

**SMART RED CLEARANCE EXTENSIONS
TO REDUCE RED-LIGHT RUNNING
CRASHES**

Final Report

SPR 773



Oregon Department of Transportation

SMART RED CLEARANCE EXTENSIONS TO REDUCE RED-LIGHT RUNNING CRASHES

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by

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16. Abstract Red-light running (RLR) is a safety hazard at signalized intersections in Oregon and around the United States. RLR crashes occurring during phase transitions at signalized intersections can be addressed through the use of Red Light Extension (RLE) systems. This research project evaluated the accuracy, efficiency, and operations of three RLE system alternatives in Oregon. A field study was conducted at five intersections in Oregon with high rates of RLR vehicles. To understand driver behavior in response to the onset of the yellow change interval (on major approaches) or the green indication (on minor approaches), 149 h of video data were transcribed. Together with signal timings, geometries, peak volumes, and operating speeds, the video data were used to calibrate a VISSIM hardware-in-the-loop (HIL) simulation of one intersection. The HIL simulation was used to code and evaluate the performances of three alternative RLE systems. Novel code was developed in R to process the VISSIM output into time-space diagrams (TSDs) for further analysis. RLR vehicles triggered the highest rate of RLE events when the downstream detection alternative was used. However, an examination of the TSDs showed that the RLE events triggered by the smart upstream speed-conditional detection alternatives (at 215 ft. & 475 ft.) consistently provided a greater degree of safety for vehicles that received a RLE while no significant increase to intersection delay occurred. Use of the 2070 controller and the NWS Voyage TM software allows the logic to be field implemented in the future.					
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*SI is the symbol for the International System of Measurement

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1.0 INTRODUCTION

Red-light running (RLR) is a safety hazard at signalized intersections in Oregon and throughout the United States. The Federal Highway Administration (FHWA) reports that there are more than 3 million intersections in the United States alone, at least 300,000 of which are signalized (FHWA 2014c). In 2013, the National Highway Traffic Safety Administration's Fatality Analysis Reporting System reported 697 deaths caused by RLR crashes, according to the National Coalition for Safer Roads (National Coalition for Safer Roads 2014a). An estimated 127,000 people are injured each year due to RLR (FHWA 2014b). In Oregon, on average, 72 fatalities (including both RLR and non RLR events) occur at intersections each year (ODOT 2014a). One study in Arlington, Virginia, found an average of three RLR vehicles every hour (equivalent to one RLR vehicle every 20 min) (Retting et al. 1998). Other studies reported RLR violation rates as high as 18 violations per hour at a single intersection (ITE 2003). A report recently completed by the National Coalition for Safer Roads (National Coalition for Safer Roads 2014b) looked at RLR trends in 2013 using data from 2,216 red-light safety cameras in 20 states. In a single year, these cameras captured 3,560,724 RLR violations (an average of 9,755 violations per day). **Error! Reference source not found.** Figure 1.1 shows a crash that occurred as a result of a vehicle running a red light.

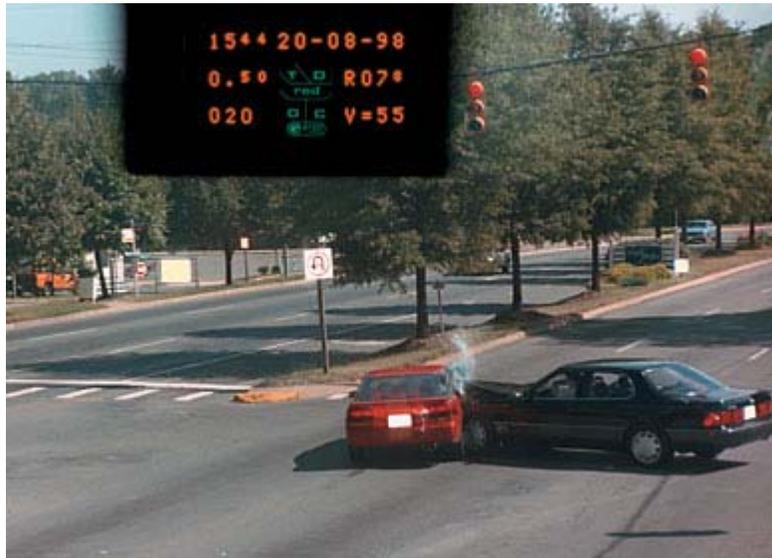


Figure 1.1: RLR crash (Retting et al. 1995)

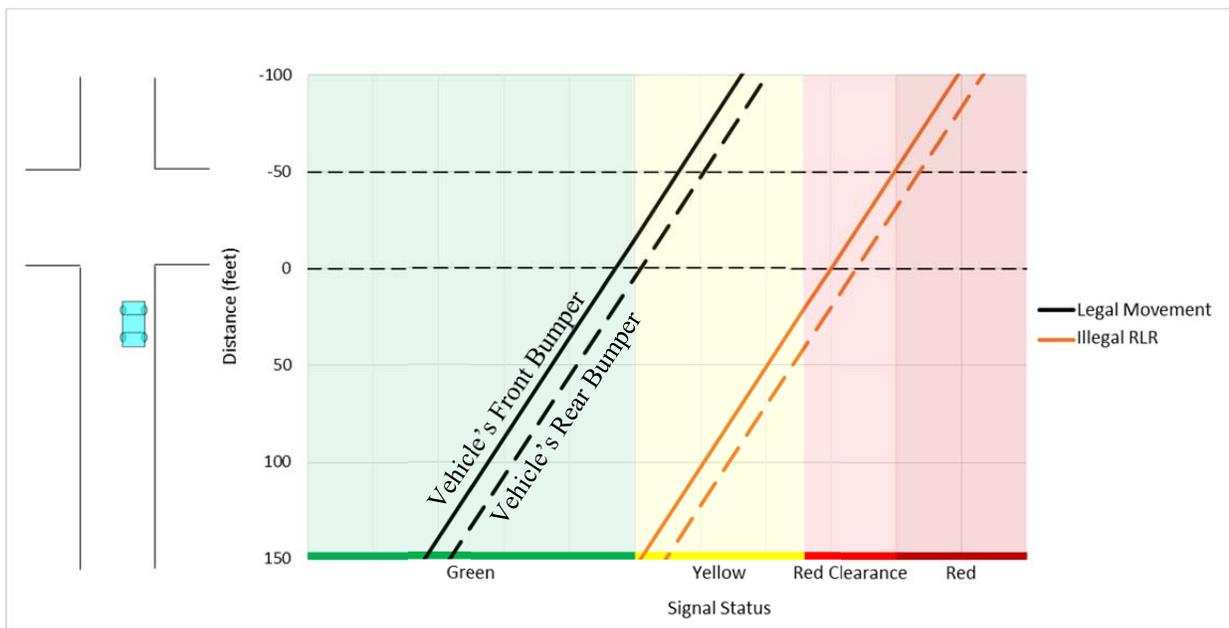
Various countermeasures have been proposed to mitigate factors contributing to RLR, but few of these countermeasures address RLR by implementing protection when a RLR vehicle is detected. Red clearance extension is one possible countermeasure that provides intersection protection by extending the red clearance (all-red) interval if a RLR vehicle is detected, allowing the RLR vehicle to clear the conflict zone with opposing traffic. The City of Portland installed red clearance extension systems at eight different intersections between 2005 and 2009 (Olson 2012). The Oregon Department of Transportation (ODOT) currently uses VoyageTM traffic

controllers, which have the ability to trigger a red clearance extension. However, research is needed to determine the best practices for detecting and predicting RLR vehicles and for extending the red clearance interval.

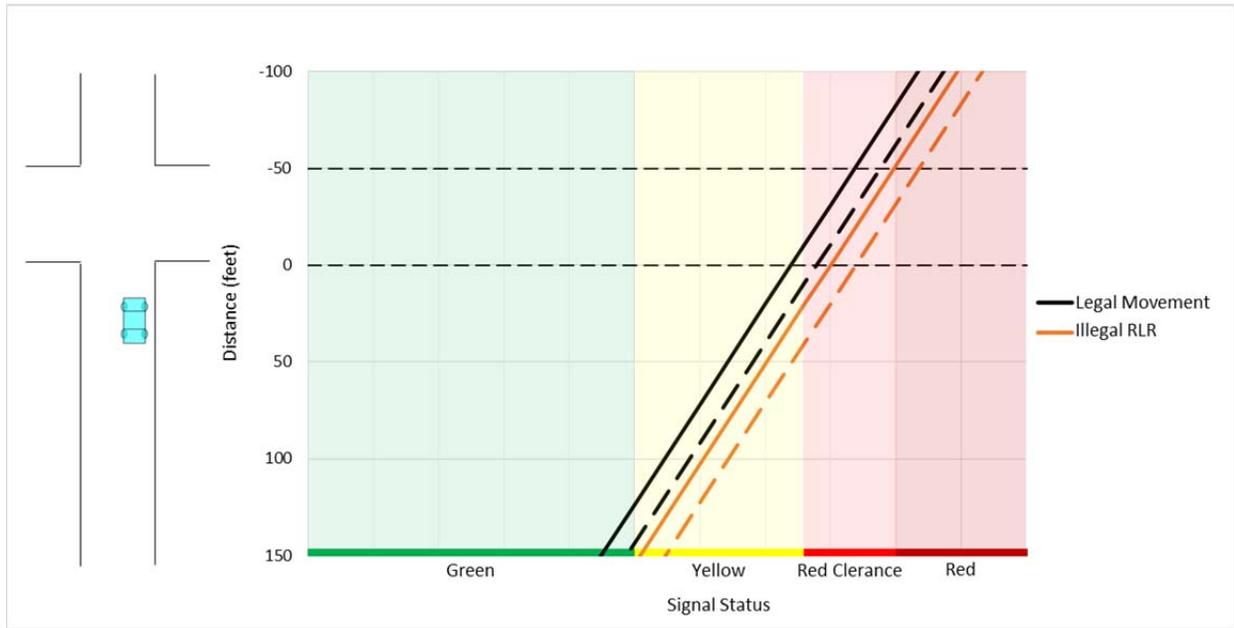
1.1 DRIVER RESPONSE TO YELLOW CHANGE AND RED CLEARANCE INTERVALS

The correct driver response to the yellow change interval is dependent on the type of yellow law. States with “permissive yellow” laws allow drivers to enter the intersection legally at any time during the yellow interval. In this case, vehicles are deemed to be RLR if they enter the intersection during a red indication. States with “restrictive yellow” laws, such as Oregon, require drivers to clear the intersection before onset of the red indication, if it is safe to do so (FHWA 2014a).

Figure 1.2 provides a classification scheme for legal and illegal movements in response to yellow change intervals, using time-space diagrams (TSDs) to distinguish between permissive and restrictive yellow laws. The example intersection has a width of 50 ft and an approach speed of 35 mph. A plan view of the intersection is shown to the left of the TSD. The y-axis displays the distance from the stop line, starting 150 ft upstream of the intersection. The x-axis displays the signal status. The yellow duration is 3.6 s, and the red clearance interval is 1.4 s. An individual vehicle trajectory is represented with a solid line for the vehicle’s front bumper and a dashed line for the vehicle’s rear bumper. Vehicle trajectories for the latest possible legal movement and an illegal RLR movement are included as they relate to restrictive and permissive yellow laws.



(a) Legal and illegal vehicle trajectories in Oregon (restrictive yellow laws)



(b) Legal and illegal vehicle trajectories in other states with permissive yellow laws

Figure 1.2: Legal and Illegal movements with restrictive vs. permissive yellow laws

McGee et al. (McGee et al. 2012) discussed the impact of differing yellow signal laws on timing practices for yellow change and red clearance intervals. According to the definitions above, states with restrictive yellow laws, such as Oregon, would ideally use the yellow change interval to provide the yellow change and red clearance durations; these states would not use a red clearance interval. In states with permissive yellow laws, the red clearance interval provides an additional buffer for vehicles in the intersection as the light turns red. Many states with restrictive yellow laws follow permissive timing recommendations, using both yellow change and red clearance intervals.

Oregon State regulations and the *Oregon Driver Manual* instruct drivers to stop at a circular yellow indication unless it is unsafe to do so, reflecting a restrictive yellow law (ODOT 2014). However, Oregon is bordered entirely by states with permissive yellow laws (Figure 1.3). Drivers entering Oregon from adjacent states may be unaware of the difference in laws and may assume that a permissive law governs yellow lights in Oregon. Table 1.1 reports the language used in the regulations and driver manuals in Oregon and its adjacent states.

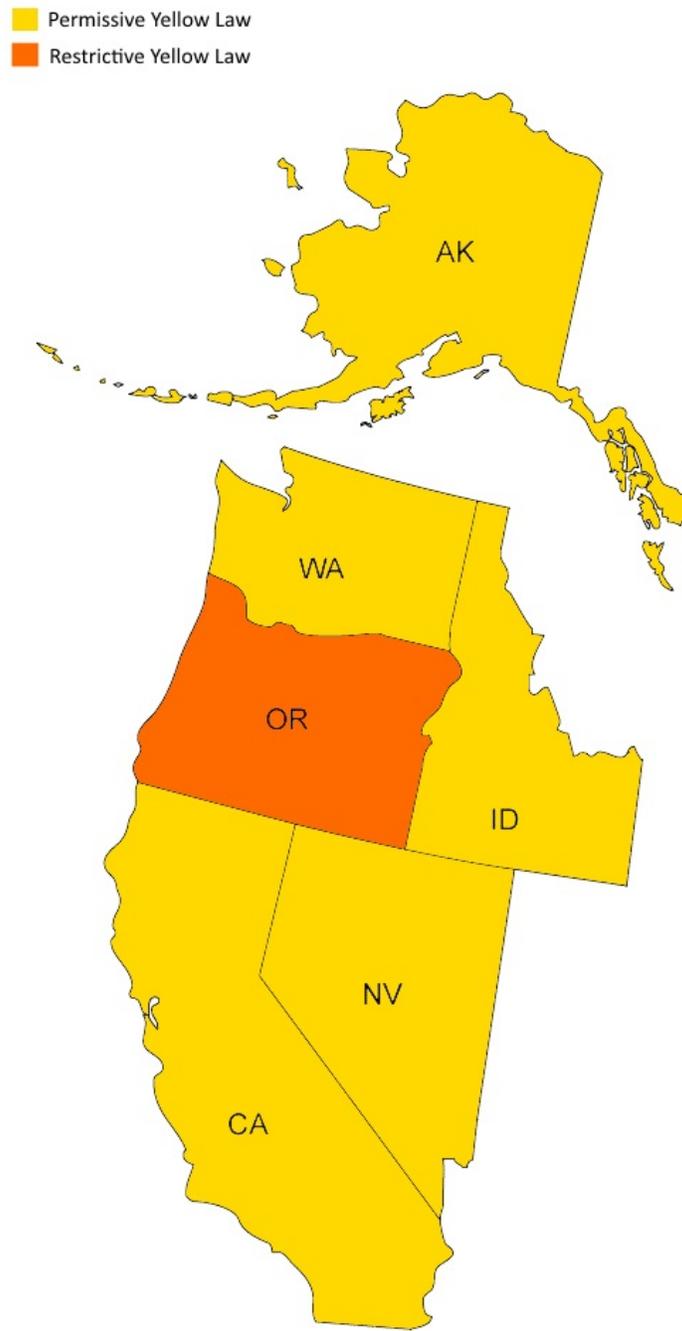


Figure 1.3: Permissive vs. restrictive yellow laws by state

Table 1.1: Yellow signal indication language and categories for Oregon and surrounding states

State (Law)	Steady Yellow Language	Driver Manual Language
Oregon (Restrictive)	<p>“Steady circular yellow signal. A driver facing a steady circular yellow signal light is thereby warned that the related right of way is being terminated and that a red or flashing red light will be shown immediately. A driver facing the light shall stop at a clearly marked stop line, but if none, shall stop before entering the marked crosswalk on the near side of the intersection, or if there is no marked crosswalk, then before entering the intersection. If a driver cannot stop in safety, the driver may drive cautiously through the intersection.” ORS 811.260 <i>(Oregon State Legislature 2013)</i></p>	<p>“Steady Yellow - A steady yellow signal warns you that the signal is about to turn red. Stop before entering the intersection. If you cannot stop safely, you may then drive cautiously through the intersection. Cautiously means slowly and carefully. Pedestrians facing a yellow light must not start across the street unless a pedestrian signal directs otherwise.” <i>(ODOT 2014)</i></p>
Washington (Permissive)	<p>“Vehicle operators facing a steady circular yellow or yellow arrow signal are thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection.” Revised Code of Washington (RCW) 46.61.055 <i>(Washington State Legislature undated)</i></p>	<p>“A steady yellow traffic light means the traffic light is about to change to red. You must stop if it is safe to do so. If you are in the intersection when the yellow light comes on, do not stop but continue through the intersection.” <i>(Washington State Legislature undated)</i></p>
California (Permissive)	<p>“A driver facing a steady circular yellow or yellow arrow signal is, by that signal, warned that the related green movement is ending or that a red indication will be shown immediately thereafter.” California Vehicle Code 21452 <i>(California Department of Motor Vehicles 2015)</i></p>	<p>“Solid Yellow – A yellow signal light means ‘CAUTION’. The red signal is about to appear. When you see the yellow light, stop if you can do so safely. If you cannot stop safely, cross the intersection cautiously.” <i>(California Department of Motor Vehicles 2014)</i></p>
Alaska (Permissive)	<p>“Steady yellow indication (A) vehicular traffic facing a steady yellow signal is warned that the movement allowed under [green indication] of this section is being</p>	<p>“YELLOW BALL A red light is about to appear. Stop unless you are already within the intersection, or so close to the intersection that you</p>

	<p>terminated and that a red indication will be exhibited immediately following the yellow indication” 13 Alaska Administrative Code (AAC) 02.010 (<i>Alaska Department of Administration undated</i>)</p>	<p>cannot stop safely. If the light changes to yellow as you enter the intersection, you may proceed with extreme caution.” (<i>Alaska Department of Administration 2013</i>)</p>
<p>Idaho (Permissive)</p>	<p>“Steady yellow indication: (a) A driver facing a steady circular yellow or yellow arrow signal is being warned that the related green movement is ending, or that a red indication will be shown immediately after it.” IC §. 49-802 (<i>Idaho State Legislature undated</i>)</p>	<p>“Yellow Light: Means caution. An amber or yellow circular indication warns that the signal is about to change to red. If you have not entered the intersection and can come to a safe stop, you should do so. If you are already in the intersection, you should continue moving and clear it safely.” (<i>Idaho Transportation Department 2014</i>)</p>
<p>Nevada (Permissive)</p>	<p>“Where the signal is a steady yellow signal alone: (a) Vehicular traffic facing the signal is thereby warned that the related green movement is being terminated or that a steady red indication will be exhibited immediately thereafter, and such vehicular traffic must not enter the intersection when the red signal is exhibited.” NRS 484B.307 (<i>Nevada State Legislature undated</i>)</p>	<p>“A yellow light means CAUTION. A steady yellow light is a warning that the light will be turning red. If you have not entered the intersection, you must stop. If you are already in the intersection, you should continue moving and clear it safely. DO NOT speed up to “beat the light.” (<i>Nevada Department of Motor Vehicles 2013</i>)</p>

1.2 ORGANIZATION OF THE REPORT

This Final Report is divided into five Chapters, which discuss the development of a novel red clearance extension system for signalized intersections in Oregon. Chapter 2 provides a review of the literature and current practice. Chapter 3 describes the results of video, speed, and intersection inventory data collected at five signalized intersections in Oregon. Chapter 4 presents the results of a hardware-in-the-loop (HIL) simulation conducted by using existing conditions at one of the signalized intersections evaluated in Chapter 3. Finally, Chapter 5 synthesizes the results of Chapters 3 and 4 into recommendations for red clearance extension systems in Oregon and presents opportunities for future research.

2.0 LITERATURE REVIEW

2.1 OBJECTIVES

This Final Report is divided into five Chapters, which discuss the development of a novel red clearance extension system for signalized intersections in Oregon. Chapter 2 provides a review of the literature and current practice. Chapter 3 describes the results of video, speed, and intersection inventory data collected at five signalized intersections in Oregon. Chapter 4 presents the results of a hardware-in-the-loop (HIL) simulation conducted by using existing conditions at one of the signalized intersections evaluated in Chapter 3. Finally, Chapter 5 synthesizes the results of Chapters 3 and 4 into recommendations for red clearance extension systems in Oregon and presents opportunities for future research.

2.2 FACTORS RELATED TO RED-LIGHT RUNNING

An important first step in determining strategies to mitigate RLR is identifying factors that play a role in RLR. Bonneson et al. (*Bonneson et al. 2002*) proposed two categories of factors that contribute to RLR: exposure and contributory factors. A third category, conflict factors, was proposed for vehicles that run the red light. The following subsections discuss the individual factors and supporting literature that fall into each of these three categories.

2.2.1 Exposure Factors

Exposure factors are precursor events that expose a driver to a situation where he or she must make a decision either to stop or proceed through an intersection (*Bonneson et al. 2002*). In their report, Bonneson et al. (*Bonneson et al. 2002*) found that the following exposure factors can affect RLR rates:

- **Flow Rate of the Subject's Approach:** Three studies (*Kamyab et al. 2000, Baguley 1988, Mohamedshah et al. 2000*) reported sufficient data supporting an increase in RLR frequency as the approach flow rate increases.
- **Number of Signal Cycles:** Longer cycle lengths decrease the frequency per unit of time that a circular yellow indication is presented. (*Bonneson et al. 2002*) recommended that RLR statistics be normalized by the cycle frequency.
- **Phase Termination by Max-Out:** Green-light extension systems are used to extend the green phase if the approach is occupied (*Kyte and Urbanik 2012, FHWA 2009*). Green-light extension reduces the number of vehicles that are presented with a circular yellow indication. The green phase is extended to some maximum limit, at which point it is forced to end ("max-out"), regardless of the presence of vehicles in the approach. Exposure to the circular yellow indication can lead to RLR situations. Pretimed signals have a similar effect, as they end regardless of vehicle presence.

- **Flow Rate of the Conflicting Approach:** A study by Mohamedshah et al. (*Mohamedshah 2000*) found that the probability of RLR crashes on the major street increased with increasing volume on the minor street.

2.2.2 Contributory Factors

Unlike exposure factors that create opportunities for RLR, contributory factors can directly contribute to RLR events. The literature identifies the following contributory factors:

- **Probability of Stopping:** A driver's probability of stopping in response to a circular yellow indication is dependent on numerous factors, including the travel time to the stop line at the onset of the circular yellow indication, headway between vehicles ahead and behind, signal coordination, signal actuation, approach grade, speed, and duration of the yellow change interval. Consequences of not stopping include threats of a right-angle crash and citation. Consequences of stopping include the threat of a rear-end crash and expected delay (*Bonneson et al. 2002*).
- **Duration of the Yellow Change Interval:** An improperly timed yellow change interval, specifically one timed too short, can contribute to RLR. In this case, a Type I dilemma zone is created, in which a vehicle can neither safely clear the intersection nor come to a comfortable stop at the stop line (*Gazis et al. 1960*). Long yellow change intervals can lead to disobedience; drivers are tempted to enter the intersection later in the yellow (*Awadallah 2009*) when they are not "rewarded" with a circular red indication if they come to a stop at the stop line (*Bonneson et al. 2002*).

2.2.3 Conflict Factors

It is possible for a conflict to occur if a driver makes an incorrect decision during the yellow change interval and runs the red light. Three factors related to this conflict are as follows:

- **Duration of the Red Clearance Interval:** The *Manual on Uniform Traffic Control Devices (MUTCD)* leaves the decision of whether to include a red clearance interval in the signal timing up to engineering judgment (*FHWA 2009*). Use and duration of red clearance intervals vary by jurisdiction. Improper timing can lead to a conflict if the red clearance interval is insufficient for a vehicle entering at the end of the yellow change interval to clear the intersection before the end of the red clearance interval (*Bonneson et al. 2002*).
- **Entry Time of the Conflicting Driver:** The entry time of a conflicting driver after being given a circular green indication can lead to a conflict with a RLR vehicle, which can be compounded by unique geometries. If the RLR vehicle is still clearing the intersection at the onset of the circular green indication for conflicting traffic, and the conflicting driver reaches the conflict zone before the RLR vehicle clears it, then a conflict will occur. This situation is especially important if the conflicting driver's vehicle is still in motion at the onset of the circular green indication, and it enters the intersection sooner than a stopped vehicle (*Bonneson et al. 2002*). This scenario is discussed further in Section 3.5.

Finally, a FHWA report on the operational guidelines of red-light camera systems (*FHWA 2005*) identified driver behavior, intersection design and operation, vehicle characteristics, and weather as additional factors contributing to RLR.

2.3 SIGNAL TIMING PRACTICES

The *MUTCD (FHWA 2009)* does not prescribe a standard method for determining the duration of the yellow change or red clearance interval. The only provided direction is that these durations should be determined by using engineering practices, and that the yellow change interval should be between 3 and 6 s in duration. The *MUTCD* includes a support statement indicating that engineering practices can be found in the Institute of Transportation Engineer's (ITE's) *Traffic Control Devices Handbook (ITE 2013)* and *Manual of Traffic Signal Design (ITE 1998)*. Guidelines of the ITE for determining the yellow change and red clearance intervals have evolved over time, based on research and practical application (*Eccles and McGee 2001*).

2.3.1 Yellow Change Interval

The 2009 *MUTCD* provides the following information on the yellow change interval:

“Steady yellow signal indications shall have the following meanings:

Vehicular traffic facing a steady CIRCULAR YELLOW signal indication is thereby warned that the related green movement or the related flashing arrow movement is being terminated or that a steady red signal indication will be displayed immediately thereafter when vehicular traffic shall not enter the intersection. The rules set forth concerning vehicular operation under the movement(s) being terminated shall continue to apply while the steady CIRCULAR YELLOW signal indication is displayed.”

And:

“Standard:

The duration of a yellow change interval shall not vary on a cycle-by-cycle basis within the same signal timing plan.”

And:

“Guidance:

A yellow change interval should have a minimum duration of 3 seconds and a maximum duration of 6 seconds. The longer intervals should be reserved for use on approaches with higher speeds.” (*FHWA 2009*)

Based on these definitions, the yellow change interval must allow the driver to see the circular yellow indication, decide whether to stop or proceed through the intersection, and comfortably stop or proceed through the intersection safely. Over the last 70 years, the ITE has published guidance for timing the yellow change and red clearance intervals. Current guidance has evolved from the 1965 guidelines, which took the form of a standard kinematic equation, to include the effects of grade, perception-reaction time (PRT), deceleration rate, and approach speed (*Eccles and McGee 2001*).

2.3.2 Red Clearance Interval

The 2009 *MUTCD* provides the following information on the red clearance interval:

“Guidance:

When indicated by the application of engineering practices, the yellow change interval should be followed by a red clearance interval to provide additional time before conflicting traffic movements, including pedestrians, are released.

Standard:

When used, the duration of the red clearance interval shall be determined using engineering practices.”

And:

“Standard:

Except as provided in Paragraph 12, the duration of a red clearance interval shall not be decreased or omitted on a cycle-by-cycle basis within the same signal timing plan.

Option:

The duration of a red clearance interval may be extended from its predetermined value for a given cycle based upon the detection of a vehicle that is predicted to violate the red signal indication.”

And:

“Guidance:

Except when clearing a one-lane, two-way facility (see Section 4H.02) or when clearing an exceptionally wide intersection, a red clearance interval should have a duration not exceeding 6 seconds.”(*FHWA 2009*)

Hence, engineering judgment should be used to determine if a red clearance interval is necessary to provide additional time to allow the intersection to clear before conflicting traffic movements are given the green indication. The concept of the red clearance interval was first introduced in the ITE’s *Traffic Engineering Handbook (ITE 1950)*. Guidance for the red clearance interval has evolved from simply a 1- to 2-s interval if the calculated yellow change interval exceeds 5 s, to a choice of three equations to calculate the red clearance interval, to the current guidance, in which the red clearance interval is calculated from the intersection width, vehicle length, and approach speed (*Eccles and McGee 2001*). In addition, the *MUTCD* allows for the use of red clearance extensions (Section 4D.26).

2.3.3 Current Guidance

2.3.3.1 ITE’s *Traffic Engineering Handbook*

The 6th edition of the ITE’s *Traffic Engineering Handbook (ITE 2010)* provides the most current equation for determining the yellow change interval:

$$Y = t + \frac{v}{2a + 2Gg} \quad (2.1)$$

where Y is the yellow clearance interval (s); t is the PRT (s); v is the design speed (ft/s); a is the deceleration rate (ft/s²); g is the acceleration due to gravity (32.2 ft/s²); and G is the grade of the approach (%/100, ft/ft; downhill is negative grade). This equation accounts for the PRT of the driver and the time required for a vehicle to decelerate comfortably to a stop, considering the speed and grade of the approach. Typically, a deceleration rate of 10 ft/s² (3.1 m/s²) and PRT of 1 s are used, but engineering judgment should be applied to determine the appropriateness of these terms for a given intersection. The 15th percentile (%ile) speed should be considered, because wide intersections may require a longer yellow change interval. If the calculated yellow change interval exceeds 5 s, then a red clearance interval is typically used to provide additional time (*ITE 2010*).

The ITE (*ITE 2010*) also provides guidance for determining the red clearance interval:

$$R = \frac{w + L}{v} \quad (2.2)$$

where R is the red clearance interval (s); w is the width of the stop line to the far-side no-conflict point (ft); v is the design speed (ft/s); and L is the vehicle length (typically 20 ft).

2.3.3.2 FHWA's Traffic Signal Timing Manual

The FHWA's *Traffic Signal Timing Manual* (FHWA 2008) is a comprehensive guide to signal timing, which proposes methods to calculate timing for all phases of a signalized intersection (i.e., passage time, minimum and maximum green times, yellow change and red clearance intervals, and pedestrian timing). The manual proposes use of the change period equation from the ITE's *Manual of Traffic Signal Design* (ITE 1998):

$$CP = \left[t + \frac{1.47v}{2(a + 32.2G)} \right] + \left[\frac{W + LV}{1.47v} \right] \quad (2.3)$$

where CP is the change period (s), defined as the sum of the yellow change interval (Equation 2.1) and the red clearance interval (Equation 2.2); t is the PRT to onset of a yellow indication (typically 1 s); v is the approach speed (typically 85th %ile speed or posted speed limit, mph); a is the deceleration rate in response to onset of a yellow indication (typically 10 ft/s²); G is the grade (% grade/100, ft/ft), where downhill is defined as negative; W is the width of the intersection (ft); and LV is the length of the vehicle (typically 20 ft).

2.3.3.3 NCHRP Report 731

McGee et al. (*McGee et al. 2012*) considered driver behavior at 83 intersections with various characteristics to determine the parameters for use in equations of the yellow change (Equation 2.4) and red clearance (Equation 2.5) intervals:

$$Y = t + \frac{1.47V}{2a + 64.4G} \quad (2.4)$$

$$R = \frac{W + L}{1.47V} - 1 \quad (2.5)$$

In these equations, t is the PRT (= 1 s); a is the deceleration rate (= 10 ft/s²); V is the 85th %ile approach speed (mph); G is the approach grade (%/100, negative for downgrade); W is the intersection width, measured from the back edge of the approaching stop line to the far side of the intersection, as defined by the extension of the curb line or outside edge of the farthest travel lane (ft); and L is the vehicle length (20 ft).

These equations and recommended values are very similar to the ITE's recommended practice, with the exception of the reduction of 1 s from the duration of the red clearance interval. This reduction is based on the observed start-up delay for conflicting vehicles; the study found an average start-up time of 1.1 s for stopped and rolling vehicles (*McGee et al. 2012*). Clearance widths (from stop line to far curb) for test sites ranged from less than 48 ft to more than 120 ft, and 40% of the data set included intersections with clearance widths over 120 ft.

2.3.4 ODOT Timing Practices

The *Traffic Signal Policy and Guidelines (ODOT 2013b)* document discusses policies regarding the timing of yellow change and red clearance intervals for ODOT-maintained traffic signals. ODOT is responsible for maintaining, operating, and installing most traffic signals on Oregon's State Highway System.

2.3.4.1 Yellow Change Interval

ODOT's yellow change interval policy utilizes the following equation, found in the ITE's *Determining Vehicle Signal Change and Clearance Intervals* report (*ODOT 2013b, ITE 1994*):

$$y = t + \frac{v}{2a + 2Gg} \quad (2.6)$$

where y is the length of the yellow interval to the nearest 0.1 s; t is the PRT (recommended as 1.0 s); v is the velocity of the approaching vehicle (ft/s); a is the deceleration rate (recommended as 10 ft/s²); g is the acceleration due to gravity (32 ft/s²); and G is the grade of approach (3% downgrade would appear as -0.03). ODOT recommends minimum and maximum yellow change intervals of 3.5 and 5.0 s, respectively (*ODOT 2013b*).

2.3.4.2 Red Clearance Interval

Table 2.1 shows the minimum red clearance intervals to be used at all traffic signals on Oregon State highways, as set forth by ODOT.

Table 2.1: ODOT minimum yellow change and red clearance intervals (ODOT 2013b)

Posted Speed (mph)	Minimum Yellow Change Interval ⁽¹⁾⁽²⁾ (s)	Minimum Red Clearance ⁽²⁾ (s)
25	3.5	0.5
30	3.5	0.5
35	4.0	0.5
40	4.3	0.5
45	4.7	0.7
50	5.0 ⁽³⁾	1.0
55	5.0 ⁽³⁾	1.0

(1) Applied to approaches with a downgrade of 3% or less.

(2) Some intersections may require more than the minimum times.

(3) ODOT limits the yellow change interval to 5 s. The sum of the yellow change and red clearance intervals shall exceed the length of the yellow interval calculated from Equation 2.6.

The above red clearance intervals may be increased if engineering judgment deems it necessary, due to factors such as intersection width, conflict points, approach speed, and percentage of trucks (*ODOT 2013b*).

2.3.5 Conflict Zone Method

Other countries have developed alternative methods for calculating the yellow change and red clearance intervals. For example, the Netherlands uses the conflict zone method. Specifically, Muller et al. (2004) proposed that the yellow change interval be timed to avoid a dilemma zone, and that the red clearance interval timing be based on clearance of each conflict pair. The yellow change interval is calculated by:

$$t_{yellow} = t_r + \frac{v_{appr}}{2|a_{dec}|} \quad (2.7)$$

where t_r is the reaction time (s); v_{appr} is the approach speed (typically, 85th %ile approach speed, ft/s); and a_{dec} is the deceleration rate (ft/s²).

This formula is based on vehicles entering the intersection throughout the yellow change interval. A red clearance interval is required to clear the intersection of a potential vehicle entering the intersection at termination of the yellow change interval. Red clearance interval timing is determined by calculating the clearance interval for each conflict pair in the intersection. For this calculation, the exit time of a vehicle entering at the last moment of the yellow change interval and the entrance time of a conflicting vehicle must be determined.

Figure 2.1 shows an example conflict zone, in which vehicle 1 is entering at the last moment of the yellow indication, and vehicle 2 is approaching a red about to change to a green indication. This method of calculating clearance intervals provides different clearance times for each conflict pair. One entering traffic stream can get a green indication slightly before another, if desired, to reduce delay.

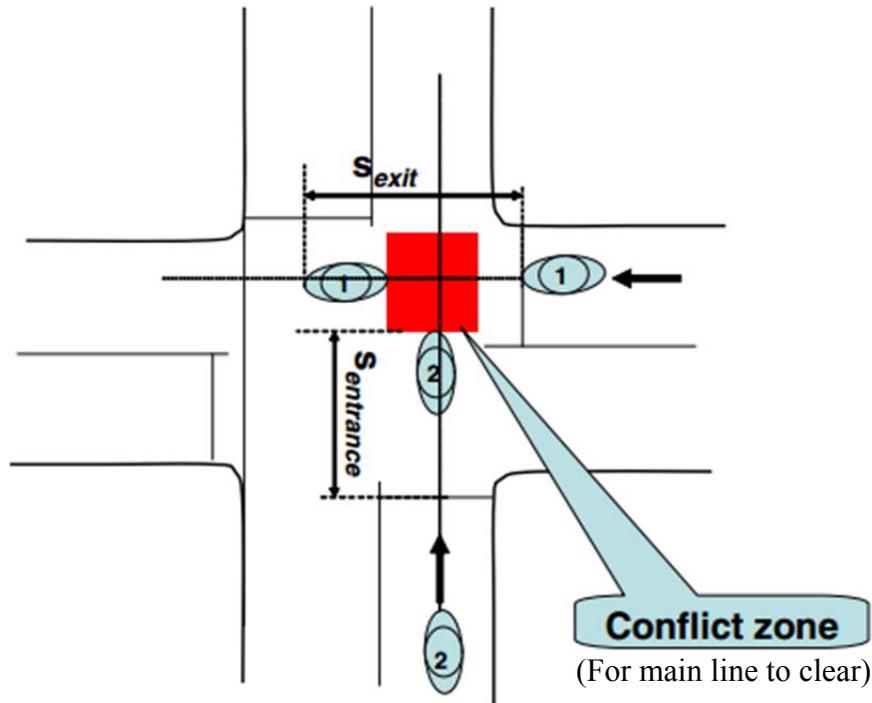


Figure 2.1: Traffic streams and conflict zones (*adapted from Muller et al. 2004*)

The exit time is calculated by:

$$t_{exit} = \frac{s_{exit}}{v_{exit}} \quad (2.8)$$

where t_{exit} is the exit time (s); s_{exit} is the distance for the vehicle to clear the conflict zone (see vehicle 1 in Figure 2.1, ft); and v_{exit} is the vehicle speed (ft/s²). Muller et al. did not provide a method for determining which vehicle speed to use. However, because a vehicle is unable to stop during the yellow change interval, it is unlikely to be traveling below the average approach speed, although a conservative value can still be used.

There is no generally accepted method for handling entrance time calculations, due to their inherently complex and variable nature. There are multiple possible vehicle trajectories that can result in various entrance times. For example, if a light turns green, an approaching vehicle that has not come to a complete stop may begin accelerating before crossing the stop bar. Except for a brief period at the beginning of the green interval, this vehicle's entrance time will be smaller than that of a vehicle that is stopped at the stop line. An example of these vehicle trajectories can be seen in Figure 2.2a. As a graphical method for calculating the minimum entrance time, Figure 2.2b shows trajectories with approach times from 0 to 5 s before the signal turns green. The minimum entrance time can be determined for any distance by using the boundary formed by the various vehicle trajectories.

Muller et al. (Muller et al. 2004) derived analytical equations for determining the minimum entrance time:

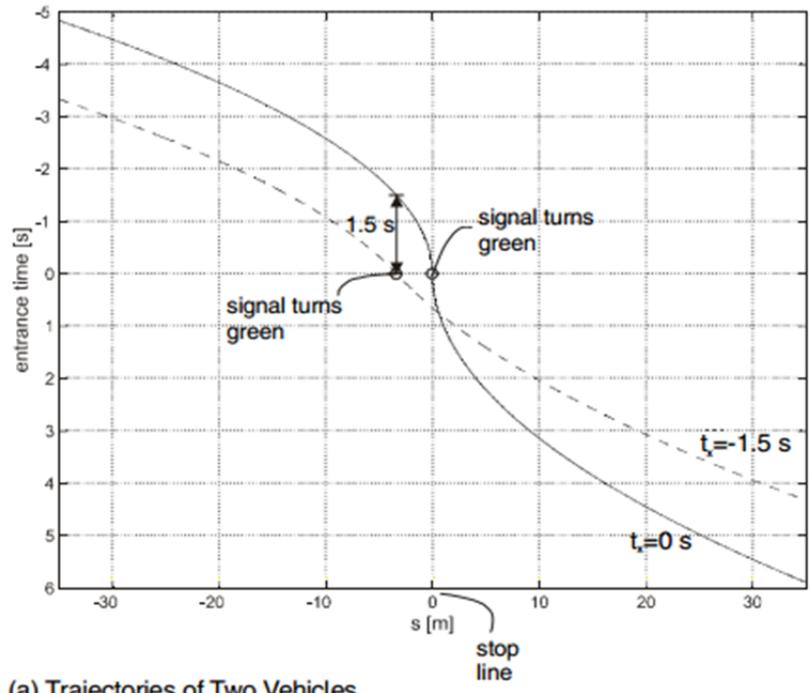
$$t_{entrance} = t_r + \sqrt{\frac{2 * s_{entrance}}{a_{acc} - a_{dec}}} \text{ if } s_{entrance} \leq s_{critical} \quad (2.9)$$

$$t_{entrance} = t_r + \frac{s_{entrance}}{v_{max}} + \frac{v_{max}}{2 * (a_{acc} - a_{dec})} \text{ if } s_{entrance} > s_{critical} \quad (2.10)$$

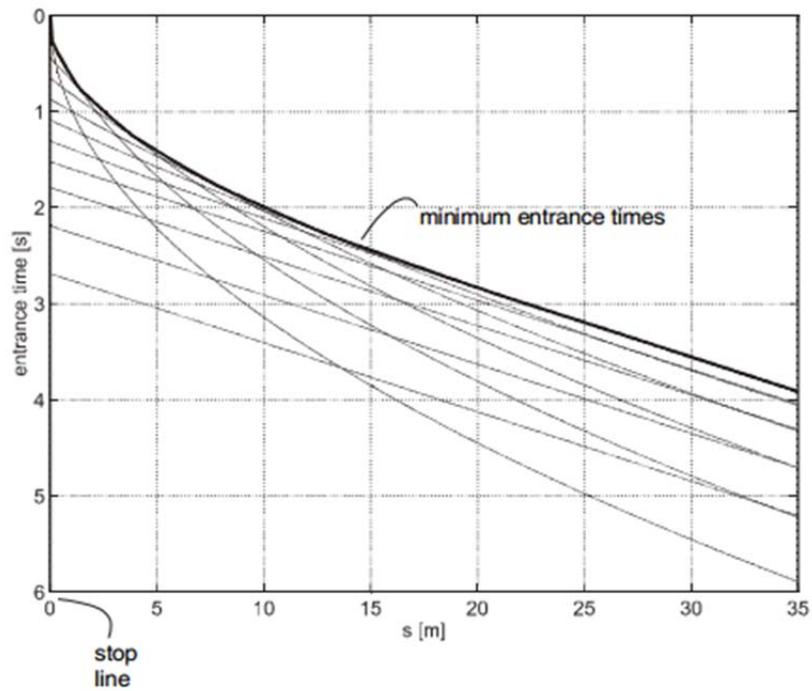
$$s_{critical} = \frac{v_{max}^2}{2 * (a_{acc} - a_{dec})} \quad (2.11)$$

where $t_{entrance}$ is the entrance time (s); t_r is the reaction time (s); $s_{entrance}$ is the distance for the vehicle to reach the conflict zone (see vehicle 2 in Figure 2.1, ft); a_{acc} is the acceleration rate (ft/s²); a_{dec} is the deceleration rate (ft/s²); and v_{max} is the vehicle running speed (ft/s). The authors calibrated the analytical method by observing and plotting trajectories of first-to-enter vehicles at the onset of the green indication at two intersections, one in Delft and one in Haarlem. The model was calibrated with the observed 2nd %ile entrance times. Due to the small sample size, the authors suggested the use of locally calibrated values. When calibration is not possible, they recommend using a value between 2.5 and 3.0 m/s² for the acceleration difference, $a_{acc}-a_{dec}$.

Finally, the authors compared red clearance interval times required for the conflict zone method and the ITE approach. Leading and lagging left-turn phasing situations were investigated for the conflict zone method because these situations result in different red clearance interval durations due to the different conflict pairs. When the conflict zone method was used for an intersection with a 20 m (65.6 ft) distance from the stop line to the edge of the opposite curb, a lagging left turn required only 0.4 s of red clearance per cycle, compared to 4.6 s per cycle for a leading left turn. This difference reduced intersection capacity by 4% to 6%, depending on the cycle length. The ITE method required 8.2 s of red clearance time per cycle, which reduced the capacity by 8% to 12%, depending on the cycle length.



(a) Trajectories of Two Vehicles



(b) Adjusted Entrance Time as Envelope of Trajectories

Figure 2.2: Derivation of entrance time from vehicle trajectories (Muller et al., 2004)

2.4 COUNTERMEASURES

Bonneson et al. (*Bonneson et al. 2002*) described two categories of countermeasures to prevent RLR: enforcement and engineering countermeasures. Enforcement countermeasures consist of manual or automated enforcement to discourage drivers from disobeying traffic laws by imposing a citation or fine. Enforcement countermeasures are most effective when the driver's decision to run the red light is "avoidable". Engineering countermeasures attempt to prevent drivers from "unavoidable" situations, in which they must decide whether to run or not run the red indication (*ITE 2003, Bonneson et al. 2002*).

2.4.1 Enforcement Countermeasures

Police enforcement and automated enforcement using red-light cameras are the two main methods for enforcing RLR laws. Police enforcement requires police presence at a light, which is a costly method. Automated enforcement uses cameras and signal controller phase information to capture evidence to document RLR. Legal and, in some cases, political implications of automated enforcement have prevented some states from using cameras to enforce traffic laws (*Bonneson et al. 2002*).

2.4.2 Engineering Countermeasures

Bonneson et al. (*Bonneson et al. 2002*) disaggregated engineering countermeasures into three subcategories: motorist information, physical improvements, and signal operation. Motorist information countermeasures provide enhanced signal displays or additional information about the signal ahead. They include pre-yellow signal indications, sight distance improvements, signal visibility improvements, increased signal conspicuity (e.g., backplates), and advanced warning signs. Physical improvement countermeasures aim to improve or solve safety and operation problems through intersection modification. They include removing unnecessary traffic signals, adding capacity through additional traffic lanes, flattening sharp vertical curves, and softening sharp horizontal curves. Signal operation countermeasures involve changing signal timing or phasing. They include improving signal coordination and operation, providing green-light extension systems, and increasing durations of the yellow change and red clearance intervals.

Bonneson and Zimmerman (*Bonneson and Zimmerman 2004*) found that longer yellow change intervals resulted in a decrease in RLR violations. The *Making Intersections Safer* report (*ITE 2003*) mentions using engineering countermeasures, such as improving signal visibility/conspicuity, increasing the likelihood of stopping, eliminating the need to stop, and addressing intentional violations, to reduce RLR. Similarly, Schattler et al. (*Schattler et al. 2011*) found that mast arm signalized intersections had fewer RLR vehicles compared to diagonal span wire intersections.

Awadallah (*Awadallah 2009*) discussed the use of transverse yellow decision lines and the new requirement that vehicles yield right-of-way before entering an intersection as two additional legal countermeasures for reducing RLR crashes. A transverse yellow decision line would be placed at the boundary of the zone where a vehicle traveling at the speed limit would be able to clear the intersection at the onset of the yellow change interval. This line would provide drivers with information to help them decide whether to stop or proceed through the intersection.

Drivers upstream of the decision line traveling at or below the speed limit at the onset of the yellow indication must stop at the stop line, while drivers downstream would be able to proceed safely through. Awadallah suggested that a regulatory sign be posted at the decision line location, to convey the meaning of the transverse line. Requiring vehicles to yield the right-of-way before entering the intersection when presented the green indication would allow a conflicting vehicle to clear the intersection. Both methods would require education and enforcement to change how people react to the yellow indication.

Engineering and enforcement countermeasures can be used in conjunction to reduce RLR and related crashes. The *Red Light Camera System: Operational Guidelines (FHWA 2005)* document states that before applying any countermeasure, an agency should perform engineering studies to determine the factors contributing to RLR, to ensure that appropriate countermeasures are selected based on the identified problems.

2.5 RED CLEARANCE EXTENSION

As a form of dilemma zone protection, red clearance extension attempts to mitigate the problem of avoidable or unavoidable RLR, which occurs when a driver cannot decide whether to stop or go at the onset of a circular yellow indication. Dilemma zone protection systems use vehicle detection to reduce driver exposure to the dilemma zone or to offset the impacts of dilemma zone indecision. The goal of a red clearance extension system is to detect a vehicle approaching an intersection near the onset of the circular yellow indication and to predict if the vehicle will safely stop, safely clear the intersection, or be in the intersection at the end of the red clearance interval. If a RLR vehicle is predicted, then a call is placed to the traffic controller to extend the red clearance interval, giving the vehicle time to clear the intersection before releasing opposing traffic. This Chapter considers aspects of previously documented red clearance extension systems, including the predictive models used to determine if red clearance extension is required, vehicle detection considerations, and performance measures to determine system effectiveness.

2.5.1 Predictive Models

To predict whether a vehicle will require a red clearance extension, various models using different prediction methods have been applied. Predictions have been made on the basis of the arrival time at the stop line (including car-following information to predict stop vs. go behavior), bivariate stop-go models, least-squares support vector machine models, multistep zonal classification, identification of vehicle presence in a multi-segment detection zone, stopping-speed prediction algorithms, and minimum speed boundaries.

Wang et al. (*Wang et al. 2012*) and Zhang et al. (*Zhang et al. 2009*) used probabilistic models to predict RLR vehicles. Wang et al. (*Wang et al. 2012*) used Autoscope™ cameras to build a model for constructing vehicle trajectories to predict arrival time at the stop line, on the basis of inductive loop detector input. They created a model to account for car-following information, predict stop vs. go behavior and enable better prediction rates. In an earlier report, Wang et al. (*Wang et al. 2009*) used a “last second” approach, which would allow a red clearance extension to be triggered just before the end of the yellow change interval. Zhang et al. (*Zhang et al. 2009*) used Autoscope™ cameras as discrete sensors to create a bivariate stop-go model using acceleration and average speed data from vehicle trajectories. Chen et al. (*Chen et al. 2014*)

applied a least-squares support vector machine model to predict RLR using continuous vehicle trajectories from radio detection and ranging (RADAR) hardware.

Gates (*Gates 2007*) developed an algorithm (Figure 2.3) that used a multistep zonal classification process to determine the need for a red clearance extension, based on the approaching vehicle's speed and position from the stop line.

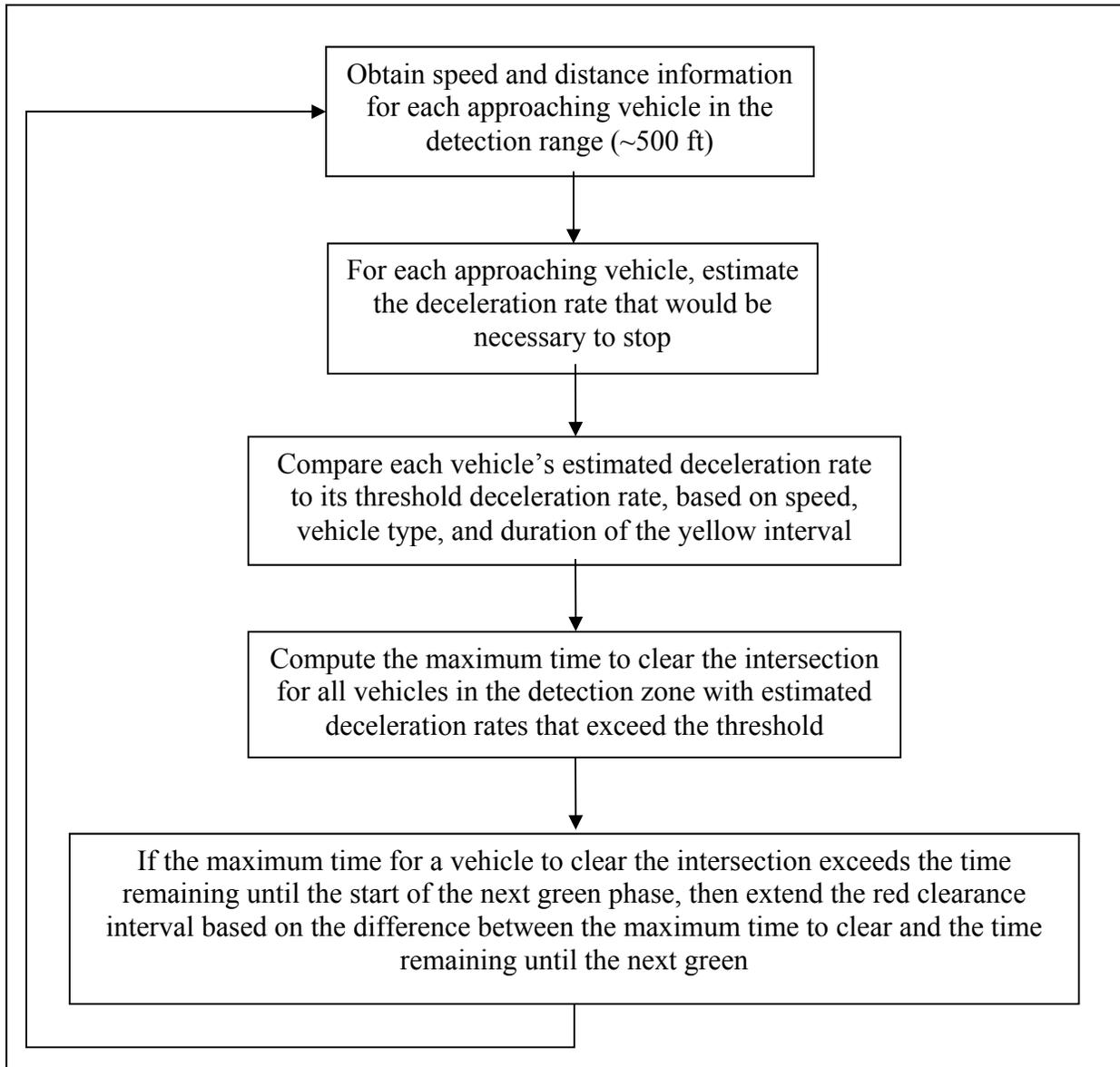


Figure 2.3: Flowchart of concepts for extension of the red clearance interval (*Gates 2007*)

Zonal classification was used to distinguish vehicles that required a red clearance extension from those that did not. An approaching vehicle was classified according to whether it was predicted to stop before entering the intersection, clear the intersection prior to the start of the conflicting green phase, or not clear the intersection prior to the start of the conflicting green phase and thus provided with extended red clearance time (*Gates 2007*). Examples of these three zones organized on a space-speed diagram can be seen in Figure 2.4.

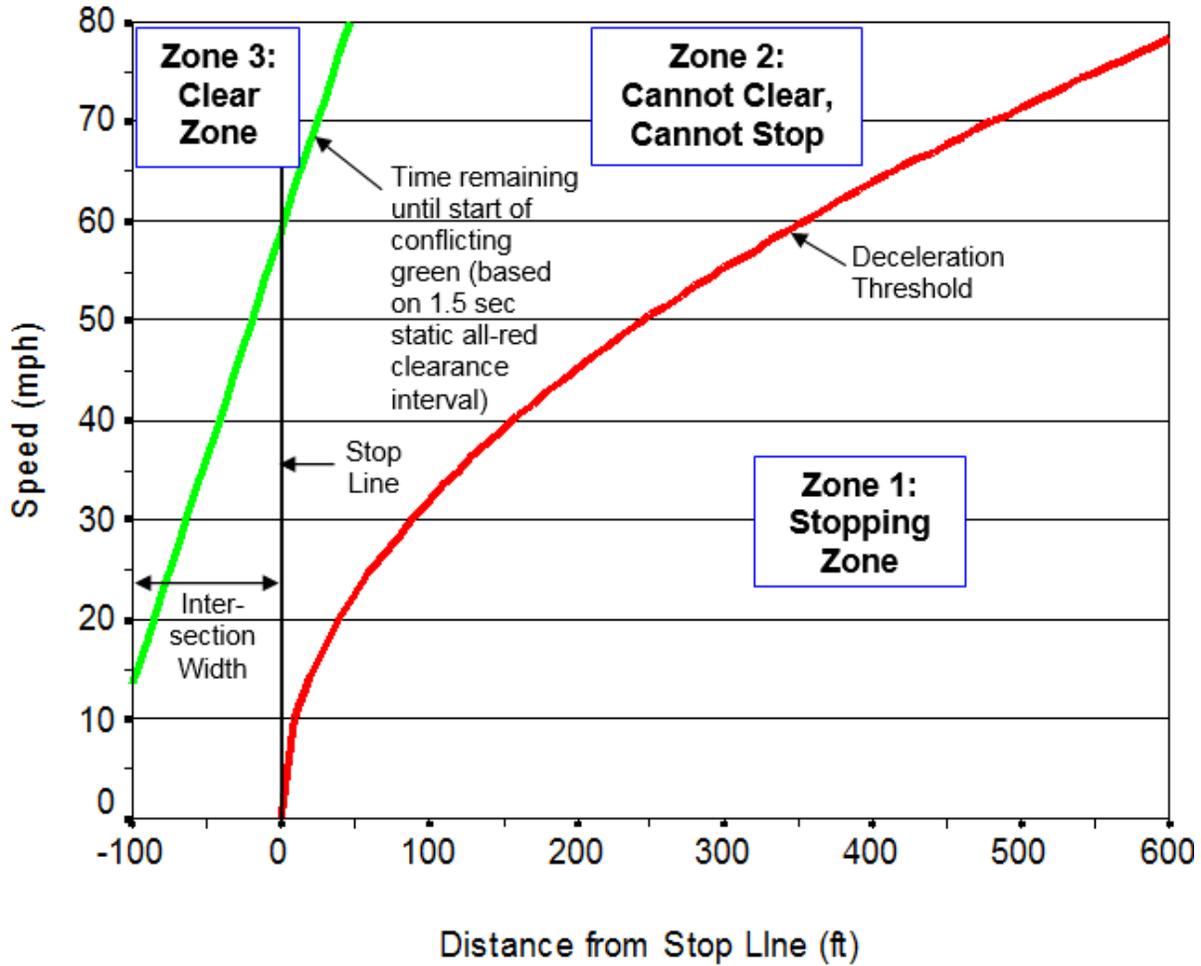


Figure 2.4: Conceptual zonal classification space-speed diagram (start of red) (Gates 2007)

Gates (Gates 2007) determined that a static red clearance interval should be included in the signal timing to reduce the incidence of false alarms. An extension can be called after expiration of the yellow change interval for late-arriving stopping vehicles that have not begun to decelerate. Continued use of a static red clearance interval reduces the number of and the delay caused by extensions. The static red clearance interval should be calculated by recommended guidelines. Due to the difficulty in differentiating late-arriving stopping from RLR vehicles by the threshold deceleration rate alone, a minimum speed threshold was used to reduce false calls from stopping vehicles:

For $t = 0$ to the preprogrammed static red clearance time T ,

$$\text{Let } V_{min} = V_{mean} - 2\sigma_v - \sigma_v \frac{t}{T} \quad (2.12)$$

where v_{min} is the minimum speed to be provided extended red clearance time (mph); v_{mean} is the mean approach speed before start of the yellow (mph); σ_v is the standard deviation (SD) of the speed (mph); and t is the time after start of the red clearance interval (s). Gates (2007) calculated the duration of the red clearance extension using the following equation:

$$AR_{extended} = TTC - TTG \quad (2.13)$$

where $AR_{extended}$ is the extended red clearance interval (s); TTC is the estimated time for the vehicle to clear the intersection (s); and TTG is the time until the next conflicting green phase (s). If multiple vehicles are detected, then the maximum time to clear all vehicles is used.

Awadallah (Awadallah 2013) proposed using vehicle presence in three segments leading up to the stop line to determine the need for a variable yellow change (currently not allowed by the MUTCD) or red clearance interval. The intersection was broken into three segments with four detectors, to determine the location of a vehicle within a segment. The detector configuration can be seen in Figure 2.5.

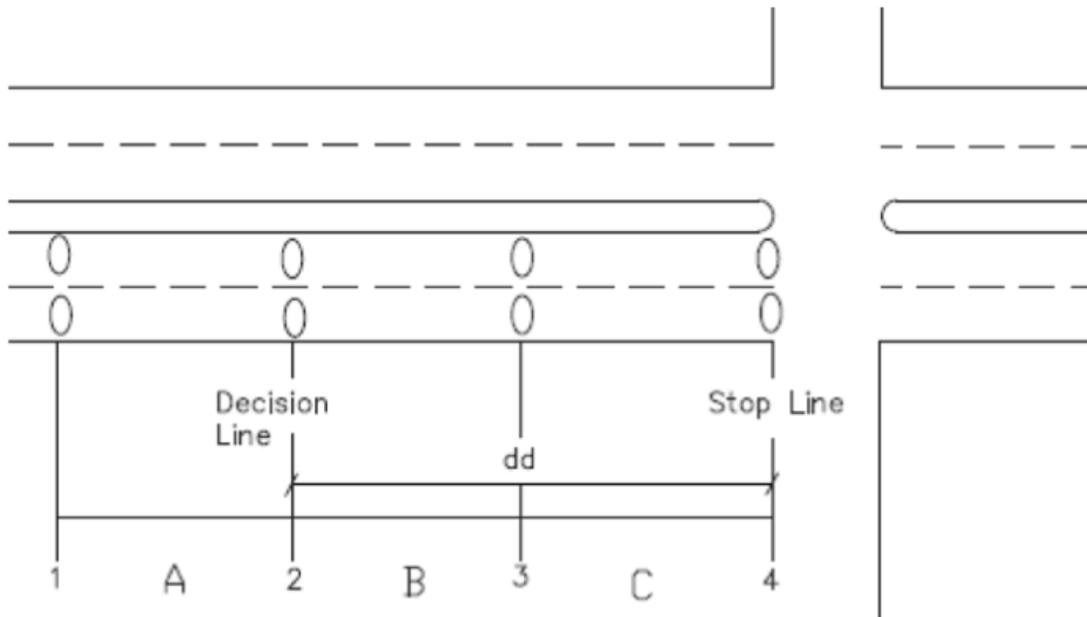


Figure 2.5: Location of detectors and decision line (Awadallah 2013)

The first detector was located at 1.5 times the decision distance, the second detector at the decision line, the third detector at halfway between the decision line and the stop line, and the final detector at the stop line. This configuration considered the vehicle approach speed in determining placement of the decision line. Four cases were developed based on possible vehicle location combinations, to determine the need for a red clearance extension (Table 2.2).

Table 2.2: Cases used to determine the variable yellow or red clearance interval (Awadallah 2013)

Case	Vehicle Presence	Timing	Notes
I	No vehicle present in segment A, B, or C at onset of yellow change interval	Use preset minimum yellow change interval	Speeding vehicles that enter segment A should be able to stop safely.
II	Vehicles present in segment A or B at onset of yellow interval	Use default yellow change and red clearance interval duration, unless case IV applies.	Not applicable
III	No vehicles present in segment A or B, but vehicles present in segment C, at onset of yellow change interval	If no vehicles enter segment A, B, or C between onset of yellow indication and minimum yellow change interval, then the speed and time that the last vehicle entered segment C are recorded. Yellow/red times should be recalculated based on the time required for the vehicle to clear the intersection.	If a new vehicle enters segment A, B, or C after onset of the yellow indication, then the default yellow change and red clearance intervals are maintained.
IV	All vehicles crossing detector four or intending to proceed after entering segment C are considered	Provide extension of red clearance interval up to preset maximum based on vehicle speed, location, and time present at detector. Vehicles entering segment C or crossing stop bar need more than designed red clearance interval.	Applicable with Cases II and III. Only vehicles at or above the design speed when entering segment C are considered for red clearance extension.

Xu (Xu 2009) generated a stopping-speed prediction algorithm using VISSIM, an ASC/3 Controller, MATLAB, and the Advanced Traffic Analysis Center controller interface device (CID) to develop the HIL interface. Their algorithm compared vehicle speed at a detector to the stopping speed calculated from a stopping sight distance equation:

$$V = \sqrt{\frac{30 * a * SSD}{32.2}} \quad (2.14)$$

where V is the vehicle speed (mph); a is the deceleration rate (ft/s^2); and SSD is the distance of the detector from the stop line (ft). The goal of the algorithm was to determine if the vehicle would stop or proceed through the intersection. A vehicle traveling faster than the stopping speed for a given detector distance from the stop line would be unable to stop in time and would

require a red clearance extension. The required extension time was calculated from the speed at the detector, distance to the stop line, and remaining time of the yellow change interval, assuming the use of a normal red clearance time for the vehicle to clear the intersection.

Chang et al. (*Chang et al. 2013*) used minimum speed boundaries for vehicles approaching an isolated intersection with a posted speed limit of 55 mph at the onset of the red clearance interval to determine the need for a red clearance extension. Vehicles detected within 500 ft of the stop line traveling faster than 56 mph at the beginning of the red change interval would trigger a red clearance extension, the duration of which would be calculated from the vehicle’s speed and distance from the stop line. Vehicles traveling faster than 67 mph at distances between 500 and 875 ft would also trigger a red clearance extension. Threshold speeds were determined from field observations. Vehicles traveling under these speeds were assumed to stop at the stop bar. Detection for the red clearance extension began within 3 s of onset and was updated every 0.1 s until termination of the red clearance interval. As the end of the red clearance interval approached, a final decision was made on whether an all-red extension was required.

Olson (*Olson 2012*) used crash data and an intersection simulation model, created in the statistical software program R, to determine the effectiveness of red clearance extension systems installed in Portland, Oregon. Modeling was necessary because upgrades were being performed on red clearance extension systems together with other intersection improvements, which made it difficult to quantify the safety impact of the extension systems alone. The model tested whether there was a difference in crash incidence between intersections with or without red clearance extension systems, while holding all other factors constant. The intersection of Powell Blvd. and 82nd Ave. in Portland was used as a guide for the model due to the availability of data (e.g., start-up times for vehicles reaching the inductive red clearance extension loops at the start of the green, red clearance extension activations during a 48-h period, and traffic volume for 82nd Ave.). A go/no go probability function for the model was created based on research by Hurwitz et al. (*Hurwitz 2012*):

$$ProbGo = 1 - \frac{1}{1 + e^{\left(6.34 - 1.36 \cdot \frac{Distance\ from\ Stop\ bar}{Vehicle\ Speed}\right)}} \quad (2.15)$$

For each approaching vehicle, the probability of stopping was compared to a random value between 0% and 100% selected from a uniform distribution. If the randomly generated value was less than the probability of stopping, then the vehicle would proceed through the intersection; otherwise, the vehicle would stop.

The time that it takes conflicting vehicles, waiting at the stop line, to reach the red clearance extension inductive loops was determined by using historic data from the City of Portland. A normal distribution was fitted to the first peak of the histogram and used to generate a random start-up time for the conflicting vehicle. The conflict point was monitored to determine if a crash occurred between a RLR vehicle and a conflicting vehicle. According to the simulations, vehicles that triggered red clearance extensions were not usually “saved” by the extension, although the extension could “save” other RLR vehicles that entered after the initial extension. None of the RLR vehicles in the simulation that resulted in a crash would have been “saved” by triggering a red clearance extension. Among the 12 RLR crashes that occurred during the 48-h

interarrival rate simulation, it is anticipated that 7 crashes (58%) could have been prevented if red clearance extension technology was active.

2.5.2 Vehicle Detection

There are many aspects to consider with regards to vehicle detection for a red clearance extension system, including detector layout, operation, and measurement. Various vehicle detection methods are currently available, each with different characteristics. Detector choice for a red clearance extension system depends on the requirements of the prediction algorithm. This section discusses vehicle detection selection and considerations from prior research.

2.5.2.1 Layout

Detector placement is an important consideration in the development of a red clearance extension system because accuracy rates depend on where detection occurs in the vehicle trajectory. The closer a vehicle is to the stop line, the more accurately its stop vs. go behavior can be detected (*Wang et al. 2012*). One trade-off in placing detectors very close to the stop line is that fewer RLR vehicles will be detected, as these vehicles reach the stop line after onset of the red clearance interval (*Wang et al. 2012*).

Xu (*Xu 2009*) found that as the detector distance upstream from the stop line increased, the system was more likely to extend the red clearance interval when using the stopping-speed prediction algorithm. Longer detector distances increased the incidence of false alarms. A balance was needed between correctly identifying RLR vehicles and false alarms. Of the detector locations tested, detector placement 150 ft upstream of the stop line minimized both missed RLR vehicles and false alarms. Zhang et al. (*Zhang et al. 2009*) based detector placement on the requirement that 85% of RLR vehicles be detected before onset of the red change interval. Detector placement was configured by using multiple sensors placed along a 200-ft segment approaching the stop line. Empirical data collected from these sensors were used to determine placement of the advance sensor. Because acceleration data need to be calculated from the advance sensor, the two sensors should be placed approximately 30 to 60 ft apart.

2.5.2.2 Operation

Depending on the type of detector, its placement can play an important role in the effectiveness of a red clearance extension system. Detector placement is particularly important for point sensors, because information is needed at important points in the dilemma zone where all-red extension decisions are made. Current ODOT practices for traffic signal design, as well as typical loop and video detection layouts, are described in the *Traffic Signal Design* manual (*ODOT 2013a*).

Error! Reference source not found. Figure 2.6 shows a common inductive loop advance detector system, used by Wang et al. (*Wang et al. 2012*) to develop a red clearance extension system, with a design speed of 45 mph and passage time of 2 s. The system contained two advance loops (~6 ft × 6 ft) and one presence loop (~6 ft × 60 ft), with S_R and S_P representing timestamps of a vehicle arriving at an advance loop and at the

presence loop, respectively. The first advance detector was placed upstream of the dilemma zone. The second and, possibly, third advance detectors were placed between the first detector and stop line. The speed of approaching vehicles and passage time of the controller governed placement of detectors on the intersection approach. For the purposes of a red clearance extension system, only the second advance detector and presence loop are used.

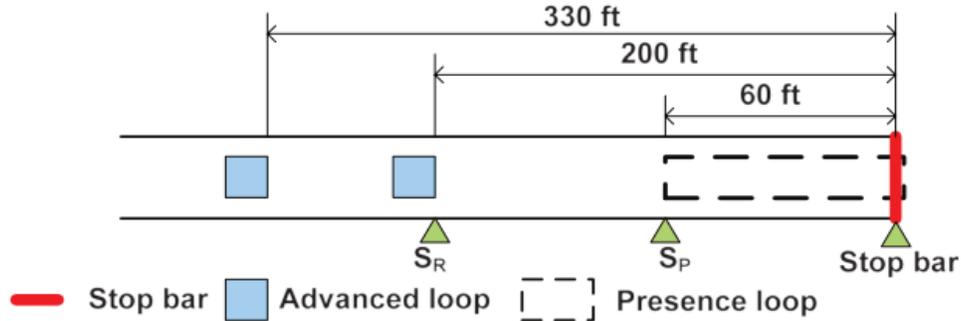


Figure 2.6: Example of a multiple advance detector system (Wang et al., 2012)

Other presence loop configurations (Figure 2.7) include multiple short loops, LHOVRA, and red-light camera RADAR systems (Wang et al. 2012). Using multiple short loops allows collection of speed data, which can aid in RLR prediction. LHOVRA is a Swedish traffic signal control technique (Young and Archer 2009). Each letter in the LHOVRA acronym stands for a different type of functionality: L, heavy vehicle priority; H, mainline vehicle priority; O, accident reduction; V, variable yellow indication; R, red-light violation control; and A, red clearance turn-around. Use of the LHOVRA system configuration in a red clearance extension system is not very useful, because the beginning of the presence detector is ~90 ft upstream of the stop line, which leads to lower correct detection rates (Wang et al. 2009). A RADAR camera system with no presence detection before the stop line would not work as a red clearance extension system. This system would only be able to detect if a vehicle ran a red light, which is too late to trigger a red clearance extension. In this example, RADAR is only used at the stop line to trigger a RLR camera; this is a choice, not a limitation, of the RADAR sensor.

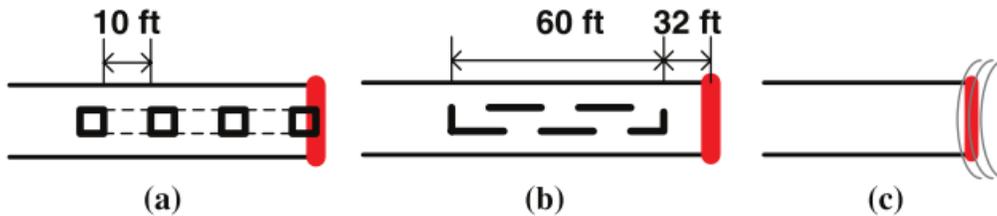


Figure 2.7: Types of presence loops: (a) multiple short loops, (b) LHOVRA, and (c) RADAR camera (Wang et al. 2012)

2.5.3 Measurement

Each prediction algorithm for RLR vehicle detection uses different variables to calculate whether a vehicle will be in the intersection during the red clearance interval. Variables requiring measurement are important in determining the appropriate detector system, as some systems are better suited for certain measurement types. Previous prediction algorithms have used vehicle location, speed, the timestamp that a vehicle crosses a detector, and vehicle classifications.

Wang et al. (Wang et al. 2012) recorded the timestamp of the vehicle arriving at the advance loop (S_R) and the first timestamp showing vehicle presence at the presence detector (S_P in Figure 2.6). Although data from the stop line did not aid in RLR prediction, they were collected as historic information. Gates (Gates 2007) used RADAR, due to its more accurate real-time measurement of speed, distance, and vehicle classification. RADAR collects data continuously for an entire approach, rather than at fixed locations. Using RADAR allowed for more accurate red clearance extensions and reduced the number of false-alarm and missing errors. Gates' RADAR system had a detection range of 500 ft and allowed for eight user-defined detection zones. RADAR performance is not affected by wind, weather, light, or temperature changes. Front-firing RADAR systems can distinguish between basic vehicle types (large vs. small), based on the surface area of the front of the vehicle, permitting estimations of vehicle length.

According to Chang et al. (Chang et al. 2013), the key to a successful red clearance extension system is vehicle detection capable of monitoring the speed and location of vehicles in an 880-ft target zone. After reviewing various detection systems, these authors selected a microwave detection method, which allowed for time- rather than distance-based tracking. Dilemma zone protection was able to account for the evolution of a vehicle's speed in the dilemma zone.

The City of Portland's red clearance extension systems use inductive loop detectors in the intersection, downstream of the stop line, to trigger a red clearance extension during the last half of the yellow change interval and the red clearance interval (Olson 2012). The goal of this layout is to detect vehicles that enter the intersection, in order to reduce the false-positive rate.

2.5.4 Red Clearance Extension Timing

The City of Portland uses 3.6 s for the yellow change, 1 s for the red clearance, and 1.8 s for the red extension intervals at intersections with red clearance extension systems (Olson 2012). Per the City of Portland's traffic signal controller firmware, Northwest Signal (NWS) Voyage™, a red clearance extension can be activated during the last half of the yellow change or during the red clearance interval (i.e., 1.8 s of yellow and 1.0 s of red clearance). **Error! Reference source not found.** Figure 2.8 shows the red extension activation period and signal timing with red clearance extension activation.

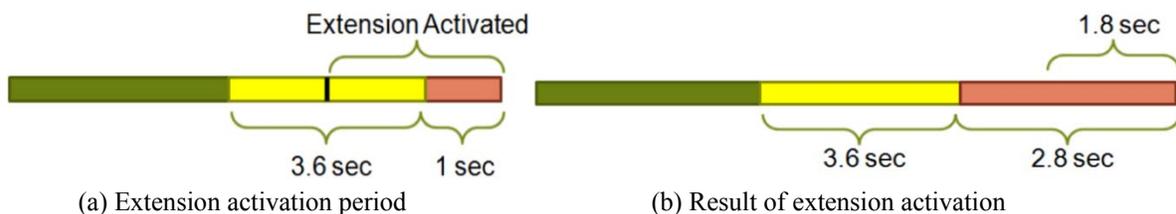


Figure 2.8: All-red extension activation period and signal timing change

The 1.8-s period was chosen for red clearance extension because a vehicle traveling at 35 mph for 1.8 s will travel 90 ft, which is the furthest distance between the red clearance extension inductive loop and the far side of the intersection.

2.5.5 NWS Voyage™ Red Clearance Extension

The NWS Voyage™ Software Operating Manual (*Northwest Signal Supply, Inc. 2012*) provides details on the use and programming of the red clearance extension feature in the Voyage™ firmware. Specifically, the red clearance interval can be extended based on the presence of a late-arriving call, if the call occurs during the last 50% of the yellow change interval or any time during the red clearance interval. The programmable value for the red clearance extension timer ranges from 0 to 25.5 s. The timer can be disabled based on time-of-day operations.

2.5.6 Performance Measures

2.5.6.1 Detection Theory

It is important to measure the effectiveness of red clearance extension systems for correctly predicting RLR vehicles. Detection theory can be applied to RLR prediction models (Table 2.3). Missing and false-alarm errors are important error types in RLR prediction (*Wang et al. 2012*). Missing errors occur when a vehicle is predicted to stop, but instead runs the red light without the additional safety benefit of red clearance extension (*Zhang et al. 2009*). False-alarm errors occur when a vehicle is predicted to run the red light, but it instead stops at the stop line, thereby increasing intersection delay by adding additional time to the cycle without progressing vehicles (*Zhang et al. 2009*).

Table 2.3: Detection theory for RLR prediction

Vehicle Type	Go Prediction	Stop Prediction
RLR Vehicle	Correct Extension	Missing Error
Stopping Vehicle	False Alarm	Correct

Xu (*Xu 2009*) used modified detection theory to determine optimal detector placement. The red clearance extension result was broken into four types (Table 2.4), corresponding to the four prediction types in Table 2.3.

Table 2.4: Red clearance extension types and corresponding theory prediction (*Xu 2009*)

Type	Description	Detection theory prediction
1	Red extended, RLR vehicles in intersection	Correct Extension
2	Red not extended, RLR vehicles in intersection	Missing Error
3	Red extended, no RLR vehicles in intersection	False Alarm
4	Red not extended, no RLR vehicles in intersection	Correct

Studies on red clearance extension methods by Zhang et al. (*Zhang et al. 2009*) and Wang et al. (*Wang et al. 2012*) used system operating characteristics to measure system performance and quantify the tradeoff between false-alarm and missing errors. Wang et al. (*Wang et al. 2012*) normalized the probability of false alarms to the false-alarm rate per cycle. Chen et al. (*Chen et al. 2014*) used receiver operating characteristics to measure the performance by graphically showing how the false-positive rate changes with the correct extension rate. Zimmerman and Bonneson (*Zimmerman and Bonneson 2005*) used control delay, stop frequency, RLR frequency, and crash frequency to measure the performance of a detection-control system that provided dynamic dilemma zone protection at signalized intersections.

2.5.6.2 Cost-Benefit Analysis

Cost-benefit analysis can be used in conjunction with detection theory to determine appropriate and effective countermeasures to combat RLR. This analysis can be used to monetize and compare the benefits of a particular countermeasure (e.g., RLR crash reduction) against its design and installation costs. ODOT provides a framework for cost/benefit analyses (*ODOT 2014b*), including comprehensive economic values for crashes by crash type (Table 2.5).

Table 2.5: ODOT crash values (*ODOT 2014b*)

Comprehensive Economic Value per Cash		
Highway type	Urban	Rural
PDO ³		
All facilities	\$19,400	\$19,400
Moderate (Injury B) and Minor (Injury C) Injury⁴		
Interstate	\$69,300	\$79,200
Other state highway	\$70,600	\$81,900
Off system	\$72,400	\$83,900
Fatal and Severe (Injury A) Injury⁴		
Interstate	\$1,150,000	\$2,330,000
Other state highway	\$1,170,000	\$1,680,000
Off system	\$870,000	\$1,670,000

In addition to any cost savings associated with crash reductions, the design and installation costs should be considered. Table 2.6 shows an example calculation of the costs associated with the design and installation of a red clearance extension system.

Table 2.6: Engineering costs of a red clearance extension system (*Quayle 2014*)

Component	Unit Price
2070 Signal controller with NWS Voyage TM firmware	~\$4500
Inductive loops	~\$1000 per approach ¹
RADAR	~\$6500 per approach
Engineering time to configure settings and validate operations	~\$1500

¹ Assumes conduit in place, otherwise cost per approach could be an additional \$5k-\$20k.

Together, these cost calculations can be used to compare the performances of red clearance extension systems with alternative RLR countermeasures.

2.6 EXPLORATION OF AVAILABLE DATA

In addition to our primary data collection, we investigated other possible data sources to aid in the evaluation of RLR in Oregon and the design of the Smart Red-Light Extension (RLE) System. Possible data sources included RLE event logs from NWS VoyageTM traffic controller software and red-light photo enforcement records. Details of these records and their availability are discussed in the following subsections.

2.6.1 Red-Light Extension Logs

Currently, RLE treatments are used at a handful of intersections in Oregon. All of these intersections are currently using NWS VoyageTM traffic controller software and the red clearance extension feature of the VoyageTM firmware. The VoyageTM firmware version 5.3.1 or higher allows for RLE events to be recorded and archived. The RLE event logger records the beginning and end of the red extension phase, as well as which detector triggered the extension. An example of this output can be seen in Figure 2.9. We investigated the potential for this information to be used together with our data collection efforts, to provide additional information about the red clearance extension feature and RLR vehicles (*Quayle and Marnell, unpublished data*).

View Schedule A Logs For 109 - Roy Rogers 24 - Scholls-Sherwood Road

Export to Comma Delimited Text File

Print Messages

Select Search Date: All Dates

Select Message Type(s): All Messages

NORTHWEST SIGNAL			
Schedule A Logs For - 109 - Roy Rogers 24 - Scholls-Sherwood Road			
Tuesday, July 22, 2014 16:11			
	Date	Time	Message
967	Friday, June 13, 2014	21:06:09	Start Red Extension Phase 2
968	Friday, June 13, 2014	21:06:12	End Red Extension Phase 2
969	Friday, June 13, 2014	21:07:04	Start Red Extension Phase 6
970	Friday, June 13, 2014	21:07:07	End Red Extension Phase 6
971	Friday, June 13, 2014	21:07:34	Start Red Extension Phase 6
972	Friday, June 13, 2014	21:07:37	End Red Extension Phase 6
973	Friday, June 13, 2014	21:08:06	Start Red Extension Phase 6
974	Friday, June 13, 2014	21:08:09	End Red Extension Phase 6
975	Friday, June 13, 2014	21:08:58	Start Red Extension Phase 6
976	Friday, June 13, 2014	21:08:59	End Red Extension Phase 6
977	Friday, June 13, 2014	21:09:46	Start Red Extension Phase 6
978	Friday, June 13, 2014	21:09:49	End Red Extension Phase 6
979	Friday, June 13, 2014	21:10:17	Start Red Extension Phase 6
980	Friday, June 13, 2014	21:10:20	End Red Extension Phase 6
981	Friday, June 13, 2014	21:10:46	Start Red Extension Phase 2
982	Friday, June 13, 2014	21:10:46	End Red Extension Phase 2
983	Friday, June 13, 2014	21:12:46	Start Red Extension Phase 6
984	Friday, June 13, 2014	21:12:56	End Red Extension Phase 6

Figure 2.9: Example of NWS Voyage™ red extension log data

Along with the planned video data, the collection of Voyage™ red extension logs at one site would provide useful supplemental information. The project team worked with agencies to get this data, but were unable collect this data for inclusion in this report.

2.6.2 Photo Enforcement

RLR photo enforcement collects data (e.g., digital images of vehicle license plate and driver, timestamped in relation to the traffic signal phasing) on RLR vehicles for the purpose of ticketing RLR drivers. Red-light cameras capture photo or video evidence of RLR, and the driver is sent a ticket through the mail. The City of Portland currently operates 11 red-light cameras at 10 intersections. The City of Portland’s 2011 *Red-Light Running Camera Program Biennial Report (Burchfield 2011)* showed a reduction in RLR violations per hour of between 69% and 93% at recently installed RLR camera locations. There were 48% fewer injuries and 42% fewer injury-related crashes at intersections over the 4-year period with camera operation compared to the 4 years before camera installation. Table 2.7 summarizes the crash history for all 10 intersections before and after photo enforcement.

Table 2.7: Crash history before and after photo enforcement (Burchfield 2011)

Total for Operation Intersections	Enforced Direction			Intersection Total		
	Total Crashes	Disregard	Other	Total Crashes	Disregard	Other
48 months before activation of enforcement	223	63	160	312	102	210
Most recent 48 months after activation of enforcement	117	32	85	180	50	130

Four of the 10 intersections only included crashes from 1 year before and after the analysis period. Total Crashes: all crashes for all 10 intersections over 48 months before or after installation of RLR cameras; Disregard: crashes attributed to disregard of the traffic signal, over the same time period.

RLR photo enforcement data potentially contain extensive information about RLR vehicles. City of Portland officials were contacted to determine the viability of using these data for this study, but city traffic engineers do not have access to data from individual RLR events (*Rotich, unpublished data*).

2.7 SUMMARY

This literature review can be summarized into the following key points, which guided the development of a red clearance extension system for ODOT and the NWS Voyage™ traffic controller firmware:

- Various factors, including traffic signal settings and probability of stopping, influence RLR. Conflict factors, such as flow rate, traffic signal timing, and driver behavior,

may lead to a crash for a RLR vehicle. Field observations will be made to determine the effects of these factors on the development of a red clearance extension system.

- Although Oregon has a restrictive yellow law, all of its bordering states have permissive yellow laws. This fact will likely impact vehicle trajectories during the yellow change interval and, therefore, will be considered when developing timing suggestions for a red clearance extension system.
- Multiple engineering and enforcement countermeasures have been proposed to reduce the incidence of RLR. An engineering study should be completed to determine the appropriate countermeasures for an intersection. Engineering and enforcement countermeasures can be used together to promote RLR reduction.
- Important considerations for red clearance extension systems include prediction algorithms, vehicle detection, red clearance interval timing, and performance measures.
- Many of the available prediction algorithms for red clearance extension require complex procedures, making them difficult to implement at various intersections. There is a need for a simplified algorithm that is effective at protecting RLR vehicles, while reducing false-alarm and missing errors, to streamline installation of red clearance extension systems and improve intersection safety.
- NWS Voyage™ requires users to define the maximum duration of red clearance extension to be accepted, by approach (up to eight phases), and the detector inputs to activate/actuate red clearance extension.
- Detector selection and placement are critical aspects in accurately detecting RLR while reducing false-alarm and missing errors. Analysis of field data from intersections in Oregon will aid in detector placement and selection strategies.
- There is little documentation on red clearance extension timing parameters. Current signal timing practices and the conflict zone method can provide insights for developing timing methods for red clearance extension, to provide vehicle clearance while reducing unnecessary delays.
- Computer simulation and HIL have been used to model the impact of red clearance extension at intersections. Simulation models will be used to model driver behavior in Oregon and to test the effectiveness of red clearance extension algorithms, detector placement, and red clearance extension timing.
- False-alarm and missing-alarm rates are important considerations because they can impact safety and vehicle delays. Detection theory provides a likely candidate for evaluating red clearance extension systems. Crash rates and vehicle delay measurements will be used to determine the potential effects of red clearance extension at an intersection.

Little field testing has been documented on red clearance extension systems to determine system effectiveness. As such, empirical observations of contributing factors, such as the first headways of conflicting vehicles, will be conducted.

3.0 FIELD STUDY OF RED-LIGHT RUNNING IN OREGON

With guidance from the literature review, we developed a field study to investigate factors contributing to RLR, with the goals of calibrating a HIL simulation study and improving red clearance extension systems in Oregon. This Chapter discusses the methodology used to collect speed and video data and to extract the RLR rate, the time to conflict (TTC) of the first opposing vehicle, and the location of RLR vehicles on signalized intersection approaches at the onset of yellow and red indications.

3.1 POSSIBLE INTERSECTION MEASUREMENTS

McGee et al. (*McGee et al. 2012*) recorded data on the last vehicle to go through an intersection and the first vehicle to stop in each lane, excluding turning vehicles. Recorded data included the initial speed, the time and location at the start of the yellow indication and of brake-light illumination, the action of the vehicle (stop or go), the time that the vehicle stopped, the intersection time after start of the red indication (for RLR vehicles only), the platoon leader (follower or non-platooned), the presence of an opposing left-turning vehicle, the time of day, and the vehicle type. Depending on the approach speed limit, cameras were placed to allow a viewing range of 300 to 600 ft along the intersection approach, with a clear view of the signal indication. Lane-line striping measurements were used as a reference for determining a vehicle's distance from the stop line.

Guided by McGees's study, we determined that the following measurements should be extracted from on-site video footage:

- TTC for the first vehicle on the minor approach at the onset of the green indication,
- Location on the approach where 95–99% of vehicles traveling at a speed threshold (e.g., posted speed) will continue through the intersection (e.g., run the red light),
- Frequency of RLR drivers, and
- Crashes and near-miss occurrences.

Additionally, the green interval, yellow change interval, red clearance interval, and cycle length were measured to verify the signal timing plans from ODOT. Intersection drawings from ODOT were used to determine locations of and distance to conflict zones.

3.1.1 Time to Conflict

Similarly to the conflict method (*Muller et al. 2004*) discussed in Chapter 2, we measured the TTC of the conflict zone between the RLR vehicle and the first vehicle on the minor approach at the onset of the green indication. The TTC was calculated from Equations 3.1 and 3.2 (Figure

3.1), by using the speed (v), dimensions (l and w), and distance (d) from the conflict point of both vehicles (*van der Horst 1990*).

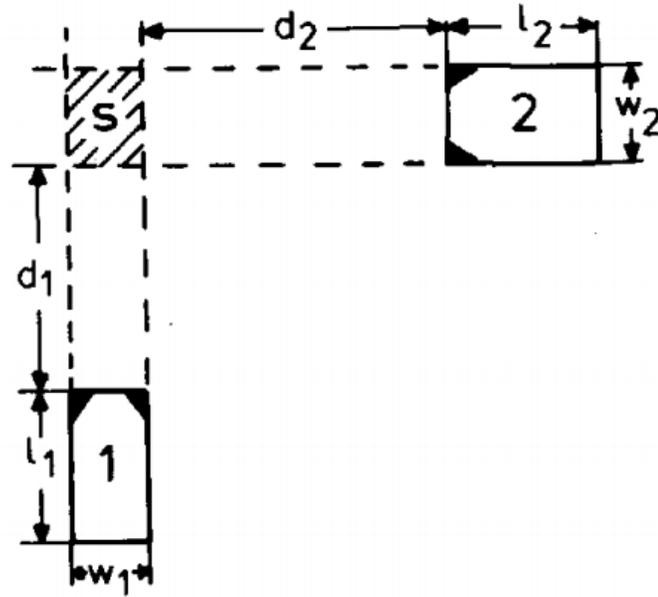


Figure 3.1: TTC for perpendicular vehicles on a collision course (*van der Horst 2010*)

$$TTC = d_2/v_2, \quad \text{if } \frac{d_1}{v_1} < \frac{d_2}{v_2} < \frac{d_1+l_1+w_2}{v_1} \quad (3.1)$$

$$TTC = d_1/v_1, \quad \text{if } \frac{d_2}{v_2} < \frac{d_1}{v_1} < \frac{d_2+l_2+w_1}{v_2} \quad (3.2)$$

where, d_i is the distance from the front of vehicle i to area S; and v_i , l_i , and w_i are the velocity, length, and width, respectively, of vehicle i . At the onset of the green indication, the minor approach vehicle will begin to accelerate into the intersection. These equations assume a constant velocity. They cannot be used to calculate the TTC directly when one vehicle is accelerating from a stop. An average TTC for various distances from the stop bar of the minor approach to the conflict zone was calculated from the collected data, by measuring the time it takes for the first minor approach vehicle to travel the distance to the conflict zone.

3.1.2 Speed Threshold

When developing a RLE system, it is necessary to know the location on the approach where 95–99% of vehicles traveling at the speed threshold will run the red light, in order to determine appropriate detector footprints. To achieve this goal, Xu (*2009*) compared the vehicle speed at a detector to the stopping speed calculated from a stopping-sight distance equation:

$$V = \sqrt{\frac{30 * a * SSD}{32.2}} \quad (3.3)$$

where, V is the vehicle speed (mph); a is the deceleration rate (ft/s²); and SSD is the detector distance from the stop line (ft). We used a similar approach, combining data from stopping-speed studies to determine the actual approach speed of vehicles, and data from videos to determine i) the vehicle location at the onset of the yellow and red indications, and ii) the stop vs. go behavior of the vehicle.

3.1.3 Red-Light running Frequency

The frequency of RLR vehicles was measured by counting the number of vehicles that ran the red indication compared to the total number of first-to-stop and RLR vehicles. Video data were collected to determine the frequency of RLR vehicles at each site.

3.1.4 Vehicle Classification

Approaching vehicles were classified to determine whether vehicle type influenced RLR. For example, trucks have different operating characteristics than passenger cars and may obstruct a following vehicle’s view of the signal heads, leaving the driver unsure of the signal status.

3.2 SITE SELECTION

Table 3.1 summarizes the factors for site selection described by McGee et al. (2012).

Table 3.1: Site selection factors (McGee et al. 2012)

Factor	Categories
Speed limit	≤ 40 mph, 45 mph, or ≥ 50 mph
Area type	Urban (downtown), Suburban, or Rural (outside of incorporated area)
Intersection clearing width (from stop line to far curb)	≤ 48 ft, 48–72 ft, 72–96 ft, 96–120 ft, or ≥ 120 ft
Proximity to upstream signal	Upstream signal within 0.5 mi, or Upstream signal within 0.5 mi
Cycle length	< 90 s, 90–120 s, 120–180 s, or > 180 s
Yellow interval duration	≤ 4.0 s, 4.1–4.5 s, 4.6–5.0 s, or ≥ 5.1 s
Red interval duration	None, < 1.0–2.0 s, 2.1–3.0 s, or > 3.0 s
Opposing left-turn signalization	Protected only, Permissive only, Protected-permissive (leading left-turn), Permissive-protected (lagging left-turn), or None/prohibited
Approach grade	Level (between -3% and +3%), Upgrade (greater than +3%), or Downgrade (greater than -3%)
Existence of red-light camera enforcement	Camera enforcement at the intersection, or No camera enforcement program within jurisdiction
Time of day for sampling	Weekday peak (7–9 AM, 4–6 PM), Weekday lunch (11 AM–1 PM), Weekday off-peak (all other weekday times), or Weekend periods
Vehicle type	Passenger vehicle, Motorcycle, Bus, Recreational vehicle, Single-unit truck, or Multiunit truck

These factors can be disaggregated into characteristics related to the site or to the sampled vehicles. Other criteria included site adequacy for camera placement, the presence of relatively straight approaches, two through lanes on the intersection approach, and intersections with ~90° approaches, as well as agency cooperation/assistance (McGee et al. 2012). Due to similarities between our study and that of McGee et al. (McGee et al. 2012) in terms of the nature of the problem and the collected data, we considered these site selection factors when evaluating potential data collection sites for the RLE project.

Eleven possible sites were suggested by ODOT staff, based on a history of RLR-related crashes, presence of RLR cameras, or use of red clearance extension systems. To determine the best data collection sites for this project, we analyzed the characteristics of the suggested sites and collected additional data, consistent with recommendations from McGee et al. (McGee et al. 2012). Finally, five distinct sites, with one intersection per site, were selected for field data collection (Figure 3.2 and Table 3.2 **Error! Reference source not found.**; detailed site information in Appendix B).

Table 3.2: Summary of selected sites

Site	City	Intersection (Abbreviation)	RLE Active	RLR Camera
1 (Beta)	Corvallis	OR-99W at Circle Blvd. (OR-99W–Circle)	N	N
2	Salem	OR-99E at Broadway NE (OR-99E–Bdwy)	-	N
3	Woodburn	OR-99E at Mt. Hood Ave. (OR-99E–Mt. Hood)	N	Y
4	Unincorporated Multnomah County	US30 at Cornelius Pass Rd. (US30–Cornelius Pass)	Y	N
5	Beaverton	US26WB at 185 th (US26WB–185 th)	Y	N

Sites were located in various geographic areas to aid in the collection of driver behavior from different regions in Oregon, which is necessary for designing a RLE system that can be widely adopted across the state. Sites with pretimed signals were not selected, because Bonneson et al. (Bonneson et al. 2001) showed evidence of differences in driver behavior at pretimed and actuated intersections (i.e., drivers approaching an actuated intersection are less likely to stop). As red clearance extension systems require detection, they are much more likely to be installed at actuated intersections; pretimed intersections would require additional capital and maintenance investment to add vehicle detection. Site 1 in Corvallis was selected as the Beta test site due to its proximity to Oregon State University (OSU). Field data collection procedures were tested at Site 1 to ensure that the desired measures could be effectively collected.

A robust evaluation of various video data collection tools was performed, resulting in the selection of the CountCam Duo 40 (CountCam). A single CountCam Duo camera can simultaneously record two camera feeds for 40 h on a single battery charge. Use of a 60-ft telescoping aluminum pole allowed cameras to be mounted directly to on-site infrastructure, resulting in a relatively quick equipment installation. Video footage was saved to SD cards, which were later uploaded to a computer for data reduction and analysis. Before installation, CountCams were programmed with the time and date, recording times, video size (60 min), and recording quality. Figure 3.3 shows a CountCam system installed at the OR-99W–Circle intersection in Corvallis, Oregon.

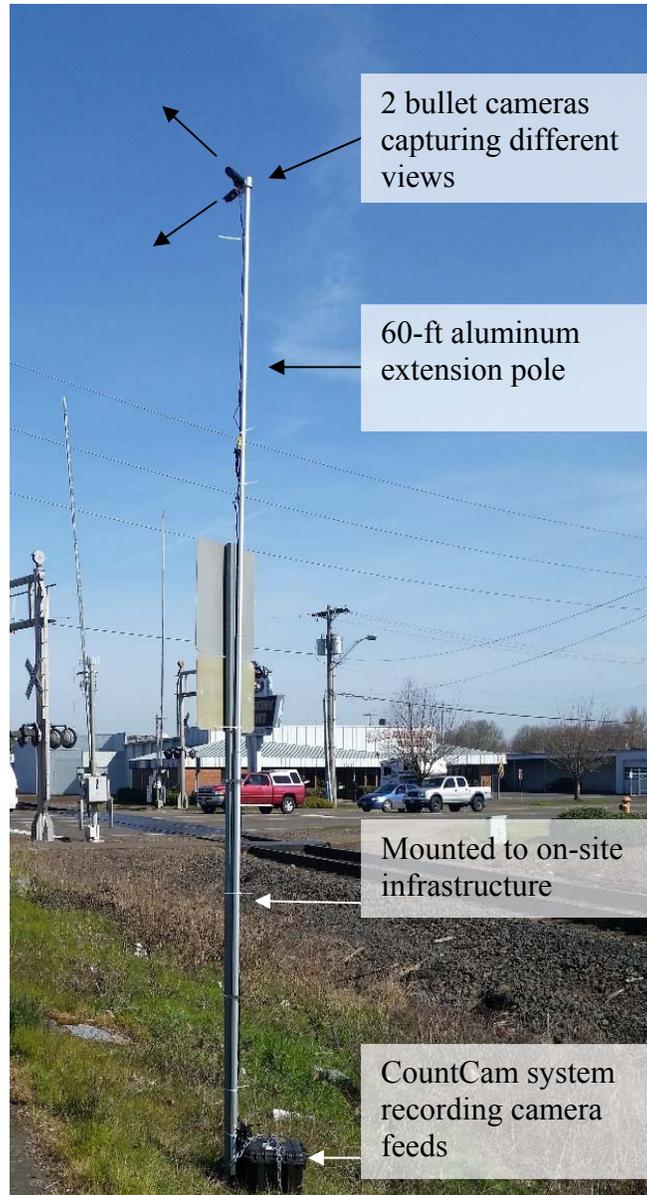


Figure 3.3: Example installation of CountCam Duo 40

At two of the five intersections, CountCam systems were installed at two opposing corners of the intersection. At the remaining three intersections, only one CountCam system was installed due

to site geometry (i.e., T-intersection, curved approach, or inadequate infrastructure for camera installation). Figure 3.4 provides a plan view of typical placements for an installation with two CountCam systems.

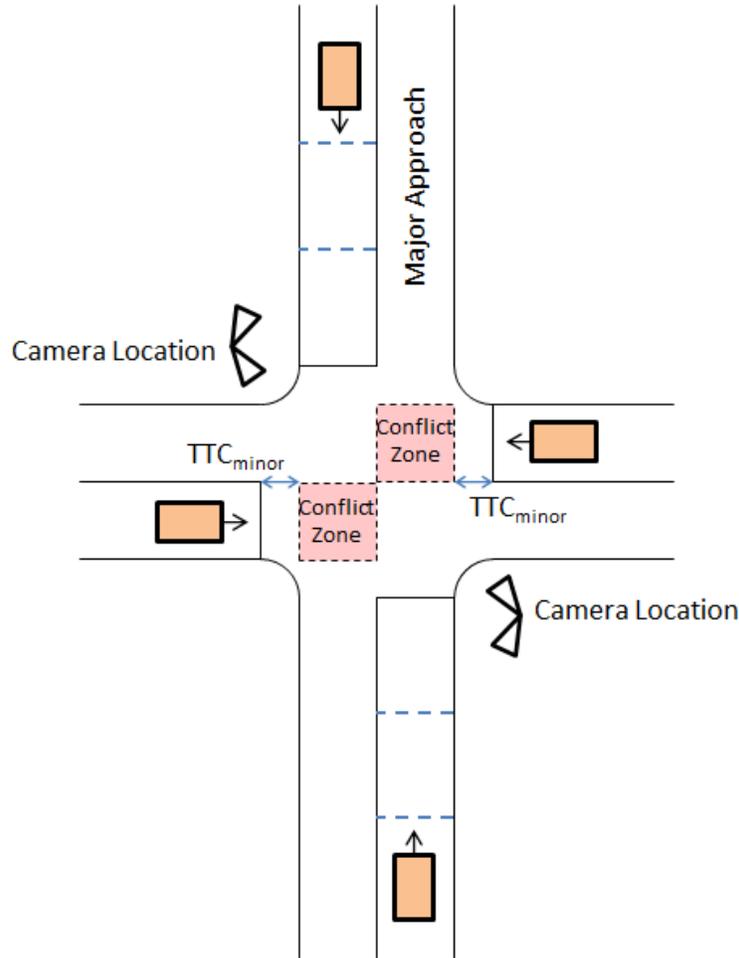


Figure 3.4: Plan view of typical equipment placement

3.4 MANUAL DISTANCE MEASUREMENTS

In addition to collecting video data in the field, a measuring wheel was used to determine distances on the major and minor approaches of each intersection. Distance measurements were captured on video to aid in the video reduction and transcription process.

3.5 FIELD WORK SAFETY PROTOCOLS

ODOT safety protocols required that students wear close-toed shoes, ANSI Class 2 safety vests, and hats while installing equipment on-site. Jobsite Hazard Assessment Worksheets were completed for each location, to ensure that potential hazards and mitigations were identified before arrival. Hazards were reviewed before each installation to ensure that all students were

aware of the potential hazards and the mitigations/safeguards to prevent them. Figure 3.5 shows an OSU student installing a CountCam system on-site, while wearing the required safety gear.



Figure 3.5: Camera installation with student in reflective safety vest and hat

3.6 VIDEO DATA REDUCTION

Distance measurements on the major and minor approaches made in the field were overlaid on video data. Google Earth satellite images and intersection plan drawings were used to verify the field measurements. Paint.net, a free image-editing software, was used to make the “transparent” images that were used as distance overlays in the video reduction process. Data reduction was completed by using VirtualDub, a free video capture/processing utility software that allows captured video data to be viewed frame-by-frame with the video timestamp displayed to the millisecond. CountCams were set to record at a rate of 10 frames per second (i.e., accuracy of 0.1 s). The Image Overlay Utility program was used to display transparent images with distance markings over the video files. The process of setting up the raw video footage to be transcribed is shown in Figure 3.6.

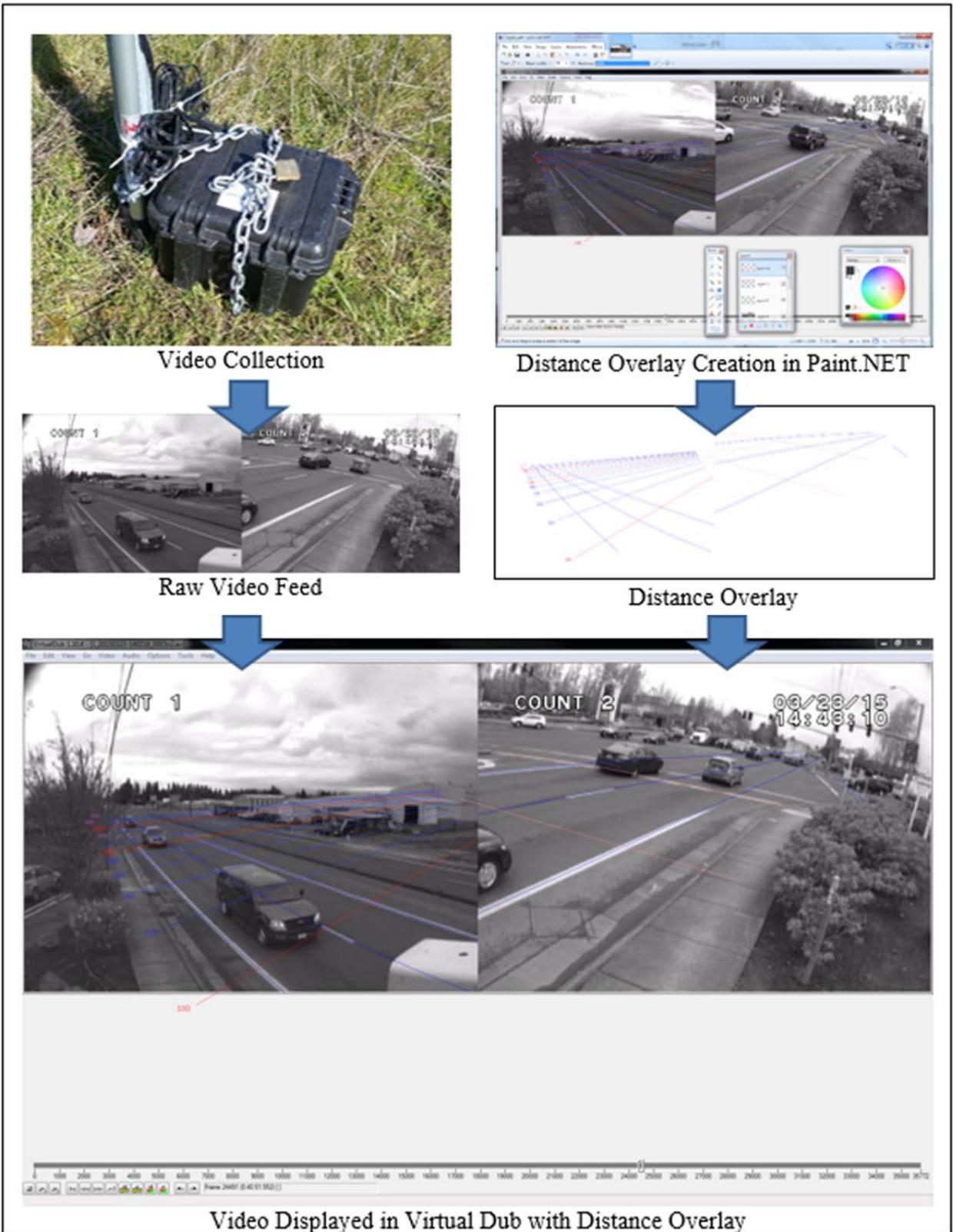


Figure 3.6: Data transcription process

3.6.1 Summary Database

In total, 252 h of video data were collected across all five intersections. File errors and equipment tampering reduced this total to 234 h of usable data (Table 3.3). Due to the position of the cameras, some mainline vehicles were occluded from view resulting in a higher number of minor street vehicles.

Table 3.3: Summary of collected data

Intersection	Hours Recorded (Usable)	Hours Transcribed	Vehicles per day (Major)	Vehicles per day (Minor)	Cycles per day	RLR events per day
OR-99W–Circle	72 (71)	47	N: 844	W: 925	390	N: 3
			S: 640	E: 1035		S: 3
OR-99E–Bdwy	36	23	1141	697	372	0
OR-99E–Mt. Hood	72	36	N: 625	W: 401	403	N: 1
			S: 571	E: 779		S: 0
US30–Cornelius Pass	36	24	984	1439	844	24
US26WB–185 th	36 (19)	19	1350	2180	501	5
Total	252 (234)	149	6155	7456	2510	36

3.6.2 Video Data Transcription

Seven students (2 undergraduate and 5 graduate students) were trained on the software and the transcription process during one 2-h training session, to ensure consistent data transcription. EXCEL templates were created to provide an outline for transcribed data. Additional pictures and notes were included and available throughout the transcription process to promote consistent data transcription. All transcribed data were reviewed by the lead graduate research assistant to ensure consistency of all entries. Video data transcriptions were divided into data from the major and the minor approaches. Types of data transcribed are described in Table 3.4.

Table 3.4: Data transcribed from digital video files

Major Approach	Minor Approach
Timestamps at onset of green, yellow, and red indications	Timestamp at onset of green indication, calculated by major approach data and signal timing plan
Vehicle location at onset of yellow indication	Timestamp of first vehicle per lane to reach stop bar and each conflict zone boundary at onset of minor movement green indication
Vehicle location at onset of red indication	Vehicle classification
Vehicle decision (stop or go)	
Number of RLR vehicles	
Vehicle classification (see Error! Reference source not found.)	

3.7 DATA ANALYSIS

Data from collected videos were categorized into major and minor approaches, plotted, and visually inspected. Major approach data contained a discrete dependent variable and were analyzed by using visual graphics and basic statistics to draw conclusions. Minor approach data contained repeated-measures data with a continuous dependent variable. All datasets were created in Microsoft EXCEL and saved as comma-separated value (.csv) files for importing into R. Data visualization and statistical analysis were performed in both EXCEL and R.

3.7.1 Major Approach

The major approach was observed to gather data on driver decisions to stop or go through the intersection at the onset of the yellow or red indication. Table 3.5 summarizes data for the observed vehicles, categorized by vehicle type.

Table 3.5: Frequency of vehicles of different types

Location		Vehicle Type						Total
		M	PC	LT	B	T	ST	
OR-99W-Circle	NW	1 (0.1%)	682 (81%)	124 (15%)	2 (0.2%)	17 (2%)	18 (2%)	844 (14%)
	SE	3 (0.5%)	497 (78%)	114 (18%)	1 (0.2%)	15 (2%)	10 (2%)	640 (10%)
OR-99E-Bdwy.		0 (0%)	789 (69%)	232 (20%)	10 (0.9%)	27 (2%)	83 (7%)	1141 (19%)
OR-99E-Mt. Hood	NW	0 (0%)	394 (63%)	180 (29%)	2 (0.3%)	15 (2%)	34 (5%)	984 (16%)
	SE	0 (0%)	346 (61%)	164 (29%)	0 (0%)	29 (5%)	32 (6%)	1350 (22%)
US26WB-185 th		1 (0.1%)	841 (85%)	106 (11%)	19 (1.9%)	9 (1%)	8 (1%)	625 (10%)
US30-Cornelius Pass		3 (0.2%)	952 (71%)	306 (23%)	4 (0.3%)	27 (2%)	58 (4%)	571 (9%)
Total		8 (0.1%)	4501 (73%)	1226 (20%)	38 (0.6%)	139 (2.3%)	243 (3.9%)	6155

*Motorcycle (M); Passenger car (PC); Light truck (LT); Bus (B); Truck (T); Semi-Truck (ST)

To facilitate analysis, data were disaggregated into observations at the onset of the yellow or red indication, because driver responses to the onset of the yellow and red indications will differ. Figure 3.7 shows the frequency of all major approach vehicle locations, with respect to distance from the stop line. The data are further broken down by indication onset and location.

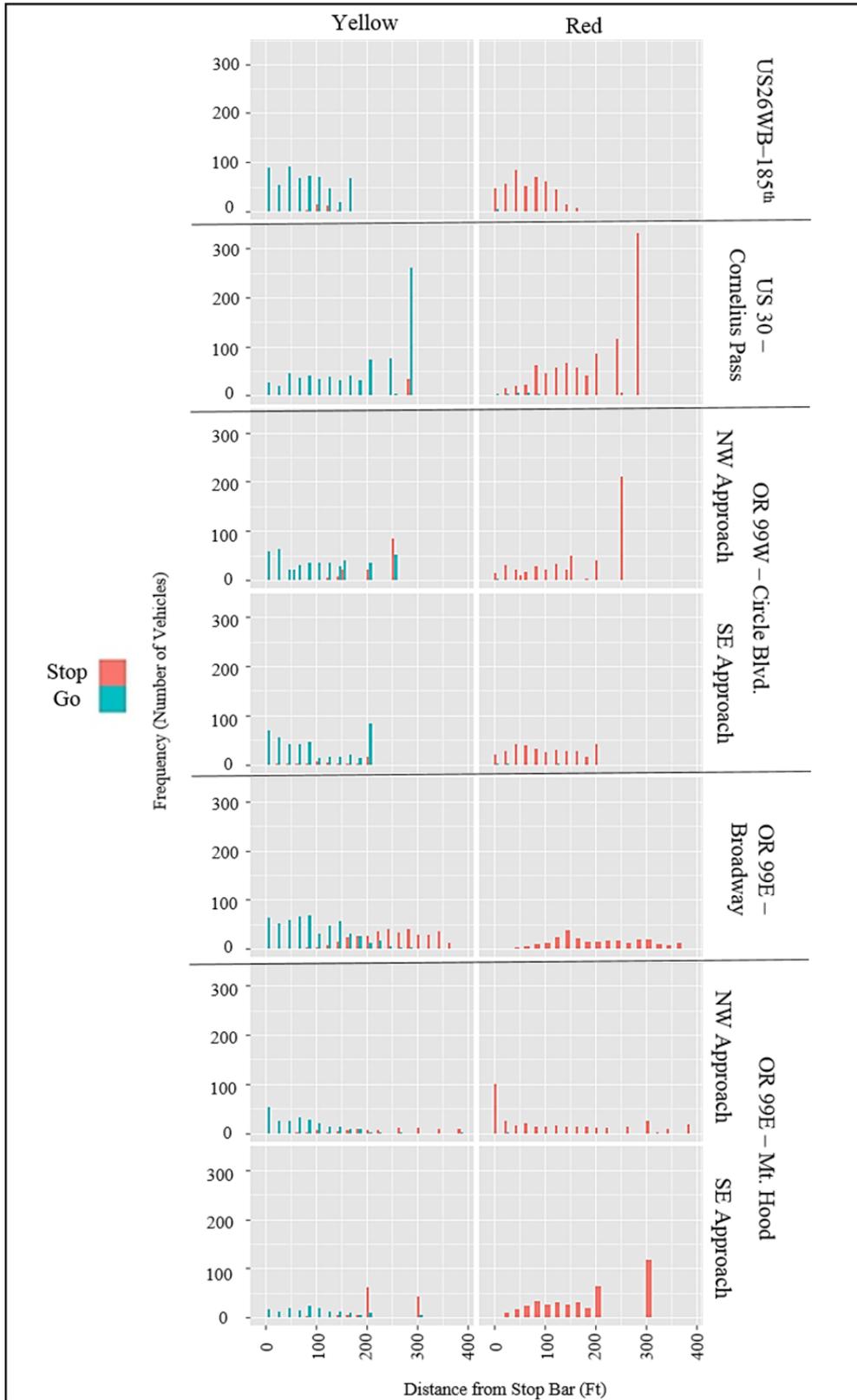


Figure 3.7: Summary of major approach data by indication and location/camera

Drivers frequently proceeded through the intersection in response to a yellow indication. This overrepresentation was a result of field-of-view limitations of the cameras. The OR-99E–Mt. Hood (Woodburn) site had a longer observation zone than other sites, resulting in a more robust distribution of vehicles that stopped or proceeded through the intersection (Figure 3.8).

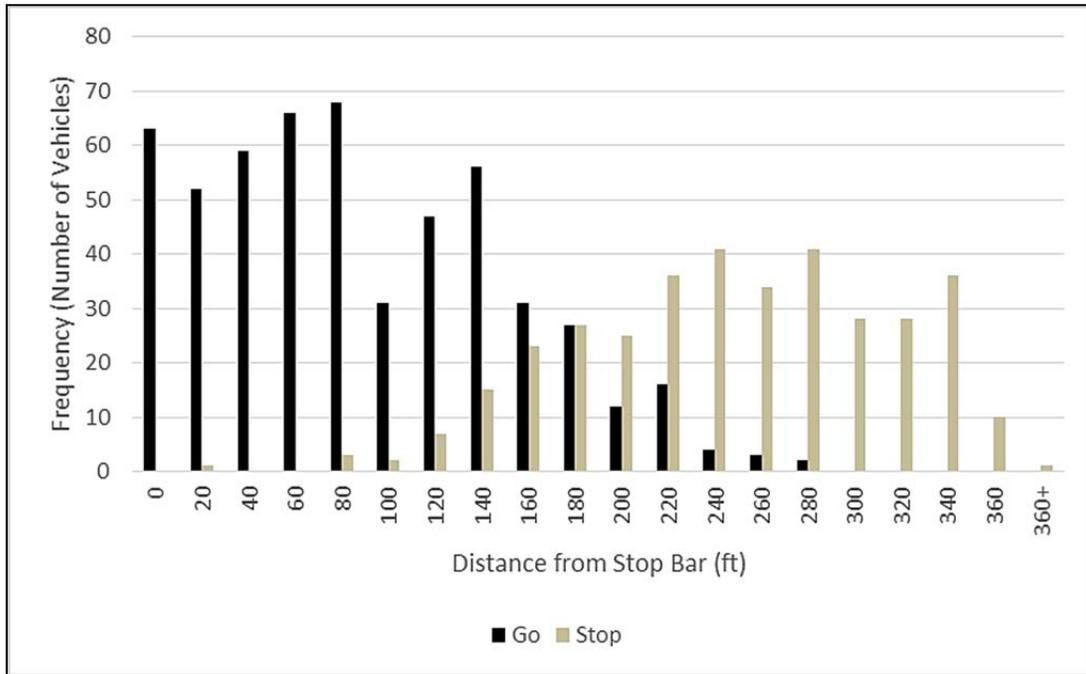


Figure 3.8: Frequency of vehicles that stop or go, based on distance from the stop bar, at onset of the yellow indication at OR-99E–Broadway (Salem)

Vehicles closer to the stop line tended to go through the intersection, whereas vehicles further from the stop line tended to stop. Figure 3.9 investigates this trend by displaying the cumulative frequency of stopping and continuing vehicles, referenced by vehicle location from the stop line. Approximately 85% of observed vehicles that went through the intersection were located within 160 ft of the stop line, and ~85% of vehicles that stopped were located more than 160 ft from the stop line, at the onset of the yellow indication. In accordance with traffic regulations, most vehicles stopped in response to onset of the red indication.

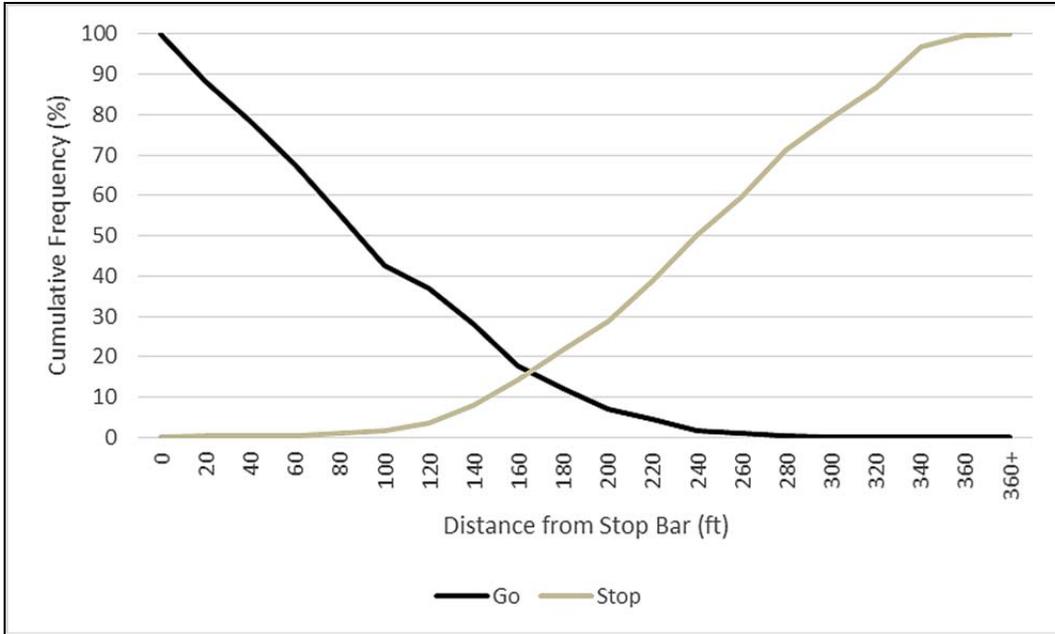


Figure 3.9: Cumulative frequency of vehicles that stop or go, based on distance from the stop bar, at onset of the yellow indication at OR-99E–Mt. Hood (NW Woodburn)

The US30–Cornelius Pass intersection experienced higher rates of RLR vehicles than other test sites. Figure 3.10 shows the frequency of decisions to stop or proceed, with respect to the distance from the stop line, at onset of the red indication at this intersection.

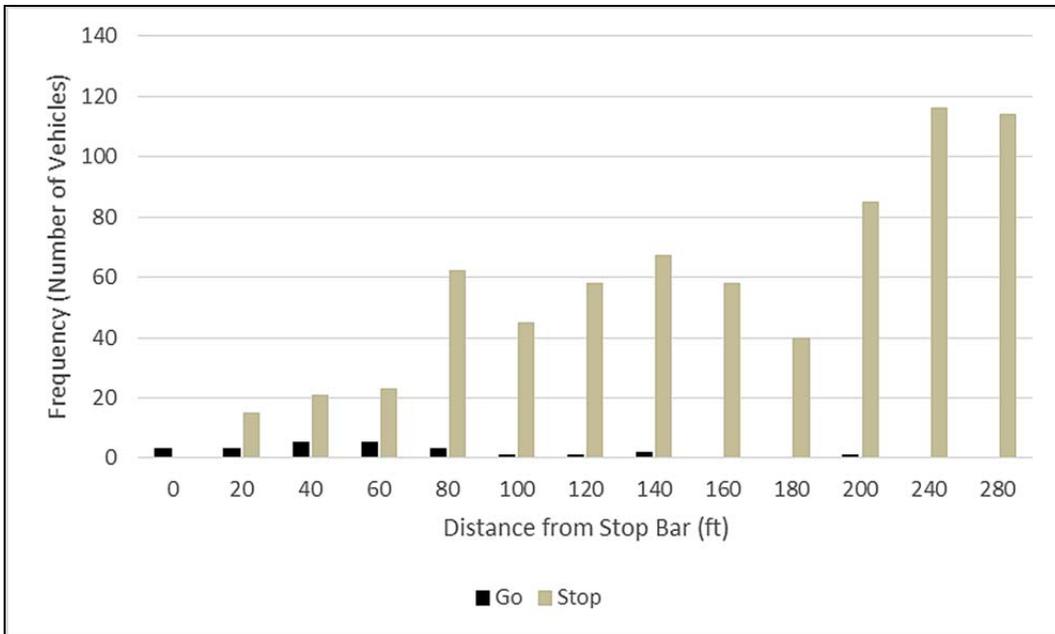


Figure 3.10: Frequency of vehicles that stop or go, based on distance from the stop bar, at onset of the red indication at US30–Cornelius Pass

Figure 3.10 again demonstrated that vehicles that went through or stopped at the intersection were clustered closer to or further from the stop line, respectively. Figure 3.11 investigates this

observation by displaying the cumulative frequency of stopping and continuing vehicles, based on vehicle location. Approximately 80% of vehicles that ran the red light or that stopped were within 100 ft or were over 100 ft from the stop line, respectively, at the onset of the red indication.

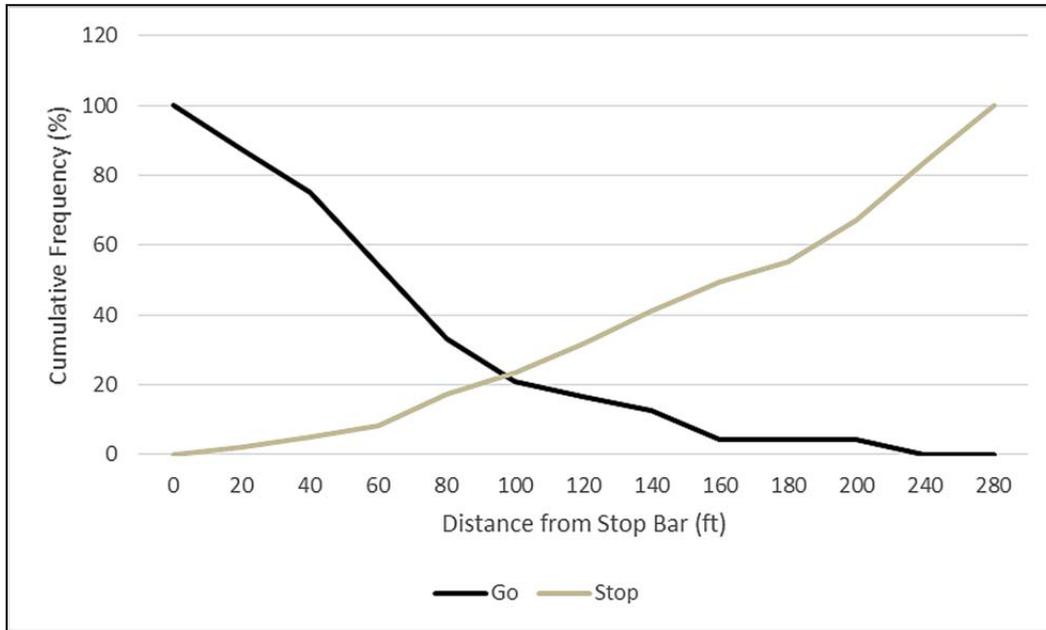


Figure 3.11: Cumulative frequency of vehicles that stop or go, based on distance from the stop bar, at onset of the red indication at US30-Cornelius Pass

Figure 3.12 shows the frequency of RLR vehicles, with respect to distance from the stop bar, at the onset of the red indication.

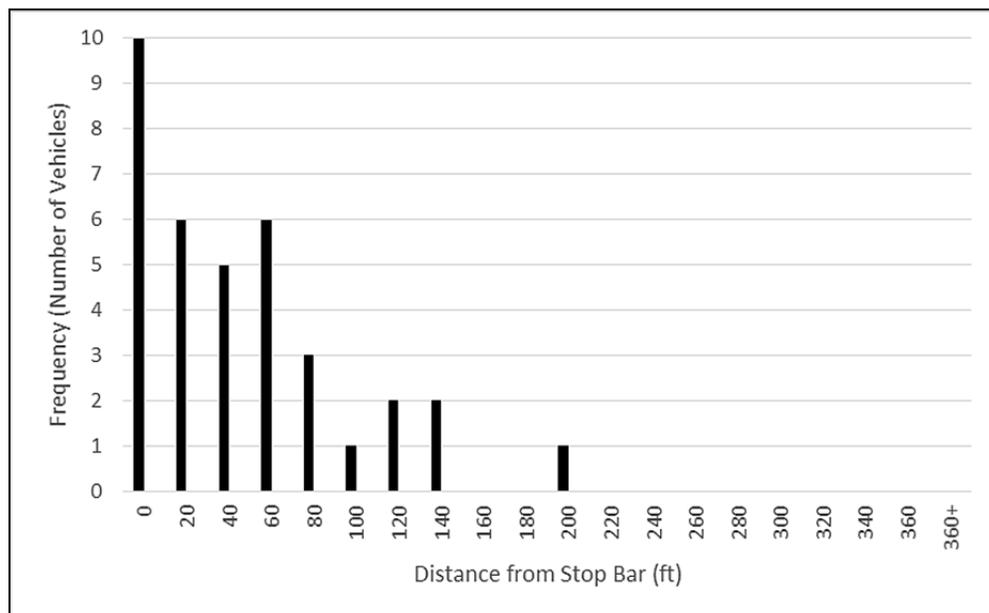


Figure 3.12: Frequency of all observed RLR vehicles, based on distance from the stop bar, at onset of the red indication

RLR vehicles were observed between 0 and 200 ft from the stop line. Among the observed RLR vehicles, 33% were light trucks and 11% were semi-trucks. These vehicle types were overrepresented as RLR vehicles, as they represented only 20% and 3.9%, respectively, of the total volume of approaching vehicles.

As the highest frequency of RLR occurred at the US30–Cornelius Pass intersection, the characteristics of RLR vehicles at this location were examined in detail (Table 3.6).

Table 3.6: Details of RLR vehicles at US30–Cornelius Pass

No.	Date	Time	Vehicle Type	Speed (mph)	Speed (ft/s)	Distance to SB (ft)	Latency (s)
1	3/24/2015	07:13:09.590	LT	60.7	89.1	4.27	0.048
2	3/24/2015	07:42:27.147	PC	52.1	76.4	9.45	0.124
3	3/24/2015	08:02:50.470	PC	56.1	82.2	11.21	0.136
4	3/24/2015	08:29:42.182	PC	58.3	85.5	18.62	0.218
5	3/24/2015	09:48:58.138	PC	56.0	82.2	64.65	0.787
6	3/24/2015	10:51:22.783	ST	52.0	76.3	70.53	0.924
7	3/24/2015	11:07:32.452	PC	73.0	107.0	45.14	0.422
8	3/24/2015	14:27:37.758	PC	63.4	93.0	86.48	0.930
9	3/24/2015	15:05:24.424	LT	60.7	89.1	40.00	0.449
10	3/25/2015	08:03:14.194	PC	66.3	97.2	117.84	1.213
11	3/25/2015	09:26:20.480	LT	66.3	97.2	117.84	1.213
12	3/25/2015	10:17:59.780	PC	66.3	97.2	40.00	0.412
13	3/25/2015	11:03:27.808	ST	40.5	59.4	63.75	1.074
14	3/25/2015	11:57:27.548	LT	54.1	79.3	75.85	0.956
15	3/25/2015	13:12:27.147	PC	60.7	89.1	40.00	0.449
16	3/25/2015	13:50:57.457	PC	34.1	50.0	200.00	4.000
17	3/25/2015	14:12:27.347	ST	52.0	76.3	124.03	1.625
18	3/25/2015	14:22:22.442	PC	63.3	92.9	128.33	1.382
19	3/25/2015	15:34:39.680	T	66.3	97.2	88.59	0.912
20	3/25/2015	15:19:52.192	LT	66.3	97.2	40.00	0.412
21	3/25/2015	16:13:24.204	LT	36.4	53.4	61.43	1.149
22	3/25/2015	17:24:56.296	PC	60.7	89.0	57.80	0.649
23	3/25/2015	18:35:42.943	PC	58.3	85.5	1.46	0.017
24	3/25/2015	18:49:54.695	LT	76.8	112.6	23.11	0.205
Min				34.1	50.0	1.46	0.017
Median				60.7	89.1	59.62	0.718
Mean				58.4	85.6	63.77	0.821
Max				76.8	112.6	200.00	4.000
SD				10.4	15.2	48.01	0.817

Over the course of 24 h of transcribed data at this intersection, 24 RLR vehicles were observed. On average, RLR vehicles proceeded through the intersection when they were ~64 ft from the stop line (0.821 s) and when they were traveling at speeds considerably above the posted speed limit (58.4 mph; Figure 3.7). Table 3.7 summarizes the frequency of RLR at this intersection with regard to recorded time periods and number of cycles. The highest rate of RLR was observed during the 8:00 to 9:00 AM and 3:00 to 4:00 PM periods.

Table 3.7: Time frequency of RLR vehicles at US30–Cornelius Pass

Time Period	# Cycles	# Leading Vehicles Recorded	# RLR	Frequency of RLR / Cycle	Frequency of RLR / # of Leading Vehicles
7–8 AM	118	122	2	0.017	0.016
8–9 AM	134	75	3	0.022	0.040
9–10 AM	147	93	2	0.014	0.022
10–11 AM	136	104	2	0.015	0.019
11 AM–12 PM	158	87	3	0.019	0.034
12–1 PM	161	90	0	0.000	0.000
1–2 PM	150	74	2	0.013	0.027
2–3 PM	144	75	3	0.021	0.040
3–4 PM	139	64	3	0.022	0.047
4–5 PM	131	55	1	0.008	0.018
5–6 PM	132	70	1	0.008	0.014
6–7 PM	150	65	2	0.013	0.031
Overall	1,700	974	24	0.014	0.025

3.7.2 Minor Approach

The determination of the TTC values for the minor street vehicles is a critical component of designing effective RLE systems, as the duration of the RLE are directly influenced by this time. The minor approach was observed to determine the TTC values for the first vehicle on the minor approach at the onset of the green indication (Figure 3.13). During 121 h of observations, 7,456 vehicles were observed (Table 3.5).

Figure 3.14 and Figure 3.15 present 1 h of minor approach data from the US26WB–185th intersection. Figure 3.14 shows the vehicle trajectories of the first minor vehicle in the left-most through lane at the onset of the green indication during a single hour of video data. Vertical lines represent the stop line (SL) and the edges of the conflict zone regions (C1, C2, and C3). The major approach is comprised of two lanes (i.e., two conflict-zone regions). Figure 3.15 shows the distributions of time to the stop line and time to the edge of each conflict zone for a full day’s worth of video data (12 h).

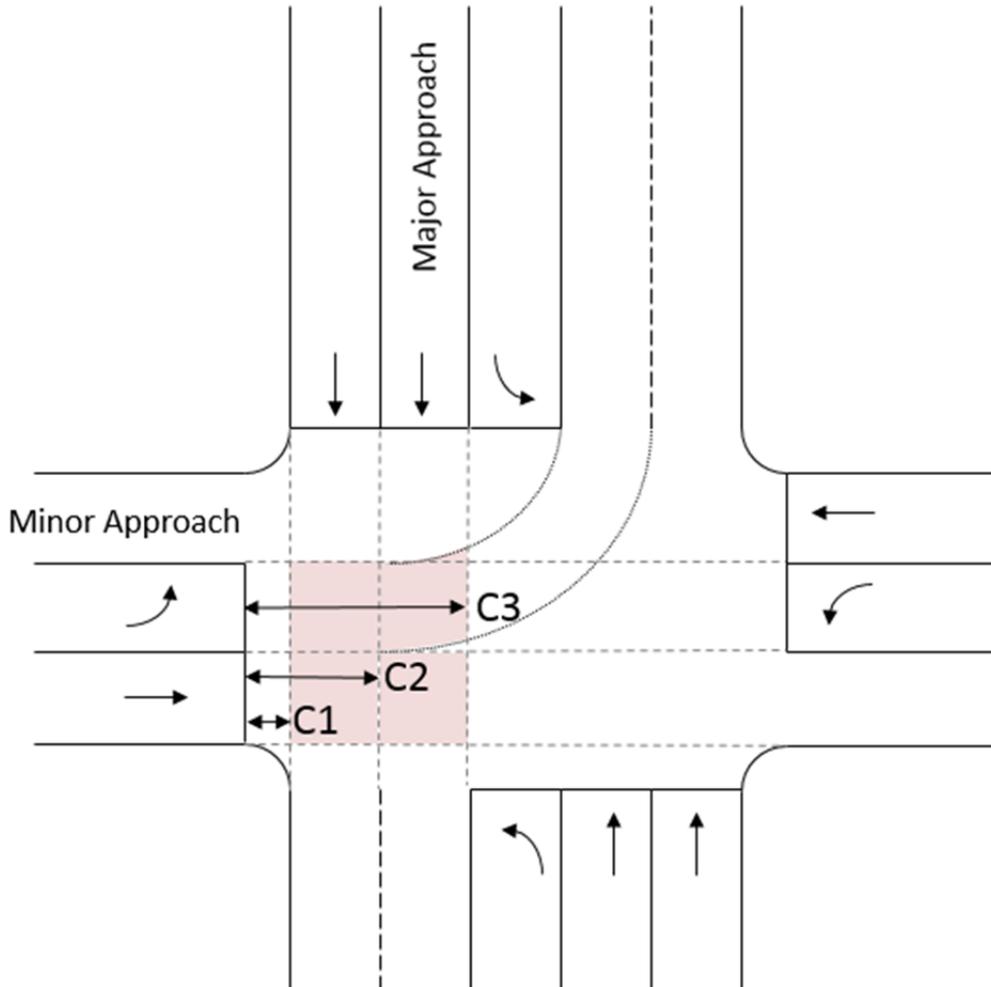


Figure 3.13: Minor approach conflict zone measurements

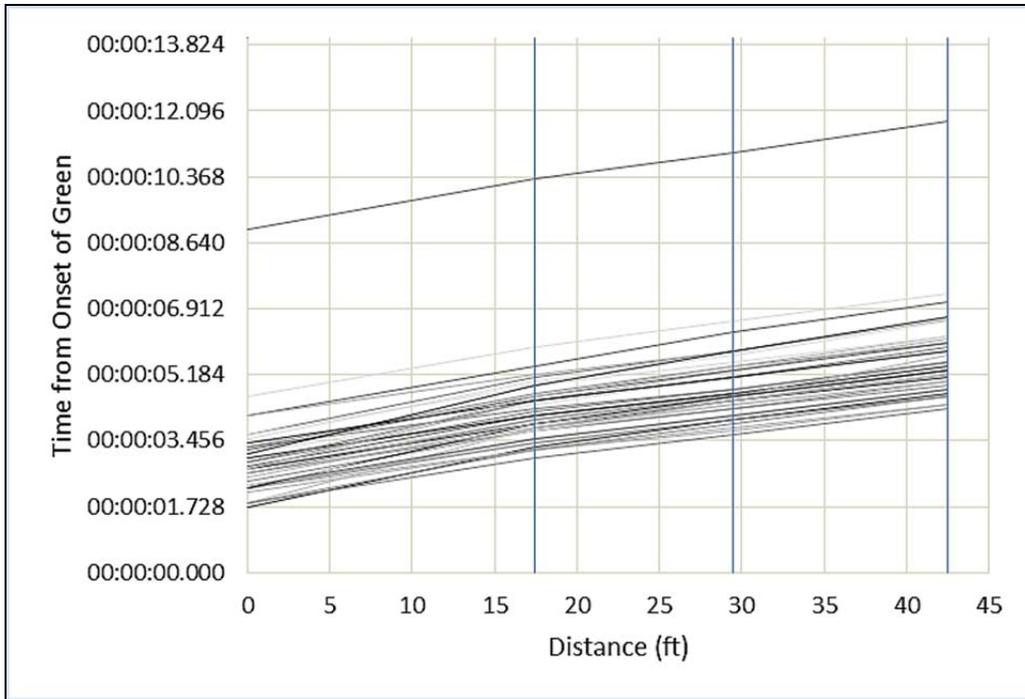


Figure 3.14: Vehicle trajectories at onset of green indication of the near lane for 1 h at US26WB-185th

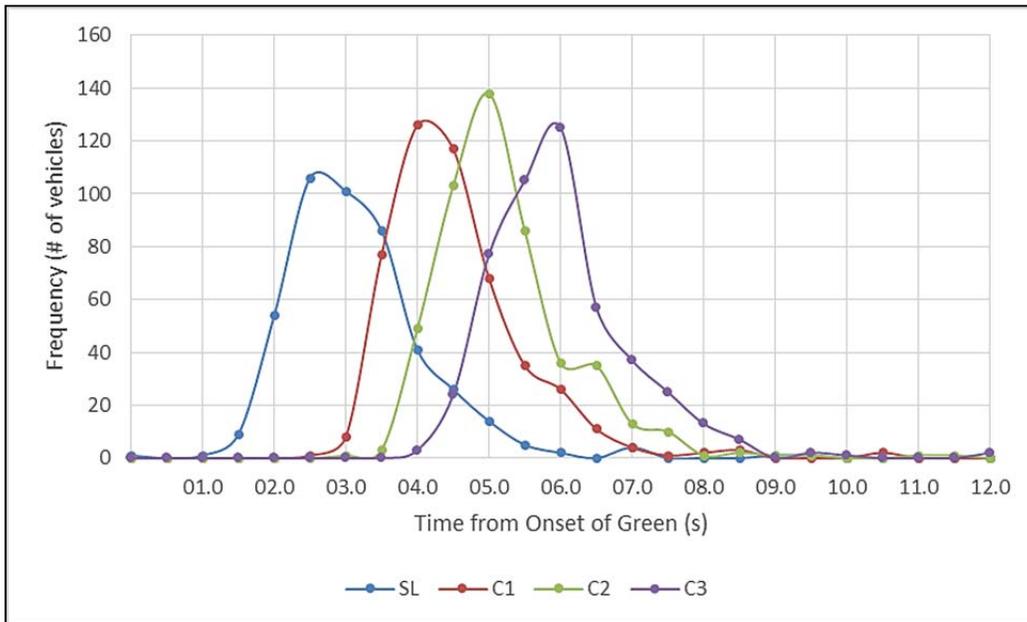


Figure 3.15: Frequency of time to stop line (SL) and each conflict zone (C1-C3) from onset of green indication for 12 h at US26WB-185th

Figure 3.15 demonstrates that a minimum of 3 seconds expires before a conflicting minor street vehicle could risk collision of a RLR on the major approach.

Visual inspection of Figure 3.15 shows that the frequency distribution is slightly skewed to the right. A log transformation was applied and a normal Q-Q plot(Figure 3.16) was created to verify the appropriateness of the long transformation of TTC values for the first conflict point (C1) at the intersection of US26WB – 185th. The majority of the points roughly fall on the Q-Q line suggesting a reasonably normal distribution though the right tail does lift away from the Q-Q line indicating that distribution is still slightly skewed to the right.

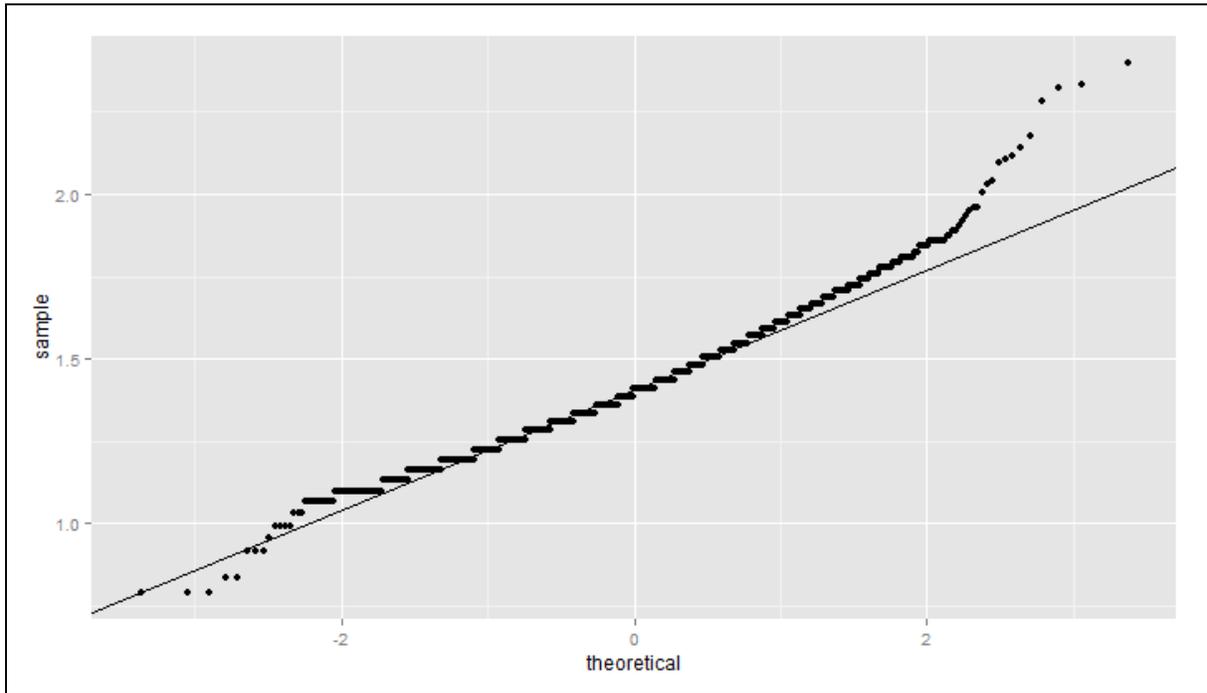


Figure 3.16: Normal Q-Q plot for TTC values at US26WB – 185th

Given the nearly normal shape after a log transformation, the 5th %ile TTC for a conflict zone can be calculated using the properties of a normal distribution. Based on the properties of a normal distribution, the 5th %ile is located approximately 1.645 standard deviations below the mean. The 5th %ile TTC can be calculated using the following equation:

$$5th\ \%ile\ TTC = \mu + (-1.645 * \sigma) \quad (3.4)$$

Where, μ is the mean TTC (s); and σ is the standard deviation TTC (s). Table 3.8 displays the mean, standard deviation and 5th %ile TTC for the first conflict point (C1 in Figure 3.13) at each location.

Table 3.8: Summary and 5th %ile of TTC at each location

Location		Mean (s)	SD	5 th %ile TTC (s)
OR-99W – Circle Blvd	NW	4.938	1.176	3.00
	SE	4.100	1.273	2.01
OR-99E – Broadway		4.865	1.237	2.83
OR-99E – Mt. Hood	NW	5.053	1.210	3.06
	SE	5.906	1.183	3.96
US26WB – 185 th		4.121	1.221	2.11
US30 -Cornelius Pass		4.811	1.243	2.77
All Locations		4.707	1.263	2.63

The 5th %ile TTC for the first conflict point varies by intersection approach from 2.01 to 3.96 seconds (Table 3.8). Figure 3.17 displays a boxplot of TTC values for the first conflict point (C1) at each location as well as the average distance from C1 to the stop line for each location.

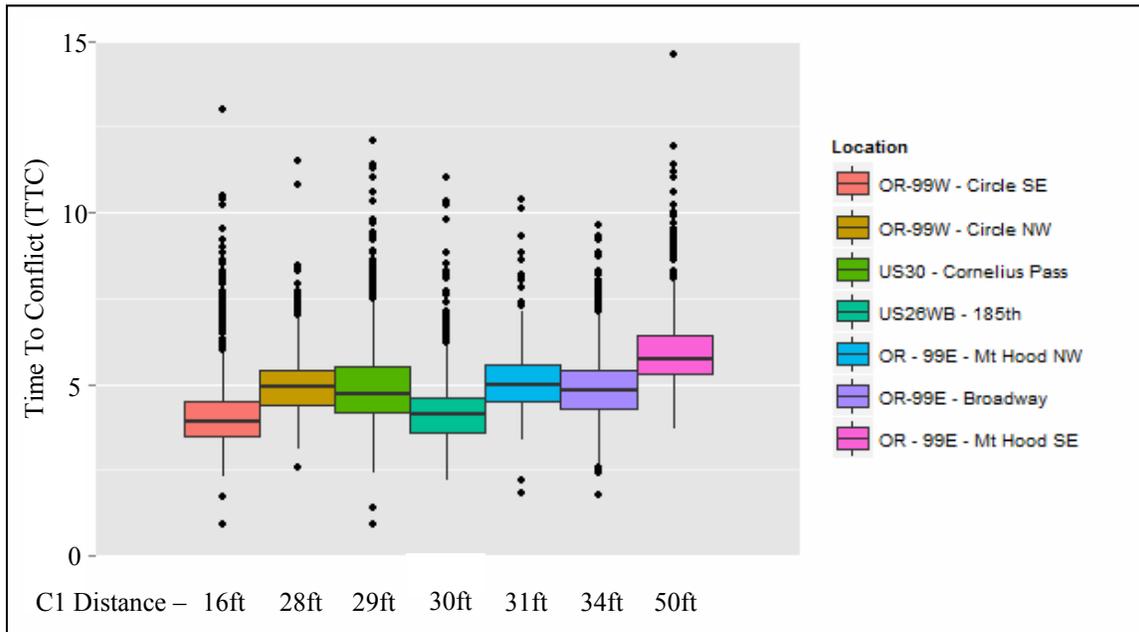


Figure 3.17: TTC at each location

Figure 3.17 demonstrates that the TTC varies for each intersection approach. This is due to differences in distance to the conflict zone. Some combination of development density and functional classification of the roadways also appears to play a role in TTC values. The intersection of US26WB – 185th is in an urban setting whereas the intersection of OR99E – Mt Hood NW is in a suburban setting. At both intersections, the distance to the conflict zone is the same, but the TTC values at the OR-99E – Mt Hood NW intersection are slightly higher. These factors show that the TTC for the minor approach should be calculated for each intersection to ensure that appropriate values are selected to both protect minor approach vehicles and to reduce delay.

To evaluate the possible effects of vehicle type on TTC, data from the US30–Cornelius Pass intersection were selected for further analysis. Movements of vehicles on the inside lane of the minor approach (first opposing left turns) were extracted, and the time to the first conflicting point (C1) was considered as the TTC. Figure 3.18 depicts the boxplot of TTC values by vehicle type. Although passenger cars reached conflict points faster, buses and semi-trucks had higher TTC values. Results of one-way analysis of variance (ANOVA) confirmed that the TTC is dependent on vehicle type (F-test = 3.05, p-value = 0.1001).

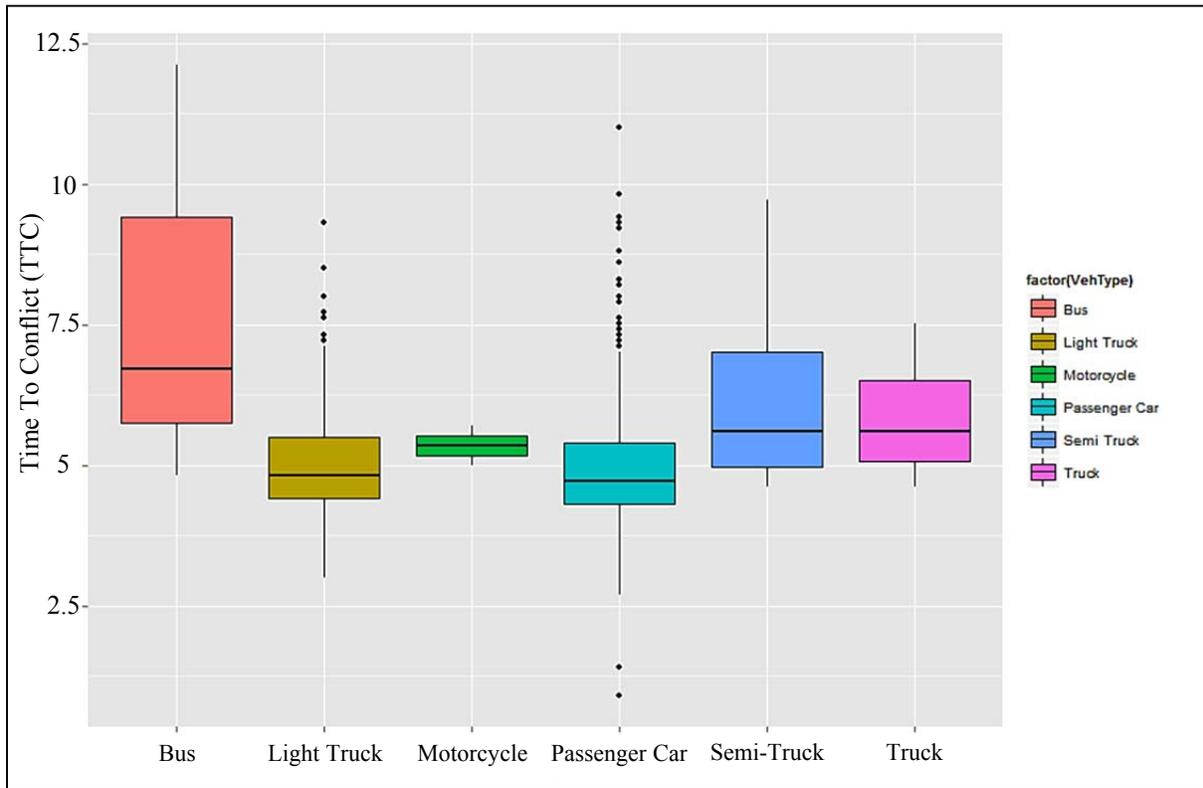


Figure 3.18: TTC, based on vehicle type, at US30–Cornelius Pass

3.7.3 Spot Speed Data

Table 3.8 summarizes spot speed data for each direction of the major approach of four intersections.

Table 3.9: Summary of spot speed data

Intersection	Approach	Count	Speed (mph)				
			Max	Mean	Median	SD	85 th %ile
OR-99W– Circle	N	195	53	40.22	40	4.86	45
	S	201	66	47.10	47	4.55	52
OR-99E–Bdwy	N	193	54	36.32	36	3.85	40
	S	205	56	43.29	43	4.47	48
OR-99E–Mt. Hood	N	197	49	38.54	38	3.94	43
	S	190	51	36.52	37	3.28	40
US30– Cornelius Pass	NW	125	68	53.50	53.00	4.77	57.40
	SE	125	81	57.51	57.00	5.06	61.00

3.8 CALCULATION OF RED LIGHT EXTENSION DURATION

An effective RLE should allow RLR vehicles to clear the furthest conflict zone before a conflicting minor approach vehicle reaches the conflict zone.

Figure 3.19 shows the furthest conflict zone for one approach at the intersection of OR-99E-Broadway (Salem). To determine the required extension time the clearance time (CT) for the major approach and TTC for the minor approach must be determined and the RLE detector location must be known.

It is important to note that the signal phasing should be considered when determining the furthest conflict zone. In particular only the conflict zone of the major approach and the following phase movement should be considered when determining the furthest conflict zone. For example if the minor approach left turning movement follows the termination of the major approach then the conflict zone of the major approach with the minor approach left turn should be considered.

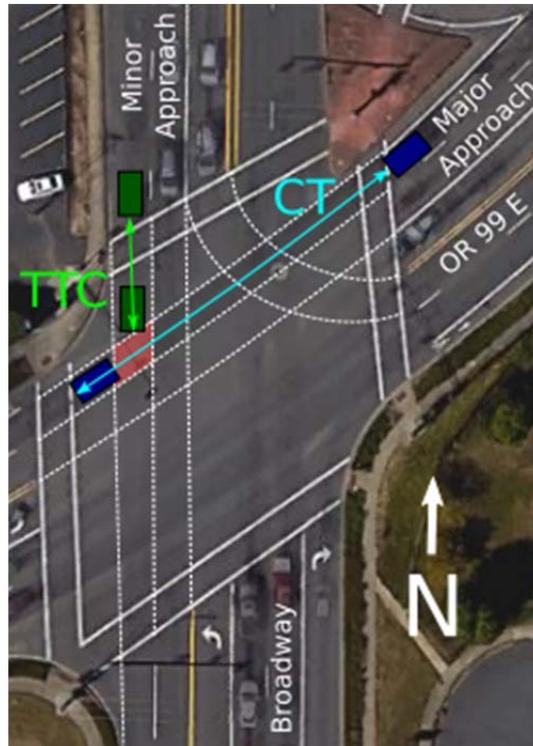


Figure 3.19: Furthest conflict zone for studied approach at OR-99E - Broadway (Salem)

To calculate the CT, the 85th %ile speed (from spot speed studies) and intersection width (from field measurements or construction documents) must be determined for each intersection approach. The CT for the major approach can be calculated using a modified version of the red clearance interval equation from the 6th edition of the ITE's Traffic Engineering Handbook (ITE 2010).

$$C = \frac{w + L}{v} \quad (3.5)$$

where, C is the clearance time (s); w is the width of the stop line to the far-side furthest conflict zone (ft); v is the 85th %ile speed (ft/s); and L is the vehicle length (typically 20 ft). For example, for the studied approach at the intersection of OR-99E-Broadway, the CT is calculated from Equation 3.5 as follows:

$$\frac{120ft + 20ft}{\left(40mph * \frac{1.47ft/s}{mph}\right)} = 2.4 s$$

Next, the 5th %ile TTC of the first conflict zone for the major approach must be determined. A field study sampling the TTC of vehicles on each minor approach should be completed to determine the mean and standard deviation of the TTC values. The 5th %ile TTC can then be calculated using equation 3.4 discussed above. As see in Table 3.8, the 5th %ile TTC for the first conflict zone at the intersection of OR-99E-Broadway (Salem) is 2.8 s.

The extension time can be calculated using the following equation:

$$Ext = \left(\frac{Det}{v} \right) + C - 5th\ \%ile\ TTC \quad (3.6)$$

where, Ext is the extension time; Det is the RLE detector location (ft) measured from the stop line; v is the 85th %ile speed (ft/s); C is the clearance time (s) calculated from Equation 3.5; and 5th %ile TTC (s), calculated from Equation 3.4. This equation adds the time it takes for the RLR vehicle to reach the stop line after being detected to the conflict time and is reduced by the time it takes 95% of minor approach vehicles to reach the conflict zone (5th %ile TTC). The extension time for studied approach of the example intersection of OR-99E-Broadway (Salem), assuming a detector located 60 ft upstream of the intersection, is calculated from Equation 3.6 as:

$$\left(\frac{60ft}{40mph * 1.47ft/s} \right) + 2.4\ s - 2.8\ s = 0.6\ s$$

3.9 DISCUSSION

Data were collected and analyzed to understand the characteristics of RLR in Oregon. This information will be utilized to calibrate a HIL simulation to test various red clearance interval extension solutions, including detection and timing strategies. Of particular importance is the TTC measurement as this ultimately bounds the usefulness of a red clearance extension for the purpose of bounding a crash.

In this Chapter, we thoroughly discussed the field study, as well as the processes of data collection and reduction, with a particular focus on driver behavior on the major and minor approaches. The complexity of the HIL simulation necessitated the selection of a subset of field locations to allow for a greater variety of alternative red clearance interval extension solutions to be modeled. Of the five field sites, one was selected for simulation testing.

The intersection of US30 and Cornelius Pass Rd. was the ideal candidate for additional modeling because it currently operates a red clearance interval extension system and is overrepresented by RLR events. The selected intersection needed to have enough traffic flow to enable the observation and analysis of complicated situations in the simulation study. With 2,180 observed vehicles, the chosen intersection had the highest traffic volume (29% of total volume) among all intersections. Second, the higher the frequency of cycles, the greater the opportunity to observe RLR events. With 844 cycles per day, the US30–Cornelius Pass intersection had the highest rate of cycles among all intersections. The most important criterion in site selection is the frequency of observed RLR events. With 24 RLR events, the intersection of US30 with Cornelius Pass had the highest rate of RLR among all intersections. Finally, the speed of vehicles on each approach is related to the potential severity of crashes. The NW and SE approaches of the intersection of US30 at Cornelius Pass had 85th %ile speeds of 57.4 and 61 mph, respectively, which were the highest operational speeds among all field sites. As such, this intersection was selected for simulation study.

4.0 SIMULATION OF RED CLEARANCE EXTENSION SYSTEMS

This Chapter describes a microsimulation analysis of different detection strategies used to trigger red clearance extensions. Specifically, the VISSIM microsimulation software with HIL was used to analyze detection strategies at the intersection of US30 with Cornelius Pass Rd. (Site 4).

4.1 SIMULATION SITE

The US30–Cornelius Pass intersection currently operates with a 2070 controller and NWS Voyage™ firmware. This intersection uses the NWS Voyage™ RLE function on through movements along US30 (NW- and SE-bound) and Cornelius Pass Rd. (NE-bound) for the left-turn movement. RLE events are triggered by loop detectors located downstream from the stop line. Appendix C includes a design drawing showing the existing detector placement. Figure 4.1 displays an aerial image of the intersection.



Figure 4.1: Satellite image of the intersection of US30 and Cornelius Pass Rd.

4.2 VISSIM MODEL

A model of the US30–Cornelius Pass intersection was developed by using VISSIM 6. Figure 4.2 displays a screen capture of the VISSIM model.

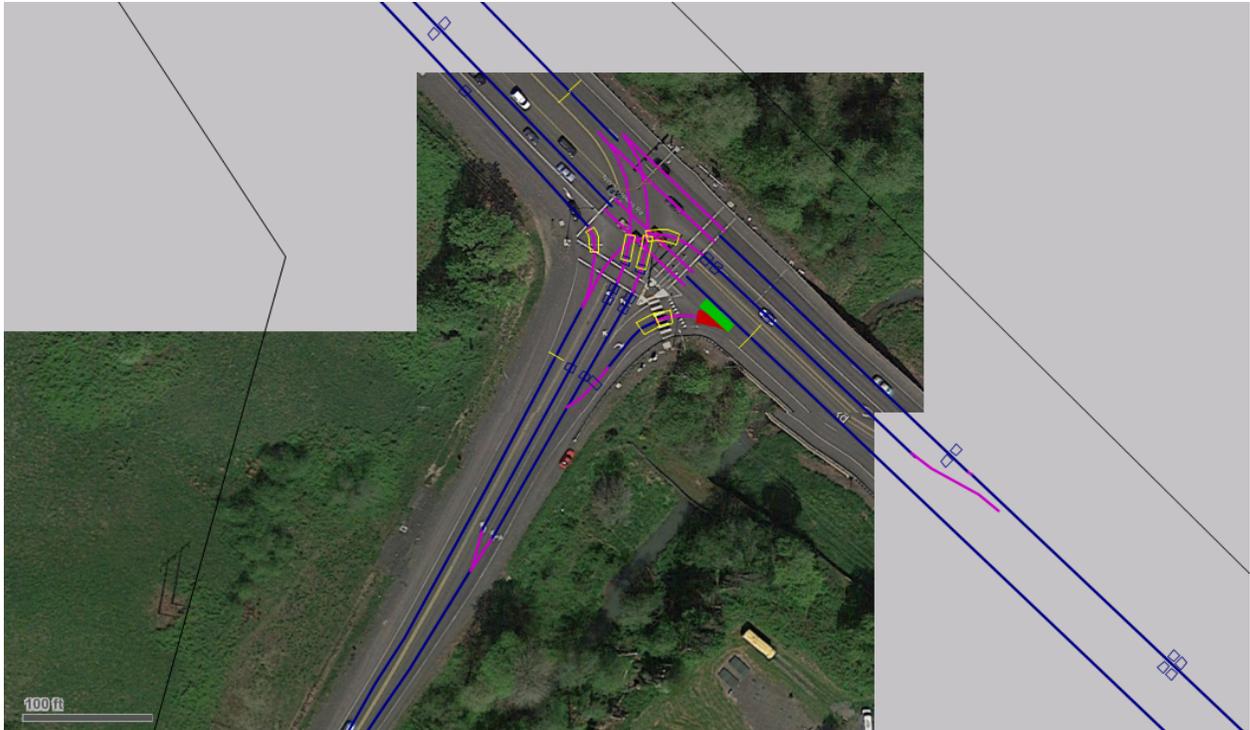


Figure 4.2: VISSIM model of the US30–Cornelius Pass intersection

The following field data described in Chapter 3 were used to create the VISSIM model:

- Link alignments and length were taken from scaled aerial images.
- Detector locations were matched to those shown in design drawings (Appendix C).
- Vehicle turning movement volumes for passenger cars and heavy vehicles were determined from video data collected at the intersection during weekday PM peak hours (Appendix D).
- RADAR-measured speed profiles were used to calibrate speed distributions along US30 (Appendix E). Speed profiles on Cornelius Pass Rd. were estimated.
- Signal heads were programmed with 99% compliance¹, to minimize RLR.

4.3 HARDWARE-IN-THE-LOOP

HIL was used to control the traffic signal in the VISSIM microsimulation. Actuations from simulated detectors were used to create inputs for a physical signal controller, which, in turn, was used to operate the simulated signals. ODOT provided the NWS Voyage™ BIN file,² containing the existing signal timing at US30–Cornelius Pass. The file was loaded and run on an Econolite

¹ Of all the vehicles in the system, 1% disobedience would take place regarding traffic signals, in which vehicles would not stop in respond to red indication.

² Two changes were made to timings in the BIN file: RLE on Cornelius Pass Rd. was disabled, and RLE logging was enabled.

2070 ATC controller. A McCain-NIATT CID was used to communicate between the signal controller and the computer running the VISSIM microsimulation (Figure 4.3).



Figure 4.3: Econolite 2070 ATC Controller (left) and McCain CID (right)

4.4 DETECTION STRATEGIES

Three detection strategies for triggering RLE were considered.

- **Downstream Detection (DD).** This detection strategy (which is currently in place at the intersection) operates RLE based on a single loop detector (per lane) located downstream from the stop bar. If the downstream detector is active during the second half of the yellow indication or a normal all-red phase, then a RLE will be triggered.
- **Simple Upstream Detection (SUD).** This detection strategy operates RLE based on a single loop detector (per lane) located upstream from the stop line. If the upstream detector is active during the second half of the yellow change indication or normal all-red phase, then a RLE will be triggered.
- **Smart Upstream Speed-Conditional Detection (SUSCD).** This detection strategy operates RLE based on a pair of loop detectors (per lane) located upstream from the stop line. Using programmable logic in NWS Voyage™, the two loops are used to differentiate vehicles at higher vs. lower speeds. If a higher speed vehicle is detected during the second half of the yellow change indication or normal all-red phase, then a RLE will be triggered.

4.5 SMART UPSTREAM SPEED-CONDITIONAL DETECTION SETUP

The SUSCD setup was accomplished in NWS Voyage™ programmable logic (Appendix F) by the process shown in Figure 4.4. A timer starts counting down when a vehicle first actuates the leading detector. If the lagging detector is actuated before the timer reaches zero, then a call is placed to the RLE detector. By adjusting the value that the timer counts down from, the minimum speed needed to trigger the RLE can be increased or decreased.

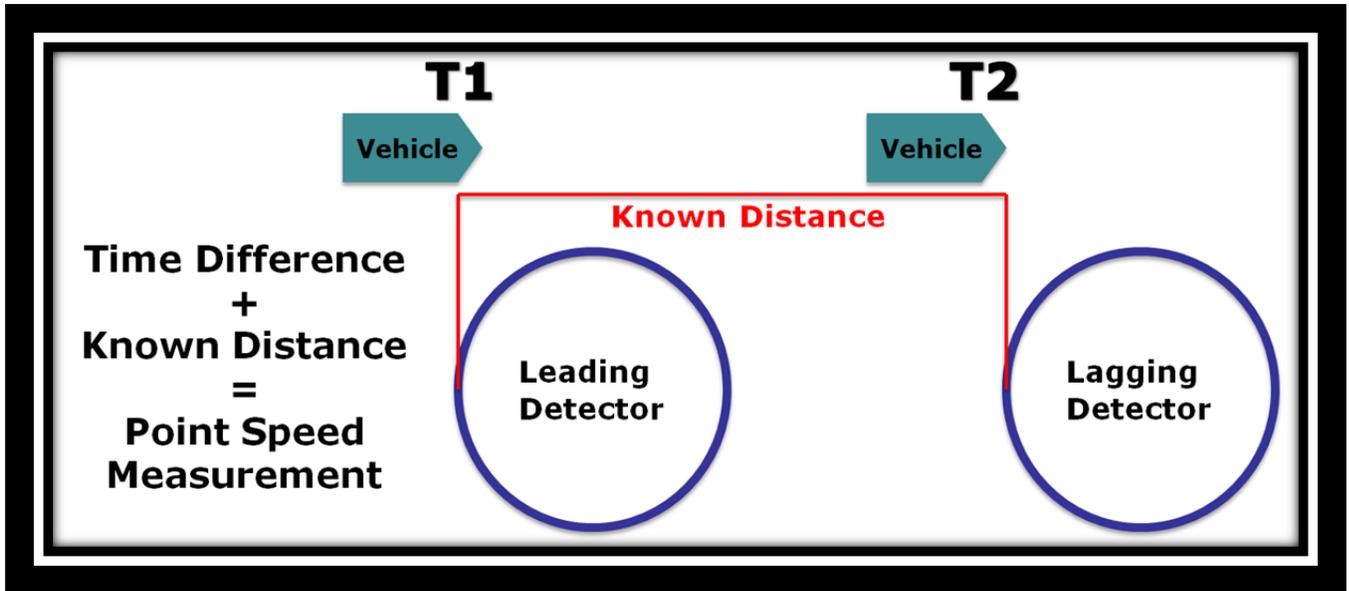


Figure 4.4: SUSCD concept

A short distance (25 ft) between two detectors (from leading edge to leading edge) was used, to reduce the possibility that two vehicles would occupy the speed-conditional detection (SCD) setup at the same time. Table 4.1 displays the calculated distances traveled at typical speeds and timer durations. Due to rounding and truncation of values in the controller software, these values should be viewed as approximations. Field observations and adjustments of the timer values should be made when this system is implemented in the field.

Table 4.1: Theoretical distances traveled at various speeds and timer durations

Speed (mph)	Duration of Timer (s)		
	0.5	0.4	0.3
35	25.7 ft	20.5 ft	15.4 ft
40	29.3 ft	23.5 ft	17.6 ft
45	33.0 ft	26.4 ft	19.8 ft
50	36.7 ft	29.3 ft	22.0 ft
55	40.3 ft	32.3 ft	24.2 ft
60	44.0 ft	35.2 ft	26.4 ft

4.6 HARDWARE-IN-THE-LOOP SIMULATION DATA COLLECTION

Data for the HIL microsimulations were collected from three main sources. **Direct VISSIM outputs** included the position data for each vehicle (.FZP file) and the chronologically sorted signal changes (.LSA file) from the microsimulation. These data sets were provided at a resolution of 0.1 s. The **VISSIM node** provided data for the total and stop delays, collected from a single node surrounding the simulated intersection. **Signal controller logs** (NWS VoyageTM RLE logs) recorded the beginning and end of each RLE event with a resolution of 1 s.

4.7 EXPERIMENTAL SCENARIOS

Six scenarios were initially evaluated, with 30 runs of an 80-min simulation (15-min seeding period, 60-min evaluation, 5-min cooldown). These initial scenarios used the *continuous check* model for driver behavior at yellow lights (see Appendix G for a description of the yellow light driver behavior models in VISSIM). As a limited number of RLE events were observed in Scenarios 2, 5, and 6, additional scenarios were run to increase the number of RLE events. Table 4.2 summarizes the characteristics of all nine scenarios.

Table 4.2: Experimental scenarios

Scenario	Detection Strategy	Detector Position	Description
1	None	N/A	RLE not active, continuous check model for YLB
2	DD	5 ft downstream	Max RLE duration = 2.0 s, continuous check model for YLB
3	SUD	215 ft upstream	Max RLE duration = 2.0 s, continuous check model for YLB
4	SUD	475 ft upstream	Max RLE duration = 5.0 s, continuous check model for YLB
5	SUSCD	215 ft upstream	Max RLE duration = 2.0 s, continuous check model for YLB, SCD timer duration = 0.3 s
6	SUSCD	475 ft upstream	Max RLE duration = 5.0 s, continuous check model for YLB, SCD timer duration = 0.3 s
7	DD	5 ft downstream	Signal head compliance rate = 90%, single decision for YLB
8	SUSCD	215 ft upstream	Signal head compliance rate = 90%, single decision for YLB, SCD timer duration = 0.4 s
9	SUSCD	475 ft upstream	Signal head compliance rate = 90%, single decision for YLB, SCD timer duration = 0.4 s

*YLB: yellow-light behavior; SCD: speed-conditional detection; SUD demonstrated significant numbers of incorrect RLE, and as such, were not considered further.

4.8 DATA REDUCTION

Useful outputs were collected from the .LSA files, .FZP files, and RLE logs. RLE data from the .LSA files (frequency of 0.1 s) matched data from the lower resolution VoyageTM RLE logs (frequency of 1.0 s). Therefore, the .LSA files were used for extension events because they provided more precise results.

The data sets were sufficiently large that the development of an efficient data reduction process was critical. Three data analysis applications, EXCEL, Access, and R, were evaluated to facilitate data reduction. R produced acceptable results in the shortest time and was selected as the analysis platform. Hundreds of lines of codes were created to automate the data reduction procedure as much as possible for each simulation trial. For each simulation run of an

experimental scenario, the files were imported into R simultaneously, and necessary calculations were facilitated with previously coded scripts.

Three sets of plots were developed for each of the nine scenarios. Figure 4.5 illustrates an example of a plot in which the RLE and signal change data were overlaid. This type of plot was primarily used to find cycles where a RLE occurred.

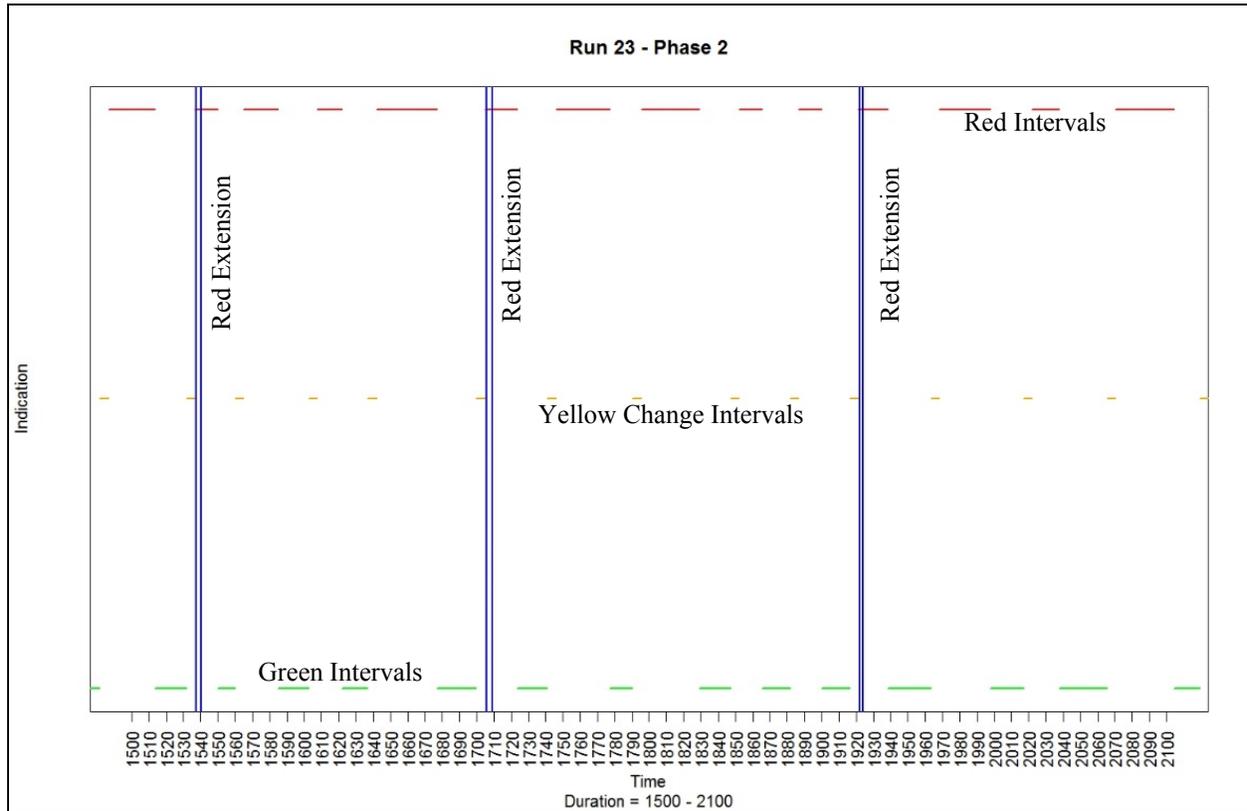


Figure 4.5: Example of a plot of signal changes overlaid with RLE data

TSDs were developed for each experimental scenario. In these diagrams, trajectories of the front and rear bumpers of vehicles were plotted against the signal changes, intersection geometry, and detector locations for cycles that included a RLE (Figure 4.6).

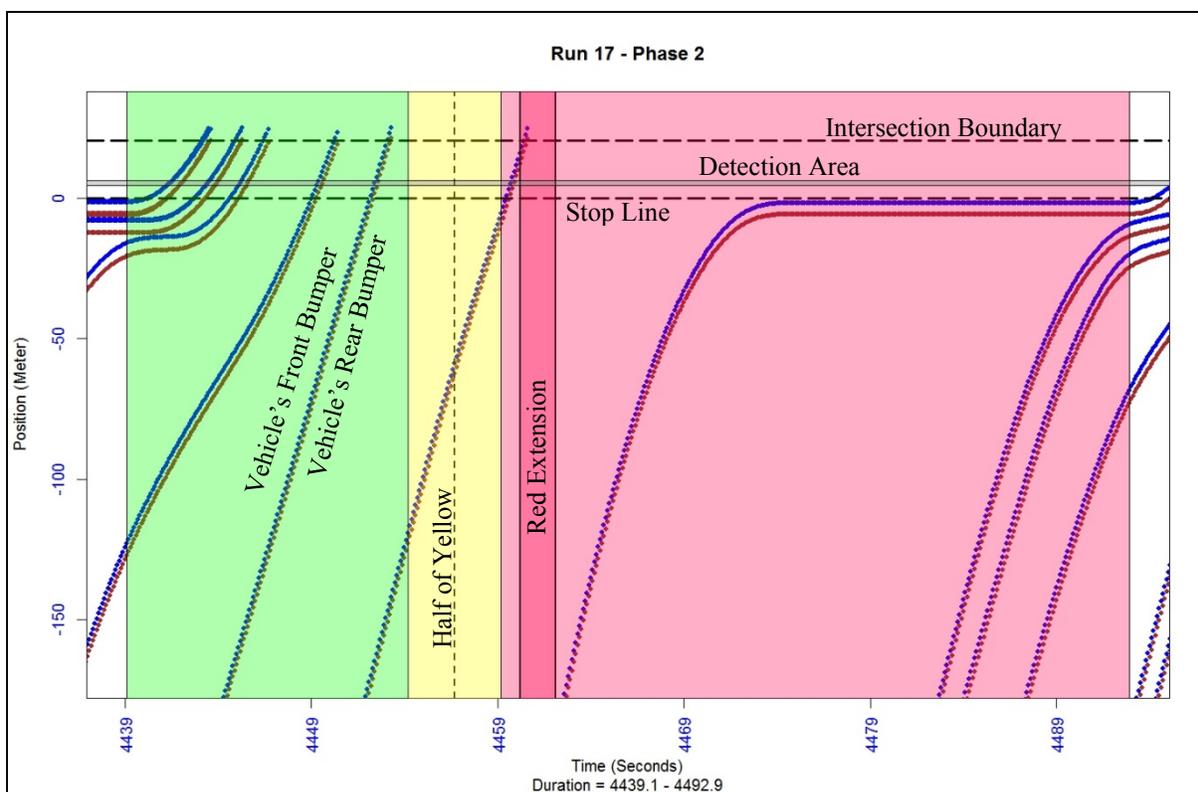


Figure 4.6: Example of a TSD showing a RLR event that triggered a RLE

TSDs were also used to identify undetected Vehicles with High Risk of Collision (VHRC) that did not trigger RLE (Figure 4.7).

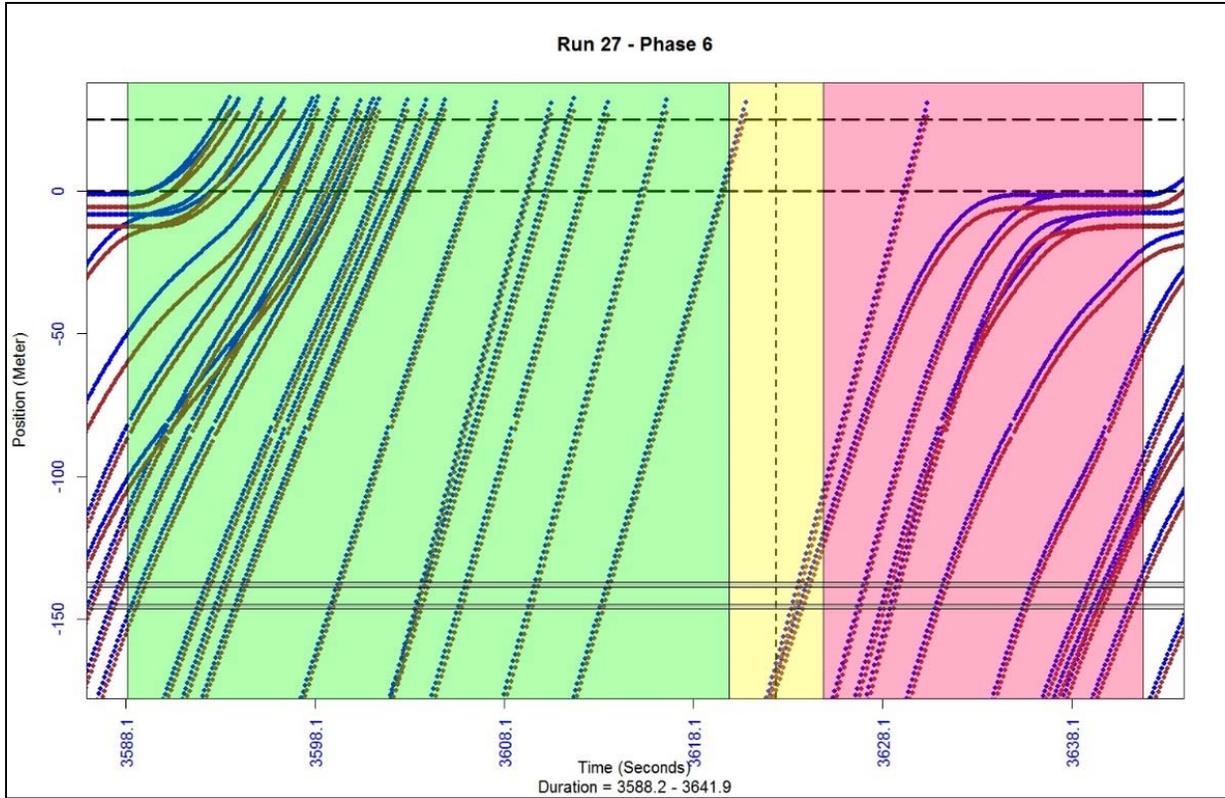


Figure 4.7: Example of a TSD showing a RLR event that did not trigger a RLE

4.9 RESULTS

HIL experimental scenarios 7, 8 and 9 were analyzed in terms of accuracy, effectiveness and operations. Evaluation of system **accuracy** was based on the number of correct calls and the number of detected VHRC. VHRCs are defined by one of two conditions: (1) if a vehicle enters intersection late in the yellow change interval it can be considered a late runner (Figure 4.8a) and (2) as a RLR (Figure 4.8b).

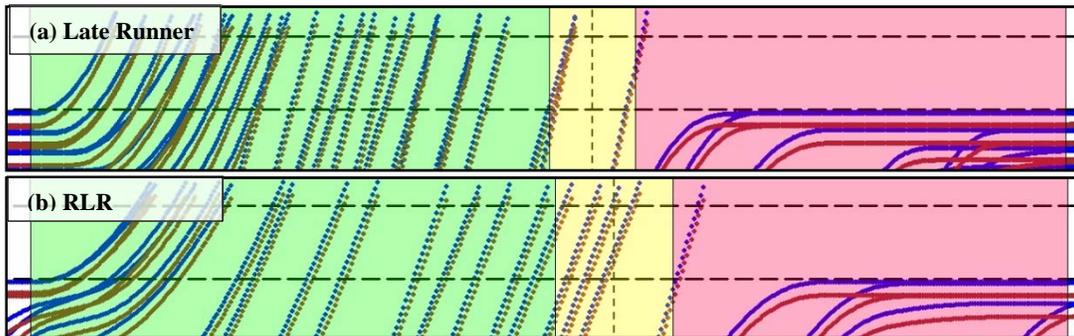


Figure 4.8: Example of a VHRC: Late Runner (a) and RLR (b)

Correct calls are explicitly defined as RLE which benefit the VHRC that triggered the RLE as it safely clears the intersection (Figure 4.9a). If an extension is triggered by a non-VHRC (Figure 4.9b), or if the RLE is triggered by a leading vehicle which does not benefit from the RLE, but a

following VHRC then uses that RLE to safely clear intersection (Figure 4.9c), it is also considered an incorrect call.

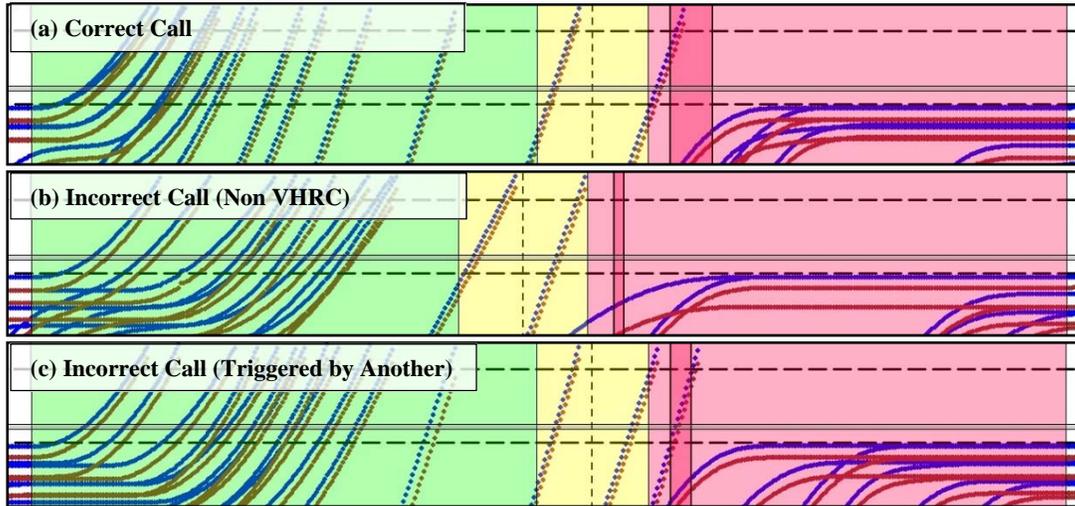


Figure 4.9: Example of a correct call (a) and incorrect call (b) and (c)

Detection accuracy is also limited to those VHRC which could have triggered the RLE in the system. If a VHRC occupies the detection area during the second half of the yellow change or All-Red interval and it triggers a RLE, it is a detected VHRC (Figure 4.10a). However, if a VHRC occupies the detection area during the aforementioned period but a RLE is not triggered, it is an undetected VHRC (Figure 4.10b). If a VHRC does not occupy the detection area during the aforementioned period, the RLE system, by design, is not capable of identifying that vehicle. Those vehicles were disregarded for the analysis of accuracy (Figure 4.10c).

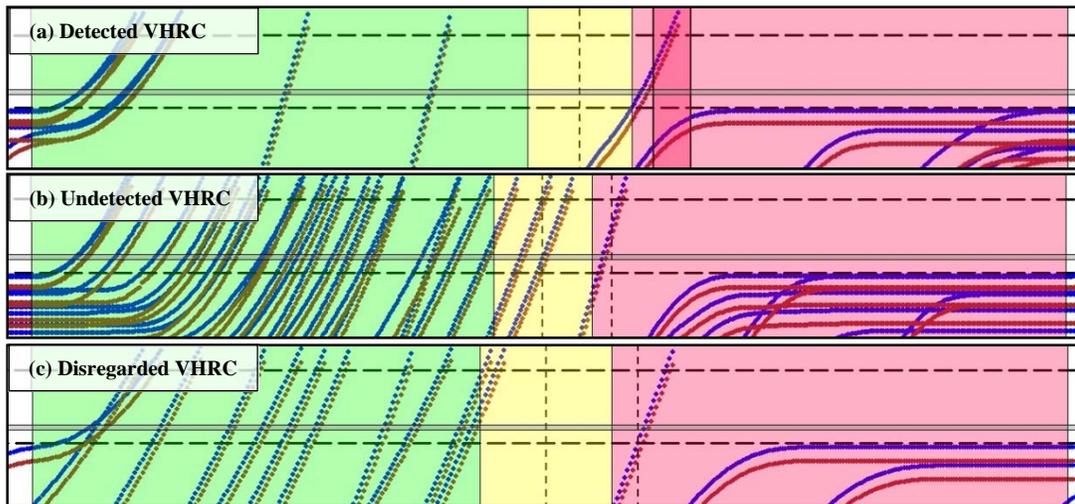


Figure 4.10: Example of detected (a), undetected (b) and disregarded (c) VHRC

Table 4.3 defines RLE system accuracy measurements in terms of the occurrence of a RLE and a VHRC.

Table 4.3: Accuracy measurements

		Extension	
		Yes	No
VHRC	Yes	A VHRC is detected and a RLE is triggered	A VHRC is not detected and a RLE is not triggered
	No	A RLE is triggered by a non VHRC	A RLE is not triggered and there is no VHRC.

RLE system **efficiency** was also considered. The purpose of a RLE system is to provide additional time for a VHRC to safely clear an intersection. The position and speed of a VHRC at the onset of the red clearance interval contributes to likelihood of that vehicle safely crossing the intersection. If a VHRC is upstream of the stop line at the onset of the red clearance interval, then the RLE is assisting a RLR completely clear the intersection. Conversely, if a VHRC is downstream of the stop line at the onset of the red change interval, then the RLE is likely helping a late runner, which has already passed greater part of intersection by the help of All-Red interval. Such that, based on the position of VHRC on the onset of red, three levels are defined to measure the efficiency of extension (Figure 4.11).

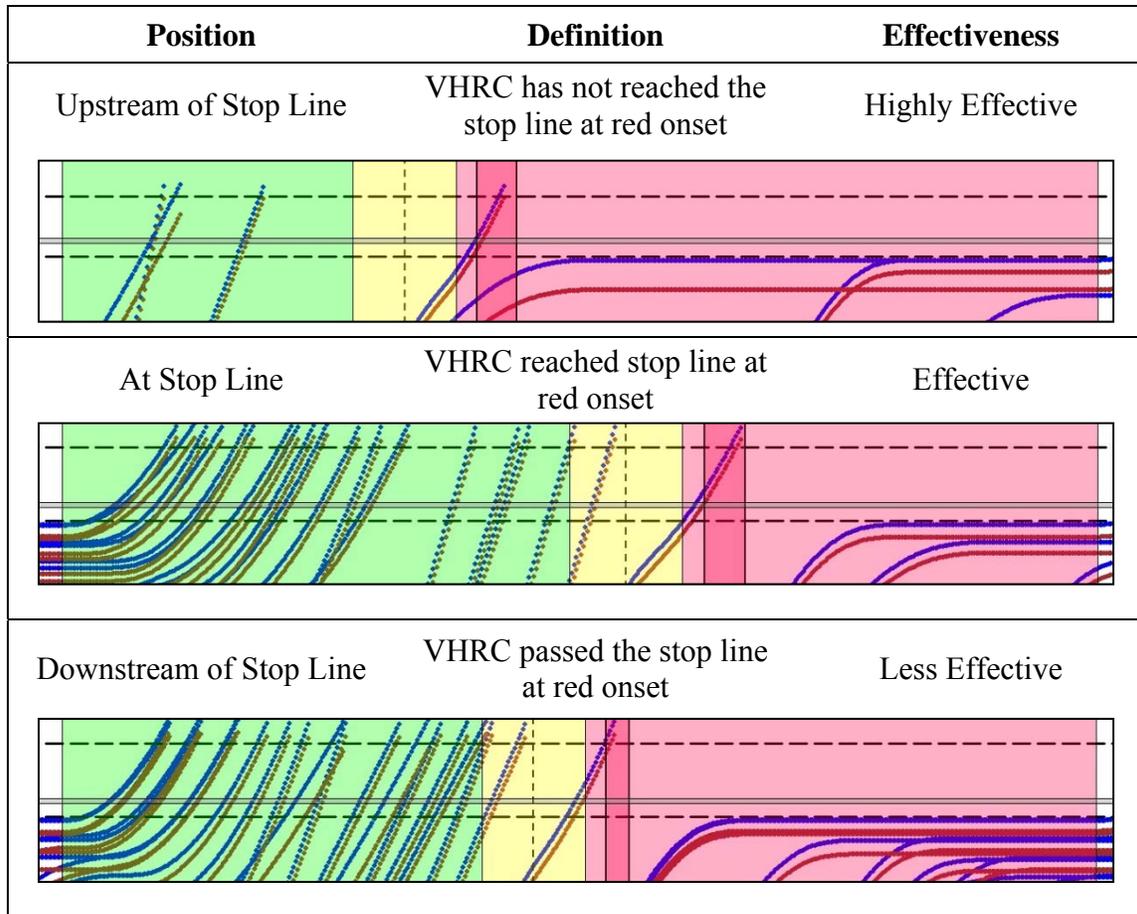


Figure 4.11: Efficiency measurement

The impact of RLE systems on signal **operations** were evaluated, the implications on delay (reported as delay per vehicle and stop delay) and extension duration were also considered.

4.9.1 Downstream Detection (Scenario 7)

As described in Section 4.4, the DD strategy (which is currently in place at the intersection) initiates a RLE based on a single loop-detector (per lane) downstream of the stop line. If a vehicle occupies the detection area during the second half of the yellow change indication or a normal all-red phase then a RLE is triggered. Figure 4.12 presents accuracy measurements for the DD system over 30 simulation runs.

Phase 2				Phase 6				Total			
		Extension				Extension				Extension	
		Yes	No			Yes	No			Yes	No
VHRC	Yes	20	7	VHRC	Yes	110	57	VHRC	Yes	130	64
	No	42	2236		No	165	1992		No	207	4228

Figure 4.12: Accuracy measurements for DD

The DD system correctly detected 130 of 194 VHRC (67.0% success rate) and correctly triggered 130 of 337 RLE (38.6% success rate). In this context, a correct call is explicitly defined as RLE which benefits the VHRC that triggered the RLE as it safely clears the intersection.

Using three levels of effectiveness (Figure 4.11), the efficiency of correctly triggered RLE by the DD system was analyzed (Table 4.4). The DD system was able to create highly effective and effective extensions in approximately 33% of all cases.

Table 4.4: Efficiency measurements for DD

RLE Effectiveness	Number	Percentage
Highly Effective <i>(Vehicle Prior to Stop Line at Onset of Red)</i>	23	17.7%
Effective <i>(Vehicle at Stop Line at Onset of Red)</i>	20	15.4%
Less Effective <i>(Vehicle Beyond Stop Line at Onset of Red)</i>	87	66.9%

Table 4.5 summarizes descriptive statistics for delay (measured as delay per vehicle and stop delay) and extension duration for downstream detection system, over 30 simulation runs.

Table 4.5: Descriptive statistics for delay and extension duration with the DD system

Statistic	Vehicle Delay	Stop Delay	Extension Duration
Min	11.94	5.35	0.10
Mean	13.08	5.35	1.25
Median	13.12	5.34	1.40
Max	13.93	5.93	1.90
SD	0.42	0.26	0.59

4.9.2 Smart Upstream Speed-Conditional Detection Setup at 215 Feet (Scenario 8)

This strategy creates the RLE based on a pair of loop detectors (per lane) positioned 215 ft (76.2 m) upstream of the stop line. In this system, if a higher speed VHRC is detected during the second half of the yellow change indication or a normal all-red phase, then a RLE is triggered. Figure 4.13 presents accuracy measurements for the SUSCD at 215 ft system over 30 simulation runs.

Phase 2				Phase 6				Total			
		Extension				Extension				Extension	
		Yes	No			Yes	No			Yes	No
VHRC	Yes	11	42	VHRC	Yes	26	81	VHRC	Yes	37	123
	No	52	2197		No	113	2044		No	165	4241

Figure 4.13: Accuracy measurements for SUSCD at 215 ft

The SUSCD system at 215 Ft correctly detected 37 out of 160 VHRC (23.1% success rate) and correctly triggered 37 out of 202 RLE (18.3% success).

The efficiency of correct extensions triggered by the SUSCD system at 215 ft (Table 4.6) was evaluated based on the three levels of effectiveness (Figure 4.11). The SUSCD system at 215 ft triggered highly effective and effective extensions for 100% of cases.

Table 4.6: Efficiency measurements for SUSCD at 215 ft

RLE Effectiveness	Number	Percentage
Highly Effective <i>(Vehicle Prior to Stop Line at Onset of Red)</i>	36	97.3%
Effective <i>(Vehicle at Stop Line at Onset of Red)</i>	1	2.7%
Less Effective <i>(Vehicle Beyond Stop Line at Onset of Red)</i>	0	0.0%

Table 4.7 summarizes descriptive statistics for delay (measured as delay per vehicle and stop delay) and extension duration for the SUSCD system at 215 ft, over 30 simulation runs.

Table 4.7: Descriptive statistics for delay and extension duration for the SUSCD at 215 ft

Statistics	Vehicle Delay	Stop Delay	Extension Duration
Min	12.10	4.64	0.10
Mean	13.10	5.37	1.01
Median	13.13	5.40	0.95
Max	13.83	5.85	2.00
SD	0.44	0.26	0.53

4.9.3 Smart Upstream Speed-Conditional Detection Setup at 475 Feet (Scenario 9)

This strategy triggers the RLE based on a pair of loop detectors (per lane) located 475 ft (144.8 m) upstream of the stop line. If a higher speed VHRC is detected during the second half of the yellow change indication or a normal all-red phase, then a RLE is triggered. Figure 4.14 presents accuracy measurements for this system over 30 simulation runs.

Phase 2				Phase 6				Total			
		Extension				Extension				Extension	
		Yes	No			Yes	No			Yes	No
VHRC	Yes	30	37	VHRC	Yes	38	33	VHRC	Yes	68	70
	No	233	1926		No	502	1618		No	735	3544

Figure 4.14: Accuracy measurements for SUSCD at 475 ft

The SUSCD system at 475 ft correctly detected 68 out of 138 VHRC (49.3% success rate) and correctly triggered 68 out of 803 RLE (8.5% success rate).

Using three levels of effectiveness (Figure 4.11), efficiency of correct extensions in SUSCD system at 475 ft is analyzed (Table 4.8). Of the few correct extensions, SUSCD system at 475 ft is able to create highly effective extensions in 100% of cases.

Table 4.8: Efficiency measurements for SUSCD at 475 ft

RLE Effectiveness	Number	Percentage
Highly Effective <i>(Vehicle Prior to Stop Line at Onset of Red)</i>	68	100.0%
Effective <i>(Vehicle at Stop Line at Onset of Red)</i>	0	0.0%
Less Effective <i>(Vehicle Beyond Stop Line at Onset of Red)</i>	0	0.0%

Table 4.9 summarizes descriptive statistics for delay (measured as delay per vehicle and stop delay) and extension duration for the SUSCD at 475 ft, over 30 simulation runs.

Table 4.9: Descriptive statistics for delay and extension duration for SUSCD at 475 ft

Statistics	Vehicle Delay	Stop Delay	Extension Duration
Min	12.92	5.38	1.20
Mean	13.75	5.83	3.34
Median	13.72	5.83	3.40
Max	14.78	6.28	5.00
SD	0.40	0.24	0.97

4.10 DISCUSSION

Figure 4.15 provides a comparison of the accuracy and efficiency measurements and Figure 4.16 provides a comparison of operational measurements in the detection system alternatives. Although there were few variations in the operational performances of the different alternatives, there were major differences in the accuracy and efficiency measurements.

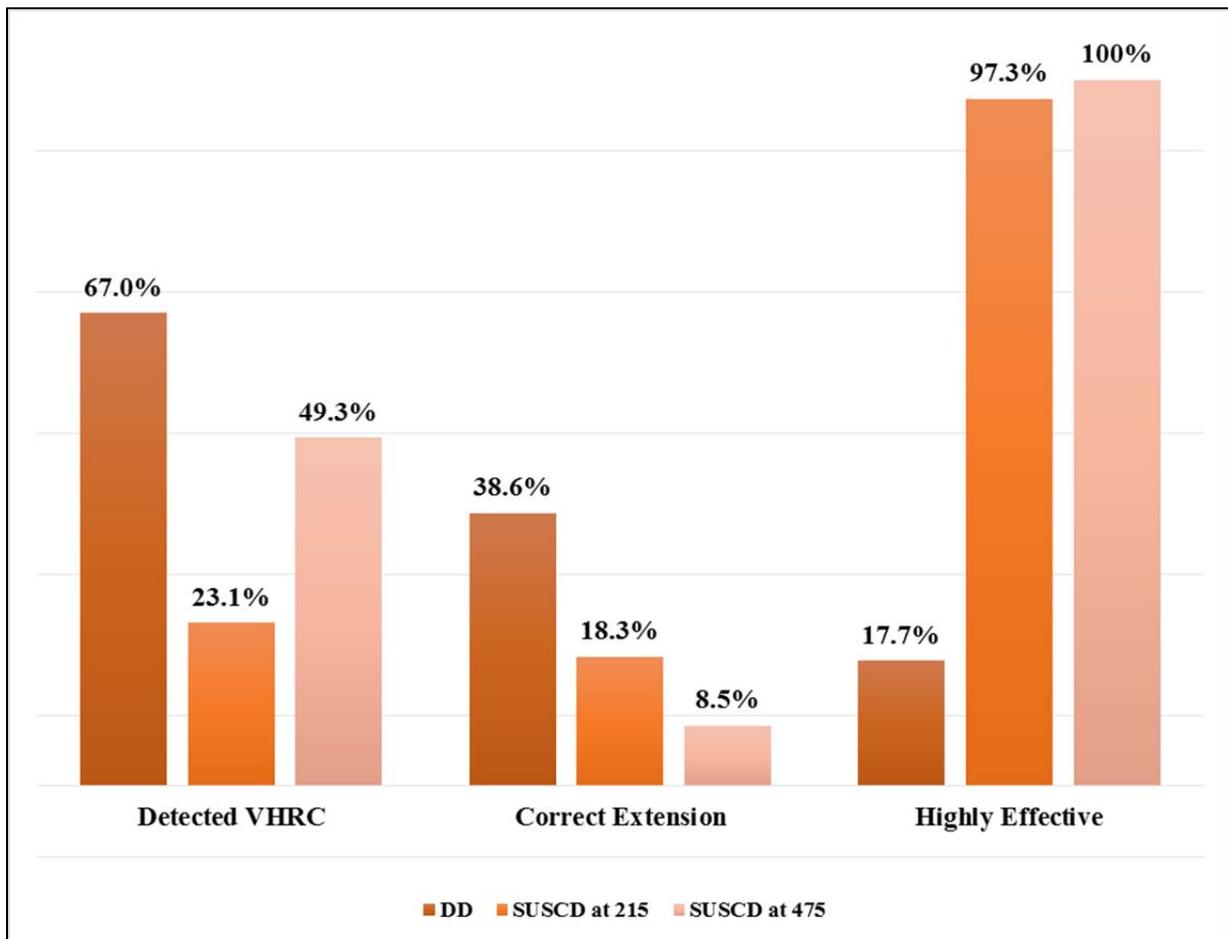


Figure 4.15: Comparison of accuracy and efficiency in detection systems

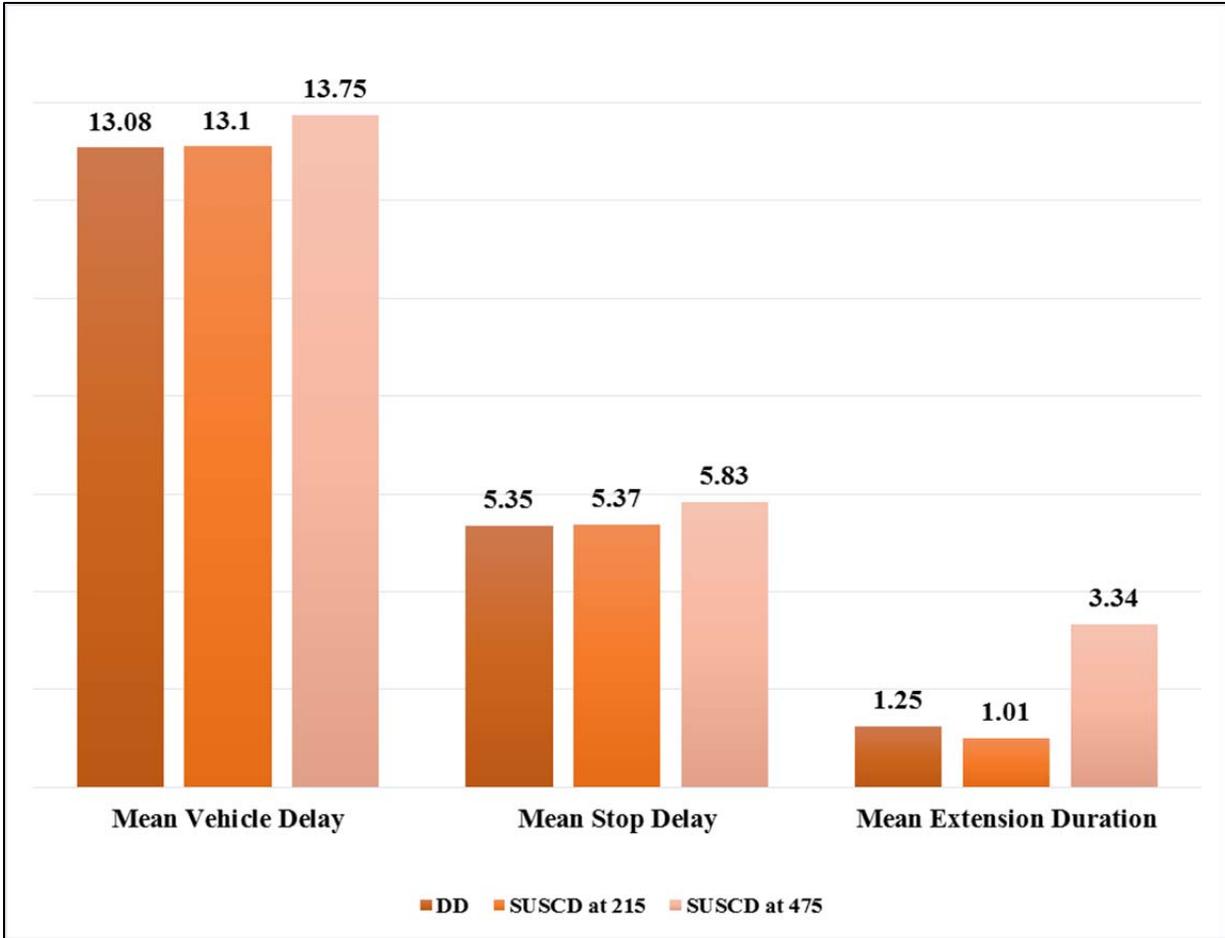


Figure 4.16: Comparison of operational measurements in detection systems

Upon initial inspection, the DD system seems to be more successful at creating extensions and identifying VHRC than the alternatives. The rate of VHRC detection for the DD system was nearly three times that of the SUSCD system at 215 ft and 1.3 times that of the SUSCD system at 475 ft. Moreover, the rate of correct extensions in the DD system was more than double that of the SUSCD system at 215 ft and about 4.5 times that of the SUSCD system at 475 ft. Therefore, the DD system that is currently in place at the intersection outperforms the accuracy of the SUSCD alternatives.

From an efficiency standpoint, SUSCD systems outperformed the DD system. While the rate of highly effective correct extensions for the DD system is approximately 18%, this rate is almost 100% for both SUSCD systems. This finding necessitated a closer examination of the trigger extensions for each RLE system.

Figure 4.17 shows a typical case of RLE produced by the DD system. By definition, the DD system calls extensions when a vehicle first occupies the detector downstream of the stop line during the second half of the yellow or a normal all-red. In other words, a vehicle that triggers an extension could be halfway or further through the intersection at the end of the typical 1-s all-red period. From a safety standpoint, drivers in opposing lanes are able to see a vehicle in front of themselves at the onset of their green.

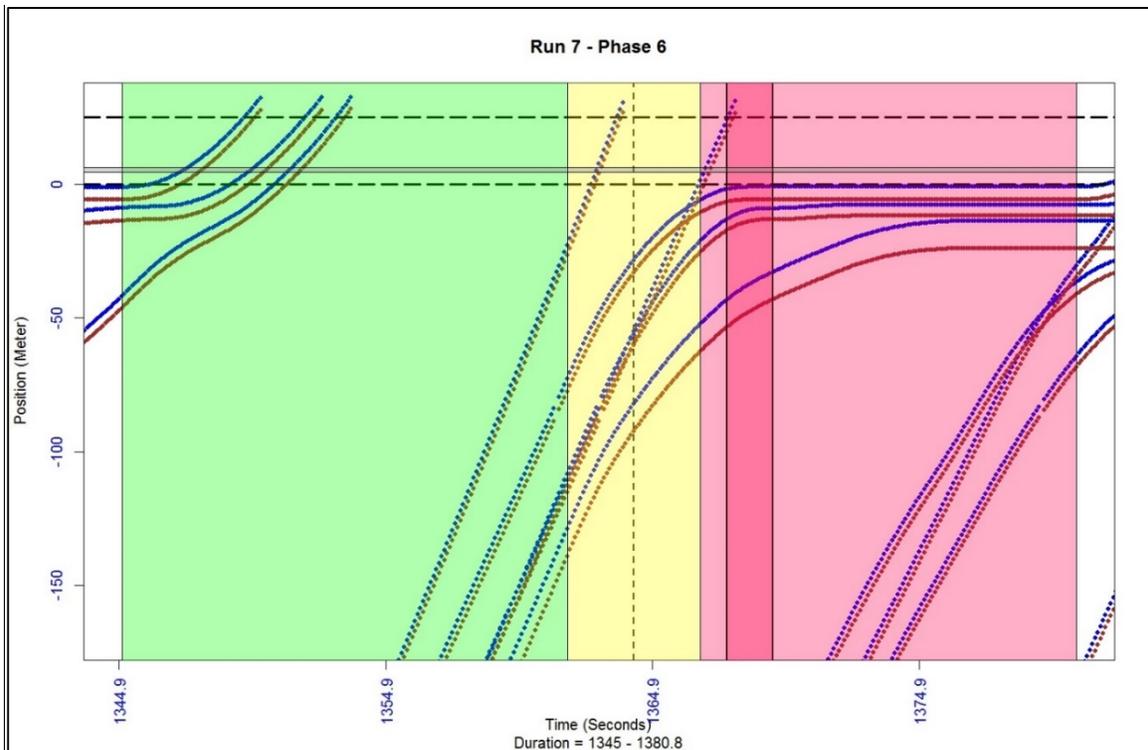


Figure 4.17: Example of a detected VHRC in the DD system

Figure 4.18 and Figure 4.19 show a common RLE event for the SUSCD system at 215 ft and 475 ft, respectively. By definition, in these systems, VHRC are detected based on their instantaneous velocity at 215 ft and 475 ft upstream of the stop line. The TSD for SUSCD at 215 ft demonstrates a VHRC that passes the stop line at the end of the 1-s all-red period and the TSD for SUSCD at 475 ft exhibits a VHRC that crosses the stop line long after the onset of red. In both cases, with the help of a correct, complete, and precise extension, the VHRC clears the intersection before any conflicting movement can occur.

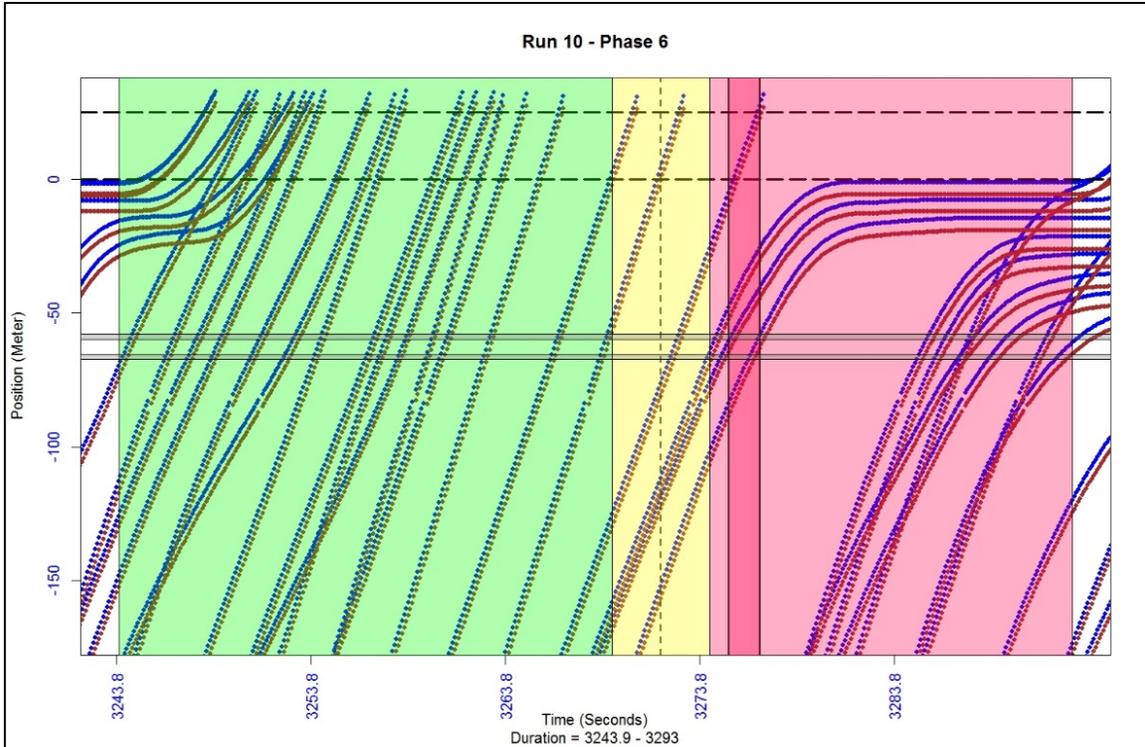


Figure 4.18: Example of a detected VHRC in the SUSCD at 215 ft

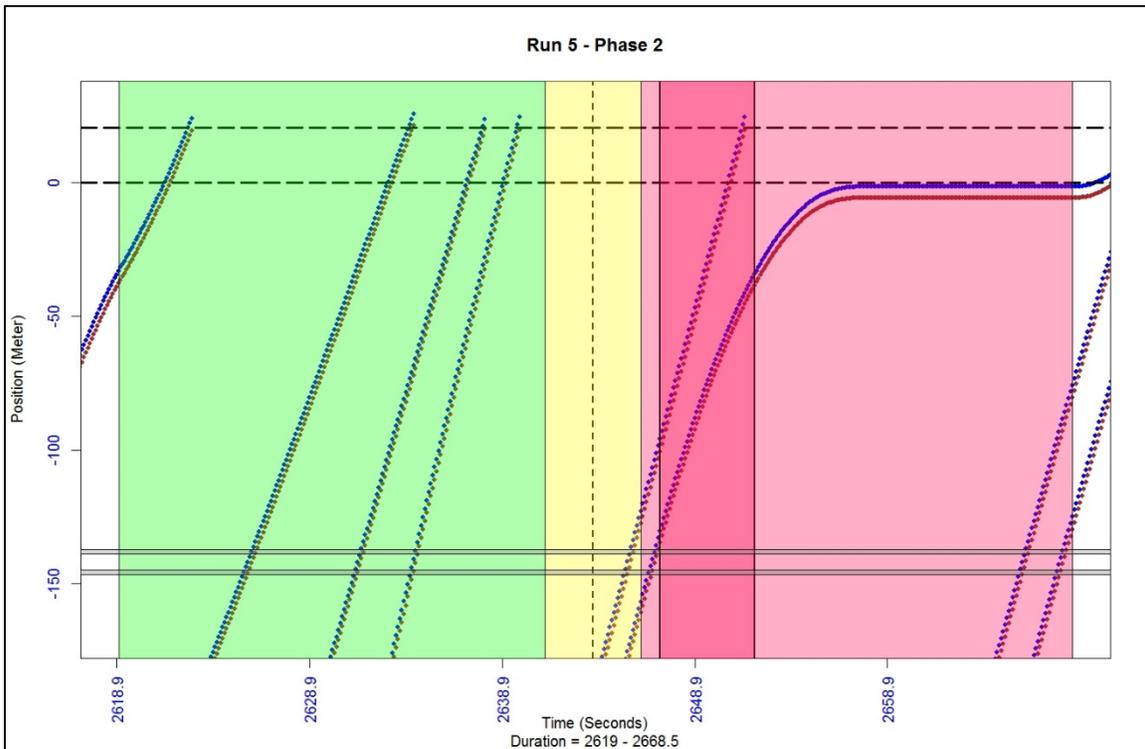


Figure 4.19: Example of a detected VHRC in the SUSCD at 475 ft

In summary, to judge the appropriateness of each alternative design, both the quantitative and qualitative performances should be considered. A comparison of demonstrated TSDs confirms

that, although SUSCD systems are less successful in detecting VHRC and making correct extensions, they are potentially strong in creating highly effective extensions. For the SUSCD systems, VHRC are detected by a single spot speed measurement made 215 ft or 475 ft before the intersection. Although speed is a crucial determinant in identifying VHRC, drivers' decisions to stop or proceed cannot be predicted by using speed alone. SUSCD systems trigger RLE events without observation of RLR. This prognostic trait in SUSCD systems justify the lower accuracy. SUSCD systems are also different, when compared to each other. While the SUSCD at 215 ft is more successful at triggering correct extensions (18.3% compared to 8.5%), the SUSCD at 475 ft outperforms in the detection of VHRC (49.3% compared to 23.1%). Triggering 803 extensions, the SUSCD at 475 ft sacrifices some efficiency. The SUSCD at 215 ft with an acceptable detection rate and fair rate of correct extensions, is situated between the performance of the DD system and SUSCD at 475. Although not comparable between the DD and SUSCD systems, extensions in the SUSCD systems are identified in a "smarter" way than those in the DD system.

5.0 CONCLUSIONS

5.1 SUMMARY OF PROJECT TASKS

Three major tasks were completed and are described in this report: a literature review of RLE systems (Chapter 2), a large-scale field study of driver behavior in response to yellow change and red clearance indications (Chapter 3), and a HIL simulation of alternative RLE system designs (Chapter 4). These major tasks resulted in the following outcomes:

- Field data of driver behavior were collected at five signalized intersections in Oregon. In total, 252 h of video data were collected, and 149 h of data were transcribed. This process resulted in the observation of 6,155 vehicles responding to yellow change indications on the major approach and 7,456 vehicle responding to green indications on a minor approach to the intersections. Exactly 36 RLR vehicles were observed on the major approach to the intersections. **The highest frequency of RLR vehicles (24) was observed at the intersection of US30 and Cornelius Pass Rd. Therefore, this location was selected for HIL simulation.**
- A novel HIL simulation was developed for the US30–Cornelius Pass intersection. A NWS Voyage™ BIN file containing the existing signal timing at this intersection was loaded and run on an Econolite 2070 ATC controller. A McCain-NIATT CID was used to communicate between the signal controller and the computer running the VISSIM microsimulation. **The HIL simulation of the US30–Cornelius Pass intersection was used as a test bed to evaluate alternative RLE system designs.**
- Within the HIL simulation environment, **logic for the SUSCD concept was developed** and used to predict RLR vehicles on the approach to a signalized intersection at 215 and 475 ft. This logic (see Appendix F) could be implemented in the field at a traffic signal with a 2070 controller and the NWS Voyage™ operating software.
- **A semi-automated procedure was coded in R to visualize VISSIM outputs in TSDs**, in which the trajectories of the front and rear bumpers of vehicles were plotted against the signal changes, intersection geometry, and detector locations, specifically for cycles that included a RLE.

5.2 EVALUATION OF SYSTEM ALTERNATIVES

Three RLE system designs were evaluated: the DD (ODOT’s existing RLE system), the SUSCD at 215 ft, and the SUSCD at 475 ft. Rates of accuracy, effectiveness and operational performance were compiled for each system, based on the HIL simulation.

The DD system successfully detected 67% of VHRC (i.e., vehicles that occupied the detector downstream of the stop line during the second half of the yellow and the all-red). Correct RLE

(i.e., the vehicle that triggered the extension benefited from the extension) occurred 38.6% of the time that an event was triggered. The average vehicle delay under the DD system operation was 13.08 s. The SUSCD system at 215 ft successfully detected 23.1% of VHRC. Only 18.3% of the triggered extensions were correct, and the average vehicle delay was 13.1 s. The SUSCD system at 475 ft successfully detected 49.3% of RLR vehicles. Correct RLE events represented 8.5% of all triggered events, and the average vehicle delay was 13.75 s.

From an efficiency standpoint, the SUSCD systems outperformed the DD system. While the rate of highly effective correct extensions for the DD system is approximately 18%, this rate is almost 100% for both SUSCD systems. This finding necessitated a closer examination of the trigger extensions for each RLE system.

Cursory examination of quantitative results leads to three general observations. (1) The DD alternative provided higher accuracy than the SUSCD system at either 215 or 475 ft. (2) The SUSCD system at either 215 or 475 ft provide higher efficiency than the DD system. (3) The average vehicle delay was relatively small and consistent across all three RLE system alternatives. Detection rates were high for the DD alternative because no prediction was made; the vehicle was already in the intersection when it was detected. Although the SUSCD system at 215 or 475 ft had a higher likelihood of false prediction of a RLR vehicle compared to the DD system, the SUSCD systems also introduced the potential for providing more robust RLE. An examination of the TSDs showed improved relationships between the vehicle trajectories, RLE events, and conflicting movements when the SUSCD systems were used.

5.2.1 Locations for Implementation

To maximize the crash reduction benefit of RLE systems, they should be installed at intersections with high rates of RLR or high rates of RLR related crashes. The costs of RLR systems are reduced at intersections already operating a 2070 Signal controller with NWS Voyage™ firmware, as these are required system elements.

5.2.2 Cost-Benefit of Implementation

The value proposition of a RLE system is high. As previously described in section 2.5.6.2, the cost of implementing a RLE system on the two major approaches of a single four-way signalized intersection is approximately \$8,000, while the cost of a single crash can range from \$69,300 for a moderate (Injury B) to minor (Injury C) severity crash to \$2,330,000 for a Fatal or severe (Injury A) crash (*ODOT 2014b*).

5.3 LIMITATIONS

Although HIL traffic simulation provides many meaningful advantages, it also requires expensive hardware and software interfaces, as well as uniquely trained staff. Perhaps the most important limitation is the requirement that the simulations take place in real-time. The process of collecting 30 individual runs, an industry standard, for each alternative scenario is very time intensive. Furthermore, 30 new runs must be produced each time the system design or a setting is modified to improve the performance measurements. Finally, the volume and structure of the

produced data require substantial programming expertise and staff time to reduce the output data into usable statistics and visualizations.

5.4 RECOMMENDATIONS

Based on the results of this study, the following recommendations should be considered:

- The duration of the yellow change and red clearance intervals have a considerable influence on driver behavior. As such ODOT should consider adopting the kinematic equations recommended in NCHRP Report 731, “Guideline for Timing Yellow and All-Red Intervals at Signalized Intersections”.
- Currently ODOT uses the posted speed limit as the approach velocity for the kinematic timing equations. ODOT should consider using an operational speed as recommended in NCHRP Report 731, which could provide more precise estimations for yellow change and red clearance durations.
- The overrepresentation of semi-trucks and light trucks in RLR events observed in the field indicates that more attention should be paid to detection strategies and timing durations which consider vehicle classification.
- The HIL simulation environment is a robust tool for testing and evaluating signal treatment alternatives and should be considered as a viable resource for ODOT.
- Adding upstream detection can enhance the efficiency of RLE systems which could in turn increase the safety of signalized intersections in Oregon.

5.5 OPPORTUNITIES FOR FUTURE WORK

As with many complex transportation problems, there exist opportunities to continue to advance the state of the practice. The development of red clearance extension systems is no different. Two recommendations can be made for future work:

- **HIL Simulations of Additional Red Clearance Extension System Designs.** The calibrated HIL model and the data analysis code developed for this project could be leveraged to test alternative SUSCD locations and to refine the logic of the RLR prediction, thereby improving the overall performance of the system.
- **Field Evaluation of Alternative Vehicle Detection Strategies.** The in-pavement loop is still widely considered to be the most accurate and commonly implemented vehicle detection strategy. However, a single point sensor has a limitation in the type of traffic data that can be extracted (presence and instantaneous speed) to support a RLR prediction algorithm. Conversely, a wide area detection system, such as that produced by a RADAR sensor, could be used to evaluate the time-to-stop-line, acceleration, or deceleration data for each approaching vehicle. These additional continuous data streams could dramatically improve the performance of a red clearance extension system.

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APPENDIX A

GLOSSARY

APPENDIX A: GLOSSARY

This appendix contains the definitions of abbreviations, acronyms, and common terms.

Table A.1: Definitions of abbreviations and acronyms

Acronym/Abbreviation	Definition
RLR	Red-light running
FHWA	Federal Highway Administration
ODOT	Oregon Department of Transportation
<i>MUTCD</i>	<i>Manual on Uniform Traffic Control Devices</i>
ITE	Institute of Transportation Engineers
NWS	North West Signal
TTC	Time to conflict
HIL	Hardware-in-the-loop
ANSI	American National Standards Institute
RLE	Red-light extension
DD	Downstream detection
SUD	Simple upstream detection
SUSCD	Smart upstream speed-conditional detection

Table A.2. Definitions of common terminology in the report

Term	Definition
Red-light running (RLR)	Vehicle enters the intersection at any point during the red indication.
Red clearance extension	Red clearance (all-red) interval is extended for some duration of time to reduce the potential for collision between a RLR vehicle. Also known as red-light extension (RLE).
Permissive yellow	Driver can enter the intersection legally at any time during the yellow interval.
Restrictive yellow	Driver cannot enter the intersection legally at any time during the yellow interval.
Probability of stopping	Likelihood that a driver will stop at the stop line in response to the onset of the yellow indication.
Yellow change interval	Warning period indicating that the green, flashing yellow, or flashing red indication has ended, and the red indication will begin.
Red clearance interval	Period of time after a yellow change interval, indicating the end of a phase and allowing additional time before the beginning of conflicting traffic.
Time-space diagram	Chart of the location of a signalized intersection along the vertical axis and signal timing along the horizontal axis.
Perception-reaction time (PRT)	Interval between obstacle appearance and driver response initiation.
Dilemma zone	A condition that occurs when yellow change and red clearance times are too short for a driver to stop or clear the intersection before the beginning of a conflicting phase. Also known as a Type I dilemma zone.
Conflict zone	Region in an intersection where two conflicting streams of traffic will produce a collision if occupied simultaneously.
Hardware-in-the-loop (HIL)	Use of a physical signal controller to operate simulated signals, and use of actuations from the simulated detectors to create inputs for the physical signal controller.
Time to conflict (TTC)	Expected time for two vehicles to collide if they remain at their present speed and on the same trajectory.
Pretimed signal	Mode of operation where every phase is on recall every cycle, regardless of changes in traffic conditions.
Actuated signal	Phase time based on either semi-actuated or fully actuated detection.
Cycle length	Duration of a complete sequence of phases in the absence of priority calls.
Downstream detection (DD)	Detection strategy operating RLE based on a single loop detector (per lane) located downstream from the stop bar. If the

Term	Definition
	downstream detector is active during the second half of the yellow indication or normal all-red phase, then a RLE will be triggered.
Simple upstream detection (SUP)	Detection strategy operating RLE based on a single loop detector (per lane) located upstream from the stop bar. If the upstream detector is active during the second half of the yellow indication or normal all-red phase, then a RLE will be triggered.
Smart upstream speed-conditional detection (SUSCD)	Detection strategy operating RLE based on a pair of loop detectors (per lane) located upstream from the stop bar. Using programmable logic in NWS Voyage TM , the two loops are used to differentiate higher from lower speed vehicles. If a higher speed vehicle is detected during the second half of the yellow indication or normal all-red phase, then a RLE will be triggered.

APPENDIX B

SITE INFORMATION

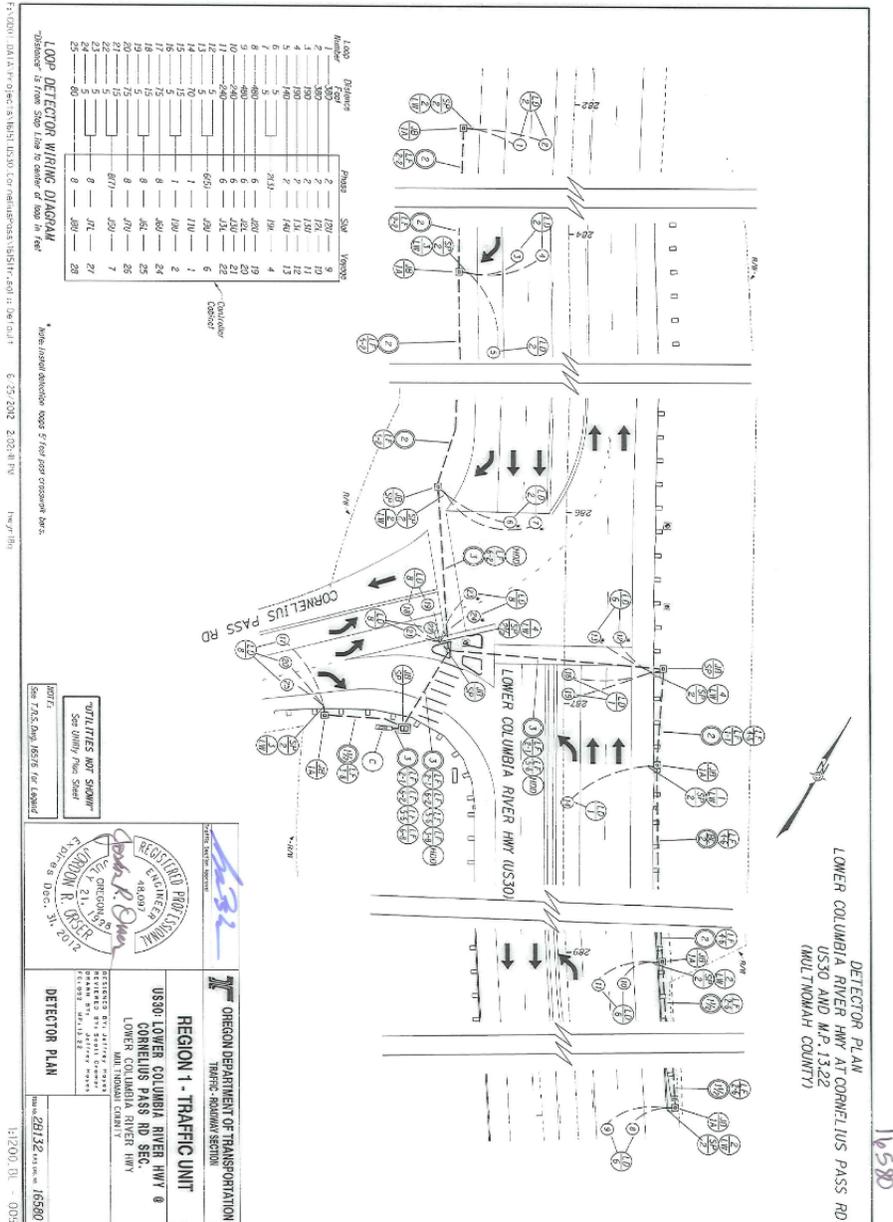
APPENDIX B: SITE INFORMATION

Site	City	Site	Notes	Red Light Cameras	Angle Crash %	Major Approach Speed Limit	Minor Approach Speed Limit	Red Extension Feature Active?	Red Extension Logging	Pretimed, Actuated, or Other?	Detection Method (loops, video, radar etc)	Traffic Controller	Firmware #?	Cycle Length	Yellow Interval Duration	Red Interval Duration	Area Type (Urban, Suburban, Rural)	Major Approach	Intersection Clearing Width Major Approach*	Intersection Clearing Width Minor Approach*	Proximity to Upstream Signal
3	Corvallis	OR-99W (91) - Circle Blvd	Higher than expected disregard of signal Railroad tracks near intersection running NS	N	36	50	25	N	N	Actuated	Loops	2070	-	105	4 sec minor, 5 sec major	.5 sec minor, 1 sec major	Urban	NB/SB	NB 129ft SB 132ft	WB 108ft EB 113ft	WB 301ft EB 630ft SB 1,727ft
7	Salem	OR-99E (72) - Broadway NE	Higher than expected angle and disregard of signal	N	34	-	-	-	-	-	-	-	-	-	-	-	Urban	EB/WB	SB 129ft NB 126ft	WB 134ft EB 137ft	WB N/A EB 881ft NB N/A SB 481ft
9	Woodburn	OR-99E @ Mt. Hood Avenue	- has red light camera on all approaches - wide variety of traffic - controller is not currently set up for truck priority but, could be added easily - approach and posted speeds on all approaches is the same at 35 MPH	Y	-	35	35	N	N	Actuated Coordinated	Loops	2070	-	-	4 sec minor, 5 sec major	0.5	Suburban				
10	Unincorporated Multnomah County	US30 @ Cornelius Pass Road		N	-	55	45	Y	N	Actuated- Uncoordinated	Loops	2070	-	n/a	varies	varies	Rural				
11	Beaverton	US26 WB @ 185th		N	-	45	off ramp	Y	Y	Actuated- Coordinated	Loops/ Radar	2070	5.3.1.	80.8 5.90	varies	varies	Suburban				

APPENDIX C

DESIGN DRAWING OF US30-CORNELIUS PASS RD

APPENDIX C: DESIGN DRAWING OF US30-CORNELIUS PASS RD.



APPENDIX D

VEHICLE TURNING MOVEMENT VOLUMES

APPENDIX D: VEHICLE TURNING MOVEMENT VOLUMES

Table D.1: Peak Hour Turning Movement Count (TMC) at US30 and Cornelius Pass Rd. in Unincorporated Multnomah County, Oregon on 5/24/2015

Groups Printed – Cars - Trucks

Start Time	None				US 30				Cornelius Pass Road				US 30				Int. Total
	From North				From East				From South				From West				
	Left	Thru	Right	Peds	Left	Thru	Right	Peds	Left	Thru	Right	Peds	Left	Thru	Right	Peds	
3:00 PM	0	0	0	0	11	198	0	0	89	0	15	0	0	114	41	0	468
3:15 PM	0	0	0	0	11	202	0	0	94	0	22	0	0	89	57	0	475
3:30 PM	0	0	0	0	23	271	0	0	125	0	16	0	0	88	46	0	569
3:45 PM	0	0	0	0	39	290	0	0	125	0	23	0	0	89	58	0	624
Total	0	0	0	0	84	961	0	0	433	0	76	0	0	380	202	0	2136
4:00 PM	0	0	0	0	16	280	0	0	127	0	25	0	0	93	57	0	598
4:15 PM	0	0	0	0	19	302	0	0	172	0	22	0	0	111	58	0	684
4:30 PM	0	0	0	0	16	312	0	0	141	0	24	0	0	99	64	0	656
4:45 PM	0	0	0	0	23	313	0	0	163	0	20	0	0	92	53	0	664
Total	0	0	0	0	74	1207	0	0	603	0	91	0	0	395	232	0	2600
5:00 PM	0	0	0	0	26	273	0	0	138	0	20	0	0	90	42	0	589
5:15 PM	0	0	0	0	25	307	0	0	129	0	15	0	0	99	68	0	643
5:30 PM	0	0	0	0	21	307	0	0	152	0	19	0	0	107	68	0	674
5:45 PM	0	0	0	0	13	247	0	0	102	0	14	0	0	86	61	0	523
Total	0	0	0	0	85	1134	0	0	521	0	68	0	0	382	239	0	2429
Grand Total	0	0	0	0	243	3302	0	0	1557	0	235	0	0	1157	673	0	7167
Apprch %	0	0	0	0	6.85	93.1	0	0	86.9	0	13.1	0	0	63.2	36.8	0	
Total %	0	0	0	0	3.39	46.1	0	0	21.7	0	3.28	0	0	16.2	9.39	0	
Cars	0	0	0	0	211	3201	0	0	1516	0	161	0	0	1076	644	0	6809
% Cars	0	0	0	0	86.8	96.9	0	0	97.4	0	68.5	0	0	93.2	95.7	0	95.0
Trucks	0	0	0	0	32	100	0	0	41	0	74	0	0	79	29	0	355
% Trucks	0	0	0	0	13.2	3.03	0	0	2.63	0	31.5	0	0	6.83	4.31	0	5.00
Bikes	0	0	0	0	0	1	0	0	0	0	0	0	0	2	0	0	3
% Bikes	0	0	0	0	0	0.03	0	0	0	0	0	0	0	0.17	0	0	0.04

Start Time	None					US 30					Cornelius Pass Road					US 30					Int. Total
	From North					From East					From South					From West					
	Left	Thru	Right	Peds	App.Total	Left	Thru	Right	Peds	App.Total	Left	Thru	Right	Peds	App.Total	Left	Thru	Right	Peds	App.Total	
Peak Hour Analysis From 3:00 PM to 5:45 PM – Peak 1 of 1																					
Peak Hour for Entire Intersection Begins at 4:00 PM																					
4:00 PM	0	0	0	0	0	16	280	0	0	296	127	0	25	0	152	0	93	57	0	150	598
4:15 PM	0	0	0	0	0	19	302	0	0	321	172	0	22	0	194	0	111	58	0	168	684
4:30 PM	0	0	0	0	0	16	312	0	0	328	141	0	24	0	165	0	98	64	0	162	655
4:45 PM	0	0	0	0	0	23	313	0	0	336	163	0	20	0	183	0	92	53	0	145	664
Total Volume	0	0	0	0	0	74	1207	0	0	1281	603	0	91	0	694	0	394	232	0	626	2601
% App. Total	0.0	0.0	0.0	0.0		5.8	94.2	0	0		86.9	0	13.1	0		0	62.9	37.1	0		
PHF	.000	.000	.000	.000	.000	.804	.964	.000	.000	.953	.876	.000	.910	.000	.894	.000	.887	.906	.000	.926	.951

Table D.1: US30 and Cornelius Pass Rd. TMC – Peak Hour (Cont'd)

Groups Printed - Trucks

Start Time	None From North				US 30 From East				Cornelius Pass Road From South				US 30 From West				Int. Total
	Left	Thru	Right	Peds	Left	Thru	Right	Peds	Left	Thru	Right	Peds	Left	Thru	Right	Peds	
3:00 PM	0	0	0	0	2	12	0	0	5	0	9	0	0	10	4	0	42
3:15 PM	0	0	0	0	1	12	0	0	5	0	5	0	0	7	2	0	32
3:30 PM	0	0	0	0	6	7	0	0	5	0	8	0	0	4	2	0	32
3:45 PM	0	0	0	0	6	11	0	0	4	0	7	0	0	7	0	0	35
Total	0	0	0	0	15	42	0	0	19	0	29	0	0	28	8	0	141
4:00 PM	0	0	0	0	5	9	0	0	6	0	14	0	0	10	5	0	49
4:15 PM	0	0	0	0	2	14	0	0	1	0	6	0	0	5	2	0	30
4:30 PM	0	0	0	0	2	6	0	0	5	0	6	0	0	13	3	0	35
4:45 PM	0	0	0	0	4	7	0	0	2	0	7	0	0	3	2	0	25
Total	0	0	0	0	13	36	0	0	14	0	33	0	0	31	12	0	139
5:00 PM	0	0	0	0	1	6	0	0	1	0	7	0	0	7	2	0	24
5:15 PM	0	0	0	0	1	4	0	0	3	0	1	0	0	5	2	0	16
5:30 PM	0	0	0	0	1	7	0	0	4	0	2	0	0	4	3	0	21
5:45 PM	0	0	0	0	1	5	0	0	0	0	2	0	0	4	2	0	14
Total	0	0	0	0	4	22	0	0	8	0	12	0	0	20	9	0	75
Grand Total	0	0	0	0	32	100	0	0	41	0	74	0	0	79	29	0	355
Apprch %	0	0	0	0	24.2	75.8	0	0	35.7	0	64.3	0	0	73.1	26.9	0	
Total %	0	0	0	0	9.0	28.2	0	0	11.5	0	20.8	0	0	22.3	8.2	0	

Start Time	None From North					US 30 From East					Cornelius Pass Road From South					US 30 From West					Int. Total
	Left	Thru	Right	Peds	App.Total	Left	Thru	Right	Peds	App.Total	Left	Thru	Right	Peds	App.Total	Left	Thru	Right	Peds	App.Total	
Peak Hour Analysis From 3:00 PM to 5:45 PM – Peak 1 of 1																					
Peak Hour for Entire Intersection Begins at 3:45 PM																					
3:45 PM	0	0	0	0	0	6	11	0	0	17	4	0	7	0	11	0	7	0	0	7	35
4:00 PM	0	0	0	0	0	5	9	0	0	14	6	0	14	0	20	0	10	5	0	15	49
4:15 PM	0	0	0	0	0	2	14	0	0	16	1	0	6	0	7	0	5	2	0	7	30
4:30 PM	0	0	0	0	0	2	6	0	0	8	5	0	6	0	11	0	13	3	0	16	35
Total Volume	0	0	0	0	0	15	40	0	0	55	16	0	33	0	49	0	35	10	0	45	149
% App. Total	0	0	0	0	0	27.3	72.7	0	0		32.7	0	67.3	0		0	77.8	22.2	0		
PHF	.000	.000	.000	.000	.000	.625	.714	.000	.000	.809	.667	.000	.589	.000	.613	.000	.875	.500	.000	.703	.760

Table D.1: US30 and Cornelius Pass Road TMC – Peak Hour (Cont'd)

Groups Printed - Bikes

Start Time	None From North				US 30 From East				Cornelius Pass Road From South				US 30 From West				Int. Total
	Left	Thru	Right	Peds	Left	Thru	Right	Peds	Left	Thru	Right	Peds	Left	Thru	Right	Peds	
3:00 PM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
3:15 PM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
3:30 PM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
3:45 PM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Total	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
4:00 PM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
4:15 PM	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	1
4:30 PM	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	1
4:45 PM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Total	0	0	0	0	0	0	0	0	0	0	0	0	0	2	0	0	2
5:00 PM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5:15 PM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5:30 PM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5:45 PM	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	1
Total	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	1
Grand Total	0	0	0	0	0	1	0	0	0	0	0	0	0	2	0	0	3
Apprch %	0	0	0	0	0	100	0	0	0	0	0	0	0	100	0	0	
Total %	0	0	0	0	0	33.3	0	0	0	0	0	0	0	66.6	0	0	

Start Time	None From North					US 30 From East					Cornelius Pass Road From South					US 30 From West					Int. Total
	Left	Thru	Right	Peds	App.Total	Left	Thru	Right	Peds	App.Total	Left	Thru	Right	Peds	App.Total	Left	Thru	Right	Peds	App.Total	
Peak Hour Analysis From 3:00 PM to 5:45 PM – Peak 1 of 1																					
Peak Hour for Entire Intersection Begins at 4:15 PM																					
4:15 PM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	1	1
4:30 PM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	1	1
4:45 PM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5:00 PM	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Total Volume	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	0	0	2	2
% App. Total	0	0	0	0		0	0	0	0		0	0	0	0		0	100	0	0		
PHF	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000	.500	.000	.000	.000	.500

APPENDIX E

SPOT SPEED STUDY AT US30-CORNELIUS PASS

APPENDIX E: SPOT SPEED STUDY AT US30-CORNELIUS PASS

To conduct this field study, observers were located ~500 ft up/downstream of the intersection of US30 and Cornelius Pass Rd. A Pocket RADAR device was used to measure the speeds of 125 vehicles for each approach (NW/SE) along US30. All vehicles were free-flowing vehicles. “Free-flow speed” was defined as the speed when there are no constraints placed on a driver by other vehicles on the road. Tables E.1 and E.2 show data collected at the site.

Table E.1. Spot speeds collected along the NW approach of the US30–Cornelius Pass intersection

Speed (mph)	Veh Class								
58	PC	52	LT	55	PC	52	LT	53	PC
47	PC	51	PC	53	LT	53	LT	51	PC
56	LT	45	T	62	ST	55	PC	56	PC
52	PC	55	LT	56	PC	52	PC	48	PC
51	PC	52	PC	50	PC	50	PC	47	LT
54	ST	49	T	51	ST	60	LT	53	PC
57	PC	50	T	51	LT	65	PC	55	PC
46	PC	54	T	52	PC	61	LT	52	LT
48	PC	57	LT	51	LT	50	ST	66	PC
58	PC	49	LT	51	LT	44	PC	51	PC
51	T	45	PC	56	LT	56	PC	59	PC
56	PC	46	ST	48	PC	50	PC	64	PC
52	LT	60	PC	52	PC	50	PC	61	PC
48	ST	57	LT	44	LT	57	PC	51	PC
56	PC	50	PC	45	PC	52	PC	54	LT
58	LT	48	PC	51	PC	63	PC	53	PC
55	PC	54	LT	58	PC	56	PC	55	LT
52	PC	55	LT	48	T	54	PC	55	LT
55	LT	68	PC	54	PC	56	ST	53	LT
53	PC	58	PC	53	LT	60	PC	51	PC
54	PC	54	PC	47	PC	52	ST	57	PC
55	PC	57	ST	45	PC	56	PC	55	PC
50	PC	53	PC	51	PC	53	LT	68	PC
62	PC	48	LT	48	ST	50	PC	56	PC
57	PC	55	LT	55	PC	52	PC	54	PC

*Technicians – Kamilah Buker and Hisham Jashami: Date & Time: 07/14/2015 from 10:45 AM to 12:00 PM; Posted Speed Limit: 50 MPH; Direction of Travel: NW; Weather: Sunny

Table E.2. Spot speeds collected along the SE approach of the US30–Cornelius Pass intersection

Speed (mph)	Veh Class								
64	PC	58	LT	51	T	51	LT	60	ST
59	PC	53	ST	56	PC	68	PC	61	PC
63	B	52	PC	54	PC	54	PC	59	LT
56	PC	57	PC	59	PC	63	PC	53	LT
51	PC	54	PC	56	PC	75	PC	54	PC
61	PC	59	LT	53	PC	61	PC	60	PC
56	LT	57	PC	56	LT	56	T	53	T
62	PC	58	LT	63	PC	53	PC	55	PC
62	PC	56	PC	58	LT	61	PC	65	PC
66	LT	49	PC	59	PC	60	PC	61	PC
61	PC	54	T	56	PC	65	PC	60	PC
56	PC	43	PC	60	PC	56	PC	63	LT
58	PC	53	PC	53	PC	60	PC	59	LT
50	ST	59	PC	61	LT	56	PC	60	PC
54	ST	52	PC	58	PC	61	PC	61	PC
81	PC	58	LT	59	PC	61	PC	51	PC
61	PC	61	LT	57	PC	61	PC	56	PC
58	PC	58	PC	54	PC	56	PC	56	LT
62	PC	59	LT	54	PC	52	PC	64	PC
54	PC	50	PC	64	LT	52	PC	59	PC
59	PC	49	T	60	PC	59	LT	54	PC
52	PC	53	PC	57	ST	55	ST	57	PC
55	LT	52	ST	54	PC	52	ST	60	PC
52	PC	55	PC	60	PC	67	PC	64	PC
54	LT	56	PC	54	ST	61	LT	50	ST

*Technicians – Kamilah Buker and Hisham Jashami: Date & Time: 07/14/2015 from 2:00 PM to 1:00 PM; Posted Speed Limit: 50 MPH; Direction of Travel: SE; Weather: Sunny

Once data were collected, the speed groups, frequencies, and cumulative frequencies were calculated for both approaches (Tables E.3 and E.4).

Table E.3. Analysis of spot speeds collected along the NW approach of the US30–Cornelius Pass intersection

Speed Groups	Number Observed	Frequency (%)	Cumulative Frequency (%)	Plotted Speed
44	2	1.6	1.6	44
45	4	3.2	4.8	45
46	2	1.6	6.4	46
47	3	2.4	8.8	47
48	8	6.4	15.2	48
49	2	1.6	16.8	49
50	9	7.2	24	50
51	13	10.4	34.4	51
52	13	10.4	44.8	52
53	10	8	52.8	53
54	9	7.2	60	54
55	13	10.4	70.4	55
56	11	8.8	79.2	56
57	7	5.6	84.8	57
58	5	4	88.8	58
59	1	0.8	89.6	59
60	3	2.4	92	60
61	2	1.6	93.6	61
62	2	1.6	95.2	62
63	1	0.8	96	63
64	1	0.8	96.8	64
65	1	0.8	97.6	65
66	1	0.8	98.4	66
68	2	1.6	100	68
Total	125	100	-	-

Table E.4. Analysis of spot speeds collected along the SE approach of the US30–Cornelius Pass intersection

Speed Groups	Number Observed	Frequency (%)	Cumulative Frequency (%)	Plotted Speed
43	1	0.8	0.8	43
49	2	1.6	2.4	49
50	3	2.4	4.8	50
51	4	3.2	8	51
52	8	6.4	14.4	52
53	8	6.4	20.8	53
54	13	10.4	31.2	54
55	4	3.2	34.4	55
56	15	12	46.4	56
57	5	4	50.4	57
58	8	6.4	56.8	58
59	12	9.6	66.4	59
60	10	8	74.4	60
61	14	11.2	85.6	61
62	3	2.4	88	62
63	4	3.2	91.2	63
64	4	3.2	94.4	64
65	2	1.6	96	65
66	1	0.8	96.8	66
67	1	0.8	97.6	67
68	1	0.8	98.4	68
75	1	0.8	99.2	75
81	1	0.8	100	81
Total	125	100	-	-

Cumulative frequency curves and histograms for the overall data for each approach are displayed in Figures E.1 and E.2.

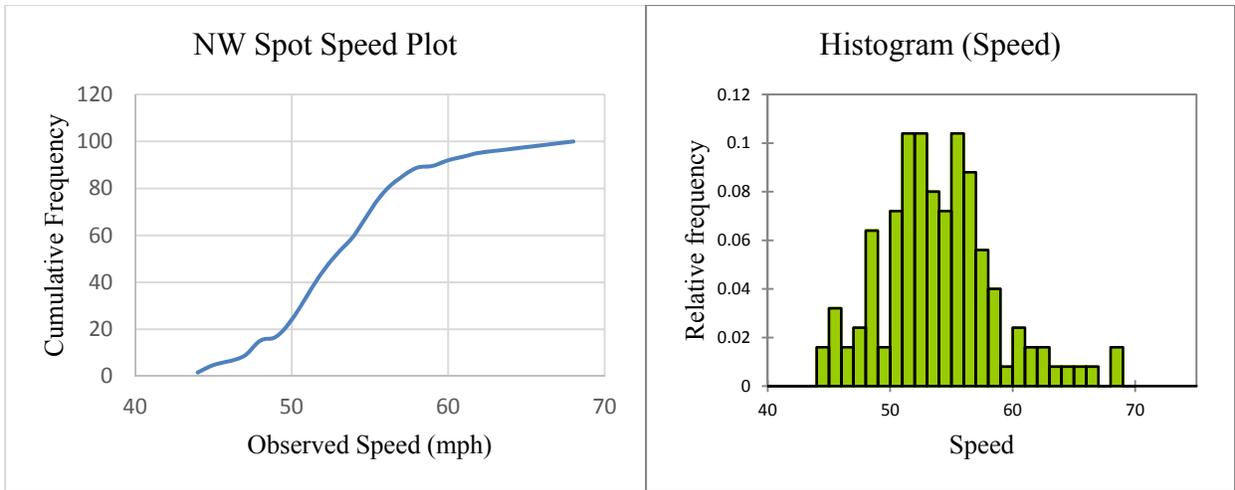


Figure E.1. Cumulative frequency plot and histogram of spot speeds collected along the NW approach of the US30–Cornelius Pass intersection

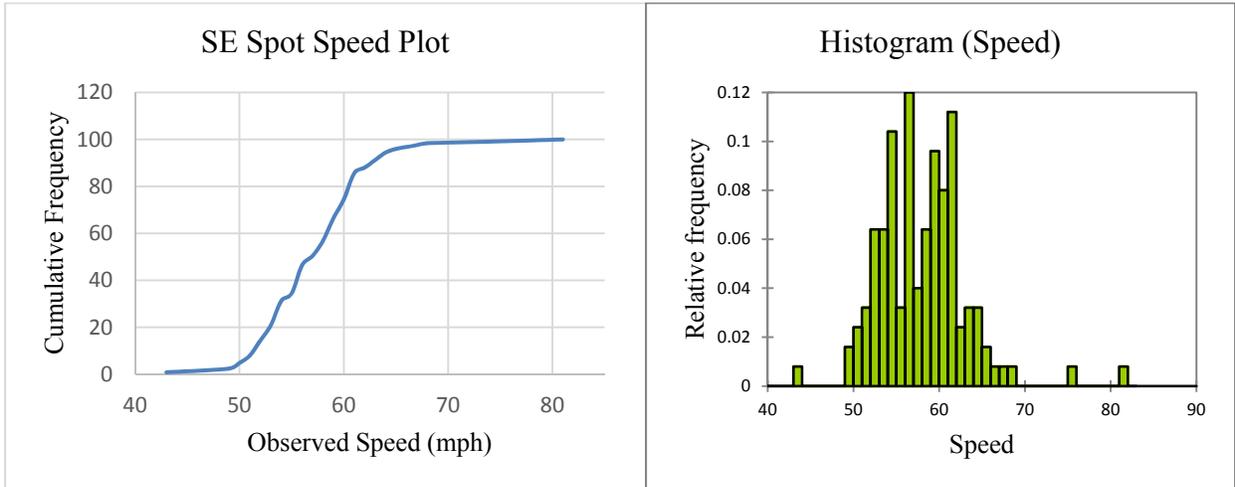


Figure E.2. Cumulative frequency plot and histogram of spot speeds collected along the SE approach of the US30–Cornelius Pass intersection

Descriptive statistics were calculated for both datasets as a whole (for each approach), as well as individually for passenger cars, light trucks, trucks, and semi-trucks (Tables E.5 and E.6).

Table E.5. Statistical results for all vehicles and by vehicle class for spot speeds collected along the NW approach of the US30–Cornelius Pass intersection

Statistic	All Vehicles	Passenger Cars	Light Trucks	Trucks	Semi-Trucks
Mean	53.50	53.96	53.45	49.50	52.40
Median (50 th %ile)	53.00	53.50	53.00	49.50	51.50
Mode	52.00	52.00	55.00	NA	48.00
Range	12.00	12.00	8.50	3.00	7.00
SD	4.77	5.17	3.57	3.02	4.90
85 th %ile	57.40	58.00	56.50	51.75	56.65
Speed range (pace)	48–58	49–59	28–58	45–55	47–57

Table E.6. Statistical results for all vehicles and by vehicle class for spot speeds collected along the SE approach of the US30–Cornelius Pass intersection

Statistic	All Vehicles	Passenger Cars	Light Trucks	Trucks	Semi-Trucks
Mean	57.51	57.95	58.36	52.60	53.70
Median (50 th %ile)	57.00	58.00	58.50	53.00	53.50
Mode	56.00	56.00	59.00	NA	50.00
Range	19.00	19.00	7.50	3.50	5.00
SD	5.06	5.35	3.55	2.70	3.09
85 th %ile	61.00	62.00	61.00	54.80	56.30
Speed range (Pace)	52–62	53–63	53–63	48–58	49–59

APPENDIX F

LOGICAL PROGRAMMING

APPENDIX F: LOGICAL PROGRAMMING

This appendix describes the logical programming used by NWS Voyage™ to operate the SUSCD setup for a single lane of traffic. SUSCD on multiple lanes was accomplished by repeating these steps while updating the timer, latch, and detector input numbers.

Table F.1: NWS Voyage™ logical programming

Step	Command	Logic Description	Function for SUSCD setup and notes
1	209	Set a Latch if Test is True	Set a latch when a vehicle first actuates the lead detector
2	1	Latch Number (Latch #1)	
3	22	Test if Input is Active	
4	106	Input Number (Leading Detector - #14)	
5	208	Load a Timer if Test is True	Load a timer when the latch is not set. Decrement the timer when the latch is set.
6	1	Timer # (Time #1)	
7	0.4	Timer Value (0.4 seconds)	
8	24	NOT	
9	26	Test if a Latch is Set	
10	1	Latch #	
11	210	Reset a Latch if Test is True	Reset the latch when the timer decrements to 0.0 s.
12	1	Latch Number (Latch #1)	
13	24	NOT	
14	27	Test if Timer is Reset/Decrementing	
15	1	Timer # (Time #1)	
16	209	Set a Latch if Test is True	Set a second latch when a vehicle first actuates the lagging detector. <i>This function is not strictly needed to operate the SUSCD setup. However, using this latch makes calibration easier when viewing the latch status in the Voyage Internal Logic Menu.</i>
17	5	Latch Number (Latch #5)	
18	22	Test if Input is Active	
19	107	Input Number (Lagging Detector - #15)	
20	210	Reset a Latch if Test is True	Reset the latch when the vehicle no longer actuates the lagging detector. <i>This function is not strictly needed to operate the SUSCD setup. However, using this latch makes calibration easier when viewing the latch status in the Voyage Internal Logic Menu.</i>
21	5	Latch Number (Latch #5)	
22	24	NOT	
23	22	Test if Input is Active	
24	107	Input Number (Lagging Detector - #15)	
25	206	Turn On an Input if Conditions are Met	Place a call on the Red Extension Detector when the two latches are active at the same time. As with standard detection for RLE, if this call is placed
26	231	Input Number (Voyage Red Extension Detector	

		- #52)	during the second half of the yellow or during the normal all red, a RLE will be triggered.
27	26	Test if a Latch is Set	
28	1	Latch Number (Latch #1)	
29	20	AND	
30	26	Test if a Latch is Set	
31	5	Latch Number (Latch #5)	

APPENDIX G

VISSIM MANUAL DESCRIPTION OF YELLOW LIGHT DRIVER BEHAVIOR MODELS

APPENDIX G: VISSIM MANUAL DESCRIPTION OF YELLOW LIGHT DRIVER BEHAVIOR MODELS

>> Base data for simulation > Defining driving behavior parameter sets > Editing the driving behavior parameter Signal Control

Editing the driving behavior parameter Signal Control

For the driving behavior at signal controls, you specify drivers' reactions to an amber and a red/amber signal light. You may also define a reduced safety distance to stop lines.

How the decision model works

For the behavior of drivers at an amber signal light, select the decision model of your choice:

➤ Continuous check

Vehicles assume that the amber light will only be visible for another 2 seconds. They then decide continuously, with each time step, whether they will continue to drive or stop.

A vehicle will not brake, if its maximum deceleration does not allow it to stop at the stop line, or if it would have to brake for longer than 4.6 m/s².

The vehicle will brake, if at its current speed, it cannot drive past the signal head within two seconds.

Both braking and stopping are possible for cases that lie in between these two scenarios. Using a normally distributed random variable, Vissim decides whether or not the driver will brake.

➤ One decision

To calculate the probability **p**, i.e. whether a driver stops at an amber light or not, the program uses a logistic regression function, with the following parameters **Alpha**, **Beta1**, **Beta2**, vehicle speed **v** and distance to stop line **dx**:

$$p = \frac{1}{1 + e^{-\alpha - \beta_1 v - \beta_2 dx}}$$

The default values of the **Probability factors Alpha**, **Beta1**, **Beta2** are based on empirical data.

The decision made is maintained until the vehicle crosses the stop line.

To produce the most accurate results, select the **One decision** option and increase the number of **Observed vehicles** accordingly ([Editing the driving behavior parameter Following behavior](#)). Internally, signal heads are modeled as vehicles and are only recognized if the number of vehicles and network objects between the vehicle in question and the signal head does not exceed the number of **Observed vehicles** minus 1.

Selecting the driving behavior parameter Signal Control

1. From the **Base Data** menu, choose **Driving Behaviors**.

The list of network objects for the network object type opens.

The list shows driving behavior parameter sets. Some driving behavior parameter sets can be predefined.

By default, you can edit the list ([Using lists](#)).

You can edit all driving behavior parameters for following behavior, lane change, lateral behavior and signal controls in the list or in tabs with the following steps.

Note: In lists, you can use the  **Select attributes** icon to show and hide attribute values ([Selecting attributes and subattributes for a list](#)).

2. Right-click the entry of your choice.

3. From the shortcut menu, choose **Edit**.

*The **Driving Behavior Parameter Sets** window opens.*

4. Select the **Signal Control** tab.

5. Make the desired changes:

Element	Description
Reaction to amber signal	<ul style="list-style-type: none"> ➤ Decision model: The decision model defines the behavior of vehicles when they approach an amber light ➤ Continuous check: Driver of vehicle continuously decides whether to continue driving or whether to stop. ➤ One decision: The decision made is maintained until the vehicle crosses the stop line. Entering probability factors: <ul style="list-style-type: none"> ➤ Alpha: default 1.59

	<ul style="list-style-type: none"> ➤ Beta1: default -0.26 ➤ Beta2: default 0.27
Behavior at red/amber signal	<p>Modeling country-specific or regional behavior at red/amber lights.</p> <ul style="list-style-type: none"> ➤ Stop: Stop as at red light ➤ Go: Go as at green light
Reduced safety distance close to a stop line	<p>Defining the behavior of vehicles close to a stop line.</p> <ul style="list-style-type: none"> ➤ If a vehicle is located in an area between Start upstream of stop line and End downstream of stop line, the factor is multiplied by the safety distance of the vehicle. The safety distance used is based on the car following model. The safety distance may be reduced via the Safety distance reduction factor attribute (Editing the driving behavior parameter Lane change behavior). For lane changes in front of a stop line, the two values calculated are compared. Vissim will use the shorter of the two distances. ➤ Start upstream of stop line: Distance upstream of the signal head ➤ End downstream of stop line: Distance downstream of signal head

Superordinate topic:

[Applications and driving behavior parameters of lane changing](#)