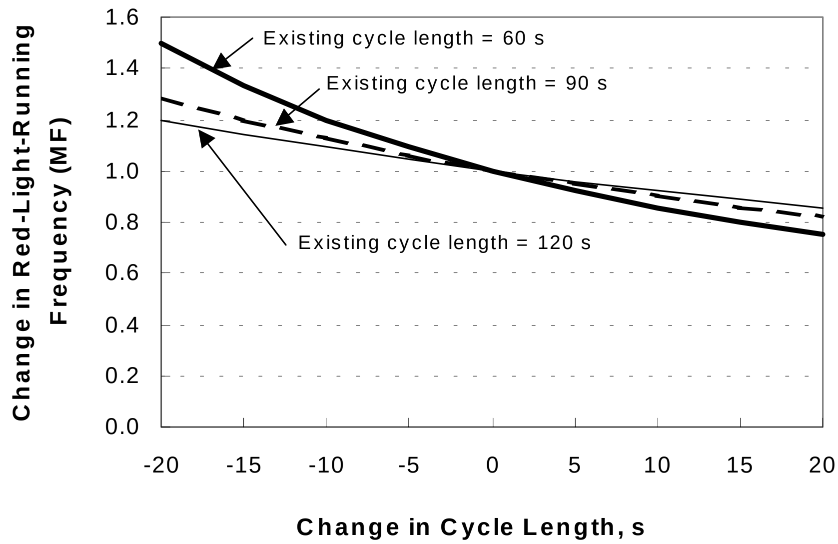

Figure A-9. Modification Factor for Cycle Length.

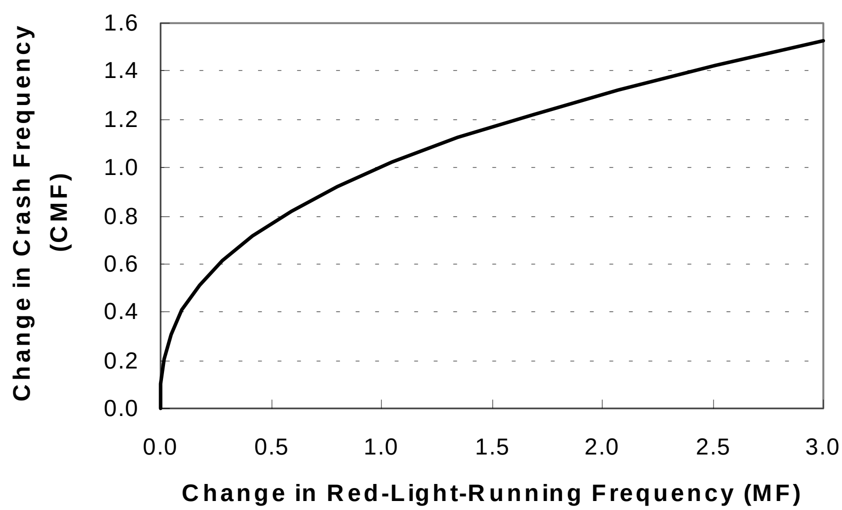

Figure A-10. Change in Crash Frequency Resulting from a Change in Red-Light-Running.

The crash modification factor *CMF* obtained from Figure A-10 can be converted into a percentage change in crashes C_r using the following equation:

$$C_r = 100 \times (CMF - 1) \tag{A-5}$$

where, C_r is the percent change in red-light-running-related crash frequency. Positive values of C_r indicate an increase in crash frequency. Negative values indicate a reduction in crashes.

Example Application

The eastbound approach to the intersection of Main Street and Spence Street is reported to have red-light-running problem. Four right-angle crashes have been associated with this approach during the previous three-year period. During a one-hour site survey, six red-light-runners were observed for the eastbound through movement (other findings from this survey were described in a preceding section).

The expected red-light-running frequency for the typical through movement was computed as 3.5 veh/h. The index value from Figure A-5 is relatively low at 0.45 but the expected red-light-running rate is still above average at 1.8 red-light-runners per 10,000 veh-cycles (= 2.8 × 3.5 × 90/500). Because this rate exceeds 1.0 and the number of crashes exceeds three, the approach is judged to be worthy of treatment. A review of possible red-light-running causes indicates that improvements to signal visibility or conspicuity could be helpful. A modification of the advance detection design, with the detectors being relocated further from the stop line, is also being considered.

A check of Figure A-6 indicates that the target modification factor is 0.82. The modification factors listed in column 4 of Table A-6 indicate that signal head back plates (*MF* = 0.75) should be able to return the frequency of red-light-running to a more typical value (i.e., 0.75 is less than 0.82). On the other hand, the use of yellow LED indications alone is not adequate because its *MF* (= 0.87) is not less than 0.82.

To illustrate the use of Equation A-4, assume that 10 red-light-runners were observed during the one-hour study. In this case, the target modification factor is 0.67. Neither back plates nor yellow LED indications alone will provide the desired reduction in red-light-running. However, if both countermeasures are used, their *JMF* of 0.65 (= 0.87 × 0.75) is less than 0.67. Therefore, the combined use of both countermeasures should provide the desired reduction in red-light-running.

As an alternative to the use of LEDs or back plates, consider a modification of the advance detection design. The engineer decides on a detection design with the most distant detector located 330 ft from the stop line. This translates into an effective yellow interval duration of 5.5 s based on the trends in Figure A-2. Given that the existing yellow duration is 4.0 s, the advance detection system effectively increases the yellow duration by 1.5 s. Based on Figure A-7, this 1.5-s change translates into an *MF* of 0.31. This countermeasure alone is more than sufficient to lower the frequency of red-light-running below the target value.

Finally, Figure A-10 is used to evaluate the effect of the countermeasures on crash frequency. The trend line in this figure indicates that the *MF* associated with the signal head back plates (0.75) should yield a crash modification factor *CMF* of 0.9. Using Equation A-5, this *CMF* translates into a crash reduction of 10 percent.

CALIBRATION PROCEDURE

Equation A-2 is intended for use by transportation agencies located throughout the state of Texas. However, levels of enforcement and driver behavior differ among the various jurisdictions for which these agencies are responsible. These differences lead to different levels of red-light-running among jurisdictions and require the calibration of Equation A-2 before it can be used for selecting and evaluating countermeasures.

The calibration process is essentially one of determining the value of the calibration factor C_f that, when used with Equation A-2, facilitates the prediction of the red-light-running frequency consistent with the typical intersection traffic movement. It is anticipated that the calibration process will be completed once, prior to first use of Equation A-2. However, the agency may find it useful to update the calibration factor periodically, especially if regional programs have been implemented to reduce red-light-running through enforcement or driver education.

A separate calibration factor can be computed for each of the following traffic movements:

- a left-turn movement in an exclusive lane,
- a right-turn movement in an exclusive lane and controlled by a signal, and
- any combination of movements that are served in a lane shared with the through movement.

A calibration factor for each movement can provide an added degree of refinement to Equation A-2. To develop these factors, the calibration process described in this section is applied once for each traffic movement for which a calibration factor is desired. As a minimum, a calibration factor should be developed for the through movement (and any turn movements that share a lane with the through movement) as this is the most common intersection traffic movement.

The calibration process consists of the following steps:

1. **Specify Subject Movement.** Identify the traffic movement for which the calibration factor will be defined. This movement represents the "subject" movement referenced in the remaining steps.

2. **Identify Typical Intersection Approaches.** Several intersections should be selected for use in the calibration process. These intersections must not be known to have an unusually large

frequency of red-light-running. Moreover, their traffic volume, signalization, and geometry should be considered typical and should reflect some range in volume level, cycle length, and number of lanes. A minimum of four intersection approaches at two or more intersections should be selected. The selection of more than four intersection approaches and more than two intersections is desirable. The subject movement must be served on each approach studied.

3. **Collect Data at Selected Intersections.** A site survey (as described previously in the section titled Site Survey) is conducted for each of the subject movements and approaches selected in Step 2.

4. **Compute Expected Frequency of Red-Light-Running.** Equation A-2 is used (with C_f = 1.0) to compute the expected red-light-running frequency $E[R]$ for each subject movement.

5. **Compute Calibration Factor.** The following equation is used to compute the calibration factor for the subject movement:

$$C_f = \frac{x_1 + x_2 + x_3 + ... + x_n}{E[R]_1 + E[R]_2 + E[R]_3 + ... + E[R]_n} \qquad \textbf{(A-6)}$$

where, x_i is the observed red-light-running frequency for the subject movement on approach i and $E[R]_i$ is the expected red-light-running frequency. The values of x_i were obtained during the site survey in Step 3. The values of $E[R]_i$ were computed in Step 4.