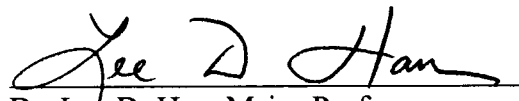
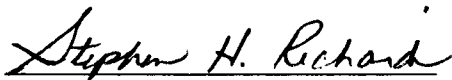


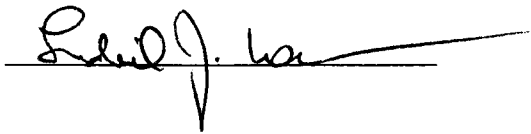
To the Graduate Council:

I am submitting herewith a thesis written by Jennifer Harper Ogle entitled "A Probabilistic Assessment of Dilemma and Option Zone Problems." I have examined the final copy of this thesis for form and content and recommend that it be accepted in partial fulfillment of the requirements for the degree of Master of Science, with a major in Civil Engineering.

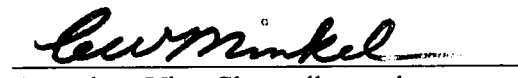

Dr. Lee D. Han, Major Professor

We have read this thesis and
recommend its acceptance:





Accepted for the Council:


Associate Vice Chancellor and
Dean of The Graduate School

**A Probabilistic Assessment
of Dilemma and Option Zone Problems**

A Thesis
Presented for the
Master of Science
Degree
The University of Tennessee, Knoxville

Jennifer Harper Ogle
December 1996

DEDICATION

This thesis is dedicated to my family:

Terry, my husband

Amber and Porter, my dogs;

Wilma Jean Harper, my mother;

Barri Lynn Harper, my brother;

Elaine Davis, Barri's girlfriend;

Larry Franklin Harper, my brother;

Sheila, Larry's wife;

Desiree and Rachel, Larry's children;

and

Gary and Sharon Ogle, my new in-laws;

who have all given me more invaluable love and support

that anyone could ever wish for.

I would also like to make a special dedication to

Harry Edsil Harper, my late father,

whom I'm sure is looking down from above with great pride.

I love you all!

ACKNOWLEDGEMENTS

I would like to express my deepest gratitude and appreciation to the members of my graduate committee, Dr. Lee Han, Dr. Stephen Richards, and Dr. Fred Wegmann, for their encouragement, guidance, patience and support throughout my graduate experience. They have given me numerous opportunities to advance in my professional career, and I will never be able to thank them enough.

A special thanks is due Dr. Lee Han, who has helped out and given sound advice when I needed it most. I greatly appreciate all of the time he has spent providing feedback during the course of preparation of my thesis.

I would also like to thank Mr. Martin Parker, of Martin Parker and Associates, for providing me with the data for which I based my assumptions for the current research.

Finally, a special debt of gratitude to my new husband, Terry, for his interest, encouragement and incredible patience throughout my graduate career. Oh, and I almost forgot, I would like to thank my dog, Amber Elle, for putting in all of those late nights with me. I couldn't have done it without her.

ABSTRACT

The Institute of Transportation Engineers currently recommends that the phase-change interval of a traffic signal be set using a constant approach speed equal to the 85th percentile approach speed, a constant deceleration of 10 ft/sec², and a constant reaction time of 1 second. Previous studies have shown that drivers do not all travel at the same speed and decelerate at the same rate, nor do they possess the same reaction time. In fact, the practice of using the 85th percentile approach speed in itself denounces the idea that all drivers would approach an intersection at a single speed. Thus, it would appear that the current practice may be inappropriate.

A probabilistic approach is developed here to estimate the effects of variance in approach speed during different periods of congestion and free-flow traffic conditions. These effects are measured by determining the magnitude of the dilemma or option zone problems for any given situation. Using previously published values of reaction time, deceleration rates and acceleration rates, and approach speed distributions, a spreadsheet calculation procedure was developed to estimate the magnitude of the dilemma/option zone problems. The following approach yields logical results regarding the probability of being caught in a dilemma zone or option zone for given distributions of approach speed at different levels of congestion or free-flow traffic conditions. As a result, the approach will enhance the practice of designing the phase-change interval to minimize these zones.

Some transportation engineers might dispute the use of the dilemma zone as a measure of effectiveness, by reason of the 1962 change in the Uniform Vehicle Code definition of the yellow signal indication. This change allows for vehicles to enter during the yellow interval and clear during the red. There is, however, no rule for providing an all-red clearance interval. This means that the driver may clear during an all-red clearance interval or during the beginning of green for the opposing traffic stream. The legality issue of clearing during the opposing green is covered by the Uniform Vehicle Code in a statement of the green indication. This statement says 'drivers facing green must yield to vehicles legally in the intersection at the time of such indication.' Hence, drivers would never be faced with the dilemma of not being able to stop before entering the intersection or not being able to clear before the onset of red.

In an informal survey of 100 people in Knoxville, TN, where the vehicle code follows the one stated above, 100 % were unknowing of at least one implication of the signalized intersection laws as stated in the Uniform Vehicle Code. According to those surveyed, it seems quite clear that they may be encountering dilemma zones because of their lack of understanding of the current laws. With the possibility of this being public perception nationwide, it seems imperative that transportation professionals assess the magnitude of problems that these drivers may encounter. For this reason, the use of the dilemma zone magnitude, as a measure of effectiveness, is warranted.

TABLE OF CONTENTS

CHAPTER	PAGE
I INTRODUCTION	1
Background	2
Rationale	9
Problem Statement	17
II LITERATURE REVIEW	19
III METHODOLOGY	45
Key Formulas and Issues	47
Model Development	49
Example Case Study	52
IV RESULTS	64
Sensitivity Analysis	82
V CONCLUSIONS.....	87
BIBLIOGRAPHY.....	90
APPENDIX A.....	94
APPENDIX B	113

LIST OF TABLES

TABLE	PAGE
II-1 Perception-Brake-Reaction Times	36
IV-1 Summary of Dilemma and Option Zone Magnitudes for Varying Phase-Change Timings - Case 1.....	70
IV-2 Summary of Dilemma and Option Zone Magnitudes for Varying Phase-Change Timings - Case 2.....	71
IV-3 Summary of Dilemma and Option Zone Magnitudesfor Varying Phase-Change Timings - Case 1 with Acceleration.....	75
IV-4 Summary of Dilemma and Option Zone Magnitudesfor Varying Phase-Change Timings - Case 2 with Acceleration.....	76
IV-5 Effects of Reaction Time Variation	84
IV-6 Effects of Length of Vehicle Variation.....	85
IV-7 Effects of Width of Intersection Variation.....	86

LIST OF FIGURES

FIGURE		PAGE
I-1	Assessment Model Development Flowchart	18
II-1	Intersection Elements	21
II-2	Diagram of Dilemma Zone	22
II-3	Diagrams of Dilemma and Option Zones	29
III-1	Plots of Stopping and Clearing Distances and Resulting Dilemma and Option Zones	50
III-2	Case 1 and Case 2 Speed Distributions	55
III-3	Plot of Clearing and Stopping Distance, $PC(v_2) = 6$ sec	57
III-4	Plot of Dilemma and Option Zone Lengths, $PC(v_2) = 6$ sec	58
III-5	Probability of Being Caught in a Dilemma Zone, $PC(v_2) = 6$ sec	59
III-6	Probability of Being Caught in an Option Zone, $PC(v_2) = 6$ sec	60
III-7	Probability of Being in a Dilemma Zone, Given Case 1 and Case 2 Speed Distributions, $P(v_2) = 6$ sec	62
III-8	Probability of Being Caught in an Option Zone, Given Case 1 and Case 2 Speed Distributions, $P(v_2) = 6$ sec	63
IV-1	Overall Probabilities of Not Being Caught - Case 1	65
IV-2	Overall Probabilities of Not Being Caught - Case 2	66
IV-3	P(OK) - Case 1	67
IV-4	P(OK) - Case 2	68

LIST OF FIGURES

FIGURE		PAGE
I-1	Assessment Model Development Flowchart	18
II-1	Intersection Elements	21
II-2	Diagram of Dilemma Zone	22
II-3	Diagrams of Dilemma and Option Zones	29
III-1	Plots of Stopping and Clearing Distances and Resulting Dilemma and Option Zones	50
III-2	Case 1 and Case 2 Speed Distributions	55
III-3	Plot of Clearing and Stopping Distance, $PC(v_2) = 6$ sec	57
III-4	Plot of Dilemma and Option Zone Lengths, $PC(v_2) = 6$ sec	58
III-5	Probability of Being Caught in a Dilemma Zone, $PC(v_2) = 6$ sec	59
III-6	Probability of Being Caught in an Option Zone, $PC(v_2) = 6$ sec	60
III-7	Probability of Being in a Dilemma Zone, Given Case 1 and Case 2 Speed Distributions, $P(v_2) = 6$ sec	62
III-8	Probability of Being Caught in an Option Zone, Given Case 1 and Case 2 Speed Distributions, $P(v_2) = 6$ sec	63
IV-1	Overall Probabilities of Not Being Caught - Case 1	65
IV-2	Overall Probabilities of Not Being Caught - Case 2	66
IV-3	P(OK) - Case 1	67
IV-4	P(OK) - Case 2	68

IV-5	Overall Probability of Not Being Caught	
	Case 1 - with Acceleration.....	78
IV-6	Overall Probability of Not Being Caught	
	Case 2 - with Acceleration.....	79
IV-7	P(OK) - Case 1 - with Acceleration.....	80
IV-8	P(OK) - Case 1 - with Acceleration.....	81
A-1	Plot of Clearing and Stopping Distance, $PC(Y) = 3.6$ sec	95
A-2	Plot of Dilemma and Option Zone Lengths, $PC(Y) = 3.6$ sec	96
A-3	Probability of Being Caught in a Dilemma Zone, $PC(Y) = 3.6$ sec	97
A-4	Probability of Being Caught in an Option Zone, $PC(Y) = 3.6$ sec	98
A-5	Probability of Being in a Dilemma Zone, Given Case 1 and Case 2	
	Speed Distributions, $PC(Y) = 3.6$ sec	99
A-6	Probability of Being in an Option Zone, Given Case 1 and Case 2	
	Speed Distributions, $PC(Y) = 3.6$ sec	100
A-7	Plot of Clearing and Stopping Distance, $PC(v_L) = 5$ sec	101
A-8	Plot of Dilemma and Option Zone Lengths, $PC(v_L) = 5$ sec	102
A-9	Probability of Being Caught in a Dilemma Zone, $PC(v_L) = 5$ sec	103
A-10	Probability of Being Caught in an Option Zone, $PC(v_L) = 5$ sec	104
A-11	Probability of Being in a Dilemma Zone, Given Case 1 and Case 2	
	Speed Distributions, $PC(v_L) = 5$ sec	105
A-12	Probability of Being in an Option Zone, Given Case 1 and Case 2	
	Speed Distributions, $PC(v_L) = 5$ sec	106

A-13	Plot of Clearing and Stopping Distance, $PC(v_1) = 5.2$ sec	107
A-14	Plot of Dilemma and Option Zone Lengths, $PC(v_1) = 5.2$ sec	108
A-15	Probability of Being Caught in a Dilemma Zone, $PC(v_1) = 5.2$ sec	109
A-16	Probability of Being Caught in an Option Zone, $PC(v_1) = 5.2$ sec	110
A-17	Probability of Being in a Dilemma Zone, Given Case 1 and Case 2 Speed Distributions, $PC(v_1) = 5.2$ sec	111
A-18	Probability of Being in an Option Zone, Given Case 1 and Case 2 Speed Distributions, $PC(v_1) = 5.2$ sec	112
B-1	Plot of Clearing and Stopping Distance, $PC(Y) = 3.6$ sec with Acceleration	114
B-2	Plot of Dilemma and Option Zone Lengths, $PC(Y) = 3.6$ sec with Acceleration	115
B-3	Probability of Being Caught in a Dilemma Zone, $PC(Y) = 3.6$ sec with Acceleration	116
B-4	Probability of Being Caught in an Option Zone, $PC(Y) = 3.6$ sec with Acceleration	117
B-5	Probability of Being in a Dilemma Zone, Given Case 1 and Case 2 Speed Distributions, $PC(Y) = 3.6$ sec with Acceleration	118
B-6	Probability of Being in an Option Zone, Given Case 1 and Case 2 Speed Distributions, $PC(Y) = 3.6$ sec with Acceleration	119
B-7	Plot of Clearing and Stopping Distance, $PC(v_L) = 3.6$ sec with Acceleration	120

B-8	Plot of Dilemma and Option Zone Lengths, $PC(v_L) = 3.6$ sec with Acceleration	121
B-9	Probability of Being Caught in a Dilemma Zone, $PC(v_L) = 3.6$ sec with Acceleration	122
B-10	Probability of Being Caught in an Option Zone, $PC(v_L) = 3.6$ sec with Acceleration	123
B-11	Probability of Being in a Dilemma Zone, Given Case 1 and Case 2 Speed Distributions, $PC(v_L) = 3.6$ sec with Acceleration	124
B-12	Probability of Being in an Option Zone, Given Case 1 and Case 2 Speed Distributions, $PC(v_L) = 3.6$ sec with Acceleration	125
B-13	Plot of Clearing and Stopping Distance, $PC(v_1) = 3.6$ sec with Acceleration	126
B-14	Plot of Dilemma and Option Zone Lengths, $PC(v_1) = 3.6$ sec with Acceleration	127
B-15	Probability of Being Caught in a Dilemma Zone, $PC(v_1) = 3.6$ sec with Acceleration	128
B-16	Probability of Being Caught in an Option Zone, $PC(v_1) = 3.6$ sec with Acceleration	129
B-17	Probability of Being in a Dilemma Zone, Given Case 1 and Case 2 Speed Distributions, $PC(v_1) = 3.6$ sec with Acceleration	130
B-18	Probability of Being in an Option Zone, Given Case 1 and Case 2 Speed Distributions, $PC(v_1) = 3.6$ sec with Acceleration	131

CHAPTER I

INTRODUCTION

The concept of the traffic signal has roots that date back to 1868, where manually operated semaphores were used in London. The electric traffic signal, known today, was developed by James Hoge in 1914. The first of which was installed in Cleveland, Ohio. Over the last 82 years, the traffic signal has evolved into a crucial element of orderly traffic flow for our increasingly busy streets and roadways (16).

One small element of this complex device is the topic of this research study – the phase-change interval. It may be known to some as the change interval, yellow interval, amber time, or clearance interval. The phase-change interval as defined for purposes of this study relates to either a yellow interval by itself or yellow interval followed by an all-red clearance interval. The *Manual of Traffic Signal Design* (16) states the following “The critical function of the phase-change interval is to warn traffic of an impending change in the right of way assignment.” While all of the other aspects of signal timing focus on the efficiency of traffic movement, the phase-change is most importantly related to safety. Many engineers have strong opinions about the timing of the phase-change interval, and these strong opinions are obvious given the array of approaches in use today to determine the interval.

If asked about the yellow interval, most transportation engineers would probably give the

quick answer of '3 seconds for urban areas and 5 seconds for rural areas.' Where did these values come from? Most practicing engineers have probably thought very little about the implications of the phase-change interval on the traffic stream. The phase-change interval has not, however, been taken lightly by all. In fact, during the last 40 years, the determination of the traffic signal phase-change interval has become a quite controversial topic. The controversy has spanned issues such as the concepts of the dilemma zone or option zone, changes in the Uniform Vehicle Code, definitions of clearance and change intervals, and even the definition of the average driver.

BACKGROUND

The initial search for the origin and reasoning behind the use of 3 to 5 seconds for the yellow portion of the phase-change interval led to a book entitled *Traffic Engineering* written by Matson, Smith, and Hurd in 1955:

"In urban areas where speeds are relatively low, yellow lights of about 3 second duration are satisfactory at most locations. At rural, high-speed locations where stopping time may have a duration of 5 to 8 seconds, road users tend to attempt to clear the intersection rather than stop. Five seconds is probably a practical maximum yellow duration in such location."

This was, as the author's literature search shows, the origin of the still common 3 or 5 second timing philosophy of the yellow signal interval. The values were determined by equations that are quite similar to those widely used in practice today. Instead of using

stopping and clearing distances, as is done today, the time to stop and time to clear were calculated as functions of distance to brake, speed of travel, perception-interpretation-emotion-volition (reaction) time, and intersection width. Probably the most important aspect of the yellow signal interval was also found here:

"...once a road user has been permitted entry into the intersection under signal control, safety demands that he be allowed comfortable journey time to traverse the intersection before cross traffic is released."

This is a concept that the author feels is sometimes compromised in order to get more out of the system. Is it time to step back and take a look at what the yellow signal interval really represents? It is an engineers responsibility to abide by the "Rules of Professional Conduct" which state:

"The registrant (engineer) shall at all times recognize the primary obligation to protect the safety, health and welfare of the public in the performance of the registrant's professional duties."

Do our current practices and laws really protect the safety, health and welfare of the public? The following background is intended to highlight several problems and questions that have been brought to attention over the last several years.

In 1960, authors Gazis et al. wrote a paper (9) entitled "The Problem of the Amber Signal Light in Traffic Flow" marking the first mention of the 'dilemma zone' problem. From their observations, the authors felt that drivers were sometimes faced with complex situations at the commencement of the yellow signal light. In this situation, the driver

must decide whether to stop quickly, possibly ending up inside the intersection, or to chance trying to go through the intersection before the onset of the red signal light. Every driver can probably attest to being caught in this situation at some point in time or another, and will most likely note that the presence of other vehicles following or leading has made the situation worse. Moreover, the involvement of these other vehicles may potentially increase the likelihood of an accident occurring if one of the drivers incorrectly judges the actions of the drivers around him. For example, if the lead car were to come to an abrupt stop at the commencement of yellow, and the following car thought that the intersection could be cleared before the onset of red, a rear-end collision will occur. On the other hand, if a driver decided to go on through the intersection, but got caught by the red light, then a driver from the cross-street may strike the driver in a right-angle accident at the onset of green for the cross-street movement. With these questions of safety in mind, many an engineer has devoted much time and research to explore different aspects of the yellow signal problem, but the controversy continues, and many questions remain unanswered.

The research done by Gazis et al. gave a clear picture of the problem of the dilemma zone. At the time of their research, the Uniform Vehicle Code gave the following definition for the amber signal light:

"Vehicular traffic facing the signal is thereby warned that the red or 'Stop' signal will be exhibited immediately thereafter and such vehicular traffic shall not enter or be crossing the intersection when the red or

'Stop' signal is exhibited."

Abiding by this definition, a motorist shall not be within the boundary of intersection at the onset of the red signal. Thus, the yellow signal interval became known as the clearance interval. The absoluteness of the definition leads to the dilemma zone-- the area on an intersection approach, determined by the yellow signal timing, speed and reaction time, where the driver cannot stop before entering the intersection nor clear the intersection before the red signal appears. Gazis et al. studied seventy intersections and found that all but one intersection had a yellow interval that was too short, thus creating a sizeable dilemma zone for the average driver. This observation led the authors to propose the idea of changing the Uniform Vehicle Code in order to allow the average driver with a solvable problem, so as to eliminate the possibility of people falling victim to an improperly designed signal.

In 1962, the Uniform Vehicle Code indicated a change in the meaning of the yellow signal interval as follows:

"Vehicular traffic facing a steady yellow signal is thereby warned that the related movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection."

This change in the code allows motorists to legally enter the intersection during the yellow phase and legally clear the intersection during the beginning of the red signal interval. The allowance of vehicles in the intersection during the red phase prompted a restatement of

the meaning of the green phase:

"Vehicular traffic facing a circular green signal may proceed straight through or turn right or left unless a sign at such a place prohibits either such turn. But vehicular traffic, including vehicles turning to the right or left, shall yield the right of way to other vehicles and to pedestrians lawfully within the intersection or an adjacent crosswalk at the time such signal is exhibited."

These changes were deemed safe by engineers who assumed that the opposing traffic stream would encounter a start-up time lag of around three seconds which would be sufficient time to allow motorists to clear the intersection. If this did not allow sufficient clearing time, the law required that the motorists facing green must yield to the clearing vehicles. These changes in the Uniform Vehicle Code turned what used to be a 'restrictive' yellow interval law into a 'permissive' yellow interval law. The law had been restrictive because vehicles had to clear the intersection before the onset of red, whereas the permissive law allows clearance on red as long as the vehicle has entered the intersection on yellow. With the permissive law in effect, the yellow interval could no longer be referenced as the 'clearance' interval and was now given the title 'change' interval.

Adolf May (15) defined a new problem zone area on the intersection approach in 1968. After surveying the 50 states as to their vehicle code laws, he found that not all states had conformed to the Uniform Vehicle Code change in definition of the yellow interval. The

two different types of laws, one of which was the restrictive law and the other being the permissive law, were studied in detail. Stopping and Clearing equations were determined for both law types given the criteria for each. Depending on the distance of the driver from the intersection, May found that 3 conditions can exist:

1. Dilemma Zone - within which the driver can neither stop nor clear the intersection. (This case was found to be more pertinent to the restrictive law and was already defined by Gazis et al.(9))
2. Free Zone - within which the drive will not encounter a dilemma zone or option zone.
3. Option Zone - within which a driver can choose between stopping and clearing the intersection.

The size of the option zone determines the magnitude of the problem for drivers. The option zone was seen as a more serious problem if it were big enough for more than one driver to be caught in it at one time. The problem, given this situation, would be encountered when the lead driver chooses to stop while the following driver chooses to go on through the intersection. The obvious result is a rear-end collision. No set of equations were developed to minimize the option zone, because the dilemma zone still seemed to be the most troublesome of the two problems.

In the late 1970's, the Green-Extension Systems (GES) came into being. These systems employed advanced detector-controller designs in order to detect vehicles upstream of an intersection in order to extend the green signal interval to allow the vehicles to pass

through the intersection, or to terminate the green interval allowing for enough time for the vehicle to come to a stop before entering the intersection. These types of arrangements have proven to work nicely at high-speed isolated intersections. Ideal system conditions are at rural intersections where traffic volumes are not very high during peak periods, traffic speeds remain high, and adequate gaps in main street traffic allow for side-street movements. While these systems have proven to work, they are not fool-proof, and a vehicle can still get caught in a dilemma zone caused by max-out of the green interval. Another problem with these systems is that they can not be used on arterials with signals that must be pre-timed for signal progression, because they rely on automation to respond to approaching vehicles.

Currently, the Institute of Transportation Engineers (ITE) recommends that the phase-change interval be set following the equation:

$$PC = t + \frac{v}{2d} + \frac{w + L}{v}$$

where, PC = phase-change period (yellow plus all-red), sec

t = perception-reaction time (usually 1 sec), sec

v = approach speed, ft/s

d = deceleration rate, ft/s²

w = width of intersection, ft

L = length of vehicle, ft.

The typically assumed values for these variables are considered to be constant approach

speed equal to the 85th percentile approach speed or the posted speed limit, a constant deceleration of 10 ft/sec², and a constant reaction time of 1 second. Previous studies and common sense have shown that drivers do not all travel at the same speed and decelerate at the same rate, nor do they possess the same reaction time. In fact, the practice of using the 85th percentile approach speed in itself denounces the idea that all drivers would approach an intersection at a single speed. Thus, it would appear that the current practice may be inappropriate. Is it reasonable to assume deterministic values to determine the phase-change interval when these values are in reality completely stochastic?

Questions such as this are what prompted the in-depth review of the literature and the concept of the following research. The background topics mentioned here are reviewed in detail in the following literature review of previous research. The background presented here only serves the purpose of providing the reader with a brief overview of relevant issues of concern for the current research.

RATIONALE

The concept of the current study was sparked by the review of literature which culminated into a barrage of contradicting results. Aspects of approach speed and traffic density were of particular interest in the literature review. In a study by Stimpson, Zador, and Tarnoff (28) entitled, "The Influence of the Time Duration of Yellow Traffic Signals on Driver Response," the authors tried to determine how intersection conflict frequency depended on yellow signal time duration under different traffic conditions of peak and off-peak

periods. It was noticed that drivers who enter the intersection during the yellow phase will either clear during the yellow phase or during the red phase, and those who clear during the red phase could cause potential conflicts by blocking the path of the opposing traffic stream. The authors felt that increasing the yellow duration slightly would decrease the number of potential conflicts, and in turn reduce the number of crashes caused by those conflicts. A potential conflict in the study was determined to be a last-to-cross vehicle that spent at least 0.2 seconds in the intersection past the onset of red. The findings show that the frequency of potential conflicts at signalized intersections were higher during peak periods than in off-peak periods. It was also determined that potential conflicts could be virtually eliminated with small increases in the duration of the yellow phase. The authors were not successful in determining the number of crashes reduced by the phase increase.

This study seemed to be, in some respects, contradictory to another study by Adolf May (15) that shows a longer yellow duration leads to motorists operating their vehicles in an unsafe or unexpected manner. May also observed in the same study, that increasing the yellow duration at a rural intersection decreased the percentage of motorists operating in an unsafe or unexpected manner. This decrease in unsafe driving with an increase in the phase-change interval is probably in some way linked to the reduction in potential conflicts with an increase in the phase-change interval. After re-examining the previous study, a possible connection was found between the rural intersections of the May study and the intersections of the Stimpson et al. study. The connection is the similarity of isolation of

the intersections. The Stimpson et al. study specified an isolated intersection in their criteria, and while the May study (15) did not specify such a requirement, it would be reasonable to assume that the rural intersection is isolated. Could this mean that motorists drive more safely through isolated intersections, or do motorists behave more aggressively/risky when faced with numerous signals and congestion in an urban setting?

Bissell and Warren (3) tried to discount the work of Stimpson et al. (28) by saying "Based on their definition of potential conflict, the results are suspect since they are not based on the legal meaning of the yellow or the observed motorist's understanding of the signal." Stimpson et al. regard a potential conflict as occurring when a last-to-cross vehicle remains in the intersection 0.2 seconds after the onset of red. Bissell and Warren understand this as meaning that all vehicles should clear the intersection before the onset of red. By examining the following statement, it appears that the intersections that the authors chose to study did not have an all-red phase, "Vehicles that have not cleared the intersection before red onset can be blocking the paths of cross-street traffic that has a green signal for varying periods of time." Looking at it from this direction, Bissell and Warren are incorrect in their assumptions. A vehicle in the intersection at the onset of red would also be in the intersection during the opposing green, and could very well cause a potential conflict if the opposing driver does not yield the right of way. Another questionable assumption of Bissell and Warren is that a motorist's understanding of the signal can be observed. This is untrue, because only a drivers reactions to the signal can

be observed. It is possible that a driver can have the correct understanding of the law and still make the wrong decision due to the inevitable factor of human error.

After reviewing the literature, and noting several contradictions, more and more questions began to emerge. Why were the potential conflicts higher in peak periods than in off peak-periods? Do longer yellow durations eliminate the potential conflicts in an intersection? Do drivers actually react in unsafe or unexpected manners when given longer yellow duration? Research (31) shows that crash rates are higher given lower speeds and wider intersections; why would this be true? If the phase-change interval is determined for the 85th percentile approach speed, what does that mean for drivers who approach at a speed higher than the 85th percentile speed? For years, engineers have used 3 seconds for urban and 5 seconds for rural intersection yellow durations, is this a good practice? Bissell and Warren (3) say that the dilemma zone problem no longer exists given the permissive definition of the yellow indication; is this true? What is the motorist's understanding of the signal?

With all of these questions in mind, it seemed logical to go to the main source of data to determine possible origins of inconsistency. Many engineers have completed study after study reviewing the process of determining the phase-change interval by investigating driver reaction to the yellow interval. Nowhere in the literature review did any evidence of driver understanding of the signal come into question. It seems logical to think that the best signal design in the history would not function properly if its' users did not understand the functional meaning. Could this be the problem causing the inconsistencies

in the previous studies?

Before any further work was done, it seemed necessary to sample the drivers understanding of the phase-change interval. In order to do so, a random and informal survey was taken of 100 drivers from Knoxville, Tennessee. These people were asked a series of four questions pertaining to their understanding of the implications of the traffic signal laws in Tennessee. The questions were asked as follows:

1. You are approaching an intersection when the yellow light appears, and you decide to go on through the intersection. You are inside the intersection when the light turns red, can you get ticketed by a police officer?
2. (If the respondent answered no to question 1 proceed with question 2, otherwise skip down to question 3.) Do you assume that the traffic signal is designed to allow for your clearance in the previous situation before the opposing traffic receives the green signal?
3. You are stopped at red light . When the green signal appears, do you have the complete right of way to enter the intersection?
4. Will you be found at fault if you hit a vehicle that is in the intersection after your green signal appears?

The *Tennessee Vehicle Code* follows the current *Uniform Vehicle Code* and therefore the correct answers are:

1. NO - It is legal to be in the intersection after the onset of red as long as you have entered during the yellow phase. (88% surveyed answered YES)
2. NO - The signal is not always designed to provide complete clearance of the intersection before the onset of green for the opposing movement. (100% of those who answered NO to the first question answered YES to this question)
3. NO - Drivers do not have complete right of way at the onset of green. The law states that motorists must yield to any vehicles or pedestrians lawfully within the intersection at the onset of green. (72% surveyed answered YES)
4. YES - As stated before, drivers are required to yield the right of way to anyone legally in the intersection. A driver, however, is not at fault if the vehicle in the intersection has run a red light, and would therefore be unlawfully in the intersection. (64% surveyed answered NO)

The results show that 88% of those surveyed believe they would be breaking the law if they were not clear of the intersection at the onset of the red interval. Of the 12% who knew it was legal to be in the intersection at the onset of red, all believed that the signal was designed to allow for their complete clearance of the intersection before the opposing traffic stream would be released. 72% of the drivers surveyed thought that they have complete right of way to enter the intersection at the onset of green. Of those people, 64% thought that they would not be liable if they struck a vehicle that was in the

intersection when they received the green signal.

Stated more simply, of 100 people surveyed, 100% either misunderstood or were unknowing of at least one implication of the permissive laws for signalized intersections. The study shows that while the law is permissive, the majority of drivers surveyed impose upon themselves the restrictive meaning of the phase-change interval. In other words, they feel that they must clear the intersection before the onset of red.

This concept of self-imposed restrictive yellow law leads back to the question 'is there a dilemma zone situation where a driver approaching a signalized intersection can neither physically stop before entering the intersection or go through the intersection before the onset of red?' If the answer to this question is yes, drivers may be more likely to make an erratic and unsafe maneuver by slamming on their brakes or speeding through the intersection. It is obvious that either of these decisions could lead to a rear-end collision. The problem of the dilemma zone has been shown to be remedied by extending the yellow time to allow for safe stopping and safe clearing. However, in extending the yellow time, a new problem could arise. Hence the second concept in question: 'is there an option zone situation where a driver approaching a signalized intersection where he can choose between either stopping or clearing before the onset of red?' If so, what is the magnitude of the problem? Is there a chance that two drivers could be in the zone at the same time? If this situation can exist, and the lead car decides to stop while the following car decides to go, a collision will most likely occur.

Some transportation engineers (3) feel strongly that dilemma zones no longer exist after the Uniform Vehicle Code change in the definition of the yellow signal indication back in 1962. This change allows for vehicles to enter during the yellow interval and clear in red. With no absolute provision for an all-red interval, this means that the driver may clear during an all-red clearance interval or during the beginning of green for the opposing traffic stream. The liability issue of allowing clearing during the opposing green is covered by the Uniform Vehicle Code in a statement of the green indication. This statement says 'drivers facing green must yield to vehicles legally in the intersection at the time of such indication.' Hence, drivers would never be faced with the dilemma of not being able to stop before entering the intersection or not being able to clear before the onset of red.

According to the 100 people surveyed, it seems quite clear that these drivers may be encountering dilemma zones because of their lack of understanding of the current laws. With the possibility of this being public perception nationwide, it seems imperative that transportation professionals assess the magnitude of problems that these drivers may encounter. It is in this respect that the following assessment tools will be of use to enhance the current practice of designing signal phase-change intervals.

PROBLEM STATEMENT

The basic purpose of this study was to develop a procedure to estimate the magnitude of the dilemma/option zone problems. The following approach attempts to determine the probability that a driver will be caught in either a dilemma or option zone. The procedure also defines a probabilistic approach to determine the effects of variance in the approach speeds, during different periods of congested and free-flowing traffic conditions, on the length of the phase-change interval. The phase-change interval as defined for purposes of this study relates to either a yellow interval by itself or yellow interval followed by an all-red clearance interval. The results of this assessment procedure may enhance the practice of designing signal change intervals to minimize dilemma and option zones.

The objective in the development of this model was to assess the probability that a driver will be caught in either a dilemma zone or option zone on an intersection approach during a phase-change interval. The method developed here is an attempt to imitate the probability that a vehicle will approach an intersection at a given speed, and also to generate the probability that the driver will encounter a dilemma/option zone during his approach. Based on these probabilities, a combined probability will result which estimates the probability that a driver will be free of any zonal problems. The assessment model process flowchart is shown in FIGURE I-1. Using this estimated probability, transportation engineers will be able to better assess the extent of the problems given various timings for phase-change intervals. This approach will more easily allow for minimization of these types of problem areas.

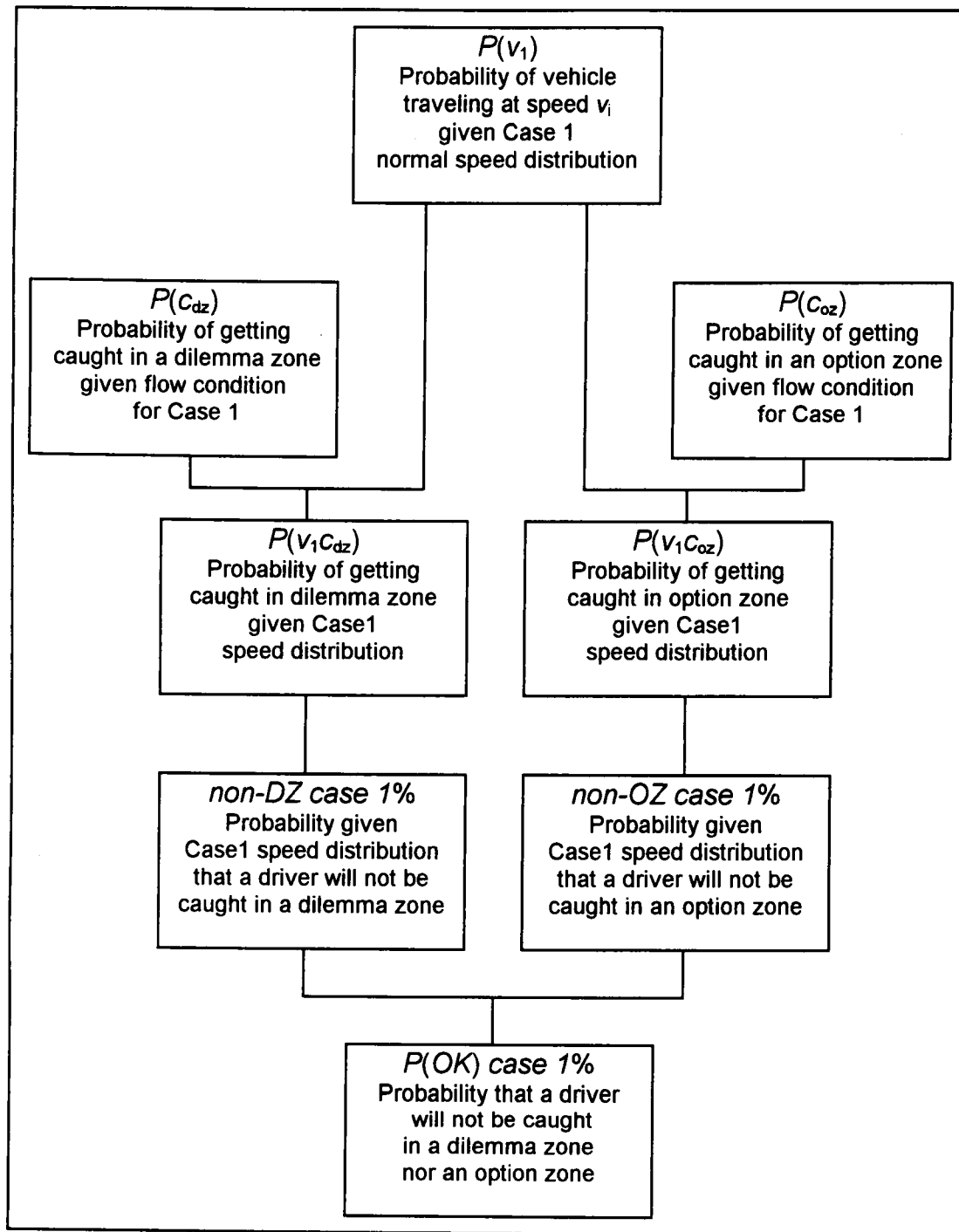


FIGURE I-1

Assessment Model Development Flowchart

**Shown for Case 1 Speed Distribution Only*

CHAPTER V

LITERATURE REVIEW

The search for literature on the determination of the yellow interval unveiled a multitude of discrepancies and varied theories. Gazis et al. (9) unleashed a monster when they questioned whether a signal could be timed so that a driver would be placed in a position to where he/she could not abide by the laws of the road. This question became known as the dilemma zone. This 1959 article was the first of an abundance of articles about problems with the yellow interval. From this point forward, transportation engineers began to question other facets of the amber signal timing such as the occurrence of an option zone, the short reaction time, and stopping probabilities.

The original concept of the 'dilemma zone' was first presented on November 27, 1959 by Denos Gazis, Robert Herman, and Alexi Maradudin (9) in a paper entitled "The Problem of the Amber Signal Light in Traffic Flow." The authors explained the dilemma zone problem as follows:

"when confronted with an improperly timed amber light phase a motorist may find himself, at the moment the amber phase commences, in the predicament of being too close to the intersection to stop safely or comfortably and yet too far from it to pass completely through the intersection before the red signal commences."

The authors felt, that under the current law, this situation could exist. The amber signal light law, current to 1959, was defined in the *Uniform Vehicle Code* of 1956 which states:

“Vehicular traffic facing the signal is thereby warned that the red or ‘Stop’ signal will be exhibited immediately thereafter and such vehicular traffic shall not enter or be crossing the intersection when the red or ‘Stop’ signal is exhibited.”

Gazis et al. set out to determine whether current criteria for setting the amber phase actually presented such a “dilemma zone” problem to motorists, and if so, to consider the function of the approach speed on the problem. An investigation of all of the mathematical relationships was completed assuming constant acceleration (a_1), constant deceleration (a_2), constant approach speed (v_0 equal to the posted speed limit), effective width of the intersection (w), length of the car (L), and the duration of the amber phase (τ). A graphical explanation of the findings are presented below. These intersection elements are shown in FIGURE II-1. Gazis et al. determined that the driver must decide between two alternatives: (1) stop at the intersection, or (2) go through the intersection. The mathematical equations for these two alternatives can be found below:

$$(1) \left(x - v_0 d_2 \right) \geq \frac{v_0^2}{2a_2}$$

$$(2) \left(x + w + L - v_0 d_1 \right) \leq v_0 \left(\tau - d_1 \right) + \frac{a_1}{2} \left(\tau - d_1 \right)^2$$

A graphical explanation is presented in FIGURE II-2. Each shaded area represents one of the alternatives facing the driver; either (1) to stop at the intersection or (2) to go through

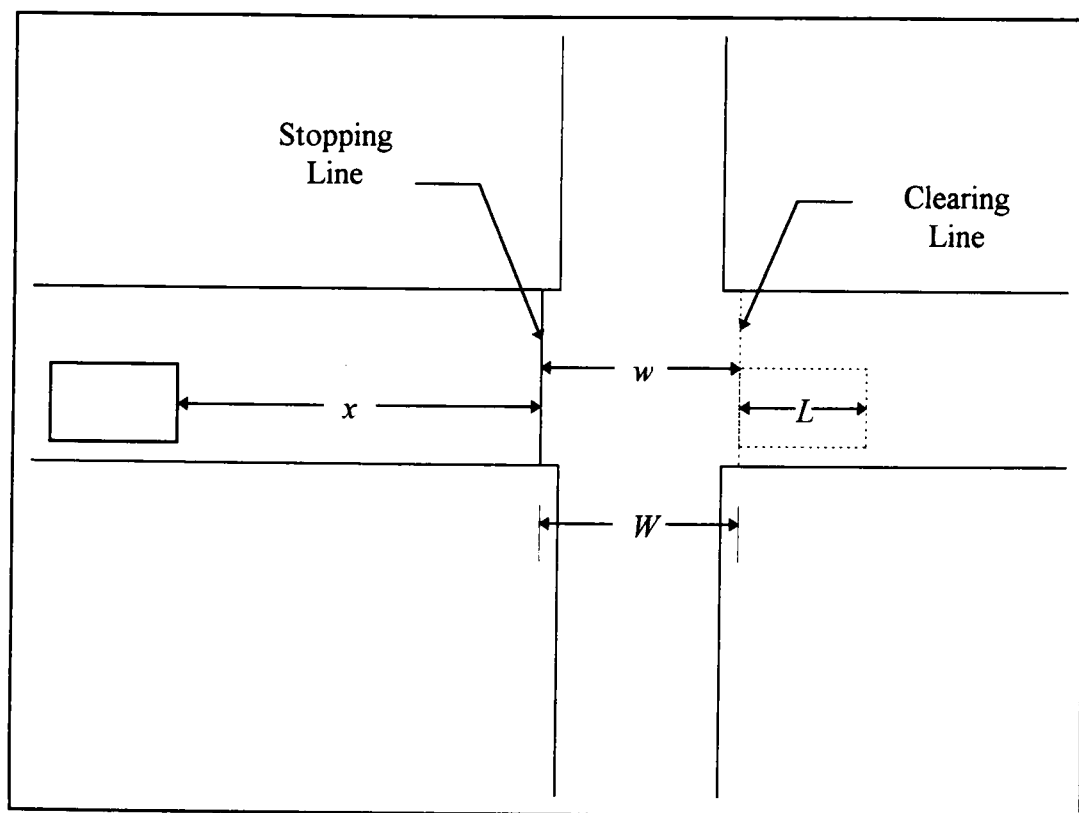


FIGURE II-1

Intersection Elements

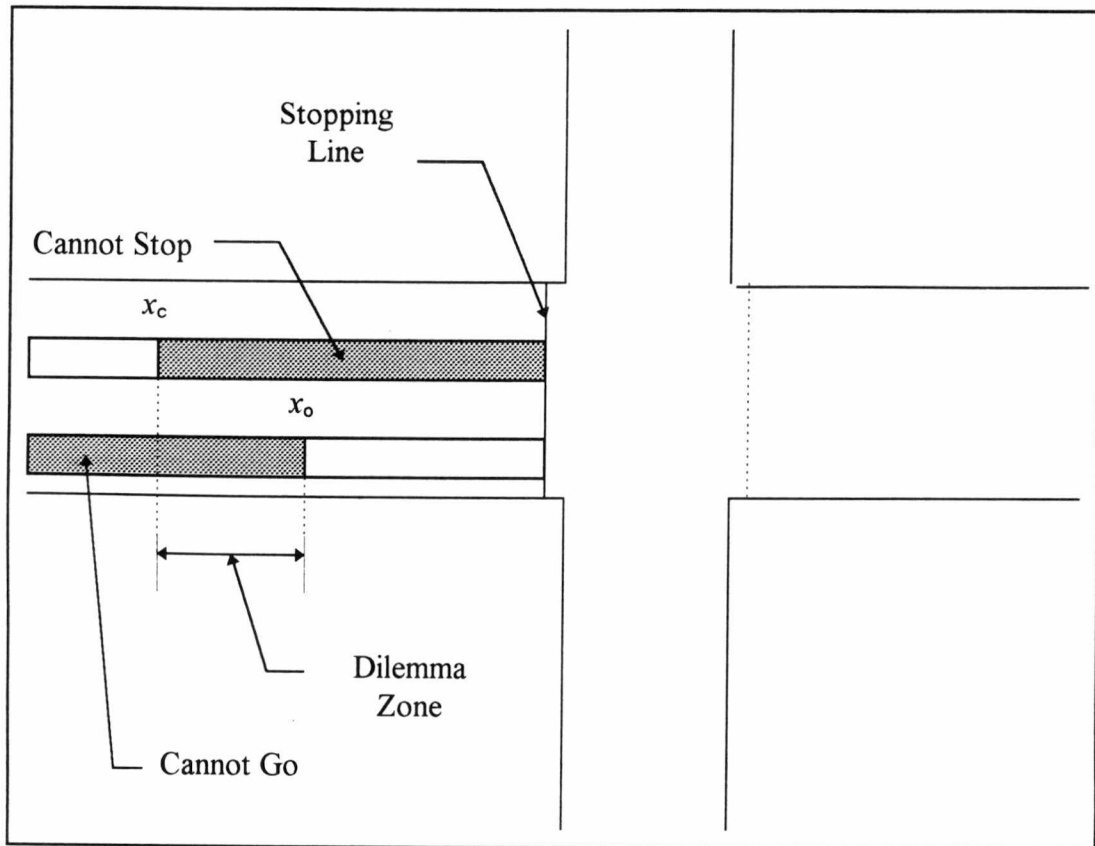


FIGURE II-2

Diagram of Dilemma Zone

the intersection. A driver at a distance from the stopping line less than x_c cannot stop safely, and a driver at a distance from the stopping line greater than x_o cannot go through the intersection without accelerating. When $x_o < x_c$, the driver could be caught in a dilemma zone if he is within the region $x_o < x < x_c$. It was noted that this situation may or may not be incurred if the driver is traveling below a constant speed equal to the posted speed limit. They also determined, through mathematical relationships, that the minimum length of the amber phase allowing safe execution of either of the alternatives corresponds to $x_o = x_c$. Therefore, the minimum amber phase was given as:

$$\tau_{\min} = \delta_2 + \left(\frac{v_0}{2a_2^*} \right) + \frac{(w + L)}{v_0}$$

where,

v_0 = approach velocity (ft/sec)

a_2^* = maximum deceleration rate

(assumed as 1/2 g to 1/3 g, or 16 to 10.7 ft/sec²)

g = gravitational acceleration (16 ft/sec²)

δ_2 = change in time after beginning of amber

(reaction time lag of driver car complex as well as decision making time of driver or 1.14 sec)

w = effective width of intersection (stop line to clear line in ft)

L = length of car (assumed to be 15 ft)

τ = duration of amber phase (sec).

In order to back up their theoretical claims, Gazis et. Al. studied 70 intersections and found that only 1 had an amber phase long enough to prevent a sizable dilemma zone.

They also noted that an average of about one car per cycle passed through the beginning of the red light phase at most intersections. The authors caution that the 1956 *Uniform Vehicle Code* assumes signals have been set so that drivers can abide by the law. The practice of the time as stated in *Traffic Engineering* by Matson et al. (18) says that a yellow duration of about 3 sec for urban and 5 sec for rural locations are probably appropriate. With this in mind, the authors stated that they "*believe that it is the duty of the traffic engineers and the drafters of the traffic ordinances to present the average, honest, driver with a solvable decision problem.*" Their paper was concluded with the two following alternatives:

1. *Design the amber-light phase according to some realistic criteria in order to guarantee that a driver can always be in a position to obey the law.*
2. *If the amber-light phases are to be kept short relative to criteria such as determined herein, it may be desirable to state the vehicle code in such a way as to make it compatible with the driver, car, road, and signal characteristics.*

In 1962, the Uniform Vehicle code was modified from a restrictive amber law to a permissive amber law. The permissive law was stated as follows:

"Vehicular traffic facing a steady yellow signal is thereby warned that the related movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the

intersection."

The permissive amber law therefore allows drivers to enter after the onset of amber and clear during the red phase. The clearance on red is supported by the *Uniform Vehicle Code* definition of a steady green indication which follows (3):

"Vehicular traffic facing a circular green signal may proceed straight through or turn right or left unless a sign at such place prohibits either such turn. But vehicular traffic, including vehicles turning right or left, shall yield the right of way to other vehicles and to pedestrians lawfully within the intersection or an adjacent crosswalk at the time such signal is exhibited."

As with most change, people were slow to realize and adopt the change in the steady yellow indication law. Thus a substantial amount of research was done on the concept of a law that no longer existed. Such research pertaining to the dated "clearance" interval, indicative of the restrictive law, will not be provided herein as it has no value to the current research.

Six years after the change, in 1968, Adolf May wrote an extensive paper (15) entitled "Clearance Intervals at Traffic Signals." One very distracting point made by May was the disagreement on amber law. From a survey, he found that 14 states had permissive laws that permit vehicles to legally enter the intersection at any time during an amber light, 27 states had restrictive laws and required vehicles to completely clear the intersection before the amber light ended, and 8 states were permissive in that they allowed vehicles to enter

on the yellow indication, but restrictive in stating that the vehicle must clear before the red. This discrepancy was with no doubt due to the 1962 change in the *Uniform Vehicle Code* (3) which allows vehicles to enter the intersection at any time prior to the termination of the amber phase. His findings also showed that most states still limited the amber phase to 3 to 5 seconds, which was appropriate in abiding by the *Manual of Uniform Traffic Control Devices* (17). In an attempt to provide a safe driving environment, states began to use of a variety of signal phasing techniques ranging from an all-red phase to a green-amber phase of 3 seconds followed by a pure amber phase of 2 seconds. The discrepancies followed by the change of laws led May to do his own theoretical analysis of amber duration according to the type of law used. He also wanted to determine for what conditions a dilemma zone might exist. When the law changed from restrictive to permissive, it was found that the intersection width need not be taken into consideration in the determination of the amber duration. However, most traffic engineers added an all-red phase to allow for safe clearance of vehicles through the width of the intersection. May looked at this change by dividing the formulas into two cases:

Case (1) Restrictive - a vehicle must either stop before entering the intersection or must clear the intersection before the end of the amber phase (this corresponds to the work done by Gazis et al. (9))

Case (2) Permissive - a vehicle must either stop before entering the intersection, or must have entered the intersection before the end of the amber phase.

The following mathematical formulas describe the above cases:

Case (1)

$$\text{stopping distance, } x_s \geq \frac{v_0^2}{2d} + v_0 t_2$$

$$\text{clearing distance, } x_c \leq v_0 T + \frac{a}{2} (T - t_1)^2 - W$$

Case (2)

$$\text{stopping distance, } x_s \geq \frac{v_0^2}{2d} + v_0 t_2$$

$$\text{clearing distance, } x_c \leq v_0 T + \frac{a}{2} (T - t_1)^2 - L$$

where, x_s, x_c = distance from stop line when the amber phase begins, (ft)

v_0 = approach speed (ft/sec)

a = acceleration rate (ft/sec²)

d = deceleration rate (ft/sec²)

W = gross width of intersection (effective width plus L , ft)

L = length of vehicle (ft)

T = duration of amber phase (sec), and

t_1, t_2 = reaction times to accelerate or decelerate respectively (sec).

Given these stopping and clearing criteria, May determined that you can have 3 different conditions:

1. $x_s > x_c$: A dilemma zone exists within which the driver could neither stop safely nor clear the intersection. This condition is more pertinent to the restrictive law.

2. $x_s = x_c$: The dilemma zone in this case is non-existent.
3. $x_s < x_c$: An option zone exists within which a driver can choose between stopping and clearing the intersection.

The dilemma zone condition follows exactly as depicted by Gazis et al. (9). The conditions of the dilemma and option zones are shown in FIGURE II-3. In the figure, x_c and x_s are defined as:

x_s = the critical stopping distance from the intersection

x_c = the critical clearing distance from the intersection.

An option zone exists when $x_s < x_c$ and a driver can choose between stopping and clearing the intersection. May thought that the condition (1) "dilemma zone" could be eliminated using the following equations:

$$T_{permissive} = t_2 + \frac{v_0}{2d} + \frac{L}{v_0},$$

assuming stop line clearance without acceleration,

$$\text{or, } T \geq t_1 + \frac{v_t - v_0}{a},$$

assuming the driver approaches the intersection at less than the speed limit and the speed limit v_t is obtained *within* the intersection and $a = 16 - 0.145v_0$

$$\text{or, } T \leq t_1 + \frac{v_t - v_0}{a}$$

assuming the driver approaches the intersection at less than the speed limit and the speed limit is obtained *outside* the intersection and $a = 16 - 0.145v_0$. The equation for the acceleration (a) was derived by May (9) from experimental data regarding the acceleration

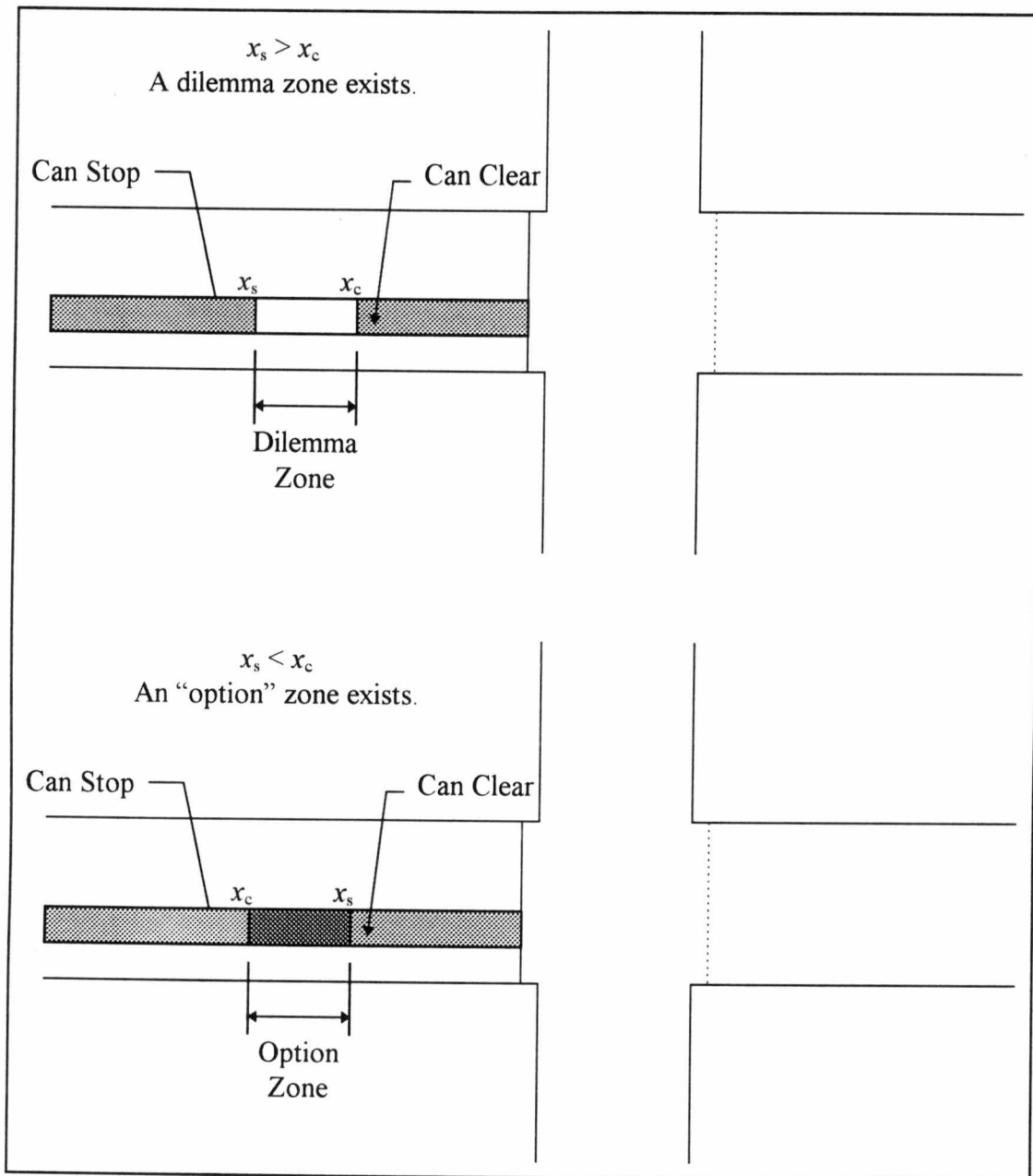


FIGURE II-3

Diagrams of Dilemma and Option Zones

of high powered modern cars.

May experimented with his theories under controlled conditions with the clearance interval set to state specifications and also with the clearance interval extended in order to minimize the dilemma zone. In order to analyze drivers reactions exclusive to the amber phase, May screened out incidents of drivers reacting to left turners and heavy traffic conditions. The study shows that in both rural and urban settings, an increase in amber duration causes a slight increase in the number of drivers operating in an unsafe or risky manner.

“Driver Behavior During the Yellow Interval” written by William Williams (26) in 1977 attempted to determine driver characteristics during the yellow interval while also examining the drivers ability to stop. Williams feels that there are two basic considerations in determining amber duration: “when a driver will choose to stop, and when a driver will choose to go through an intersection.” As such, Williams developed a method using a probability of stopping function taking into consideration distance and approach velocity. His formula of preference for setting yellow time is:

$$Y = R + \frac{v}{2a^-} + \frac{w + L}{v} - \left[K + \sqrt{\frac{2d}{a^+}} \right]$$

where, R = driver decision and reaction time (1.1 sec)

v = 85th percentile approach speed

a^- = deceleration accepted 85 percent of the time (6.5 ft/s²)

w = distance from stop line to the clear line

L = length of vehicle (17 ft for automobiles)

K = reaction time of cross-flow traffic (0.4 sec)

d = distance between vehicles and cross flow traffic, and

a^+ = maximum acceleration of cross flow traffic (16 ft/s²).

The $\left[R + \frac{v}{2a^+} \right]$ term in the above equation should be set from site specific probability of stopping vs. time charts. Thus, engineers can standardize the yellow interval for their own cities by conducting field tests to determine driver characteristics in their own areas. He does caution the use of cross-flow acceleration time lag if light jumping may be possible. The yellow computed here is time for clearance which can be provided by both yellow and all red phases.

Starting around the mid-70's, Zegeer, and Parsonson et al. (1, 4, 21, 22, 32) completed major research in the design and use of advanced detector-controllers to eliminate the dilemma zone. The detectors were designed mainly for use at intersections where approach speeds are greater than 35 mph, and where drivers usually face an appreciable dilemma zone. When a motorist is caught in a dilemma zone, a decision to stop abruptly may end in a rear-end collision, and a decision to go on through may result in a right-angle collision. If an actuated controller instead of a pre-timed controller is in place, engineers can design the detection so as to minimize the dilemma zone. The actuated system can detect vehicles before they approach the dilemma zone and either extend the green time or

provide adequate stopping distance for the drivers. While these systems have proven to work, they are not fool-proof, and a vehicle can still be caught in a dilemma zone when arriving at a signal which has reached gap-out of the green interval. For this reason, and because there are still numerous pre-timed signals on our roads, the search continues for a better understanding of the amber signal and the dilemma zone.

Up to this point, most studies proposed the use of formulae that relied on approach speeds, distance from the stop line, and intersection geometrics to determine proper yellow intervals for reduction of dilemma zones. In 1980, Stimpson, Zador, and Tarnoff (28) proposed to look at the potential conflict which results from being caught in the dilemma zone. In order to try and reduce the potential conflicts, they felt it would be necessary to change the duration of the yellow signal. They set out to determine driver response to the yellow onset for varying traffic and pavement conditions and yellow signal duration. The traffic conditions varied from peak to non-peak and pavement conditions varied from wet to dry. The experiment by Stimpson et al. shows that an increase of yellow time by 30% almost eliminates potential conflicts at one site while a 32% increase in yellow at another site reduces conflicts by 75% (the percentage of potential conflicts being the percent of last to clear vehicles that spent at least 0.2 seconds in the intersection past the red onset). The study therefore shows that the increase of yellow duration reduces the number of potential conflicts. The number of potential conflicts were also shown to be highest in peak traffic conditions and also in dry weather. The authors note that the conflict rates do not change in the same direction as approach speeds, but feel that

conflict rates should be considered in the overall determination of the appropriate yellow phase. It is noted that there must be some trade off between operational efficiency and reduction in potential conflict with the use of this method to determine a safer yellow interval. "...conflicts are dependent not only on intersection geometry and travel speed, but on traffic density and possibly pavement condition. The precise nature of this dependence remains to be determined (28)."

Authors Bissell and Warren wrote a paper in 1981 entitled "The Yellow Signal is NOT a Clearance Interval" (3). After studying the results of an FHWA survey, they found that the majority of traffic engineers were still considering the yellow signal as a clearance interval. Most of these engineers are following practice found in the Institute of Transportation Engineers (ITE) 1976 *Transportation and Traffic Engineering Handbook* (30) which is based on the 1960 work of Gazis et al. Even though the majority of states have changed their laws to coincide with the 1962 *Uniform Vehicle Code* allowing vehicles to legally clear the intersection during the red signal, traffic engineers still design for cars to clear before the onset of red. Bissell and Warren feel that the problem of the dilemma zone as stated by Gazis (9) is virtually eliminated by the use of the new definitions from the Uniform Vehicle Code. They assume that the start up delay of the opposing traffic will allow enough time for motorists to clear after the onset of red for their movement. It may seem that this practice may result in increased right angle collisions, however, the authors claim it to be safe if people abide by the definition of a green phase in the *Uniform Vehicle Code*:

"Vehicular traffic facing a circular green signal may proceed straight through or turn right or left unless a sign at such a place prohibits either such turn.. But vehicular traffic, including vehicles turning right or left, shall yield the right of way to other vehicles and to pedestrians lawfully within the intersection or an adjacent crosswalk at the time such signal is exhibited."

Note that up to this point, most research studies followed the thought that the yellow interval was the clearance interval and that all cars must be cleared from the intersection before the onset of red. Bissell and Warren use equations for stopping distance and travel time to the stop-line in their determination of an appropriate yellow interval. The stopping distance (S_s) is therefore:

$$S_s = Vt_r + \frac{V^2}{2d}$$

where, d = deceleration rate (10 ft/sec²)

t_r = perception reaction time (1 sec)

V = speed of approach vehicle (ft/sec)

Thus, the minimum yellow time (t) can be found by:

$$t = \frac{S_s}{V}$$

In closing, the authors stated:

"The legal concept, where traffic may clear at the beginning of the red signal phase as indicated in the Uniform Vehicle Code, should alleviate the fears of many traffic engineers and researchers who are concerned

with the dilemma zone associated with the yellow interval."

In 1983, Robert M. Shanteau (25) took up the discussion of reasonable drivers in his paper "Considerations in the Length of the Yellow Interval." He notes that the ITE Handbook gives reasonable driver values of one second for reaction time and 10 (or 15) ft/sec² for deceleration rate. Shanteau's main disagreement with previous research is the assumption of *constant* deceleration and *constant* reaction time for all speeds. In a look at the current procedure, anyone driving within the 85th percentile speed is considered a reasonable driver. Shanteau holds the belief that the design driver should be raised to the 90th percentile driver. His method for determining the yellow interval uses stopping probability curves. The curves are converted from percentage of stopping vs. distance to stop-line to percentage of stopping vs. time to stop-line. The yellow time is then picked right off the plot for a 90 percent stopping probability. Shanteau used pre-published stopping probability curves to analyze his method. Unfortunately, he was not able to make any concise conclusions due to the erratic behavior of the data. His results pointed toward the failure of the simple kinematic model of vehicle motion for this situation. His suggestion was to develop stopping probability curves by observing cars that do and do not stop. Shanteau notes that his method of determining the yellow interval might allow researchers to get around the problem of assuming constant reaction times and constant deceleration rates.

In almost all yellow interval research, the perception reaction time has been considered

equal to one second. Hooper and McGee (10) studied driver characteristics in 1983 and found that a one second reaction time was inadequate for use in the determination of the change interval. Earlier studies done to determine the alerted-brake reaction time did not take into account decision time or perception time. It should be noted that drivers will not brake instantaneously at the onset of yellow. Drivers must acknowledge traffic conditions, approach speed, and distance to the intersection before making a decision to apply brakes and stop or to continue through the intersection. Taking these factors into account, Hooper and McGee empirically determined values for perception-brake-reaction time as shown in TABLE II-1.

TABLE II-1
Perception-Brake-Reaction Times

	Perception-Brake-Reaction Time (s)
50th	1.13
85th	1.77
95th	2.16

Their values correspond to values found in the field by Wortman and Matthias of 1.13 sec for the 50th percentile, and 1.80 for the 85th percentile. The effect of increasing the perception reaction time on the change interval is 1:1. The authors end by noting criticism of the currently used short perception reaction time for change intervals. They recommend that there should be an effort to scientifically determine a more correct representation of the perception reaction time for use by engineers nationwide.

Zador et al. (31) studied the "Effect of Clearance Interval Timing of Traffic Flow and Crashes at Signalized Intersections" in 1985. Here, the authors refer to the clearance interval as the yellow plus all-red phases. The authors noted that earlier work by Gazis et al. (9) "*suggests that the proportion of drivers at risk of a crash can be reduced to zero for drivers traveling at similar speeds with similar perception reaction times and similar deceleration rates.*" Zador et al. looked at traffic flow and crash data for 91 intersections in eight metropolitan areas. The intersections represented diverse parameters including:

- yellow signal laws (allowed to enter vs. stop on yellow)
- average approach speed (35 to 55 mph)
- cross-street width (20 to 124 ft)
- yellow phase duration (2.8 to 5.7 sec)
- all-red phase duration (0 to 3 sec)

Information on these parameters along with the average daily traffic (ADT) for the monitored street and its' ratio with cross-street ADT were used to statistically sort the intersections into eight relatively similar clusters. The eight intersection clusters were then ranked in ascending order of crash rates and further grouped by similar crash rates. The final set of five overlapping clusters was then analyzed. In their study, they found that the timing of the change interval had an affect on accident rates at the signalized intersections. Higher crash rates were found among clusters with less adequate clearance intervals. The highest crash rate cluster was found to have an average implied deceleration of 13 ft/sec², whereas the lowest crash rate cluster had a deceleration rate of 8.5 ft/sec². Crash rates were shown to increase with an increase in ADT on the cross-street. Also, the highest

crash rate cluster had wider cross-streets and slower monitored street speeds, which leads to increased crossing time to clear the intersection.

“Our interpretation of the overall pattern of association between intersection characteristics, clearance intervals, traffic flow, and crash rates is that the increasing deficiency of clearance interval timing sharply increases the proportion of drivers who enter intersections and do not clear them during the clearance interval. The consequent reduced separation of the two traffic streams and increased braking by drivers who do not want to enter the intersection without protection from cross-street traffic leads to substantial increases in crashes.”

The authors stressed that the inadequacy of clearance interval timings shown in this study and in previous studies continues to prove that there is a need for further research into clearance interval timing.

“Setting Change Intervals at Signalized Intersections” was written by Yupo Chan (5) and Thomas Liao in 1987. Again, the change interval, in this case, is defined as yellow plus all-red. In their paper, they derive the equation for the change interval timing (Y) for a level road as follows:

$$Y = \frac{V}{2fg} + \frac{w + L}{V} + t_r$$

where, V = prevailing speed of roadway (constant)

w = width of cross-street

L = typical vehicle length (constant)

f = coefficient of friction (constant)

g = gravitational acceleration (constant)

t_r = driver reaction time

The assumptions that are built into the formula, as pointed out by the authors, are constant deceleration and constant approach speed. They have shown this equation slightly different than the one proposed by the ITE Handbook (30) in order to show the elements of brake/tire/road interaction involved in stopping a vehicle, and to emphasize the “fudge factor” (t_r) reaction time. Chan and Liao wrote a BASIC program to analyze the behavioral processes that a driver goes through at an intersection. The computer program either measures the simulated brake reaction time, or determines if an accident has occurred in the process of clearing. The program can also determine stop zone (S) and the go zone (G) from variable input done by the user. The equations used for calculating the stop and go zones are given as:

$$S = \frac{V^2}{2fg} + Vt_r, \text{ and}$$

$$G = V \left(Y - \frac{w + L}{V} \right).$$

The authors also discuss the problem of a dilemma zone (D), which is given as:

$$D = S - G, \text{ or}$$

$$D = \frac{V^2}{2fg} + Vt_r - V \left(Y - \frac{w + L}{V} \right).$$

It is easy to understand that the intersection should be designed so as to get $D=0$.

However, Chan and Liao caution that optimization of the change interval using this equation could still result in an incorrect design. It also appears to make sense why engineers still have no consensus as to how to determine the correct change interval when D is a function of so many variables:

$$D(V, f, Y, t_r, w \dots) = 0$$

There are also functions of variables inside of variables, consider friction (f). Friction is a function of driver behavior, weather, pavement type, etc. The authors used the computer program to analyze 18 signals in Washington state. It turns out that all but one signal had an inadequate change interval. This finding was significant, because the simulated reaction time is generally shorter than the one second recommended by ITE. In their analysis, they found out that as long as a “lump sum” reaction time is used in change interval formulas, there would never be an elimination of the dilemma zone as shown in the following equation:

$$D = S - G = Vt_m, \text{ or}$$

$$D = \frac{V^2}{2fg} + \frac{t_v + t_m}{V} - Vt_v - \frac{V^2}{2fg} = Vt_m$$

where t_r is broken down into components of

t_v = visual perception time

t_m = muscle-response time

The authors also recommend using deceleration rate determined from site specific parameters. This recommendation is far from current practice where constant values for

many variables are used to determine the change interval.

Lin, Cooke, and Vijayakumar (13) studied the "Utilization and Timing of the Signal Change Interval" in 1987. Again the change interval is defined as being the yellow plus all-red if any all-red is used. Methods of determining the change interval vary from agency to agency, but most use the basic formula suggested in the 1982 *ITE Transportation and Traffic Engineering Handbook*. In this study, the yellow interval requirement was defined as the "*length of a yellow interval that will allow a specified percent of signal change intervals to be free of vehicles entering the intersection on a red signal indication.*" The authors note that traffic conditions may change rapidly at a signal due to queuing of vehicles. The approach speeds in this case are also affected by traffic. From the study of five intersections and eight movements, a 95 percentile yellow interval requirement varied from three to five seconds. This variation cannot be explained by the variation in approach speed for this study. This may be due to the fact that most of the data were gathered during the peak traffic hours, where long queues would develop immediately after the change interval ended. Lin et al. believe that vehicle supply at the onset of the yellow interval is a large factor in the requirement of the yellow interval. Previous research methods of determining the change interval were analyzed and regression analysis was performed on each. However, their work proved little meaning with the resulting low R^2 values. Hence, the authors prescribe their own formula for determining the yellow interval (Y) until a consensus can be made. Their formula for satisfying the 95th percentile requirements is as follows:

$$Y = 4.0 + C_1$$

where, C_1 = correction factor between -1.0 and +1.0 seconds.

A value between +0.5 and +1.0 seconds may be chosen for C_1 if one of the following vehicle supply patterns exists:

- a) frequent carryovers of long queues from one cycle to the next; or
- b) a rapid build-up of long queues immediately after the change interval expires;
or
- c) the percent of change intervals utilized exceeds 70 percent.

On the other hand, a value of C_1 would be between -1.0 and -0.5 seconds if:

- a) the movements of concern have low flow rates and are regulated by traffic actuated controls; or
- b) the vehicle supply to the intersection at the yellow onset is frequently cut off due cyclic flow patterns created by signal coordination; or
- c) the rate of change interval utilization is less than 30 percent.

Vehicle patterns between these two extremes can use a C_1 value of 0.

Authors Joseph Prashker and David Mahalel (23) try to interpret behavioral patterns of humans in their paper "The Relationship Between an Option Space and Drivers' Indecision at Signalized Intersection Approaches" in 1989. They start by looking at the function of the signal. Ideally, the traffic signal should read either stop or go, but the driver cannot instantly change to stop or go. Therefore, the signal has been designed to have a transition mode...the change interval. The driver cannot stop instantly for two

reasons: (1) it takes time and energy to stop a car once it is in motion, and (2) it takes the human driver a certain amount of time to perceive the signal and then to take the appropriate action. Existing methods of determining the change interval use a normative, deterministic approach. The correct approach should take into consideration the dynamic variables involved. The authors of the paper wish *“to demonstrate through a behavioral model, how the risk of rear-end collisions at intersections can be theoretically evaluated.”* First they looked at the probability that the driver will cross the intersection to be equal to $[1 - P(x)]$, where $P(x)$ is the probability of stopping. This information is typically shown in a probability of stopping curve. Next, the probability that two successive drivers will have conflicting decisions about stopping was given as $P(x) * [1 - P(x)]$. This probability reaches a maximum at $P(x) = 0.5$. In order for two vehicles to have conflicting decisions, at least two vehicles must be approaching a signal at the onset of the yellow interval. The number of rear-end accidents can then be thought of as a function of traffic volume and the size of the indecision zone. Prashker and Mahalel note that option zones and dilemma zones are tools that can be used in a normative sense to determine what a driver can physically do, but in no way what he will do. Although option zones allow for drivers to stop or go through the intersection, long option zones have shown to increase the probabilities of conflicting decisions. This, in turn, creates higher probabilities of rear-end accidents. The authors came up with a probability model for this situation that follows:

$$Pr[stop] = \Phi \left[\frac{(t_i - t_{cr,i})}{\sigma} \right]$$

where, $Pr[stop]$ = probability of stopping

Φ = cumulative summation

t_i = time to stop line as calculated from constant speed

$t_{cr,i}$ = duration of the warning period for driver I

σ = standard deviation

The authors tested their behavior model at four intersections and found that when approach speeds increase, the value of t_{cr} increases while σ^2 decreases. Prashker and Mahalel thought that “the shape of the decision function is actually determined by the speed of travel of the traffic stream rather than by the speed of individual vehicles. The last explanation receives support from the fact that drivers’ decisions are based not solely on the speed and distance from the intersection, of their cars, but also on the relative speed and location of their vehicles compared to other vehicles.”

CHAPTER III

METHODOLOGY

The basic purpose of this study was to develop a procedure to estimate the magnitude of the dilemma/option zone problems. The following approach attempts to determine the probability that a driver will be caught in either a dilemma or option zone. The procedure also defines a probabilistic approach to determine the effects of variance in the approach speeds during different periods of congested and free-flowing traffic conditions on the length of the phase-change interval. The phase-change interval as defined for purposes of this study relates to either a yellow interval by itself or yellow interval followed by an all-red clearance interval. The results of this assessment procedure may enhance the practice of designing signal change intervals to minimize dilemma and option zones.

The development of this approach was based on two things: (1) the concept that drivers impose upon themselves a restrictive meaning of the yellow interval, and (2) the concept that traffic conditions and speed distributions in urban areas during congested and free-flow periods affect the determination of the phase-change interval, which ultimately determines the magnitude of dilemma and option zone problems. The concept of a self-imposed restrictive law means that drivers feel that they have to clear the intersection before the onset of red. This means that even though the law has been stated to eliminate the problems of dilemma zones for drivers, they may still experience the dilemma zone if the yellow signal is too short. For this reason, dilemma zones and option zones should be

assessed to determine the implications for those drivers which are not familiar with the current law. The second concept was based on the observation that during free-flow traffic conditions, traffic flow is generally minimal and speeds are relatively close to the speed limit. Whereas, during congested traffic conditions, traffic flow is heavy and speeds are generally much slower. The question is raised as to how these factors affect the determination of the phase-change interval, and ultimately the magnitude of the dilemma/option zone problems.

The objective in the development of this model was to assess the probability that a driver will be caught in either a dilemma zone or option zone on an intersection approach during a phase-change interval. The approach developed here is an attempt to imitate the probability that a vehicle will approach an intersection at a given speed, and also to generate the probability that a driver will encounter a dilemma/option zone during his approach. Based on these probabilities, a combined probability will result which estimates the probability that a driver will be free of any zone problems. Using this estimated probability that a driver will be free of any dilemma or option zones, transportation engineers will be able to better assess the extent of the problems for various lengths of phase-change intervals. This approach will more easily allow for minimization of these types of problem areas.

This chapter describes the procedures used in the present study. The sections include: key formulas and issues, model development, and a case study.

KEY FORMULAS AND ISSUES

Since the majority of traffic engineers use the recommended practice set by ITE (16) to determine the phase-change interval, this practice was also used herein to develop base equations for the following procedure. The phase change interval, as stated before, can consist of either a yellow change interval by itself, or both a yellow change interval and an all-red clearance interval. The phase-change interval is required to warn drivers of an impending change in right of way. States which follow a permissive yellow signal interval law only have to allow time for vehicles to come to a stop or enter the intersection before the onset of red. The equation

$$Y = t + \frac{v}{2d} \quad (1)$$

where, Y = yellow change period, sec

t = perception-reaction time (usually 1 sec), sec

v = approach speed, ft/s

d = deceleration rate, ft/s²

reflects the time it takes for a vehicle to stop, which is a representation of the minimum time setting for a yellow change interval. Vehicles in the intersection at this time can legally clear the intersection during the beginning of red, and by law are to be given the right of way by vehicles facing the green indication.

Some states use a red clearance interval to allow traffic to clear the intersection before opposing traffic receives the green signal. This interval is recommended by ITE to have a

maximum value of

$$r = \frac{w + L}{v} \quad (2)$$

where, r = red clearance interval, sec

w = width of intersection, ft

L = length of vehicle, ft

v = approach speed, ft/s.

The total phase change interval (PC) is thus a combination of yellow plus all-red or simply yellow alone. The yellow plus all-red or phase change timing is determined by the following equation:

$$PC = t + \frac{v}{2d} + \frac{w + L}{v} \quad (3)$$

The *Traffic Control Devices Handbook* (28) suggests extending the yellow to be equal to the length of the total phase-change interval if no red clearance interval is used. This method assumes that drivers do not utilize the full yellow change interval, but only the stopping time portion. States that do not have permissive laws, and require that vehicles stop or clear the intersection before the onset of red, must use formula (3) for setting the length of the yellow interval. This assures that drivers may abide by the law and will not be ticketed for no fault of their own.

The formulas used to determine the dilemma zone and option zone come from the basic kinetic formulas of stopping distance and clearing distance as follows:

$$\text{Stopping Distance} = X_s = v_0 t + \frac{v_0^2}{2d} \quad (4)$$

$$\text{Clearing Distance} = X_c = v_0 PC - w - L \quad (5)$$

In these equations, v_0 is the approach speed which will be used in the following research as a range of values from 0 to 120 fps. When $X_s > X_c$ a dilemma zone occurs, likewise when $X_c > X_s$, an option zone occurs. This is depicted graphically in FIGURE III-1.

MODEL DEVELOPMENT

The first step in the development of the model was to integrate the aspect of variation in speed for different times of day. Based on actual data collected by Martin Parker and Associates consulting firm, it was determined that the speed distributions approaching an intersection are normal. Therefore, this model assumes normal distributions for all speed distributions to be tested. The model allows for two cases (case 1 and case 2) of speed distributions to be studied at the same time. This will allow the user to more easily examine the differences for different times of the day or days of the week, or any other two situations that have varying speed distributions. If the user knows the mean speed and standard deviation, the normal probability curves $P(v_1)$ and $P(v_2)$ can be determined for the two speed distributions.

The second step in the model process is to determine the probabilities of getting caught in either a dilemma or option zone. This step requires that you have already calculated a phase-change timing (PC) and know the flow conditions (q) for the two speed distribution

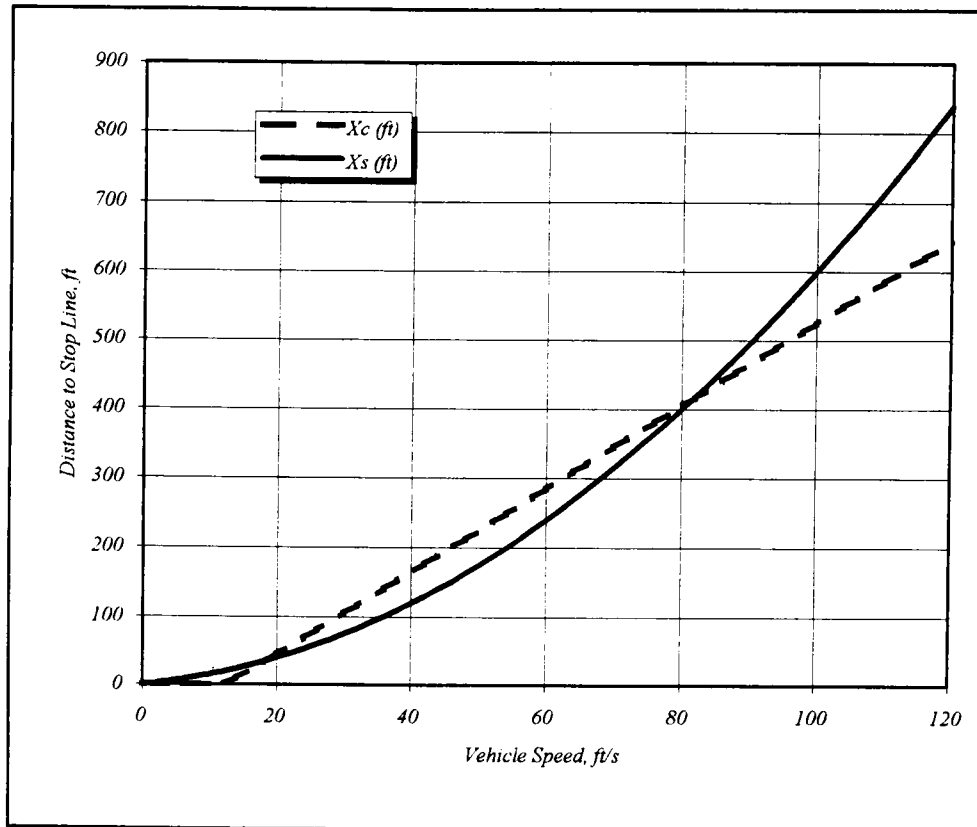


FIGURE III-1

**Plots of Stopping and Clearing Distances
and Resulting Dilemma and Option Zones**

cases. These flow values are then converted to an average headway (h) for each case by using the formula:

$$h = \frac{3600}{q} \quad (6)$$

To calculate the phase-change interval, the user must know the width of the intersection and assume a reaction time, deceleration rate, and average length of the vehicles. Given these parameters, curves can be plotted for X_s and X_c over a range speeds (v_0). From this plot, the lengths of the dilemma zone and option zone can be calculated. The resulting probabilities that a driver may get caught in a dilemma or option zone are calculated by using the following formulas:

$$\text{Probability of getting caught in dilemma zone} = P(c_{dz}) = \frac{L_{dz}}{hv_0} \quad (7)$$

$$\text{Probability of getting caught in option zone} = P(c_{oz}) = \frac{L_{oz}}{hv_0} \quad (8)$$

where, L_{dz} = length of dilemma zone, ft

L_{oz} = length of option zone, ft.

Since this procedure can be applied using any common spreadsheet program, many iterations of the model can be made quickly using different values for the phase-change interval timing.

Finally, the probabilities that a driver will get caught in a dilemma zone or option zone given the two cases of speed distributions can be determined. These probabilities are calculated by multiplying the values of $P(v_1)$ and $P(v_2)$ with $P(c_{dz})$ and $P(c_{oz})$ for each

speed v_0 . The resulting values give the curves $P(v_1c_{dz})$ and $P(v_2c_{dz})$, and also the curves for $P(v_1c_{oz})$ and $P(v_2c_{oz})$.

The overall probabilities that a driver will not get caught in a dilemma or option zone can be found by calculating the sum of the probabilities that a driver will get caught in a zone and subtracting that sum from 1. For example, the probability that a driver will not get caught in a dilemma zone given the Case 1 speed distribution and PC in seconds is:

$$\text{non-DZ Case 1\%} = 1 - \sum P(v_1c_{dz})$$

This formula can also be used to find the *non-DZ Case 2%*, *non-OZ Case 1%*, and *non-OZ Case 2%*. Users can take these percentages into consideration when designing signals to estimate what percent of drivers may be affected by a dilemma zone or option zone.

This process can be iterated time and time again using varying values of PC , t , and d to become more familiar with how each of the variables may affect the probability of a driver getting caught in a dilemma zone.

EXAMPLE CASE STUDY

The following is an example case study that was used to try and answer previous questions raised by the literature review. The assessment model was used to determine the effects of congested and free-flow traffic conditions on the probability of getting caught in a dilemma zone or option zone. The example intersection was assumed to be in an urban setting and has a crossing width of 60 feet, and an approach speed limit of 35 mph. There is no pedestrian activity in the area, and the surface street grade is negligible.

Following are assumed values for the variables that are common to all example problem calculations:

w = width of intersection (ft) fixed at 60 ft

L = length of car (ft) assumed fixed at 20 ft

t = reaction time (sec) assumed fixed at 1 sec

d = deceleration rate (ft/sec²) assumed constant at 10 ft/sec²

a = acceleration rate (ft/sec²) = $16 - 0.145v_0$

The first step in the development of the example model is to determine the speed distributions to be tested. For this example, two cases (case 1 and case 2) of normal distributions have been assumed for a 35 mph approach to an urban intersection:

- (1) Case 1 is assumed to be normal speed distribution for a non-congested traffic condition. The mean speed for the approach is 30 mph with a standard deviation of 10 mph. This distribution was chosen to depict free-flow conditions where cars generally travel near the speed limit, but are free to travel at speeds far lower or higher than the average.
- (2) Case 2 is also assumed to be a normal speed distribution, but for congested traffic conditions. The mean speed is a much slower 10 mph and the standard deviation is chosen to be 2.5 mph to show a more condensed distribution. This distribution was chosen to depict the typical congested urban setting, where vehicles are much more dependent on the speeds of the cars around them, and being constrained to little or no free-flow movement.

The plot of the distributions $P(v_1)$ and $P(v_2)$ can be found in FIGURE III-2.

The second step in the example model process is to determine the probabilities of getting caught in either a dilemma or option zone. This step requires that you have already calculated a phase-change timing for the current situation and know the flow conditions from the two speed distribution cases. Values for *non-OZ* %'s and *non-DZ* %'s will be calculated over a range of 0-8 second phase-change intervals (in 0.2 second intervals) to determine the final $P(OK)$ curve. Four phase-change settings will be examined in detail, and these are:

$PC(v_L)$ = phase-change interval determined using speed limit of 35 mph (5.0 sec)

$PC(v_1)$ = phase-change interval determined using the 85th percentile speed
determined from the Case 1 speed distribution (5.2 sec)

$PC(v_2)$ = phase-change interval determined using the 85th percentile speed
determined from the Case 2 speed distribution (6.0 sec)

$PC(Y)$ = phase-change interval equal to the minimum yellow time for a 35 mph
approach speed from the Manual of Traffic Signal Design (3.6 sec).

The values of flow (q) for the given speed distribution cases also had to be assumed since the distributions were hypothetical. If the single approach capacity is 1800 pcphpl, and capacity is reduced by $\frac{1}{2}$ at a signalized intersection (60 sec signal cycle length split 2 ways), the resulting capacity would be 900 pcphpl. Therefore one may assume that low, medium and high flow conditions might be in the ranges of 0-300 pcphpl, 300-600 pcphpl, and 600-900 pcphpl respectively. The Case 1 condition, being non-congested, was

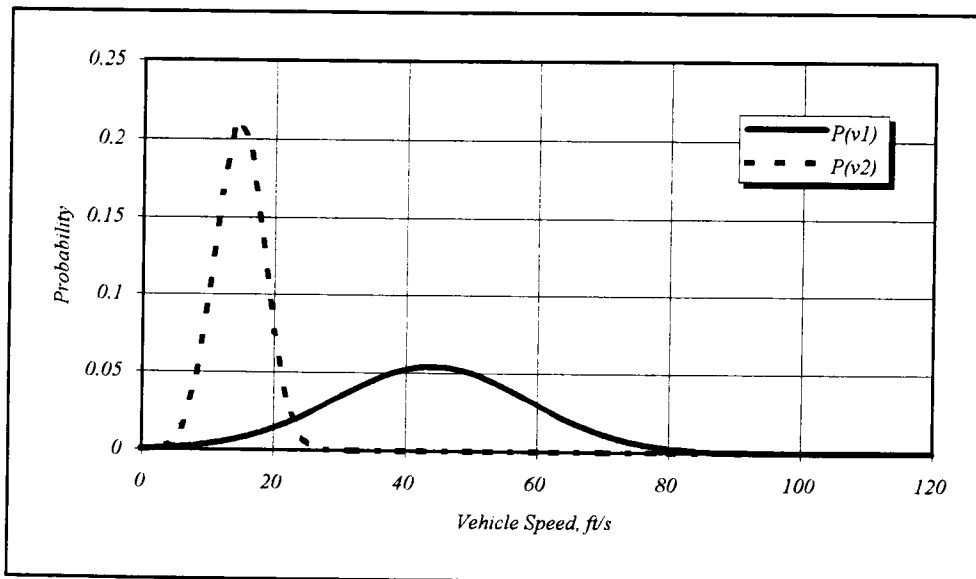


FIGURE III-2

Case 1 and Case 2 Speed Distributions

assumed to be 300 pcphpl. The Case 2 congested condition was chosen to be 750 pcphpl, which was determined from the midpoint of the high range. These flow values are then converted to an average headway for each case (h) by using the formula:

$$h = \frac{3600}{q} \quad (6)$$

Given these parameters, curves can be plotted for X_s and X_c given v_0 in the range of 1-120 ft/s as shown in FIGURE III-3. From this plot, the lengths of the dilemma zone and option zone can be calculated. An example of this graphs is shown for $PC(v_2) = 6$ sec in FIGURE III-4. The resulting probabilities that a driver may get caught in a dilemma or option zone are calculated by using the following formulas:

$$\text{Probability of getting caught in dilemma zone} = P(c_{dz}) = \frac{L_{dz}}{hv_0} \quad (7)$$

$$\text{Probability of getting caught in option zone} = P(c_{oz}) = \frac{L_{oz}}{hv_0} \quad (8)$$

where, L_{dz} = length of dilemma zone, ft

L_{oz} = length of option zone, ft.

These probabilities can be found in their graphical form for $PC(v_2) = 6$ seconds in FIGURES III-5 and III-6.

Finally, the probabilities that a driver will get caught in a dilemma zone or option zone given the two cases of speed distributions can be determined. These probabilities are calculated by multiplying the values of $P(v_1)$ and $P(v_2)$ with $P(c_{dz})$ and $P(c_{oz})$ for each

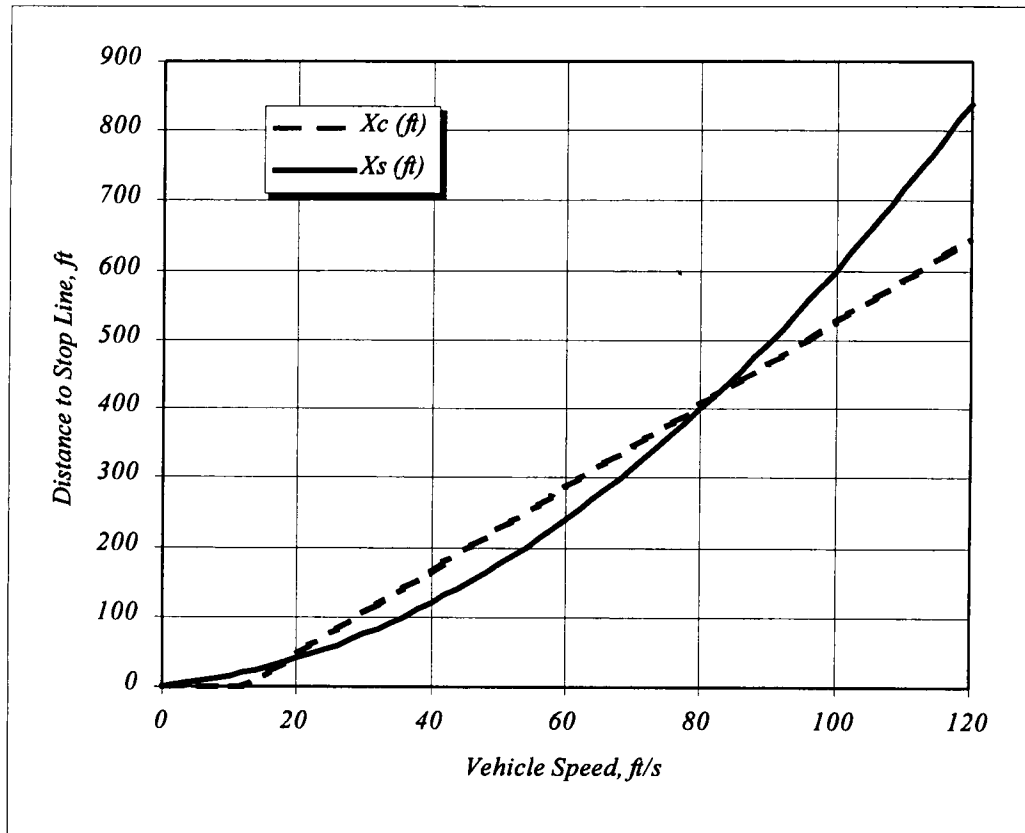


FIGURE III-3

Plot of Clearing and Stopping Distance

$$PC(v_2) = 6 \text{ sec}$$

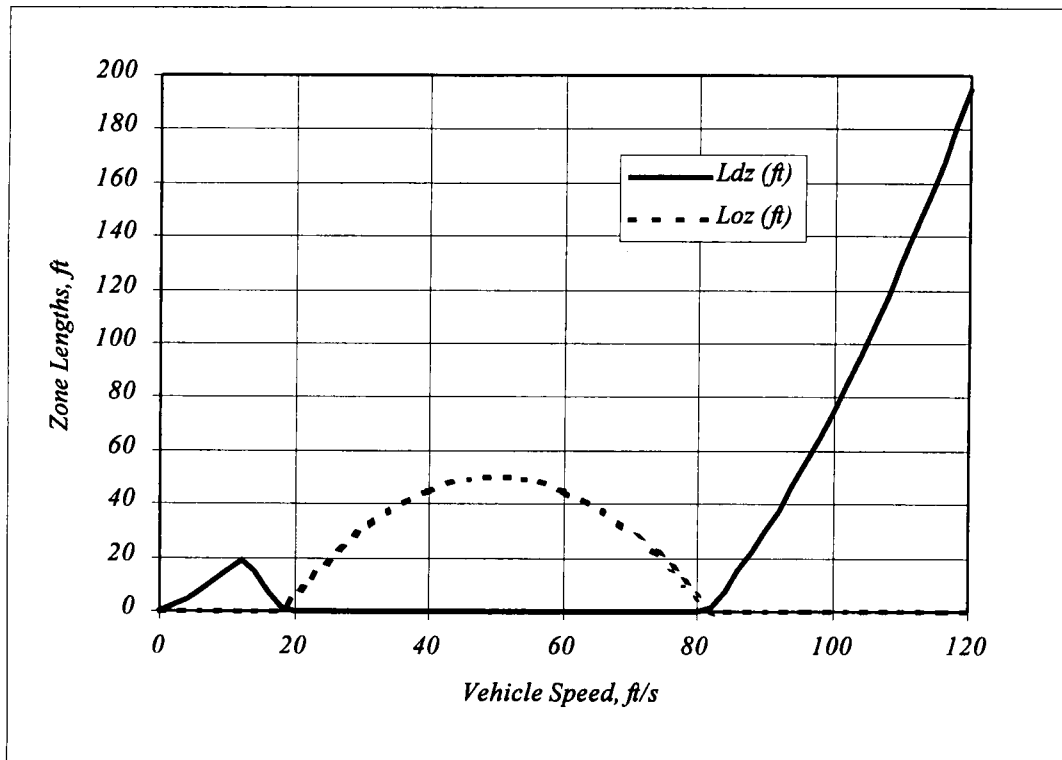


FIGURE III-4

Plot of Dilemma and Option Zone Lengths

$$PC(v_2) = 6 \text{ sec}$$

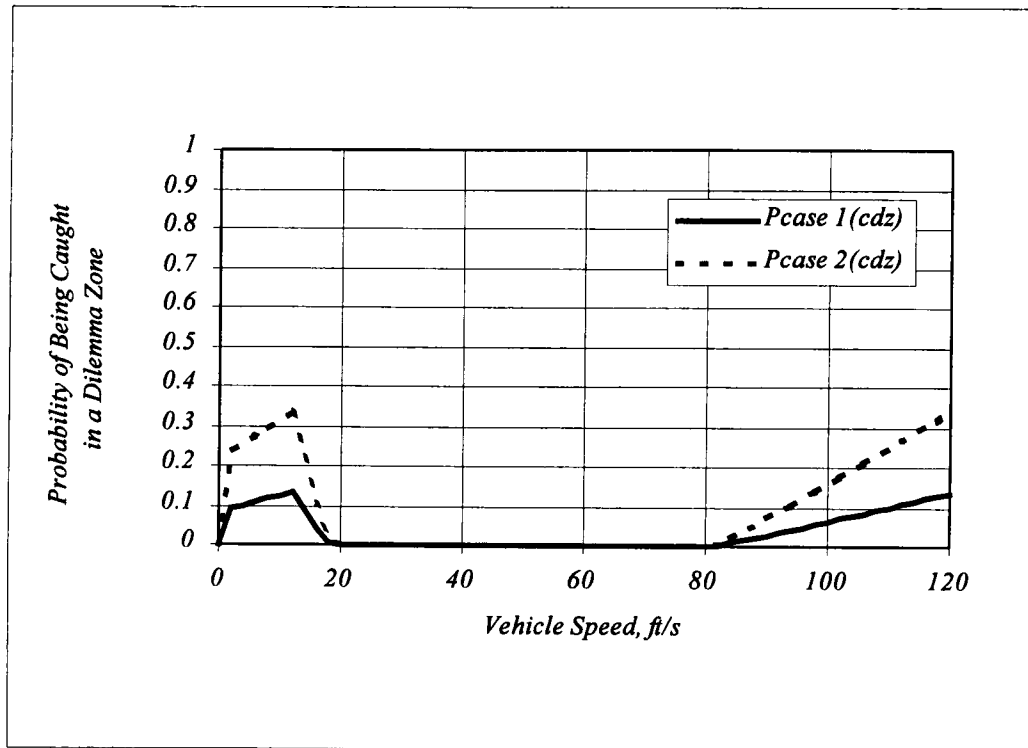


FIGURE III-5

Probability of Being Caught in a Dilemma Zone

$$PC(v_2) = 6 \text{ sec}$$

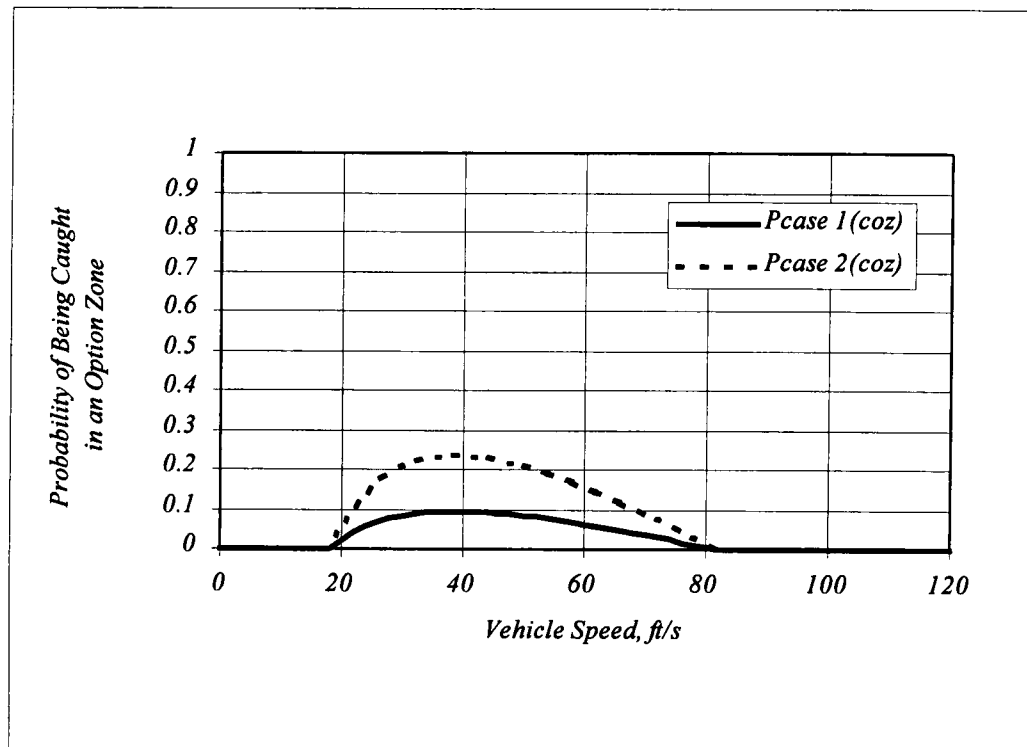


FIGURE III-6

Probability of Being Caught in an Option Zone

$$PC(v_2) = 6 \text{ sec}$$

speed v_0 . The resulting curves for $P(v_1c_{dz})$ and $P(v_2c_{dz})$ can be found in FIGURE III-7, and the curves for $P(v_1c_{oz})$ and $P(v_2c_{oz})$ can be found in FIGURE III-8.

The overall probabilities that a driver will not get caught in a dilemma or option zone can be found by calculating the sum of the probabilities that a driver will get caught in a zone and subtracting that sum from 1. For example, the probability that a driver will not get caught in a dilemma zone given the Case 1 speed distribution and $PC(v_2) = 6$ seconds is:

$$\text{non-DZ Case 1\%} = 1 - \sum P(v_1cdz) = 99.7\%$$

All of the results from this iteration are listed below:

$$\text{non-DZ Case 1\%} = 99.7\%$$

$$\text{non-DZ Case 2\%} = 82.78\%$$

$$\text{non-OZ Case 1\%} = 92.71\%$$

$$\text{non-OZ Case 2\%} = 99.12\%.$$

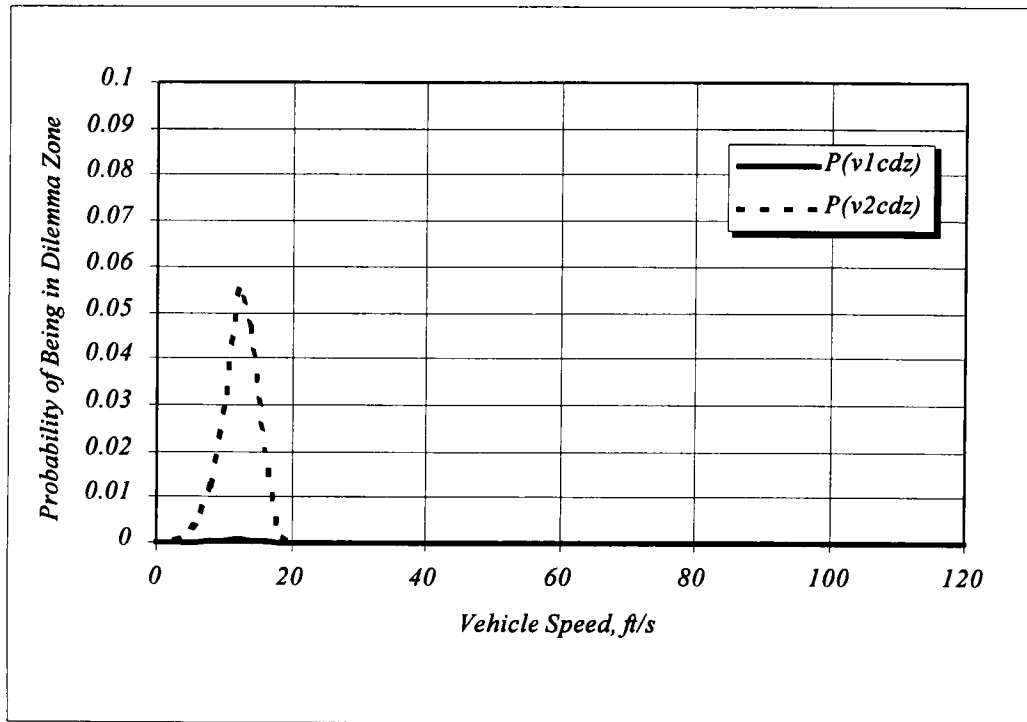


FIGURE III-7

Probability of Being in a Dilemma Zone

Given Case 1 and Case 2 Speed Distributions

$$PC(v_2) = 6 \text{ sec}$$

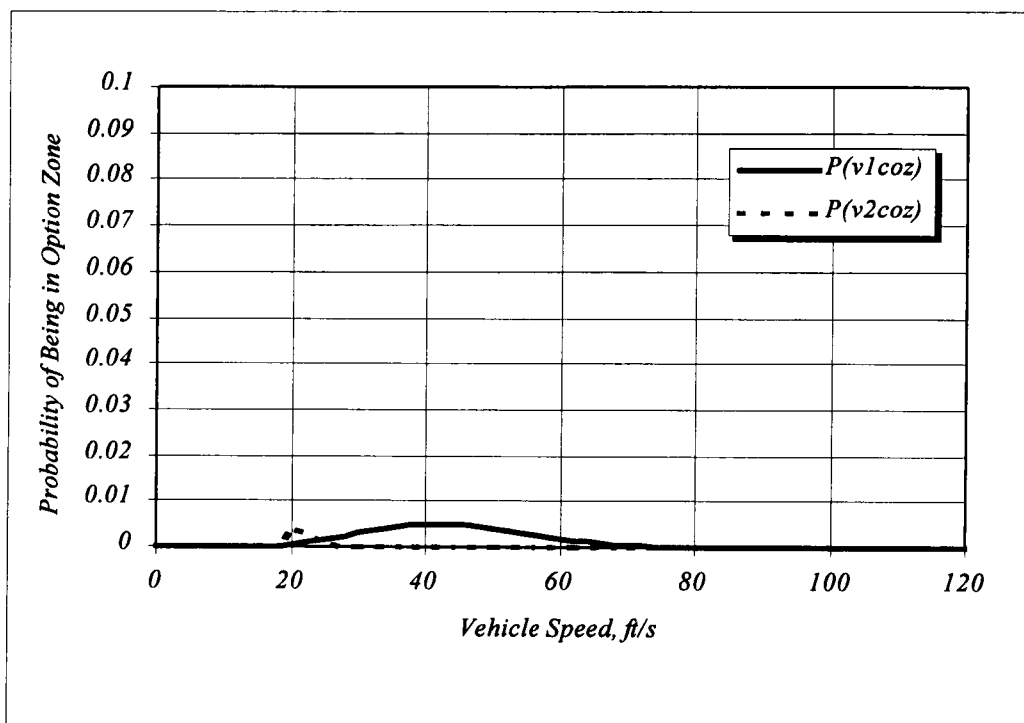


FIGURE III-8

Probability of Being Caught in an Option Zone

Given Case 1 and Case 2 Speed Distributions

$$PC(v_2) = 6 \text{ sec}$$

CHAPTER IV

RESULTS

If the procedure, detailed in the example case study, were repeated numerous times using different values of PC over the range of 2-8 seconds in 0.2 second intervals, the curves shown in FIGURES IV-1 and IV-2 would respectively result for case 1 and case 2 speed distributions. FIGURE IV-1 shows the total probability that a driver will not get caught in a dilemma zone (*non-DZ Case 1%*) curve for PC values from 2-8 seconds. The same type curve is shown for the option zone (*non-OZ Case 1%*) in the same figure. The heavy solid curve shown in FIGURE IV-3 denotes the probability that a driver will neither be caught in a dilemma zone nor an option zone situation. Looking back at FIGURE IV-1, it can be seen that the curve in FIGURE IV-3 encompasses parts of the non-dilemma zone curve and also parts of the non-option zone curve. Note that the solid curve in FIGURE IV-3 peaks at about 5 seconds. This would be the optimal phase-change timing, given the assumed driver population and speed distributions, to minimize the probability that a driver will be caught in either zone. FIGURE IV-2 and IV-4 show the results for the case 2 speed distribution. FIGURE IV-2 clearly depicts that drivers will have a much higher probability of being caught in a dilemma zone given the case 2 (congested) speed distribution than given the case 1 (free-flow) speed distribution. The optimal phase-change interval timing for the case 2 speed distribution, as portrayed in FIGURE IV-4, approaches an unrealistic value of 6.6 seconds. The case 2 results strongly reflect the idea that dilemma zone problem still exists.

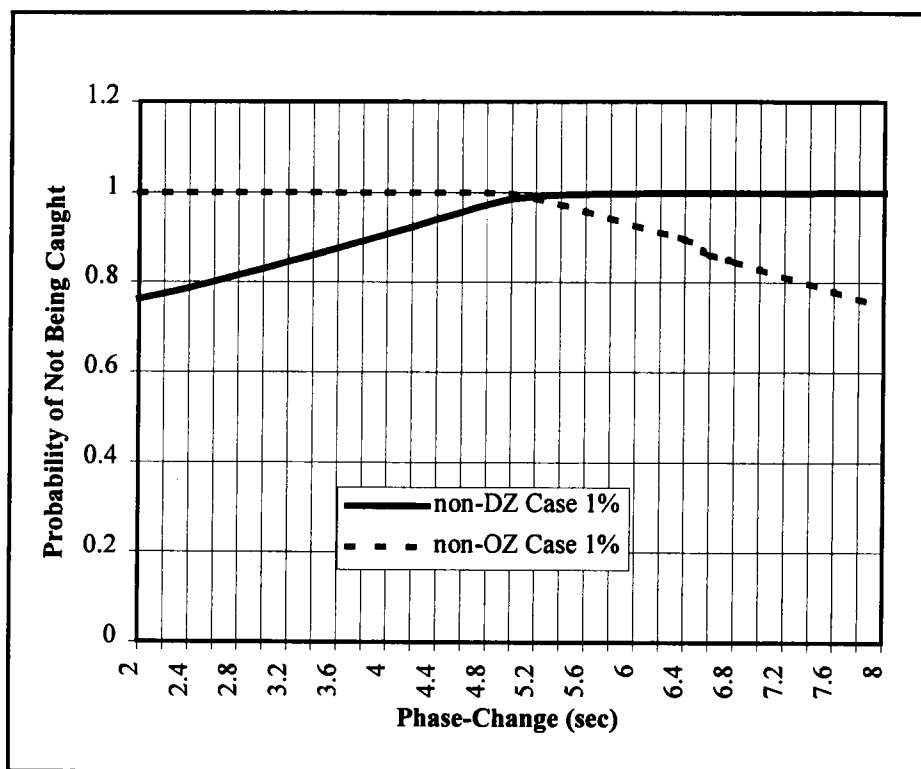


FIGURE IV-1

Overall Probabilities of Not Being Caught - Case 1

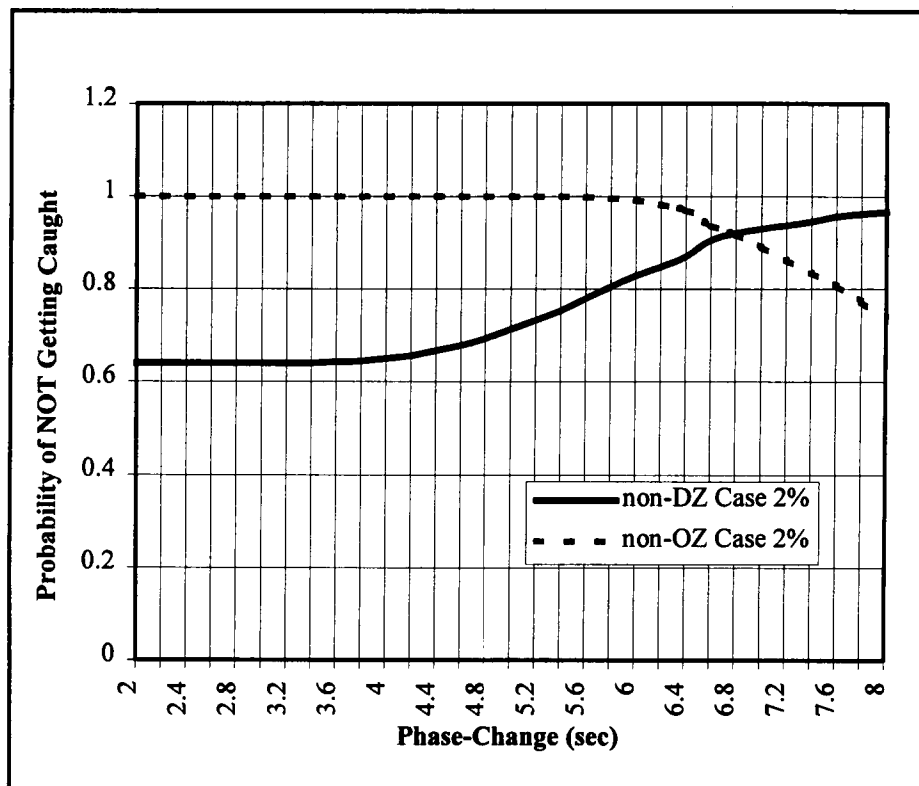


FIGURE IV-2

Overall Probabilities of Not Being Caught - Case 2

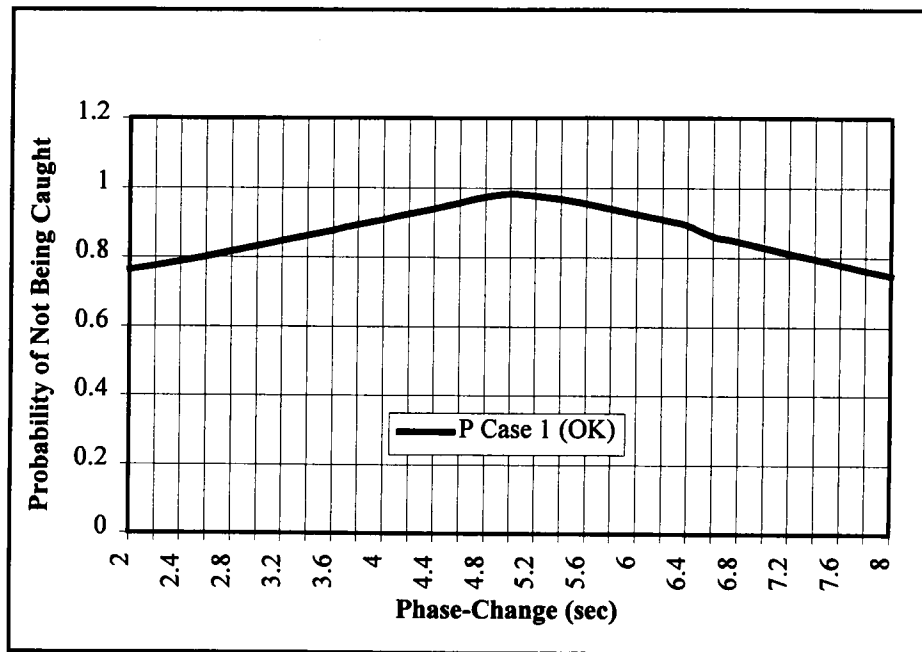


FIGURE IV-3

***P(OK)* - Case 1**

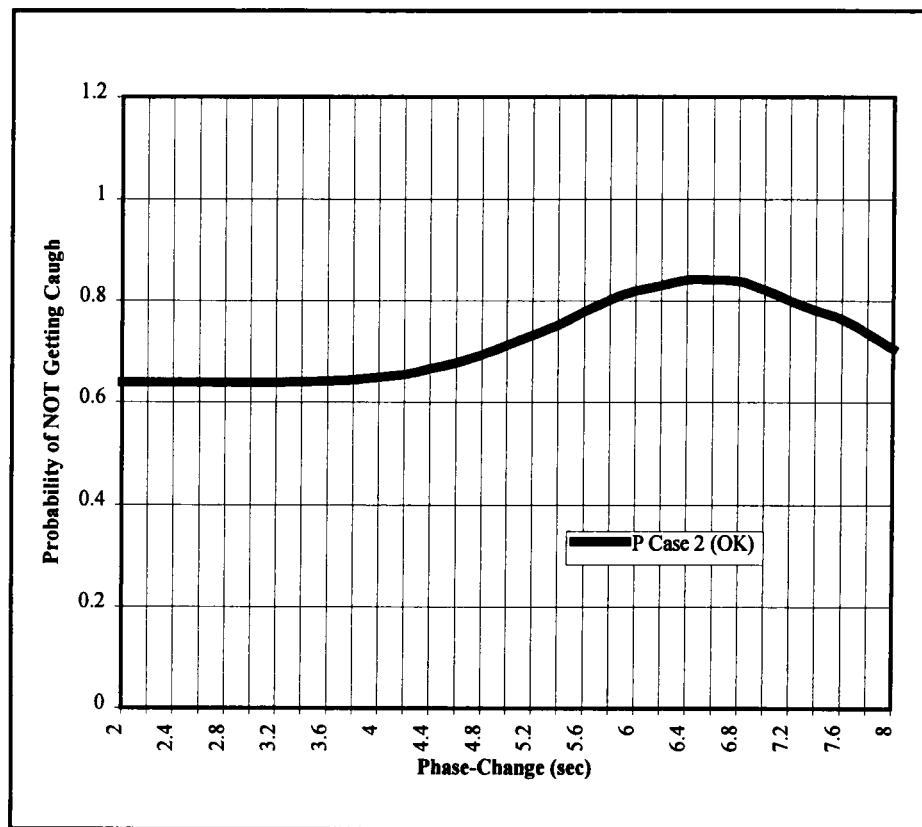


FIGURE IV-4

P(OK) - Case 2

It was mentioned in the case study that four phase-change timings would be studied in detail. These were:

$PC(v_L)$ = phase-change interval determined using speed limit of 35 mph (5.0 sec)

$PC(v_1)$ = phase-change interval determined using the 85th percentile speed
determined from the Case 1 speed distribution (5.2 sec)

$PC(v_2)$ = phase-change interval determined using the 85th percentile speed
determined from the Case 2 speed distribution (6.0 sec)

$PC(Y)$ = phase-change interval equal to the minimum yellow time for a 35 mph
approach speed from the Manual of Traffic Signal Design (3.6 sec).

The resulting plots of $P(v_2) = 6$ seconds data were given in the case study, while the resulting plots of the other three cases can be found in APPENDIX A. FIGURES A-1 through FIGURE A-6 show the resulting curves for $P(Y) = 3.6$ seconds, FIGURES A-7 through FIGURE 12 show the resulting curves for $PC(v_L) = 5$ seconds, and FIGURES A-13 through FIGURE A-18 show the resulting curves for $PC(v_1) = 5.2$ seconds. TABLES IV-1 and IV-2 summarize the percent of non-dilemma zone and non-option zone respectively for each speed distribution case over the phase-change timing range of 2-8 seconds.

Note that both of the phase-change timings determined from using the two speed distributions are larger than the phase-change timing determined from the speed limit. The 85th percentile speeds were calculated to be 40 mph for case 1 and 12 mph for case 2. Even though the percentile speeds were calculated from normal distributions whose mean

TABLE IV-1

**Summary of Dilemma and Option Zone Magnitudes
for Varying Phase-Change Timings - Case 1**

Case 1 Speed Distribution			
Phase-Change, sec	non-DZ Case 1%	non-OZ Case 1%	P Case 1 (OK)
2	76.17	100	76.17
2.2	77.35	100	77.35
2.4	78.66	100	78.66
2.6	80.04	100	80.04
2.8	81.48	100	81.48
3	82.97	100	82.97
3.2	84.48	100	84.48
3.4	86.02	100	86.02
3.6	87.59	100	87.59
3.8	89.17	100	89.17
4	90.76	100	90.76
4.2	92.36	100	92.36
4.4	93.97	100	93.97
4.6	95.58	100	95.58
4.8	97.2	100	97.2
5	98.48	99.65	98.13
5.2	99.07	98.61	97.68
5.4	99.37	97.29	96.66
5.6	99.54	95.82	95.36
5.8	99.64	94.29	93.93
6	99.7	92.71	92.41
6.2	99.74	91.11	90.85
6.4	99.77	89.5	89.27
6.6	99.82	86.25	86.07
6.8	99.84	84.62	84.46
7	99.85	82.99	82.84
7.2	99.86	81.35	81.21
7.4	99.87	79.71	79.58
7.6	99.89	78.08	77.97
7.8	99.9	76.43	76.33
8	99.9	74.79	74.69

TABLE IV-2

**Summary of Dilemma and Option Zone Magnitudes
for Varying Phase-Change Timings - Case 2**

Case 2 Speed Distribution			
Phase-Change, sec	non-DZ Case 2%	non-OZ Case 2%	P Case 2 (OK)
2	63.89	100	63.89
2.2	63.89	100	63.89
2.4	63.89	100	63.89
2.6	63.89	100	63.89
2.8	63.89	100	63.89
3	63.9	100	63.9
3.2	63.92	100	63.92
3.4	63.97	100	63.97
3.6	64.14	100	64.14
3.8	64.4	100	64.4
4	64.89	100	64.89
4.2	65.48	100	65.48
4.4	66.58	100	66.58
4.6	67.67	100	67.67
4.8	69.23	100	69.23
5	71.17	100	71.17
5.2	73.1	100	73.1
5.4	75.19	99.97	75.16
5.6	77.9	99.87	77.77
5.8	80.46	99.61	80.07
6	82.78	99.12	81.9
6.2	84.7	98.23	82.93
6.4	86.94	97.13	84.07
6.6	90.45	93.64	84.09
6.8	92.03	91.7	83.73
7	92.97	89.14	82.11
7.2	93.66	86.33	79.99
7.4	94.56	83.51	78.07
7.6	95.66	80.7	76.36
7.8	96.24	77.37	73.61
8	96.64	73.86	70.5

speeds were below the speed limit, they did not both end up with design speeds that were below the speed limit. The case 1 design speed was 5 mph faster than the speed limit. With the slower speeds of case 2, it takes more time to clear the intersection and thus a larger phase-change interval would be needed. With case 1, the approach speed is higher and more time is needed to come to a stop and thus the larger phase-change would be needed. This shows that the distributions of speed to have an impact on the design of the phase-change interval.

Going back to the resulting diagrams, FIGURES III-3, A-1, A-7, and A-13 show that the dilemma zone decreases as the phase-change increases. Whereas, the option zone increases as the phase-change increases. This fact is also evident in the Length of Zone figures (III-4, A-2, A-8, and A-14). As the zones increase or decrease, so do the probabilities of getting caught in one. It is evident, from FIGURES III-5 and III-6, that during more congested times, the probability of getting caught in a dilemma zone or option zone increases due to increased vehicular exposure to the zones.

In the range of phase-change values (3.6 to 5.2) that were studied for case 1, 87% of the drivers will not have any problems. For case 2, with phase-change values range from 3.6 to 6 seconds, the percentage of drivers that will not have problems is much lower. The percentage ranges from 64% at 3.6 seconds to 85% at 6 seconds.

One thing that is highly evident is that the dilemma zones for phase-changes set to equal the minimum yellow time, which allows time required to stop with no clearance time, yield much greater dilemma zone problems. FIGURE IV-2 shows that the use of minimum yellow time PC(Y) of 3.6 seconds for the phase-change interval yields substantial dilemma zone where only 64% of the drivers will be free of the zone. This means that close to one out of every two drivers approaching an intersection during yellow will experience the problem of a dilemma zone.

The figures, depicting probabilities of getting caught in dilemma zones, show that the probabilities are greatest at speeds of around 0 to 20 or 30 ft/s (0 to 13 or 20 mph). This is one reason why the probabilities are greater for case 2 vehicles to be caught, because the majority of the vehicles for this distribution are traveling in this speed range.

One problem that needs to be avoided is an overly long phase-change interval that results in a large option zone and inefficient signal cycle lengths. Option zones are considered to be less detrimental than dilemma zones unless two or more cars are in the zone at the same time. This would seem to worsen in case 2, where the vehicle exposure is higher. Looking at FIGURES IV-1 and IV-2, this can be seen much more easily. Surprisingly, the option zone in FIGURE IV-2 does not become a problem until it reaches a phase-change of 6 seconds. This is most likely due to the fact that most of the vehicles will experience a dilemma zone, where they will not be able to clear the intersection, due to the low speeds.

Another common driver reaction to phase-change intervals is acceleration. Some drivers may decide to accelerate during the clearance of an intersection to cut down on time delays from being stopped by a traffic signal. If acceleration is assumed, the magnitude of the option zone increases greatly, while the dilemma zone is mostly eliminated in this case for the majority of approaching vehicles traveling near the speed limit.

The four phase-changes were re-evaluated allowing for acceleration in clearance. The new equation for clearing distance became:

$$X_{ca} = v_0 PC - w - L + \frac{a}{2}(PC - t)^2 \quad (9)$$

where $a = 16 - 0.145v_0$, assuming that acceleration deteriorates with speed. Diagrams of the new curves, which depict the allowance of acceleration during the phase-change interval, can be found in APPENDIX B. FIGURES B-1 through B-6 are for $PC(Y) = 3.6$ seconds, FIGURES B-7 through B-12 are for $PC(v_L) = 5$ seconds, FIGURES B-13 through B-18 are for $PC(v_1) = 5.2$, and FIGURES B-19 through B-24 are for $PC(v_2) = 6$ seconds. The dilemma and option zone free percentages are given in TABLES IV-3 and IV-4. If acceleration is assumed, the minimum yellow time is the phase-change timing most closely related to minimizing both zones for both traffic conditions.

If the procedure, detailed in the example case study (changed only by using equation (9) instead of equation (5)), were repeated numerous times using different values of PC in

TABLE IV-3

**Summary of Dilemma and Option Zone Magnitudes
for Varying Phase-Change Timings - Case 1 with Acceleration**

Phase-Change, sec	Case 1 Speed Distribution with Acceleration		
	non-DZ Case 1%	non-OZ Case 1%	P Case 1 (OK)
2	76.69	100	76.69
2.2	78.27	100	78.27
2.4	80.1	100	80.1
2.6	82.12	100	82.12
2.8	84.32	100	84.32
3	86.68	100	86.68
3.2	89.19	100	89.19
3.4	91.82	100	91.82
3.6	94.56	99.97	94.53
3.8	96.57	99.08	95.65
4	97.93	97.41	95.34
4.2	98.81	95.14	93.95
4.4	99.34	92.43	91.77
4.6	99.65	89.44	89.09
4.8	99.83	86.23	86.06
5	99.92	82.87	82.79
5.2	99.96	79.4	79.36
5.4	99.98	75.84	75.82
5.6	99.99	72.21	72.2
5.8	100	68.53	68.53
6	100	64.8	64.8

TABLE IV-4

**Summary of Dilemma and Option Zone Magnitudes
for Varying Phase-Change Timings - Case 2 with Acceleration**

Case 2 Speed Distribution with Acceleration			
Phase-Change, sec	non-DZ Case 2%	non-OZ Case 2%	P Case 2 (OK)
2	63.89	100	63.89
2.2	63.89	100	63.89
2.4	63.89	100	63.89
2.6	63.94	100	63.94
2.8	64.41	100	64.41
3	66.55	100	66.55
3.2	72.7	100	72.7
3.4	83.09	100	83.09
3.6	96.55	99.7	96.25
3.8	99.88	87.52	87.4
4	100	70.99	70.99
4.2	100	53.45	53.45
4.4	100	35.35	35.35
4.6	100	18.33	18.33
4.8	100	6.26	6.26
5	100	1.16	1.16
5.2	100	0.11	0.11
5.4	100	0.01	0.01
5.6	100	0.01	0.01
5.8	100	0.01	0.01
6	100	0.01	0.01

the range of 2-6 seconds in 0.2 second intervals, the curves shown in FIGURES IV-5 and IV-6 would result. FIGURE IV-5 shows the total probability that a driver will not get caught in a dilemma zone (non-DZ Case 1%) curve for PC values from 2-6 seconds given the ability to accelerate to clear the intersection. The same curve is shown for the option zone (non-OZ Case 1%). A heavy solid curve, which encompasses parts of the option zone curve and dilemma zone curve, denotes the probability that a driver will neither be caught in a dilemma zone nor an option zone situation (FIGURE IV-7). Note that the curve in FIGURE IV-7 peaks at about 3.6 to 3.8 seconds for both curves. This would be the optimal phase-change timing, given the case 1 speed distribution and other driver related assumptions, to minimize the probability that a driver will get in either zone. This determination of the optimal phase-change interval most closely relates to the phase-change interval found by using ITE's recommended practice. FIGURES IV-6 and IV-8 show the same curves plotted for the case 2 speed distribution.

The resulting FIGURE IV-5 appears to be almost identical to FIGURE IV-1 in shape. FIGURE IV-1 did not take into account any acceleration, while FIGURE IV-5 did. The difference between the two sets of curves is shown in FIGURE IV-7. The resultant non-zonal curve for the case 1 speed distribution allowing acceleration peaks with an optimal phase-change interval of 3.8 seconds while FIGURE IV-3 peaks at 5 seconds. Thus, if the percentages of safe drivers stays about the same with a decrease in phase-change time needed allowing acceleration, it might appear that driver acceleration at the onset of the

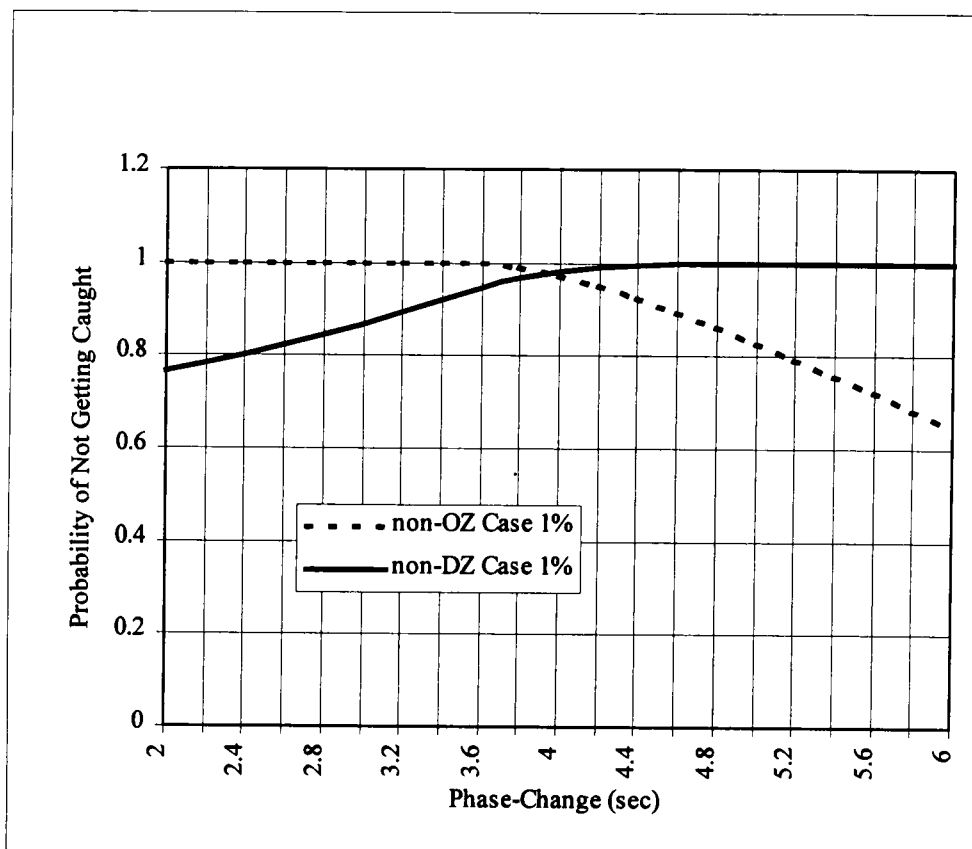


FIGURE IV-5

Overall Probability of Not Being Caught

Case 1 - with Acceleration

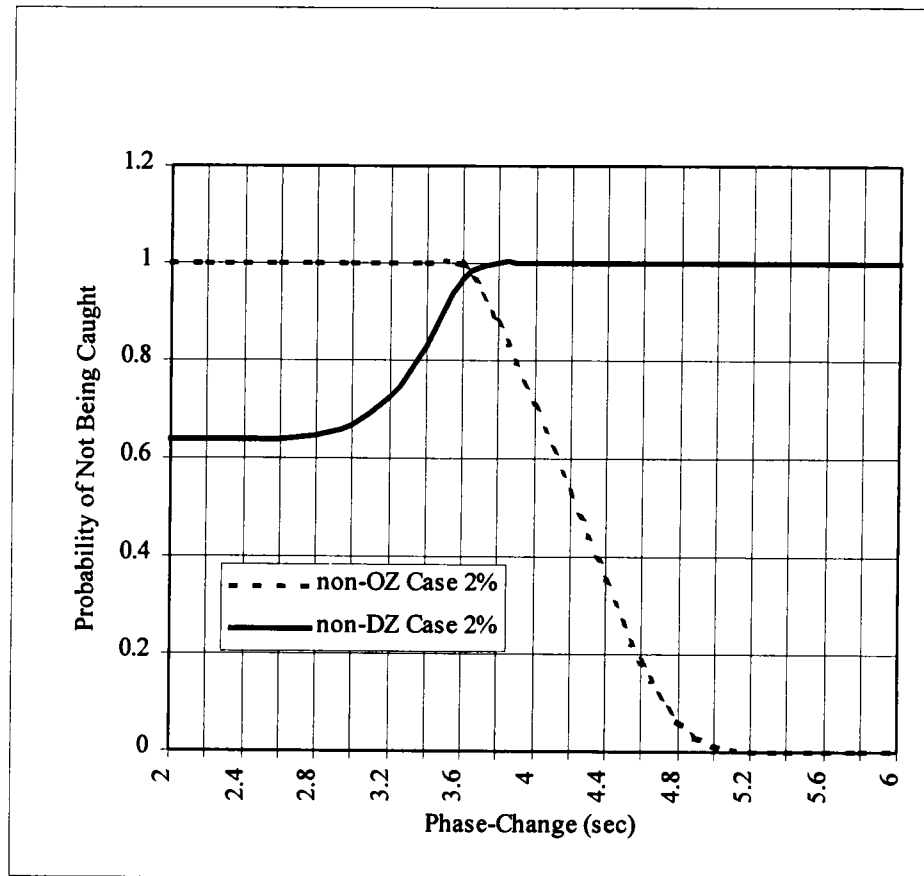


FIGURE IV-6

Overall Probabilities of Not Being Caught

Case 2 - with Acceleration

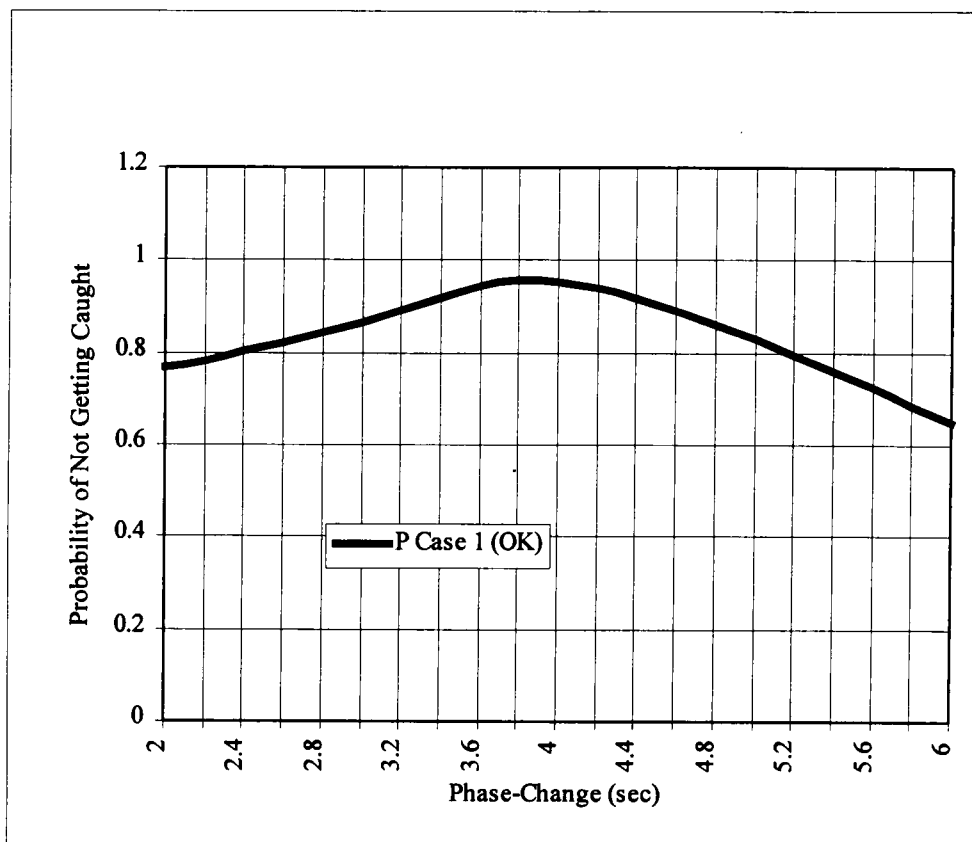


FIGURE IV-7

***P(OK)* - CASE 1 - with Acceleration**

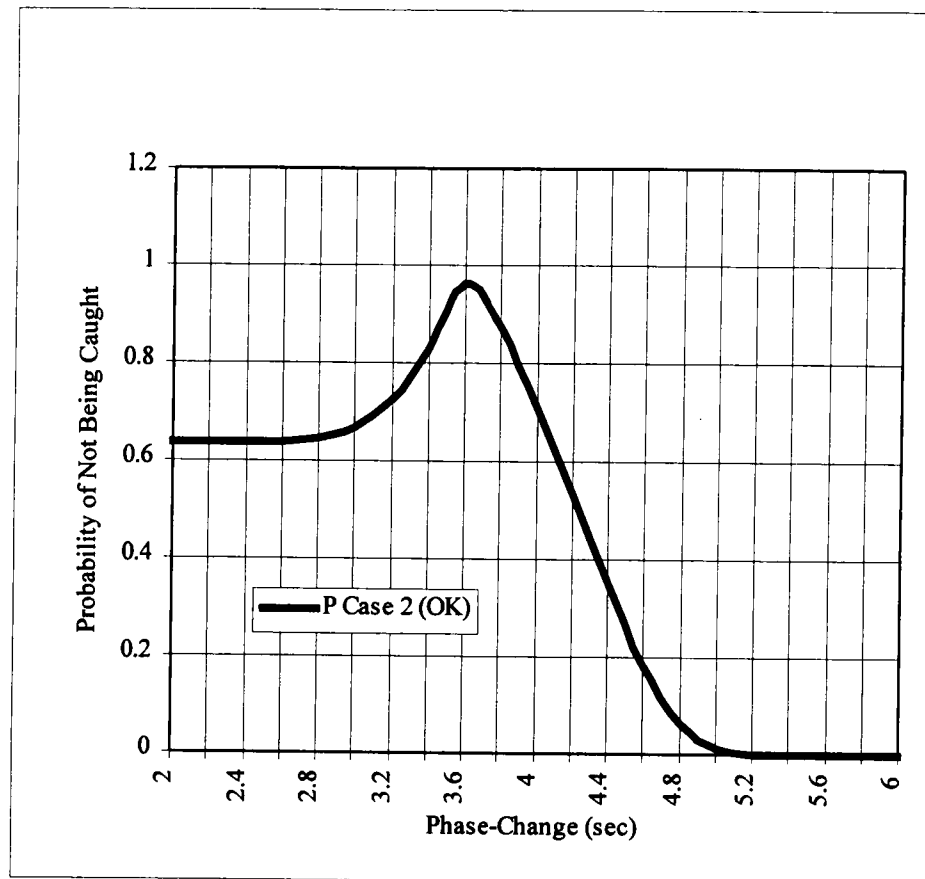


FIGURE IV-8

P(OK) - CASE 2 - with Acceleration

phase change interval would be favorable. The case 2 speed distribution curves found in FIGURE IV-6 show an erratic pattern of dilemma and option zone-free percentages. There is a definite optimal phase-change timing of 3.8 seconds for the given situation assuming acceleration at the onset of the phase-change interval. After 3.8 seconds, the magnitude of the option zone significantly increases while the dilemma zone is non-existent.

SENSITIVITY ANALYSIS

Given the somewhat erratic behavior to the changing variables of phase-change timing and acceleration, a sensitivity analysis was performed to determine the effect of reaction time, length of vehicle, and width of intersection on the resulting non-dilemma zone percentages and non-option zone percentages. The reaction time was varied from 0.75 seconds to 2.50 seconds in 0.25 second increments; the length of the vehicle was varied from 10 feet to 60 feet in 5 foot increments; and the width of the intersection was varied from 30 feet to 120 feet in 10 foot increments. The following were held constant, except for when necessary to perform sensitivity analysis on that certain variable:

$$PC = 5 \text{ sec}$$

$$w = 60 \text{ ft}$$

$$L = 15 \text{ ft}$$

$$t = 1 \text{ sec}$$

$$d = 10 \text{ ft/sec}^2$$

85th percentile speed.

TABLES IV-5 through IV-7 contain the results of the sensitivity analysis. Results can be compared against the constant scenario which was run with the preceding constant variables. The constant scenario gives results of:

non-DZ Case 1% = 98.5

non-OZ Case 1% = 99.7

non-DZ Case 2% = 71.2

non-OZ Case 2% = 100.0.

TABLE IV-5

Effects of Reaction Time Variation

Reaction Time (sec)	non-DZ case 1%	non-OZ case 1%	non-DZ case 2%	non-OZ case 2%
0.75	99.2	98.3	76.4	100
1.00	98.5	99.7	71.2	100
1.25	96.8	100	65.0	100
1.50	94.7	100	60.8	100
1.75	92.6	100	55.5	100
2.00	90.5	100	50.3	100
2.25	88.4	100	45.1	100
2.50	86.4	100	39.9	100

TABLE IV-6**Effects of Length of Vehicle Variation**

Length of Vehicle (ft)	non-DZ case1%	non-OZ case1%	non-DZ case 2%	non-OZ case 2%
10	98.9	99.1	73.9	100
15	98.5	99.7	71.2	100
20	97.8	100	68.4	100
25	96.8	100	67.0	100
30	95.8	100	65.6	100
35	94.9	100	65.0	100
40	93.9	100	64.4	100
45	93.0	100	64.2	100
50	92.1	100	64.0	100
55	91.1	100	63.9	100
60	90.2	100	63.9	100

TABLE IV-7**Effects of Width of Intersection Variation**

Width of Intersection (ft)	non-DZ case 1%	non-OZ case 1%	non-DZ case 2%	non-OZ case 2%
30	99.7	94.6	95.0	94.2
40	99.5	96.6	87.4	99.1
50	99.2	98.3	78.2	100
60	98.5	99.7	71.2	100
70	96.8	100	67.0	100
80	94.9	100	65.0	100
90	92.9	100	64.2	100
100	91.1	100	63.9	100
110	89.4	100	63.9	100
120	87.7	100	63.9	100

CHAPTER V

CONCLUSIONS

This study was undertaken to develop a probabilistic assessment technique to determine the magnitude of dilemma and option zones and also to determine the effects of speed distribution and traffic conditions on the phase-change interval. The procedure defined herein meets both of those challenges. The assessment technique requires little data, and is simple to use once the spreadsheet is set up. These qualities make this a viable tool for use by all transportation engineers. While the procedure does not result in the determination of the best phase-change interval to be used in the design of the traffic signal, used as it was intended, as a tool, it provides valuable information on the magnitude of dilemma and option zones that can lead to the enhancement of the traffic signal.

The current research has shown that dilemma zones do exist by nature of the stochastic variables involved in the determination of the phase-change interval. By way of the sensitivity analysis, it is shown that dilemma zones are negatively impacted by drivers with slower reaction times, wider intersections and longer vehicles. The opposite seems to be true for the option zone.

The question of driver understanding of the phase-change interval laws is something that should be researched further. Given the determination that drivers who accelerate under

free-flow conditions have the same probability of being in a dilemma zone as those who do not might lead to the determination that this is fundamentally more efficient, because the associated cycle length is shorter. However, acceleration through intersections could very well have a negative impact on driver safety.

The current standards for determining the length of the phase-change interval in comparison to this proposed approach tool determination are much shorter. For transportation professionals, there should be a commitment to better determine what is safe before determining what is efficient. The determination of the phase-change interval should be done over a range of variables, because it has been proven in this report to make a difference.

It has been shown also in this report that the magnitude of implications caused by the phase-change interval is affected by the demand on the traffic system or traffic flow conditions. This also means, that transportation engineers may also be able to more efficiently design traffic signal timing by varying the phase-change interval timing for varying traffic flow conditions at different times of the day.

The design of phase-change intervals is still not a perfect science, but with increasing technology, the task is being made easier every day. It has been shown here that the use of stochastic information does have an effect on the design of the phase-change interval timing. Technology currently in test phases will most likely allow future transportation

engineers to determine the distributions of stochastic variables through the use of machine vision. This technology will allow computers to read images received by a video camera placed at an intersection. Each vehicle that passes through the intersection will be traced, and its trajectory and classification will be recorded. The trajectory will allow the determination of speed, deceleration rate or acceleration rate of a vehicle approaching an intersection. These stochastic distributions of information will hopefully enable the development of a science in signal phase-change design.

BIBLIOGRAPHY

BIBLIOGRAPHY

1. Agent, K. R., and Pigman, J. G. *Evaluation of Change Interval Treatments for Traffic Signals at High-Speed Intersections*. Report KTC9426. Lexington, KY: Kentucky Transportation Center, 1994.
2. Allos, A. E., and Al-Hadithi, M. I. "Driver Behavior During Onset of Amber at Signalized Intersections." *Traffic Engineering and Control*, Vol. 33, No. 5, 1992, pp. 312-317.
3. Bissell, H. H., and Warren, D. L. "Yellow Signal is NOT a Clearance Interval." *ITE Journal*, Vol. 51, No. 2, 1981, pp. 14-17.
4. Bonneson, J. A.; McCoy, P. T.; and Moen, B. A. *Traffic Detector Design and Evaluation Guidelines: Final Report*. Report TRP-02-31-93. Lincoln, NE: Nebraska Department of Roads, 1994.
5. Chan, Y., and Liao, T. "Setting Change Intervals at Signalized Intersections." *ITE Journal*, Vol. 57, No. 2, 1987, pp. 45-50.
6. Chang, M., Messer, C., and Santiago, A. "Timing Traffic Signal Change Intervals Based on Driver Behavior." *Transportation Research Record* 1027, 1985.
7. "Determining Vehicle Signal Change and Clearance Intervals." Institute of Transportation Engineers, Technical Committee 4TF-1. In *Compendium of Technical Papers*. Washington, D. C.: Institute of Transportation Engineers, 1994.
8. Easa, Said M. "Reliability-based Design of Inter-green Interval at Traffic Signals." *Journal of Transportation Engineering*, Vol. 119, No. 2, March-April 1993, pp. 255-271.
9. Gazis, D.; Herman, R.; and Maradudin, A. *The Problem of the Amber Signal Light in Traffic Flow*. Warren, MI: General Motors Corporation Research Laboratories, 1959.
10. Hooper, K. G., and McGee, H. W. "Driver Perception-Reaction Time: Are Revisions to Current Specification Values in Order?" *Transportation Research Record* 904, 1983.
11. Kikuchi, S., and Riegnor, J. "Methodology to Analyze Driver Decision Environment During Signal Change Intervals: Application of Fuzzy Set Theory." *Transportation Research Record* 1368, 1992.

12. Lin, F. B. "Timing Design of Signal Change Intervals." *Transportation Research Record* 1069, 1986.
13. Lin, F. B., Cooke, D., and Vijayakumar, S. "Utilization and Timing of Signal Change Interval." *Transportation Research Record* 1114, 1987.
14. Mahalel, D., and Zaidel, P. "A Probabilistic Approach for Determining the Change Interval." *Transportation Research Record* 1069, 1986.
15. May, A. D. "Clearance Interval at Traffic Signals." *Highway Research Record* 221, 1968.
16. *Manual of Traffic Signal Design*. Institute of Transportation Engineers, Kell, J. H., and Fullerton, I. J., ed. Englewood Cliffs, NJ: Prentice Hall, Inc., 1991.
17. *Manual on Uniform Traffic Control Devices*. Federal Highway Administration. Washington, D.C.: US Department of Transportation, 1988.
18. Matson, T. M., Smith, W. S., and Hurd, F. W. *Traffic Engineering*. McGraw Hill Book Company, Inc., 1955.
19. McShane, W., and Roess, R. *Traffic Engineering*. Englewood Cliffs, NJ: Prentice Hall, Inc., 1990.
20. Orcutt, F. L. *The Traffic Signal Book*. Englewood Cliffs, NJ: Prentice Hall, Inc., 1993.
21. Parsonson, P. S.; Day, R. A.; Gawlas, J. A.; et al. "Use of EC-DC Detector for Signalization of High-Speed Intersections." *Transportation Research Record* 737, 1979.
22. Parsonson, P. S. "Signalization of High-Speed, Isolated Intersections." *Transportation Research Record* 681, 1978.
23. Prashker, J. N., and Mahalel, P. "The Relationship Between an Option Space and Drivers' Indecision at Signalized Intersection Approaches." *Transportation Research*, Part B, Vol. 23B, No. 6, 1989, pp. 401-413.
24. "Proposed Recommended Practice: Determining Vehicle Change Intervals." Institute of Transportation Engineers, Technical Committee 4A-16, *ITE Journal*, May, 1985.

25. Shanteau, R. M. "Considerations in the Length of the Yellow Interval." *Engineering Bulletin of Purdue University*, No 154. West Lafayette, IN: Purdue University, 1983, pp. 115-120.
26. Williams, L. W. "Driver Behavior During the Yellow Signal Interval." *Transportation Research Record* 644, 1977.
27. Wortman, R. H., and Fox, T. C. "A Reassessment of the Traffic Signal Change Interval." *Transportation Research Record* 1069, 1986.
28. Stimpson, W. A.; Zador, P. L.; and Tarnoff, P. J. "The Influence of the Time Duration of Yellow Traffic Signals on Driver Response." *ITE Journal*, Vol. 50, No. 11, 1980, pp. 22-29.
29. Tignor, S. C., and Warren, D. "Driver Speed Behavior on U. S. Streets and Highways." In *Compendium of Technical Papers*. Washington, D.C.: Institute of Transportation Engineers, 1980.
30. *Transportation and Traffic Engineering Handbook*. Institute of Transportation Engineers, Wolfgang S. Homburger, ed. Englewood Cliffs, NJ: Prentice Hall, Inc., 1982.
31. Zador, P.; Stein, H.; Shapiro, S.; et al. "Effect of Clearance Interval Timing on Traffic Flow and Crashes at Signalized Intersections." *ITE Journal*, Vol. 55, No. 11, 1985, pp.36-39.
32. Zegeer, C. V. *Effectiveness of Green-Extension Systems at High-Speed Intersections*. Lexington, KY: Kentucky Department of Transportation, 1977.

APPENDIX A

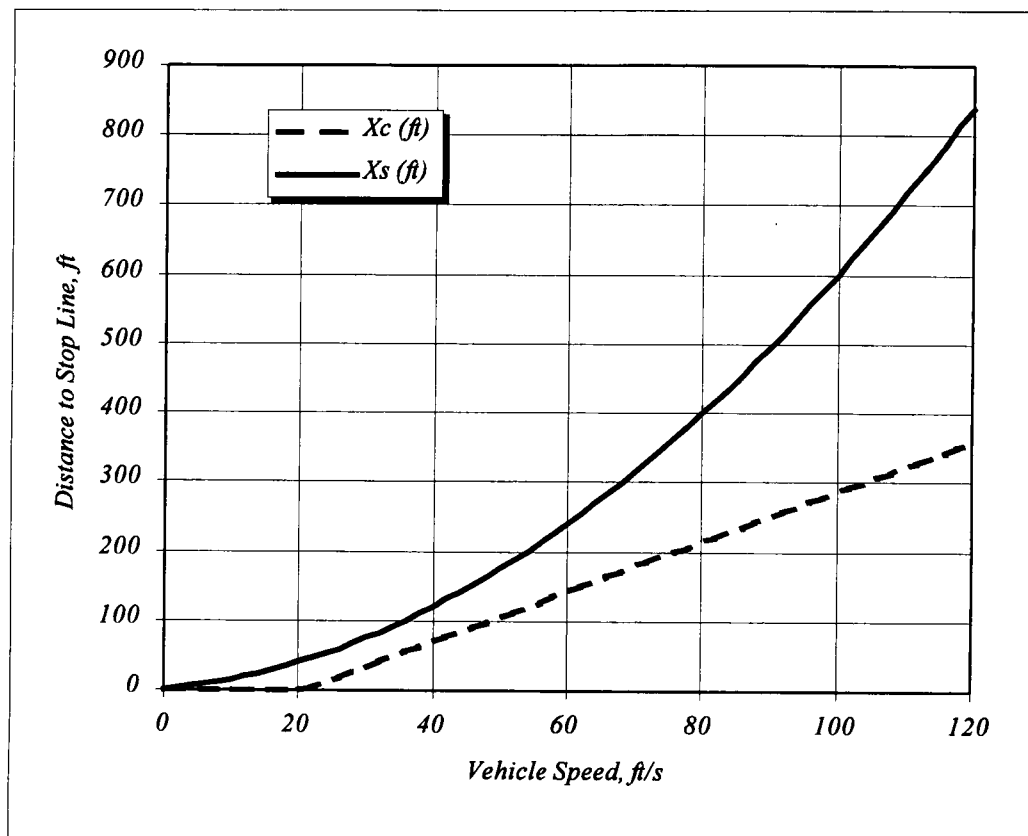


FIGURE A-1

Plot of Clearing and Stopping Distance

$$PC(Y) = 3.6 \text{ sec}$$

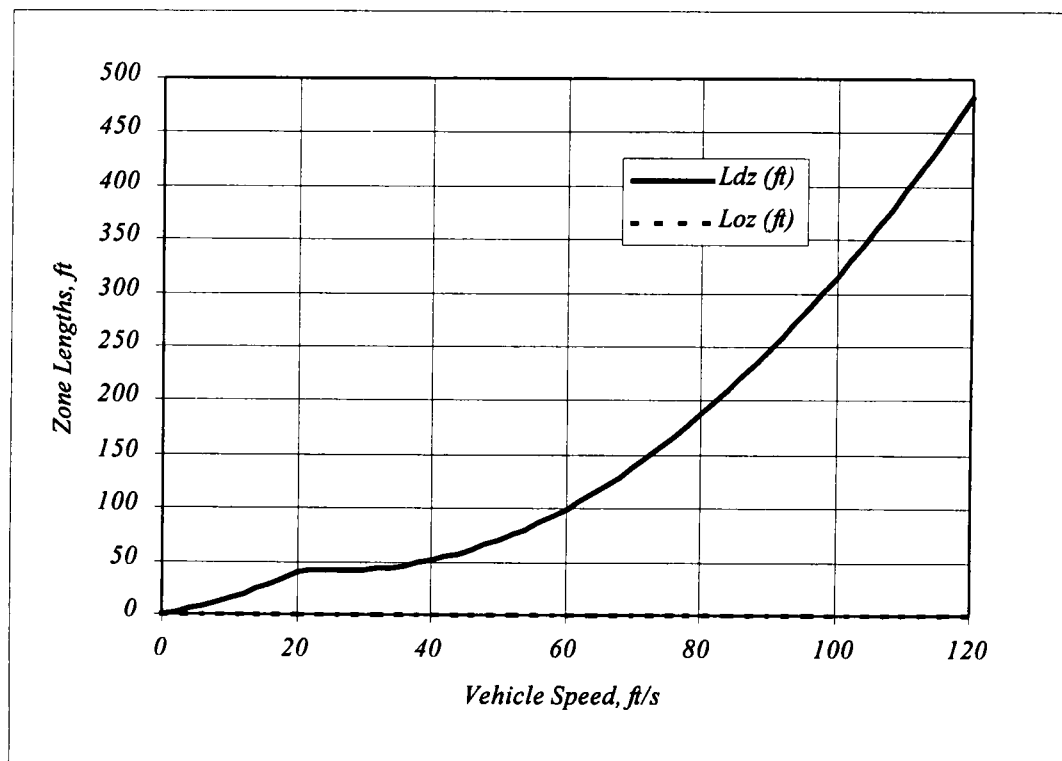


FIGURE A-2

Plot of Dilemma and Option Zone Lengths

$$PC(Y) = 3.6 \text{ sec}$$

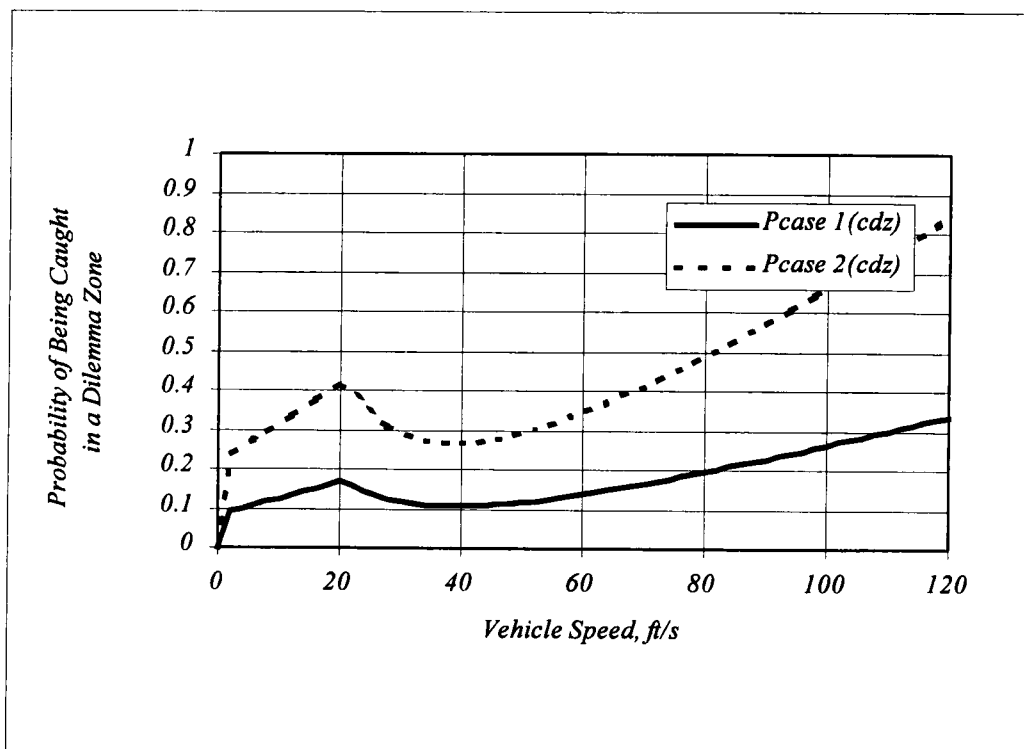


FIGURE A-3

Probability of Being Caught in a Dilemma Zone

$$PC(Y) = 3.6 \text{ sec}$$

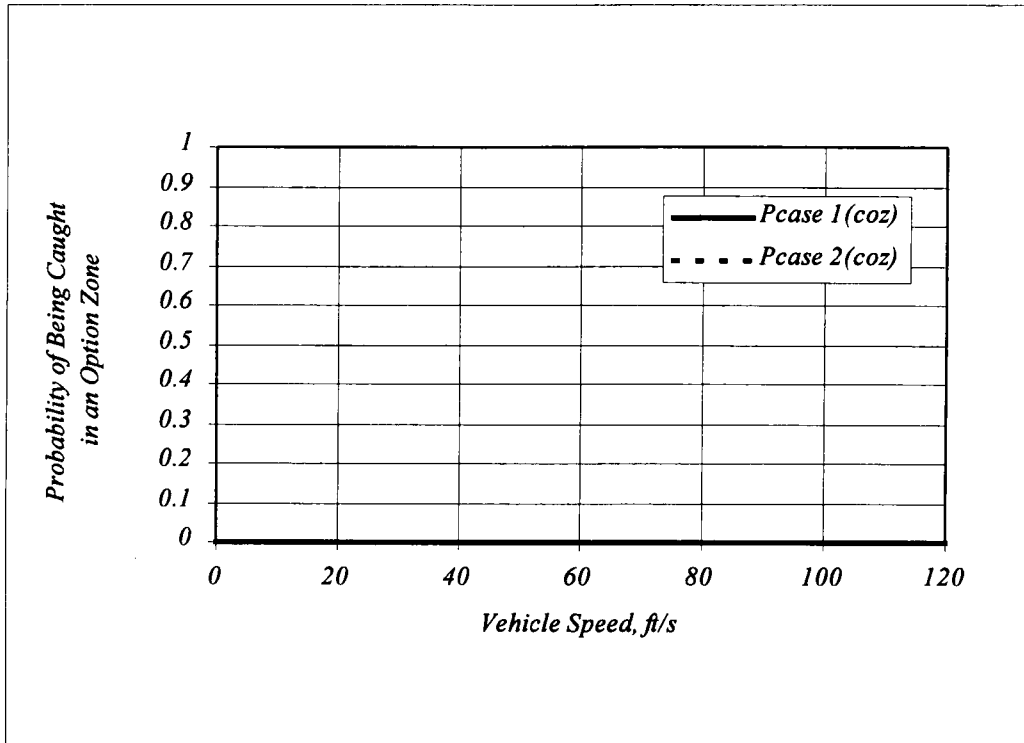


FIGURE A-4

Probability of Being Caught in an Option Zone

$$PC(Y) = 3.6 \text{ sec}$$

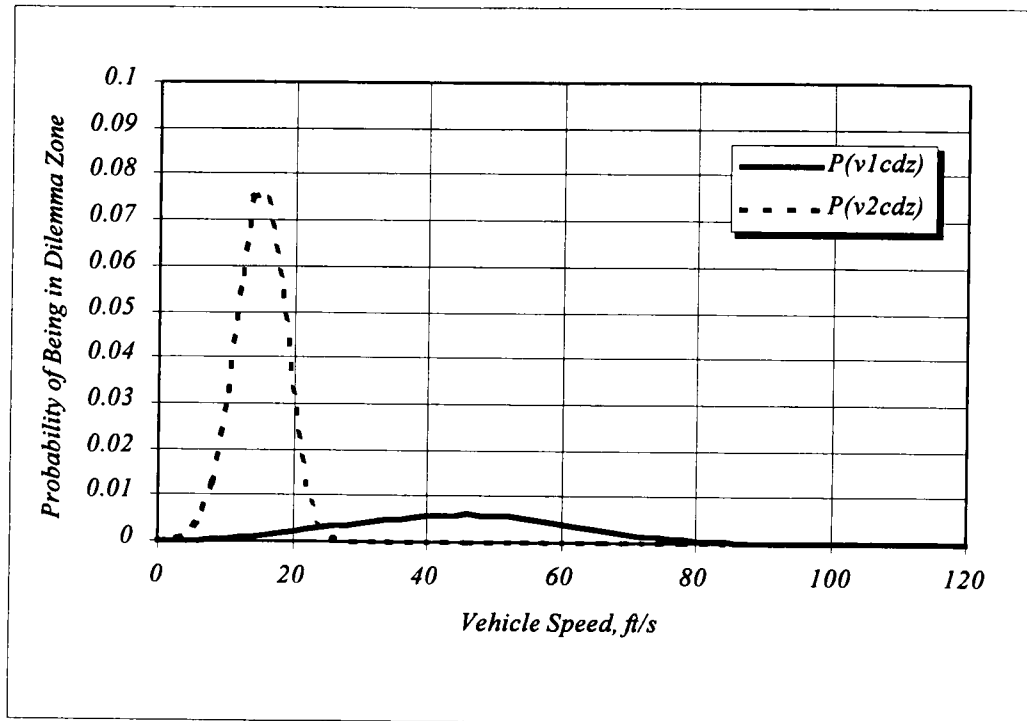


FIGURE A-5

Probability of Being in a Dilemma Zone

Given Case 1 and Case 2 Speed Distributions

$$PC(Y) = 3.6 \text{ sec}$$

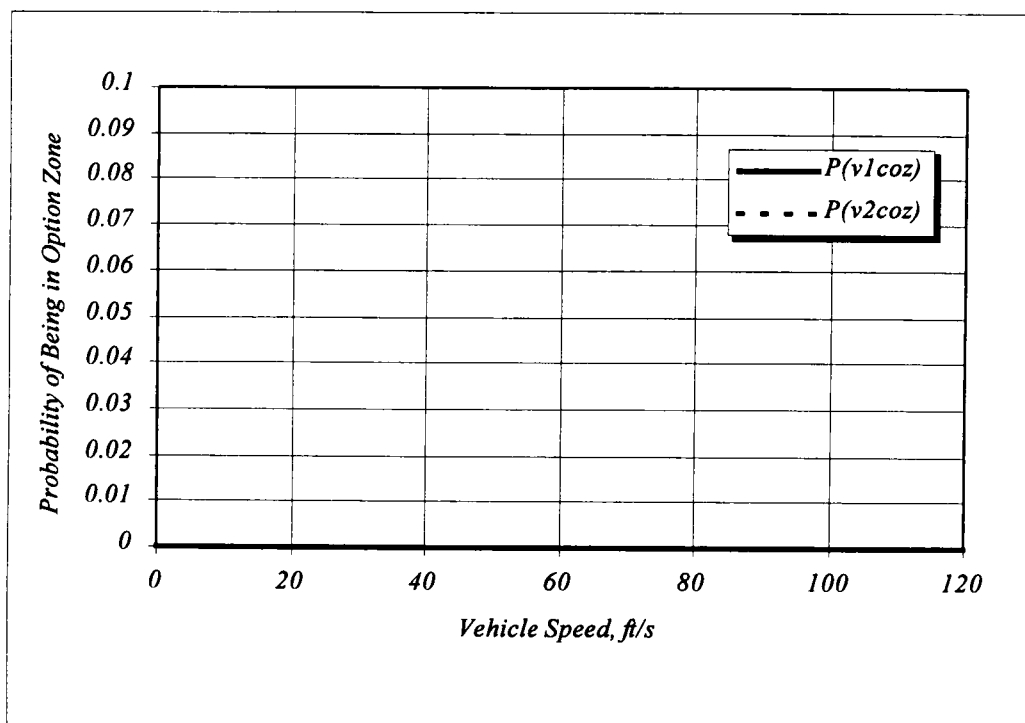


FIGURE A-6

Probability of Being in an Option Zone

Given Case 1 and Case 2 Speed Distributions

$$PC(Y) = 3.6 \text{ sec}$$

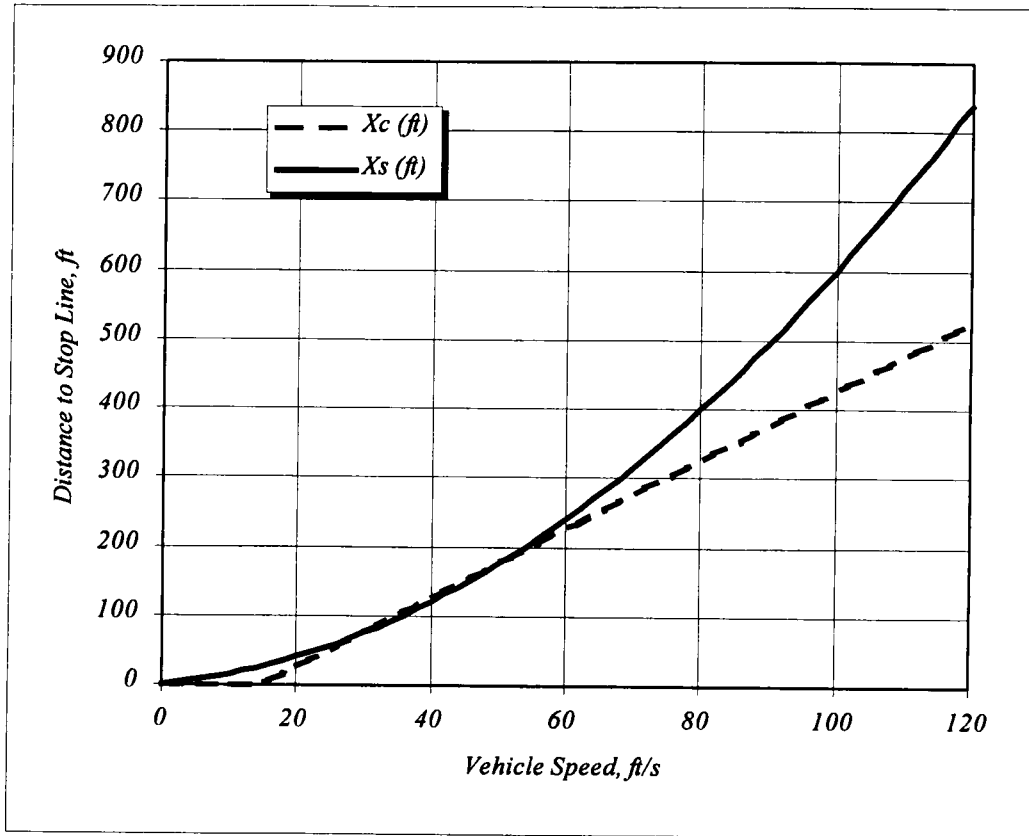


FIGURE A-7

Plot of Clearing and Stopping Distance

$$PC(v_L) = 5 \text{ sec}$$

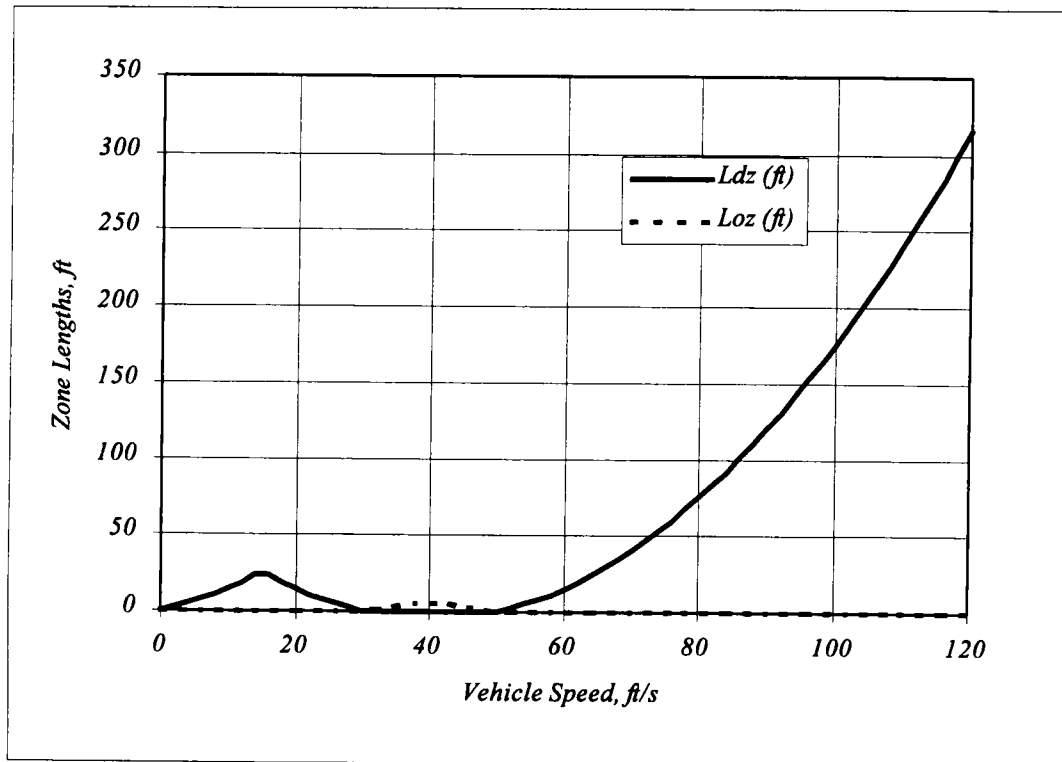


FIGURE A-8

Plot of Dilemma and Option Zone Lengths

$$PC(v_L) = 5 \text{ sec}$$

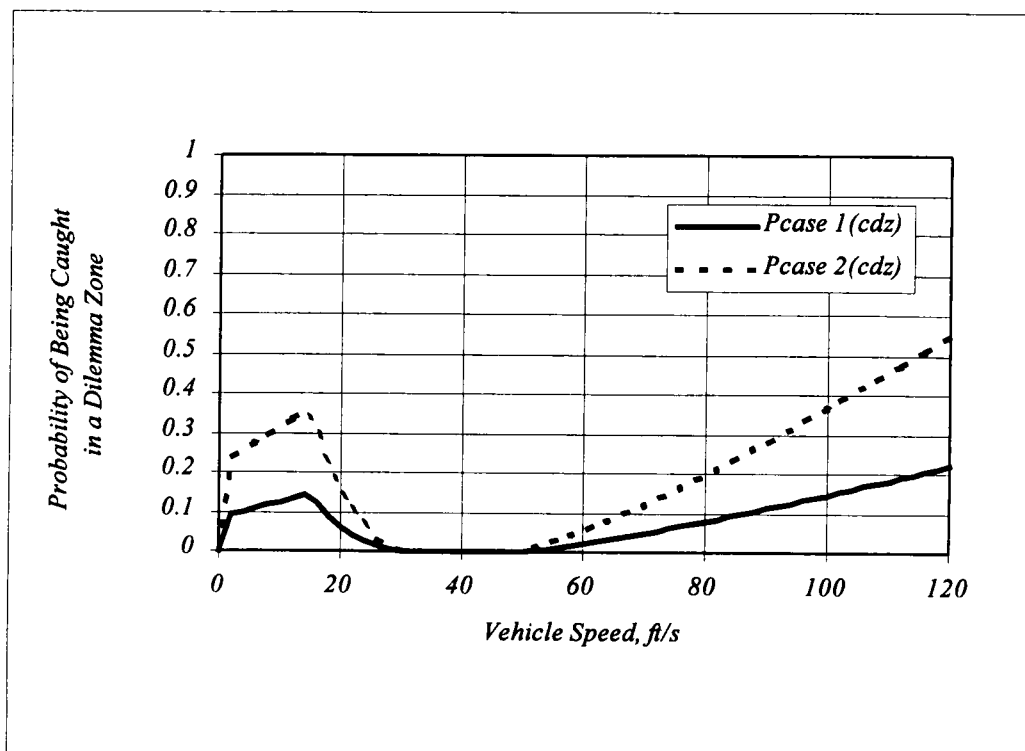


FIGURE A-9

Probability of Being Caught in a Dilemma Zone

$$PC(v_L) = 5 \text{ sec}$$

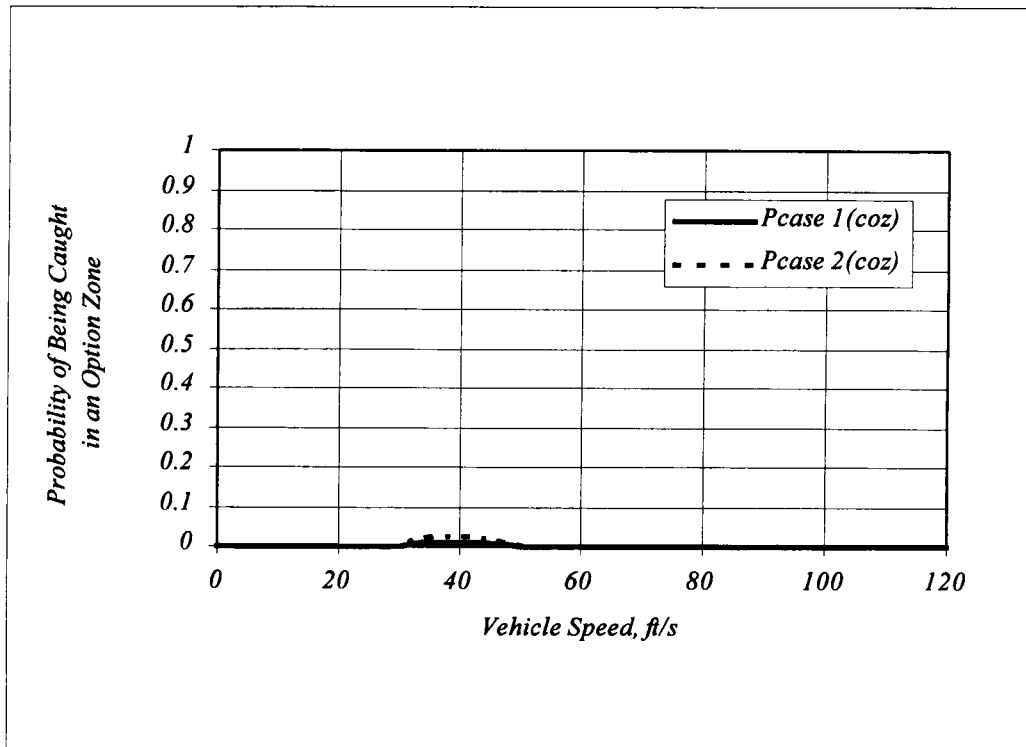


FIGURE A-10

Probability of Being Caught in an Option Zone

$$PC(v_L) = 5 \text{ sec}$$

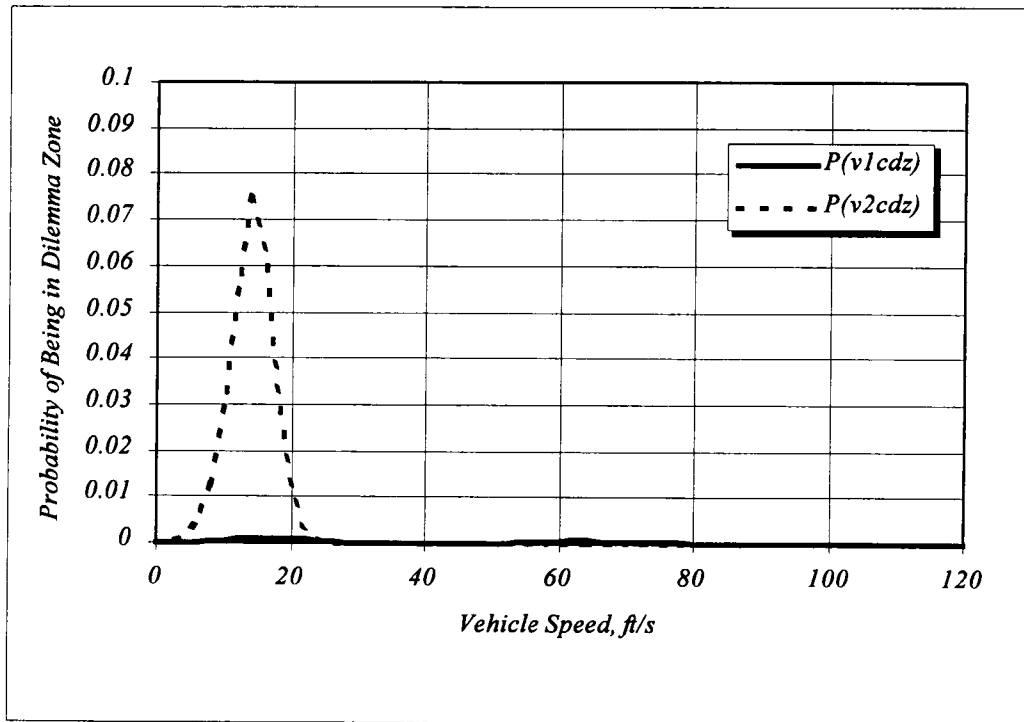


FIGURE A-11

Probability of Being in a Dilemma Zone

Given Case 1 and Case 2 Speed Distributions

$$PC(v_L) = 5 \text{ sec}$$

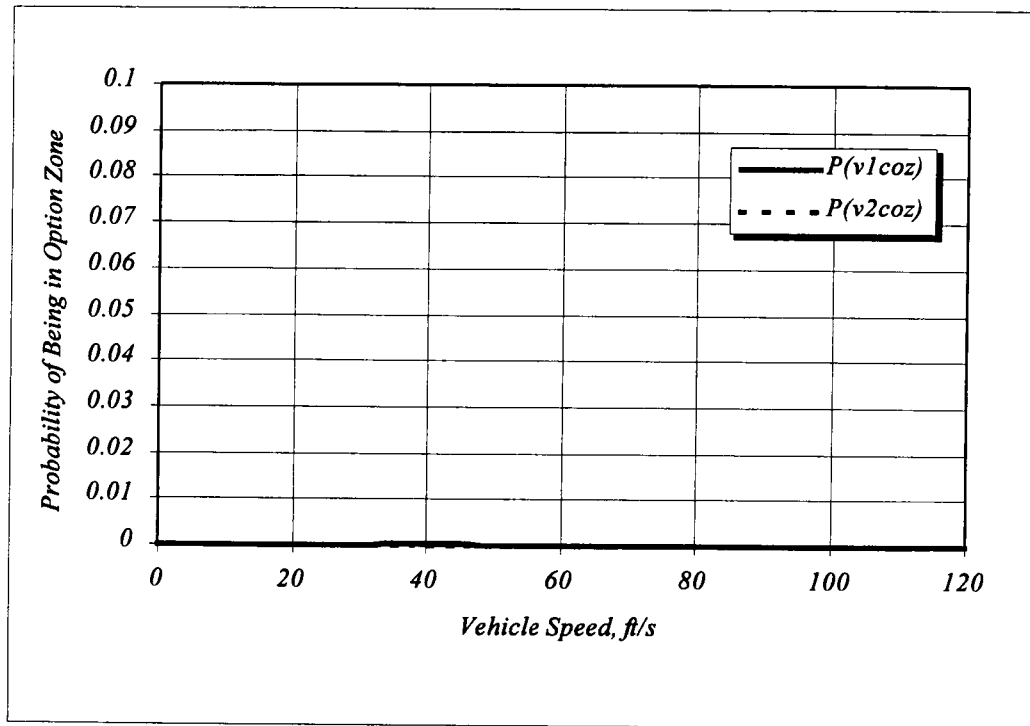


FIGURE A-12

Probability of Being in an Option Zone

Given Case 1 and Case 2 Speed Distributions

$$PC(v_L) = 5 \text{ sec}$$

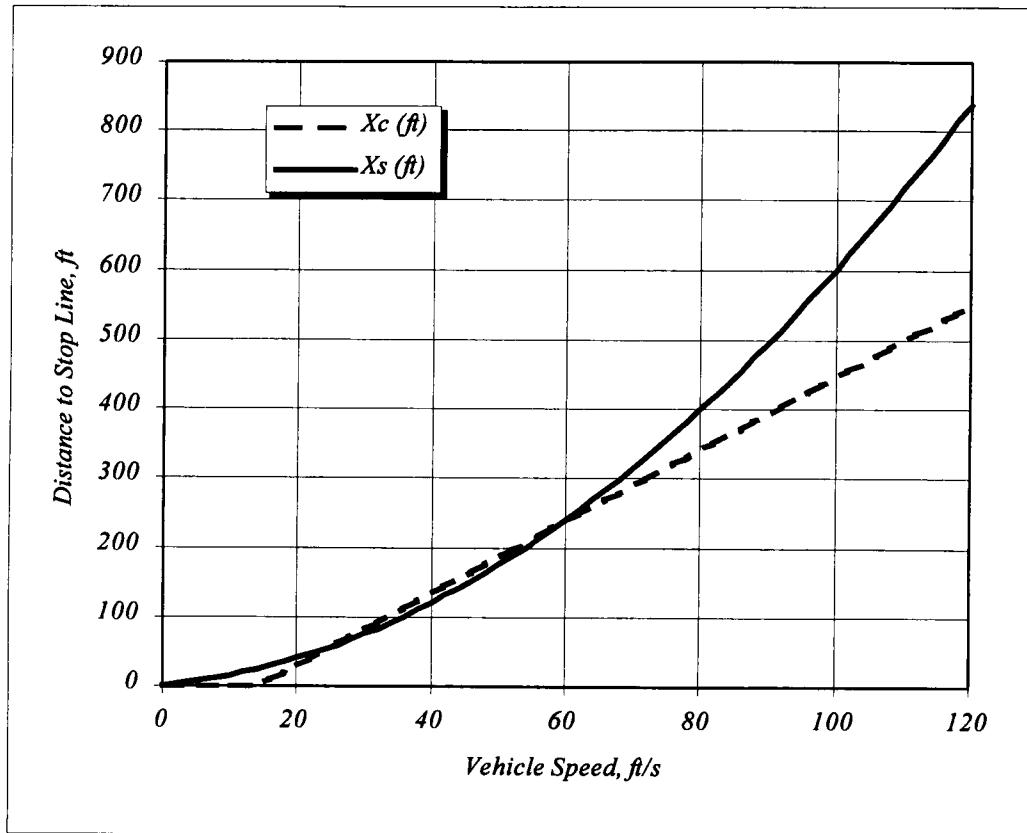


FIGURE A-13

Plot of Clearing and Stopping Distance

$$PC(v_1) = 5.2 \text{ sec}$$

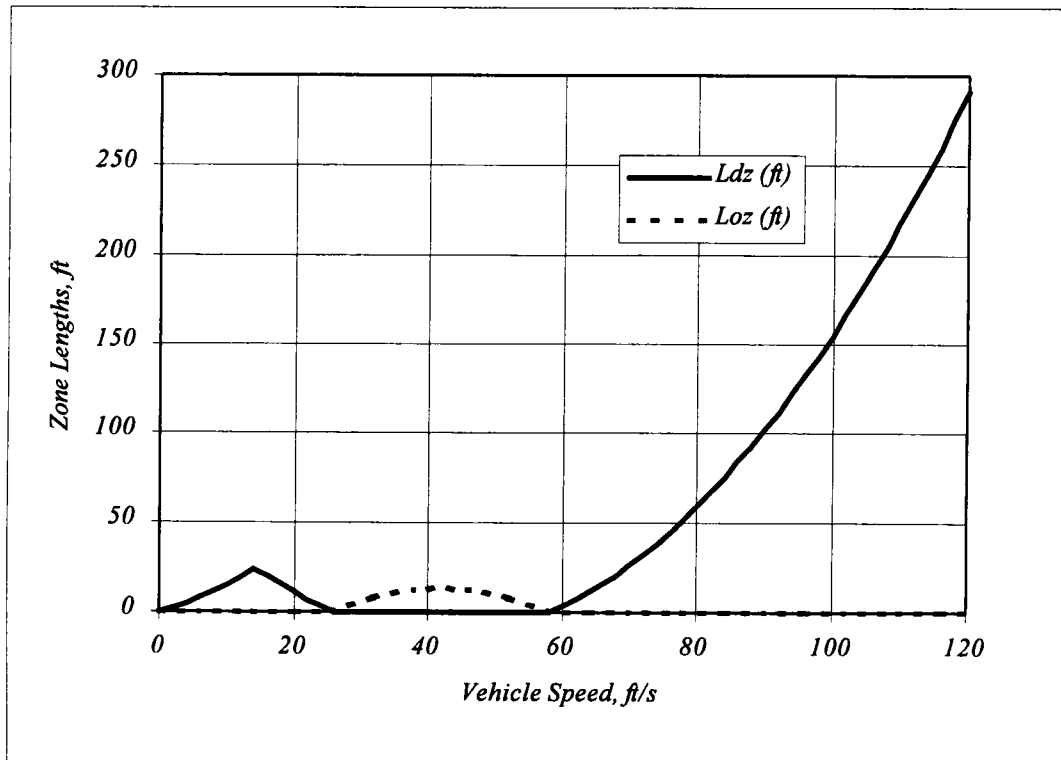


FIGURE A-14

Plot of Dilemma and Option Zone Lengths

$$PC(v_1) = 5.2 \text{ sec}$$

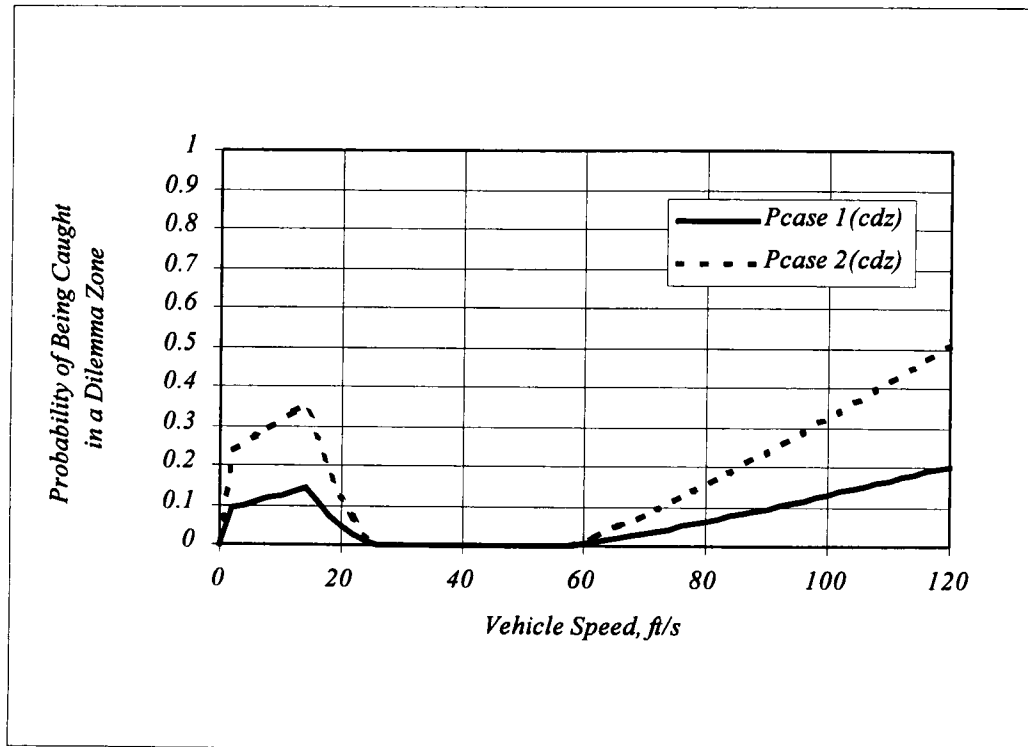


FIGURE A-15

Probability of Being Caught in a Dilemma Zone

$$PC(v_1) = 5.2 \text{ sec}$$

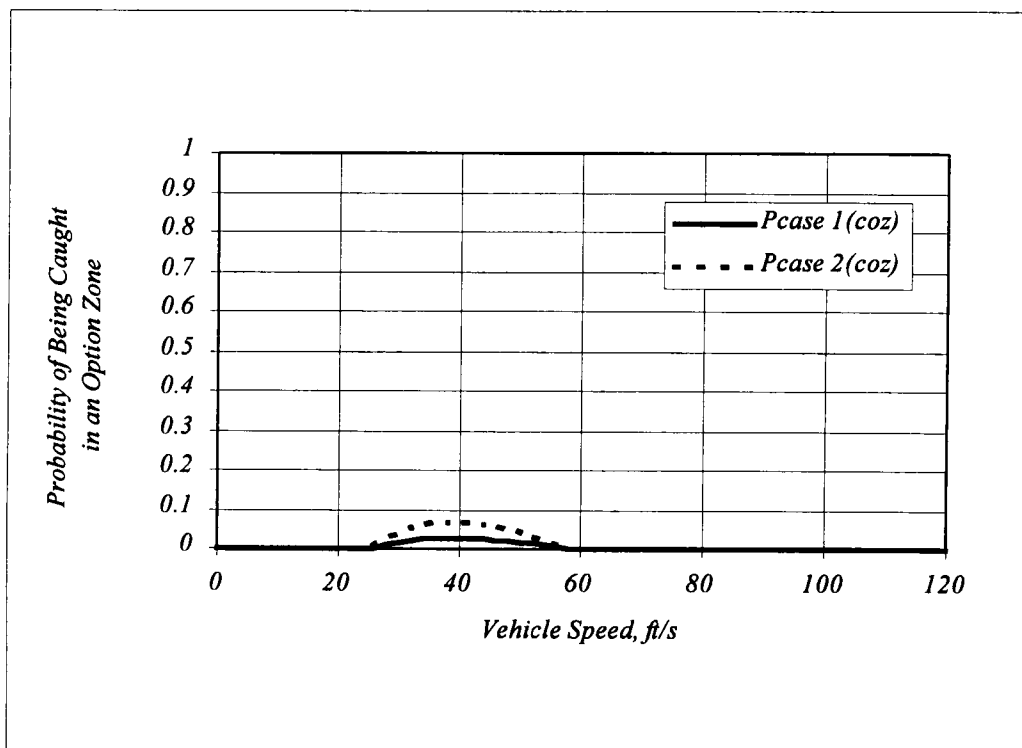


FIGURE A-16

Probability of Being Caught in an Option Zone

$$PC(v_1) = 5.2 \text{ sec}$$

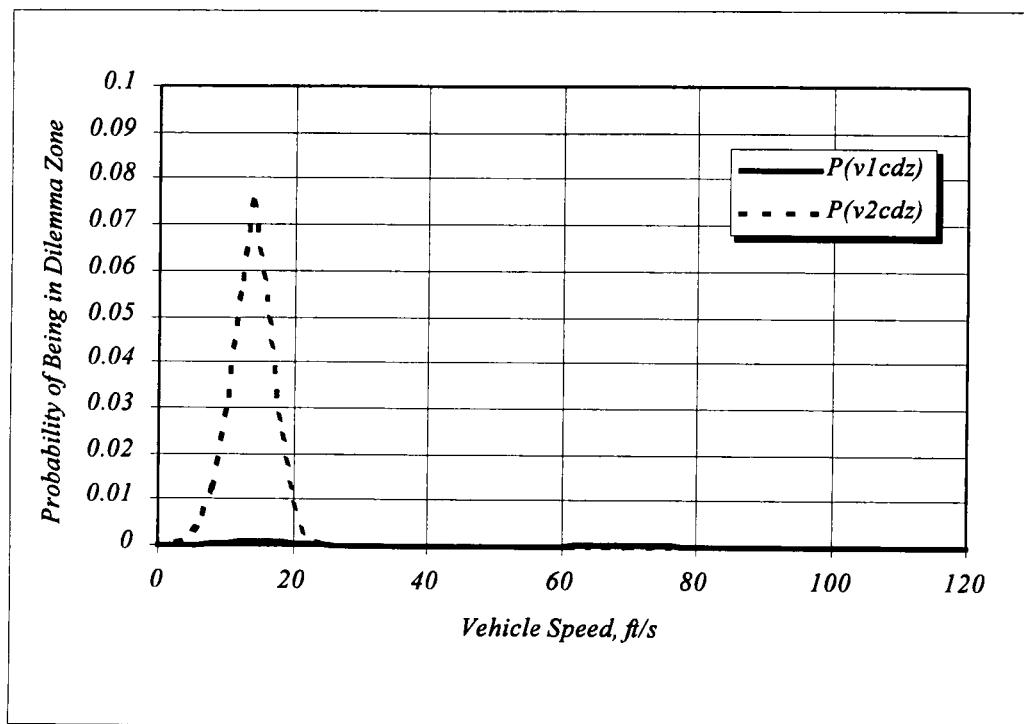


FIGURE A-17

Probability of Being in a Dilemma Zone

Given Case 1 and Case 2 Speed Distributions

$$PC(v_1) = 5.2 \text{ sec}$$

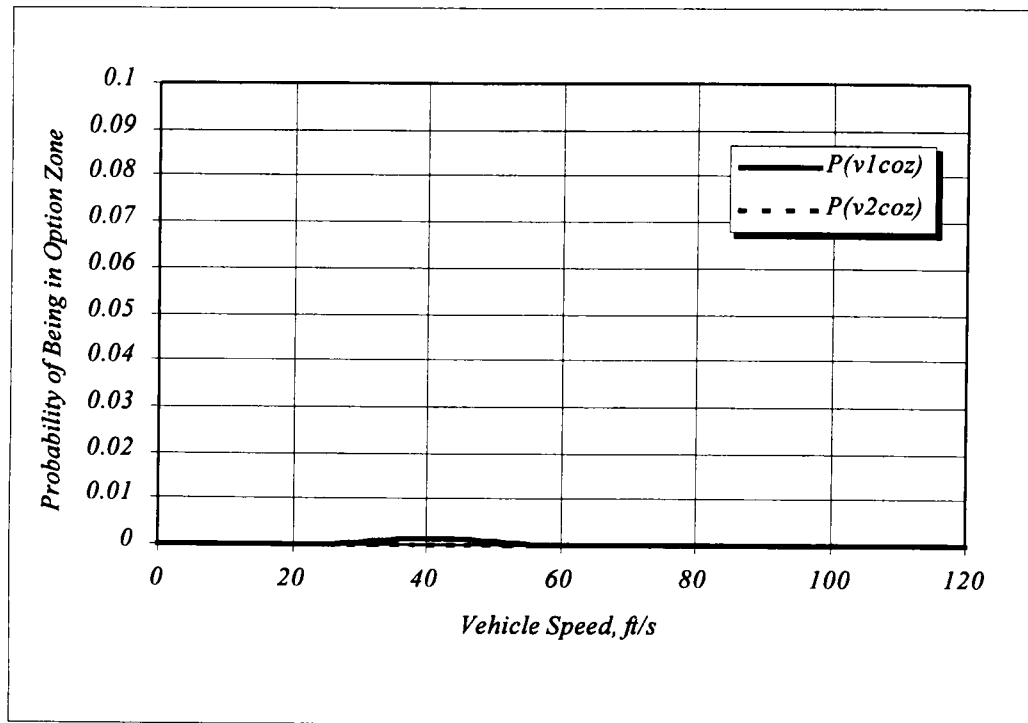


FIGURE A-18

Probability of Being in an Option Zone

Given Case 1 and Case 2 Speed Distributions

$$PC(v_1) = 5.2 \text{ sec}$$

APPENDIX B

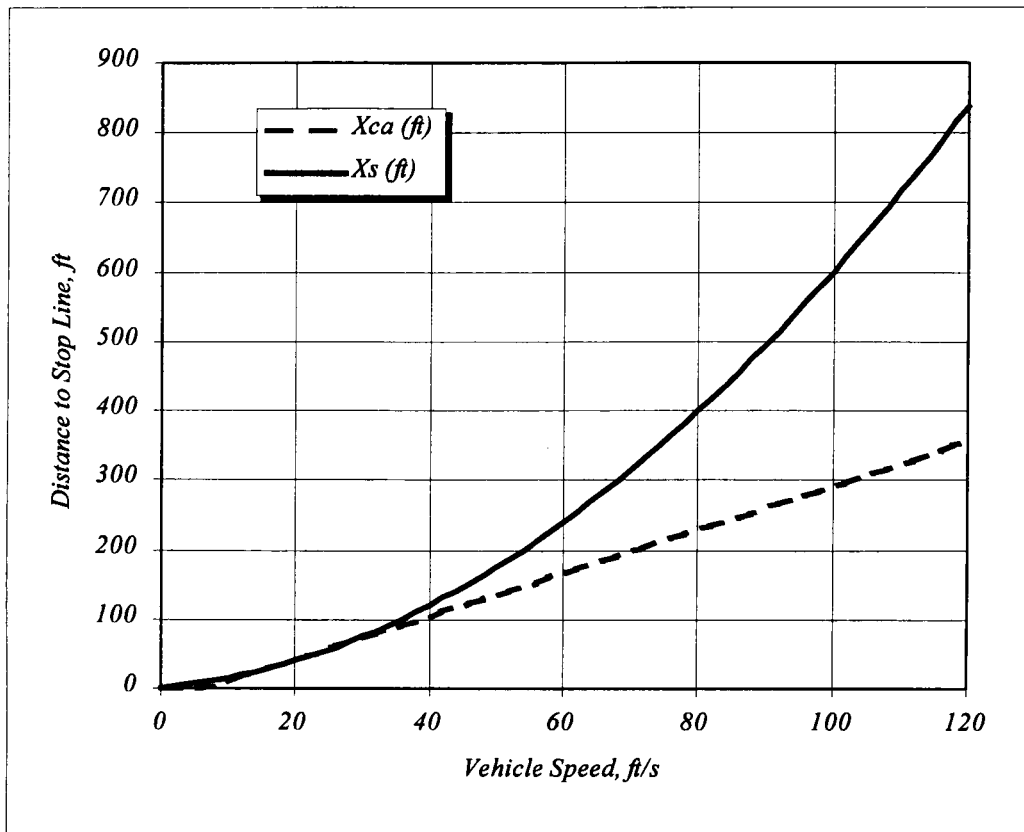


FIGURE B-1

Plot of Clearing and Stopping Distance

$PC(Y) = 3.6$ sec with Acceleration

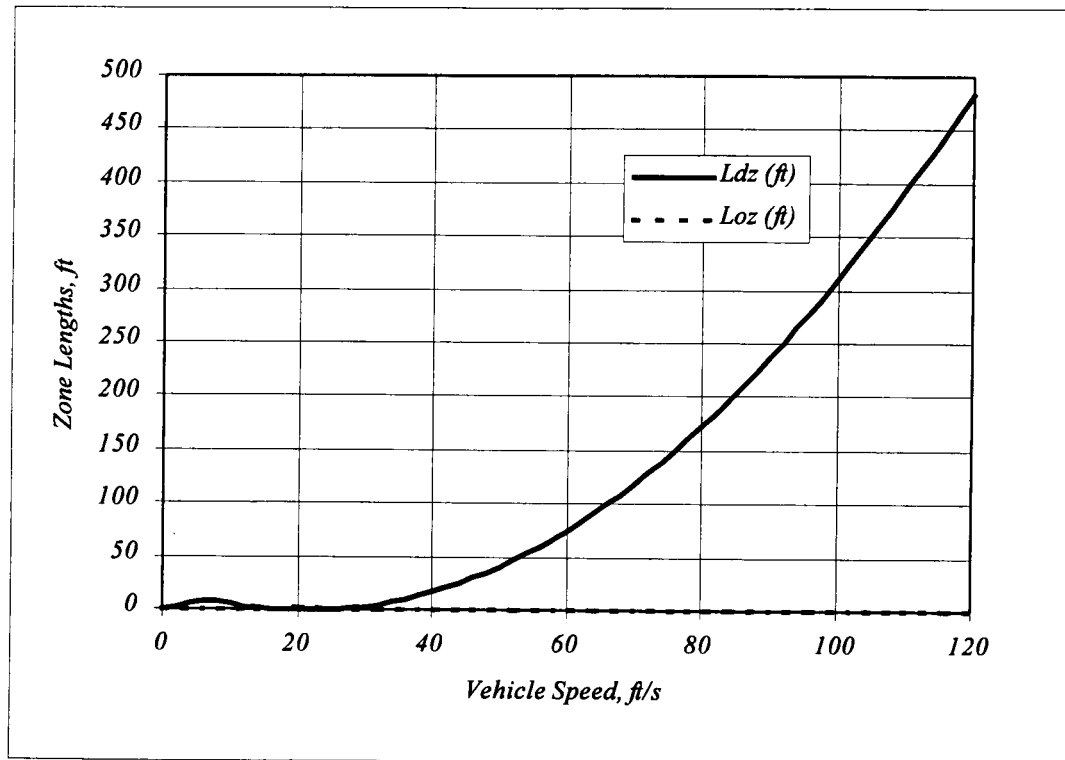


FIGURE B-2

Plot of Dilemma and Option Zone Lengths

$PC(Y) = 3.6$ sec with Acceleration

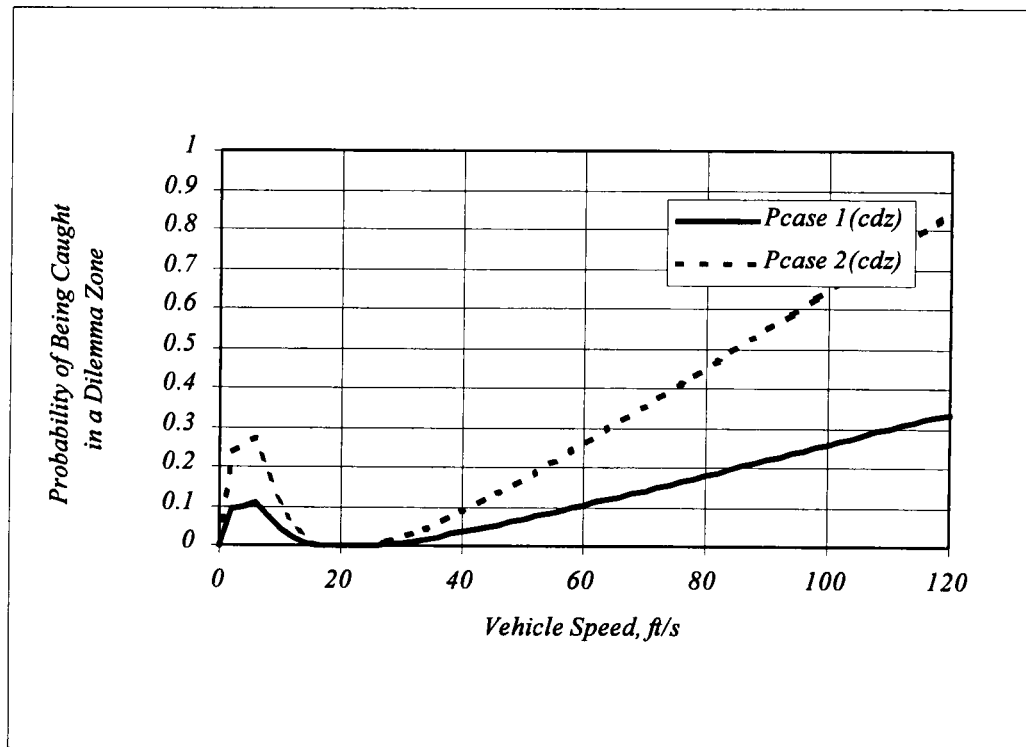


FIGURE B-3

Probability of Being Caught in a Dilemma Zone

$PC(Y) = 3.6$ sec with Acceleration

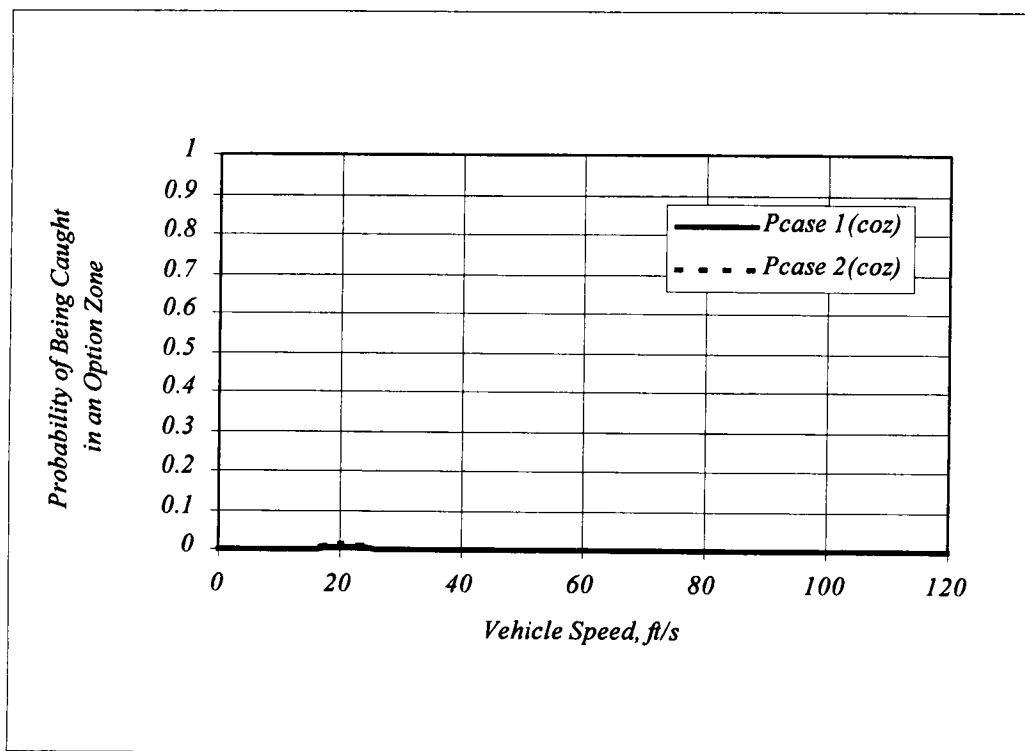


FIGURE B-4

Probability of Being Caught in an Option Zone

$PC(Y) = 3.6$ sec with Acceleration

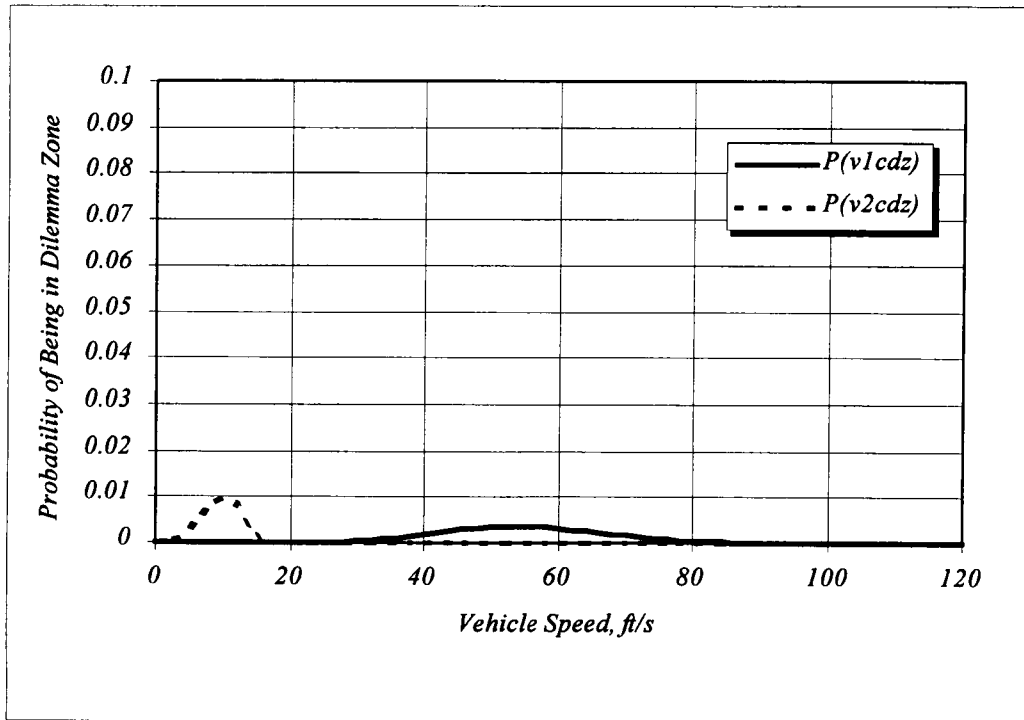


FIGURE B-5

Probability of Being in a Dilemma Zone

Given Case 1 and Case 2 Speed Distributions

$PC(Y) = 3.6$ sec with Acceleration

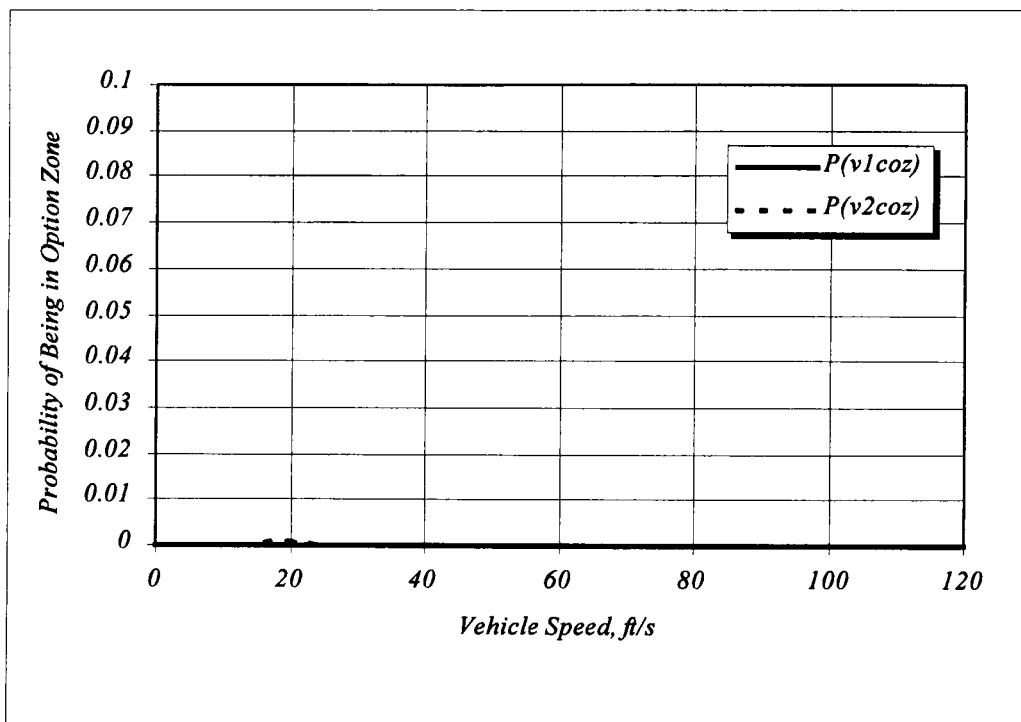


FIGURE B-6

Probability of Being in an Option Zone

Given Case 1 and Case 2 Speed Distributions

$PC(Y) = 3.6$ sec with Acceleration

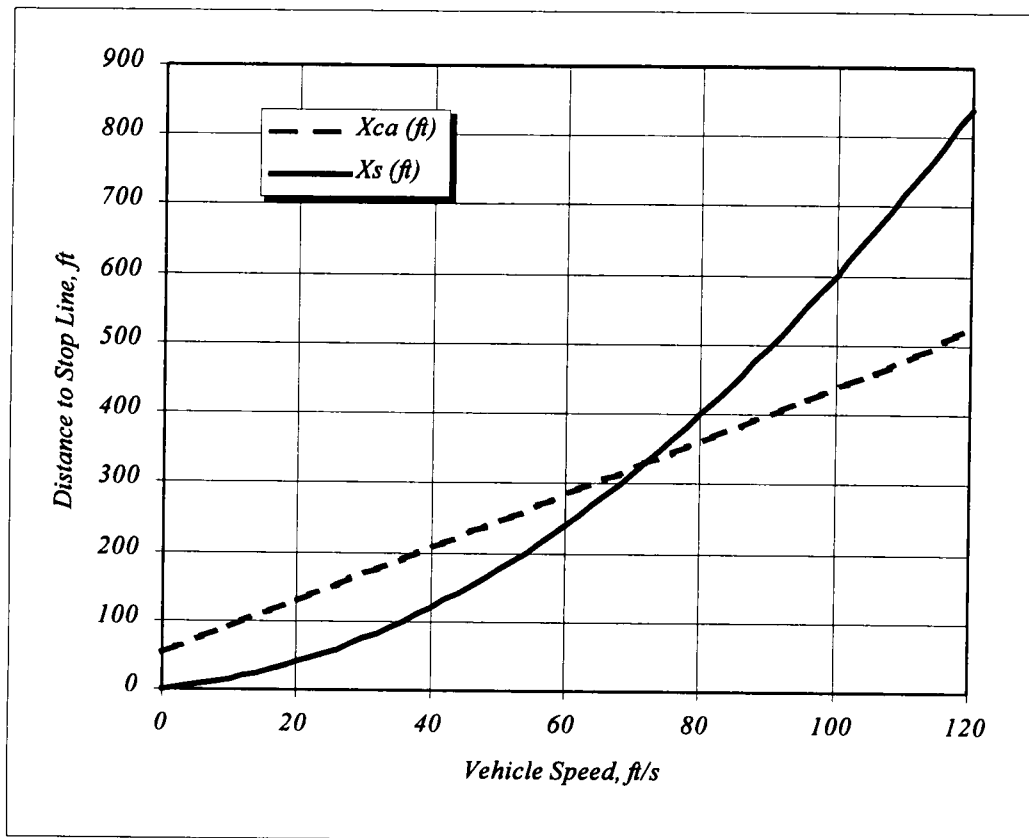


FIGURE B-7

Plot of Clearing and Stopping Distance

$PC(v_L) = 5$ sec with Acceleration

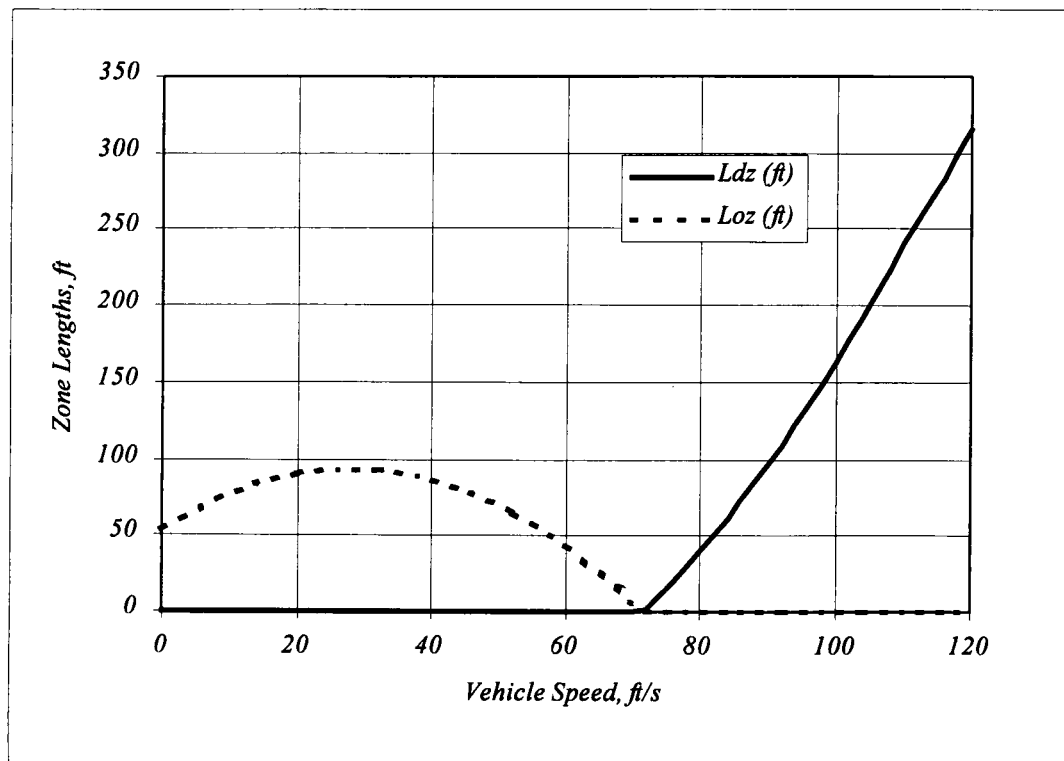


FIGURE B-8

Plot of Dilemma and Option Zone Lengths

$PC(v_L) = 5$ sec with Acceleration

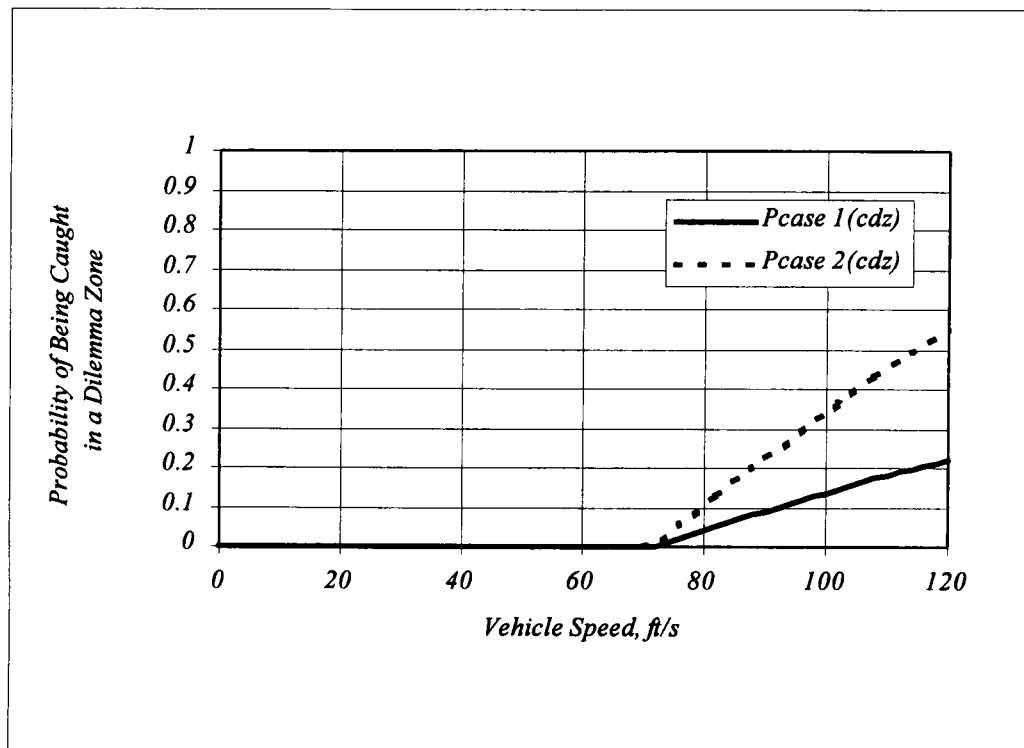


FIGURE B-9

Probability of Being Caught in a Dilemma Zone

$PC(v_L) = 5 \text{ sec with Acceleration}$

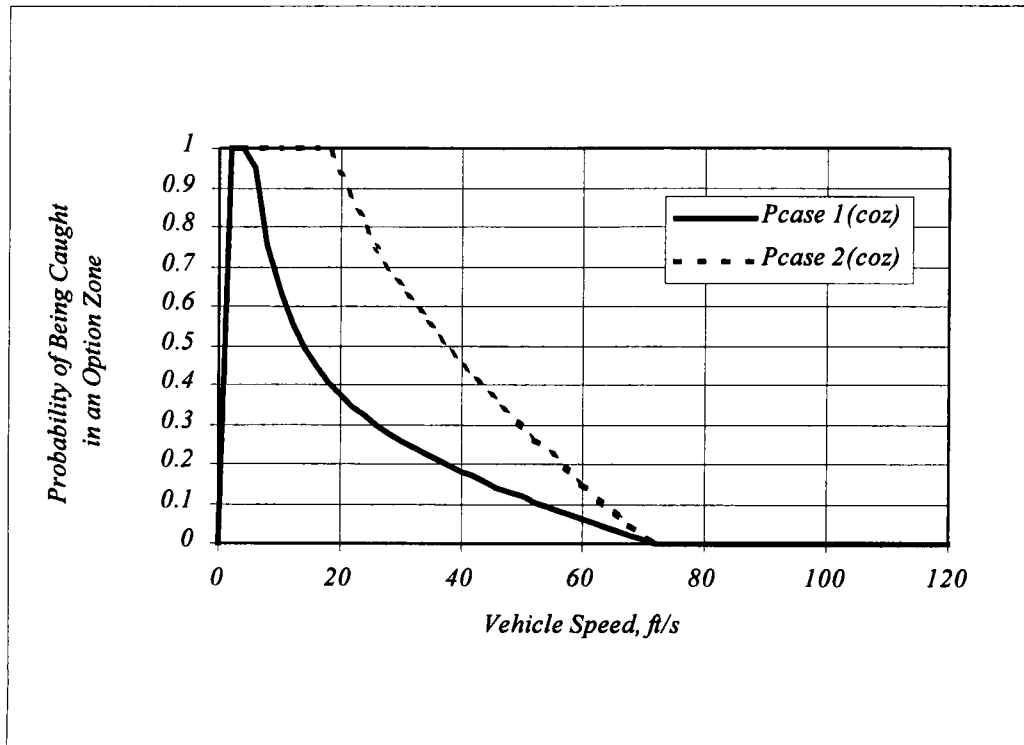


FIGURE B-10

Probability of Being Caught in an Option Zone

$PC(v_L) = 5 \text{ sec with Acceleration}$

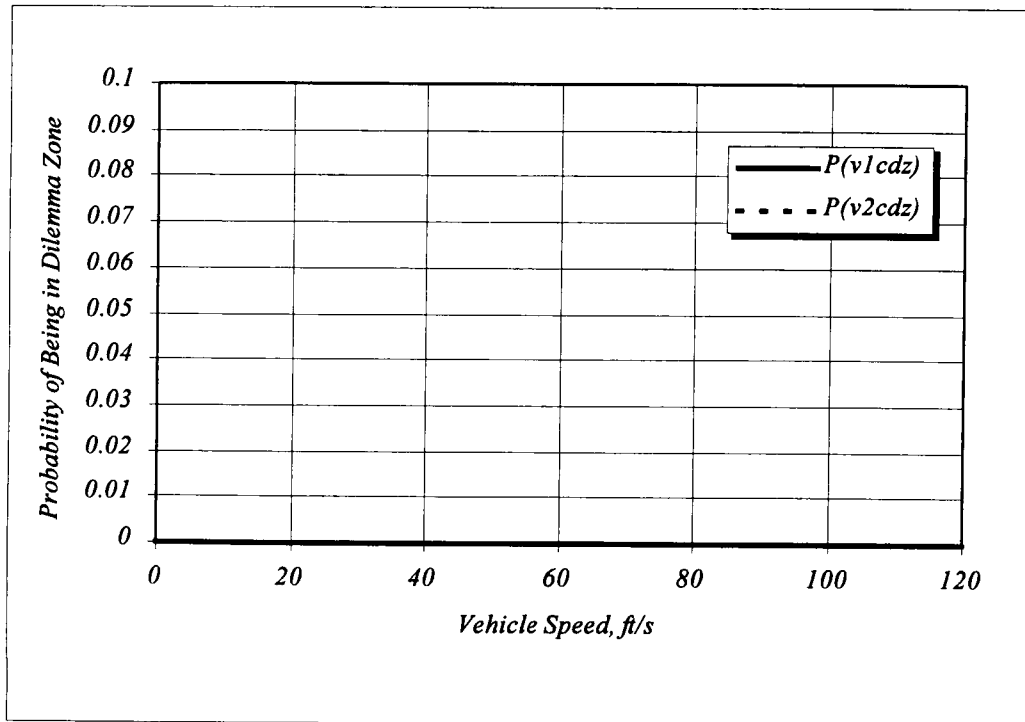


FIGURE B-11

Probability of Being in a Dilemma Zone

Given Case 1 and Case 2 Speed Distributions

$PC(v_L) = 5$ sec with Acceleration

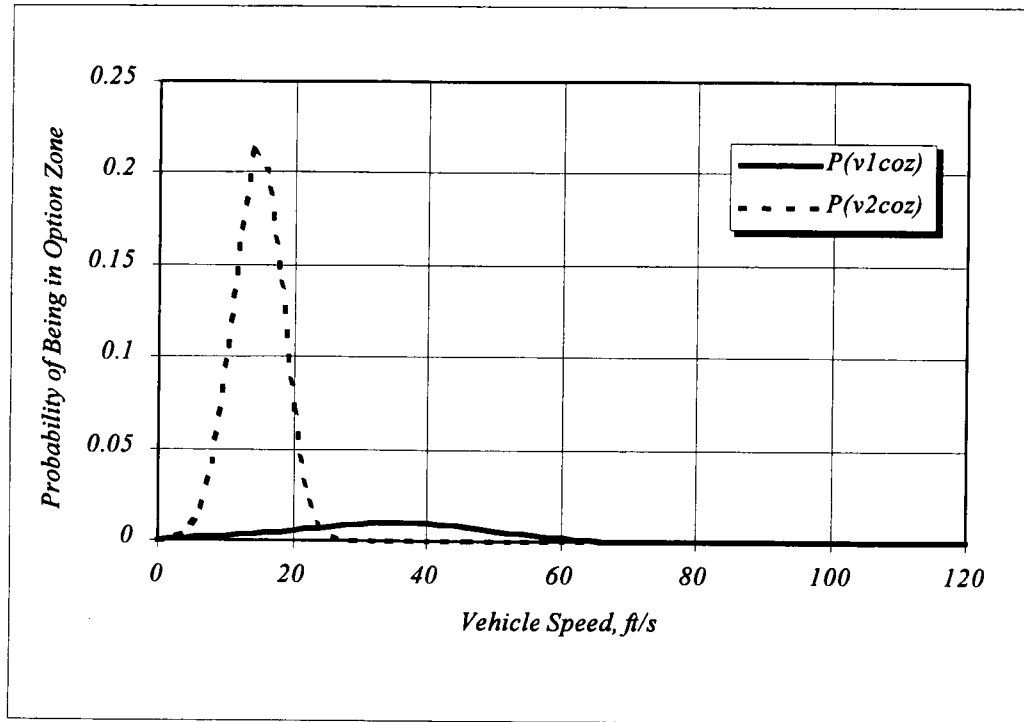


FIGURE B-12

Probability of Being in an Option Zone

Given Case 1 and Case 2 Speed Distributions

$PC(v_L) = 5$ sec with Acceleration

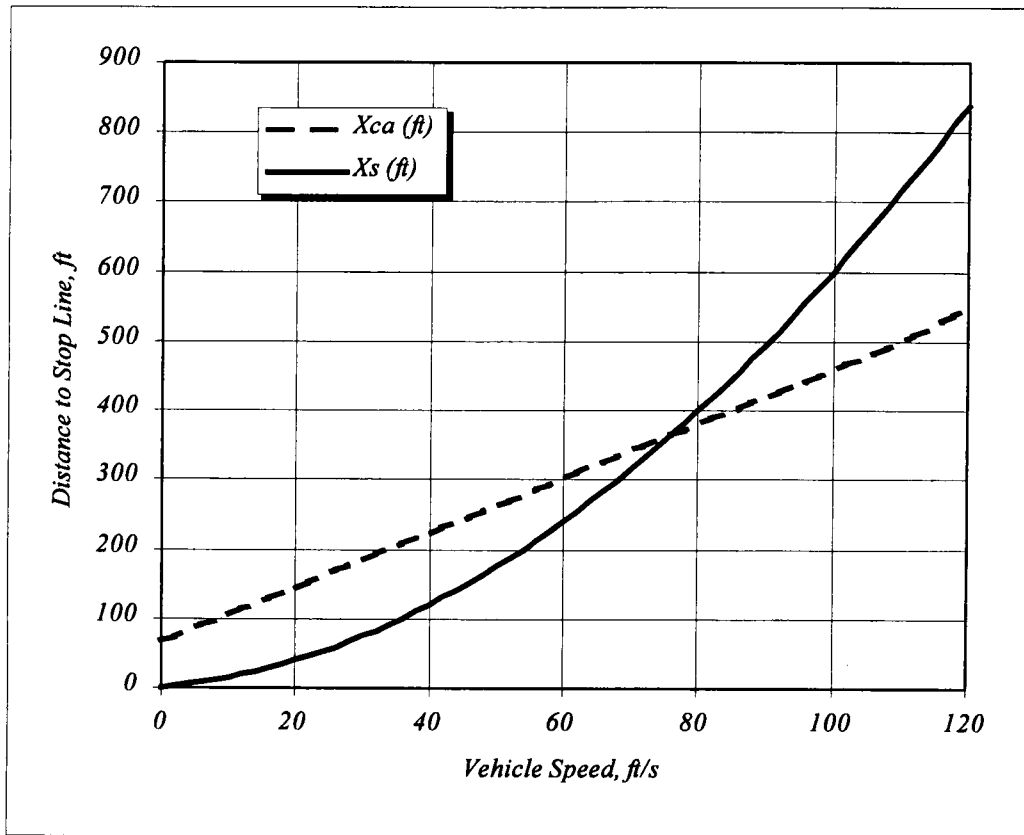


FIGURE B-13

Plot of Clearing and Stopping Distance

$PC(v_1) = 5.2$ sec with Acceleration

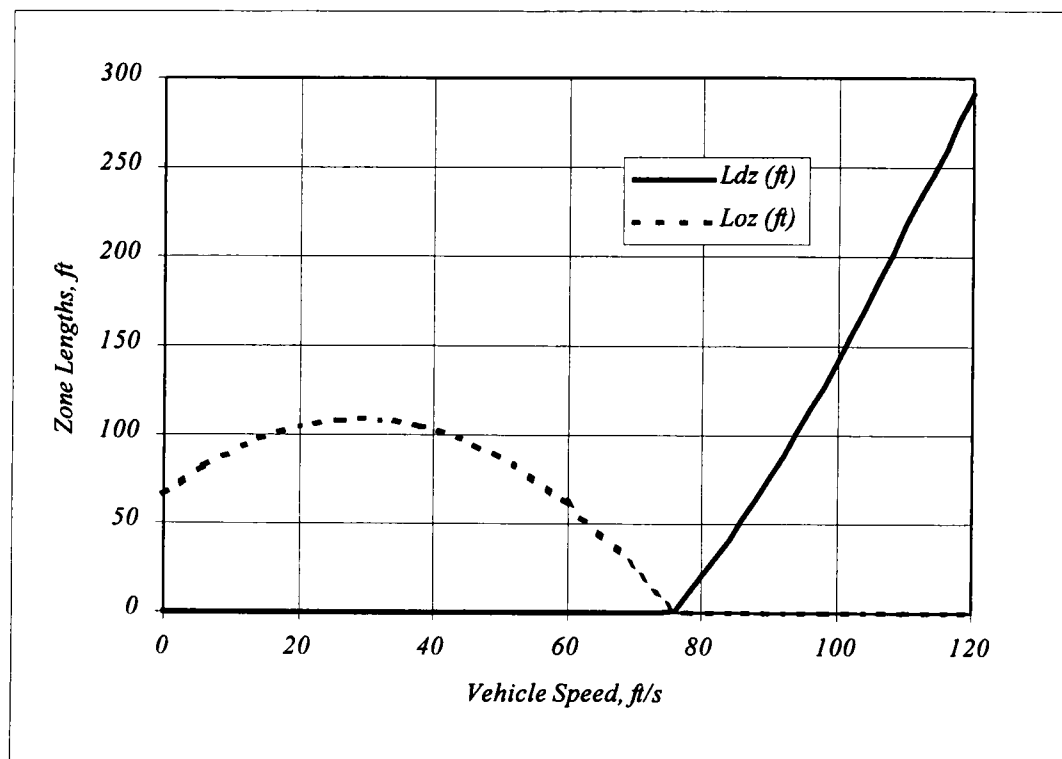


FIGURE B-14

Plot of Dilemma and Option Zone Lengths

$PC(v_1) = 5.2$ sec with Acceleration

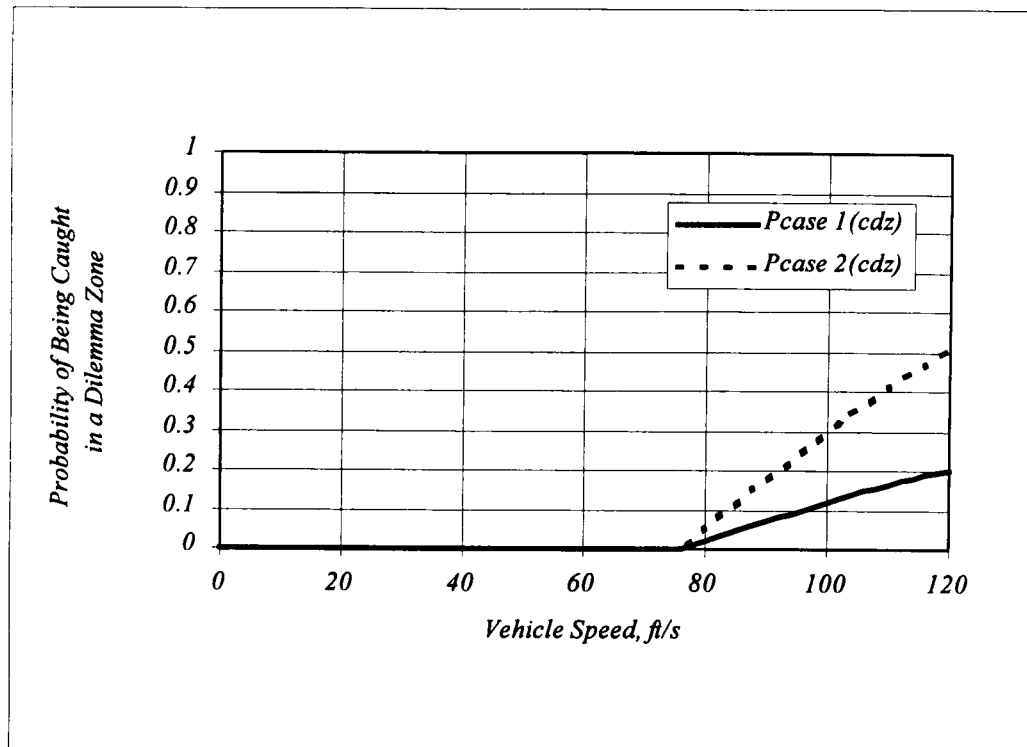


FIGURE B-15

Probability of Being Caught in a Dilemma Zone

$PC(v_1) = 5.2$ sec with Acceleration

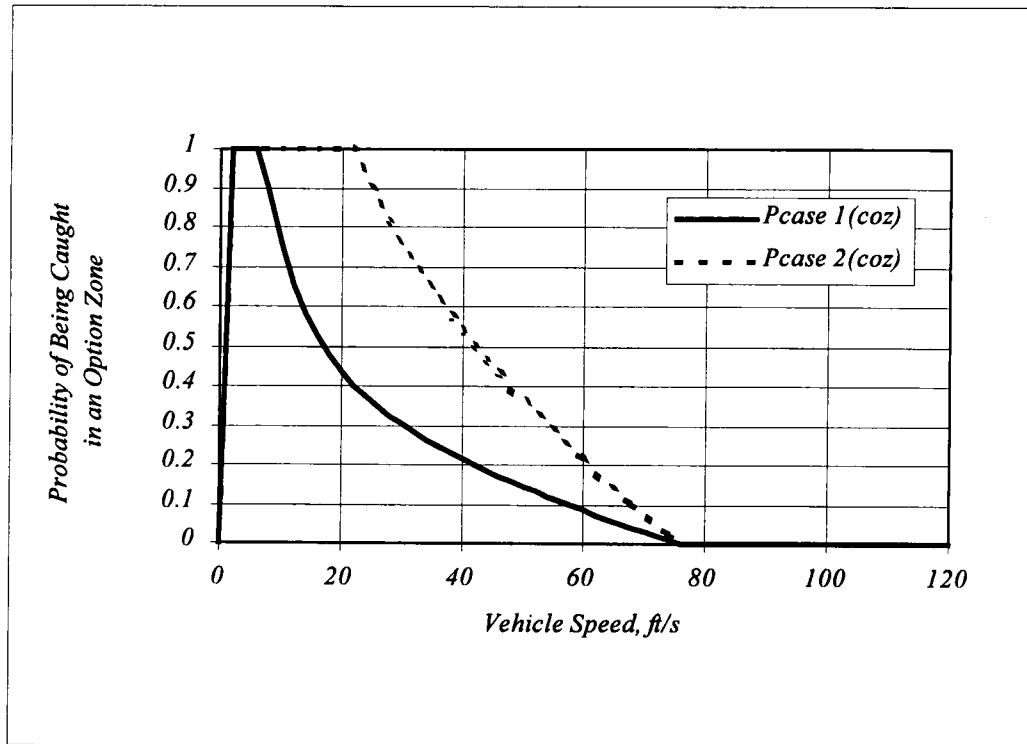


FIGURE B-16

Probability of Being Caught in an Option Zone

$PC(v_1) = 5.2$ sec with Acceleration

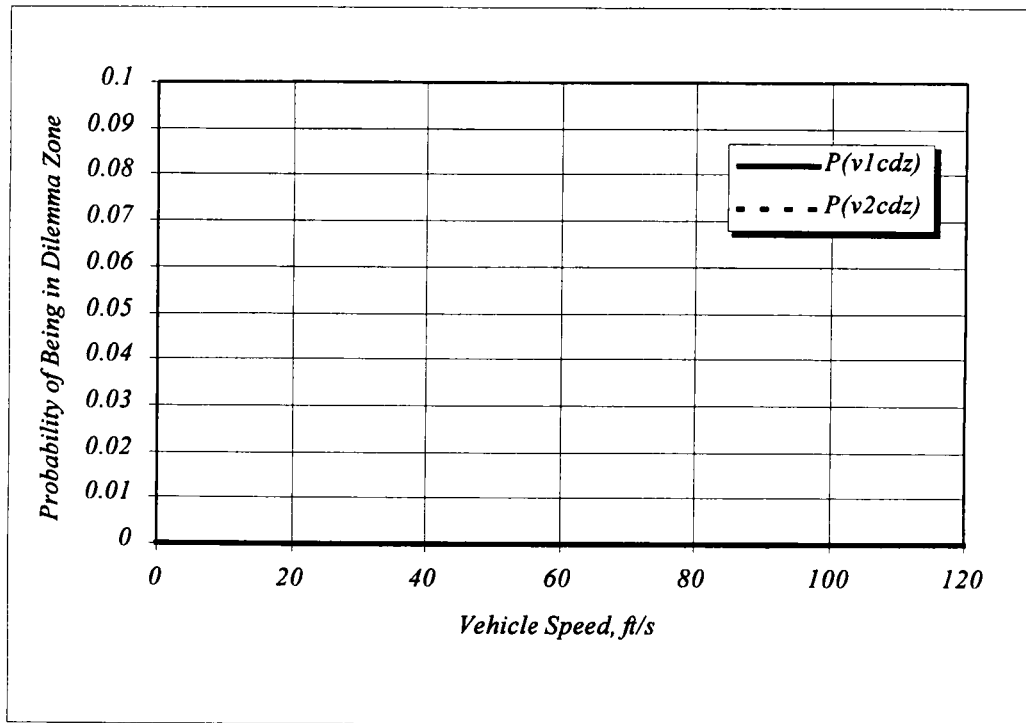


FIGURE B-17

Probability of Being in a Dilemma Zone

Given Case 1 and Case 2 Speed Distributions

$PC(v_1) = 5.2$ sec with Acceleration

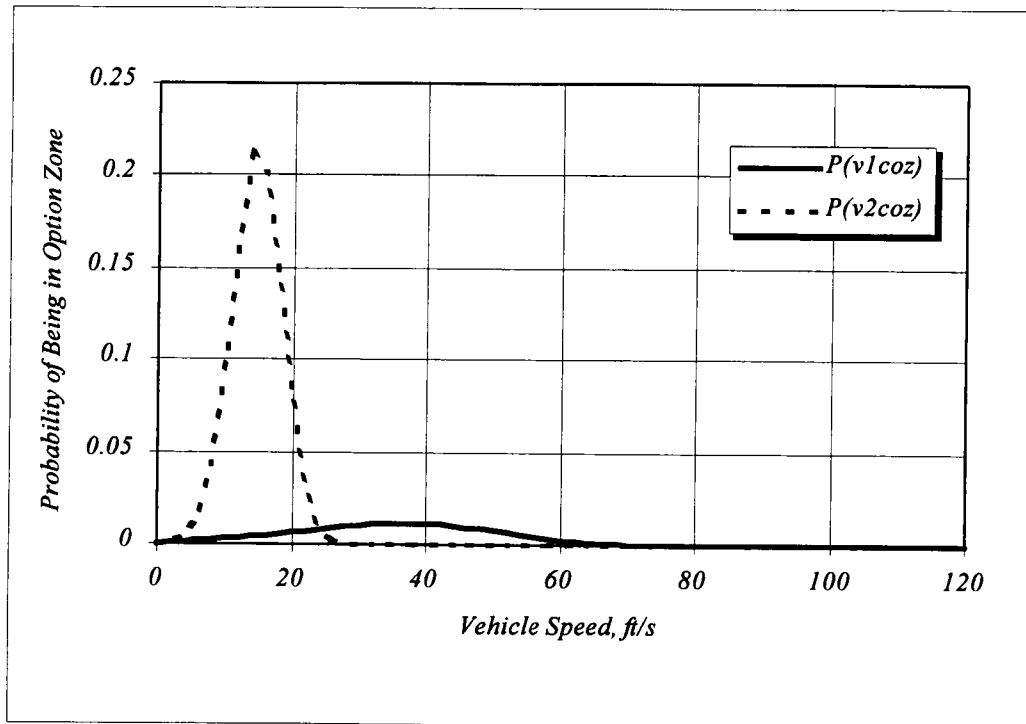


FIGURE B-18

Probability of Being in an Option Zone

Given Case 1 and Case 2 Speed Distributions

$PC(v_1) = 5.2$ sec with Acceleration

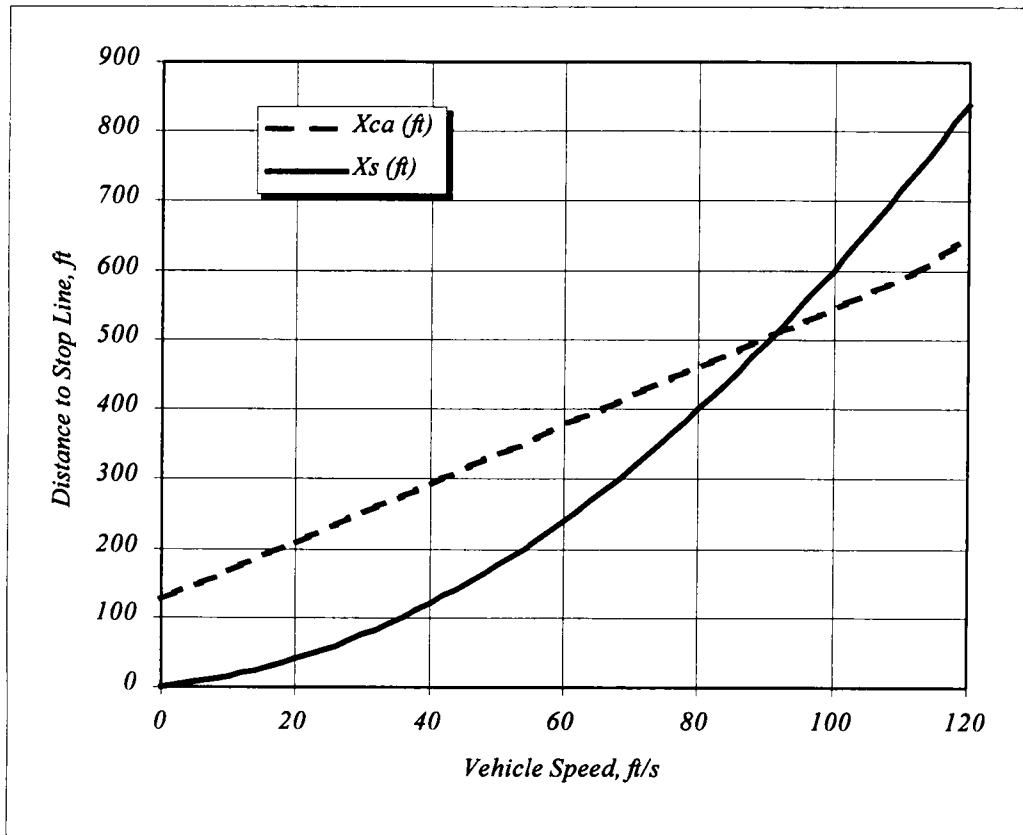


FIGURE B-19

Plot of Clearing and Stopping Distance

$PC(v_2) = 6$ sec with Acceleration

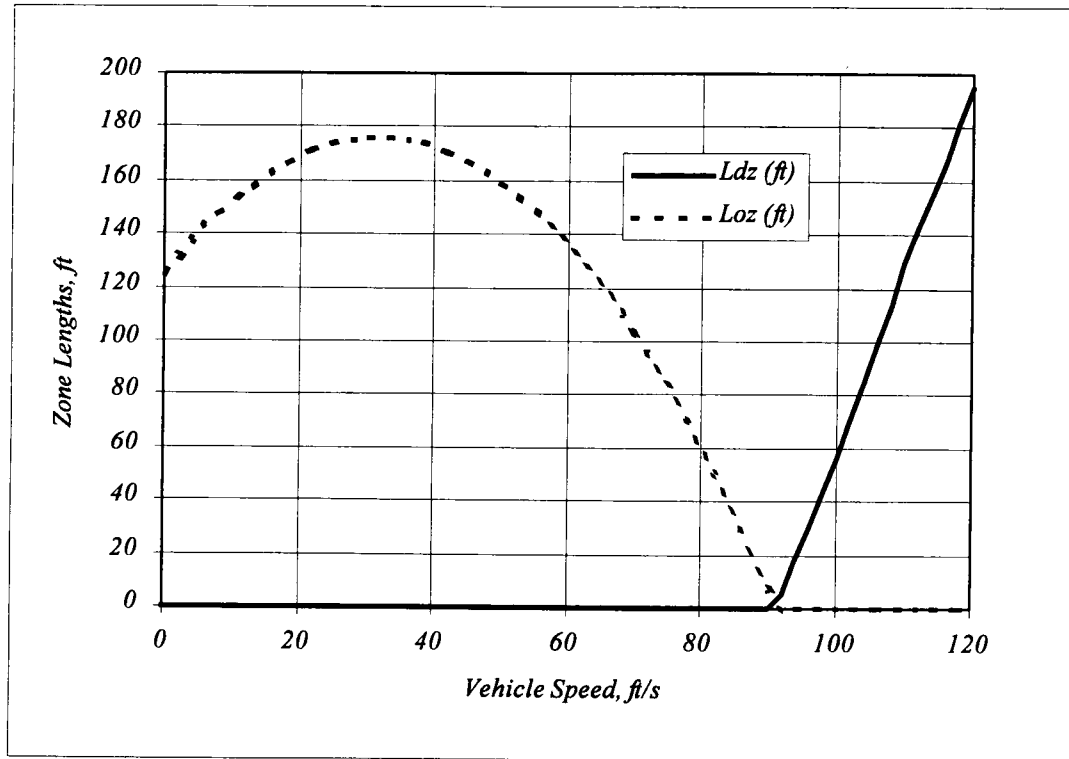


FIGURE B-20

Plot of Dilemma and Option Zone Lengths

$PC(v_2) = 6$ sec with Acceleration

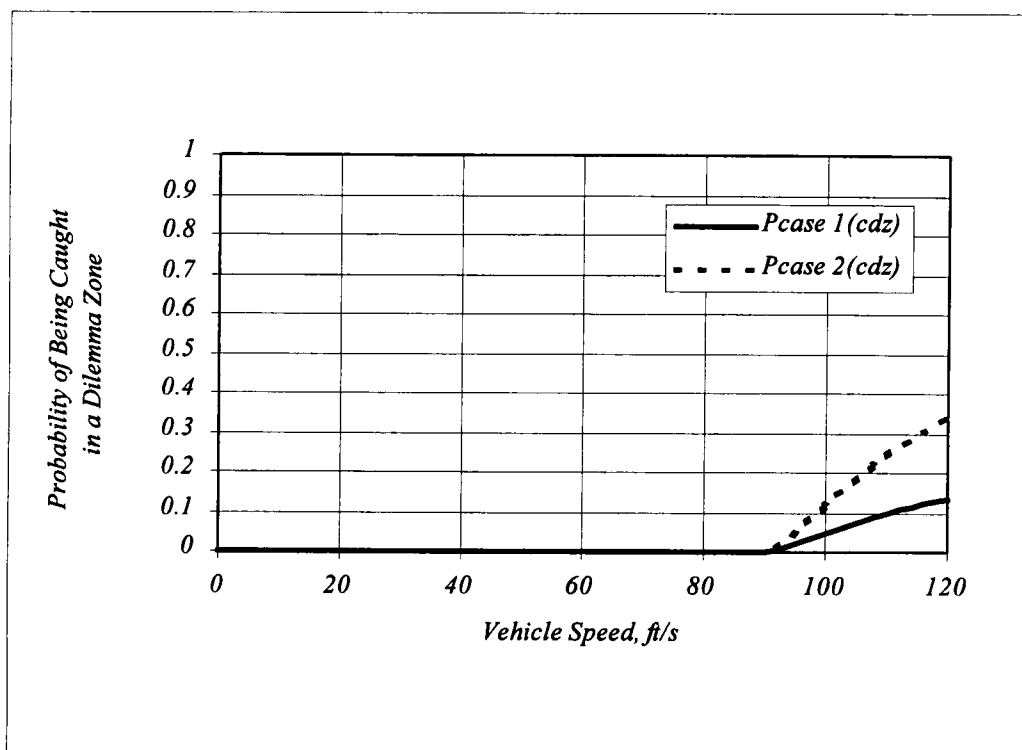


FIGURE B-21

Probability of Being Caught in a Dilemma Zone

$PC(v_2) = 6 \text{ sec with Acceleration}$

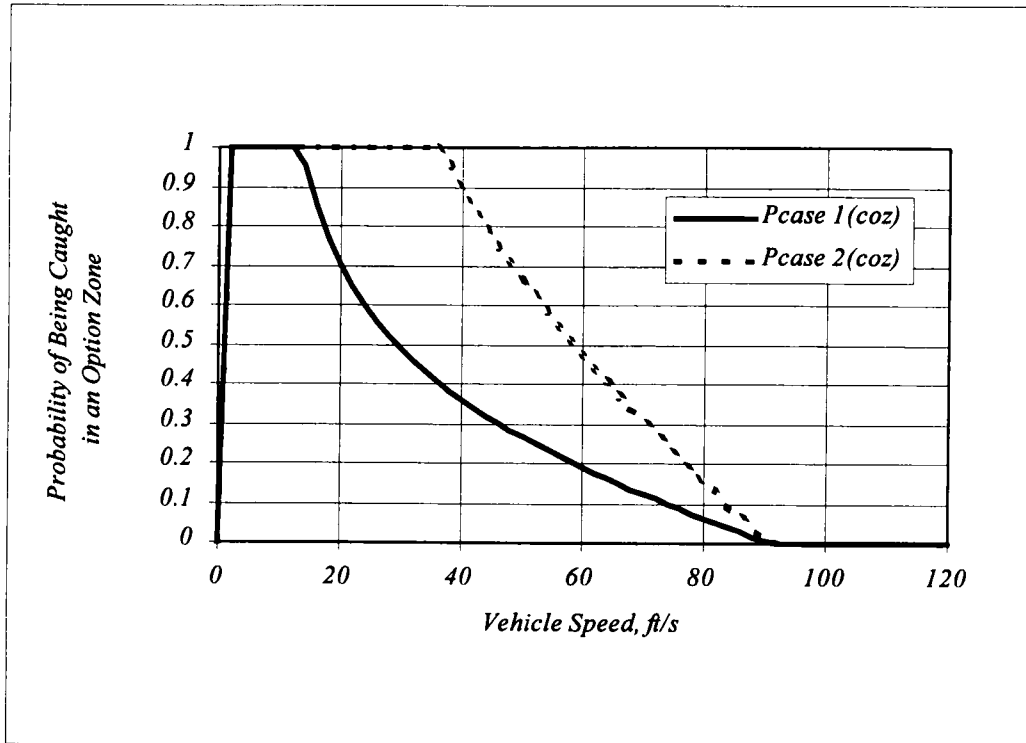


FIGURE B-22

Probability of Being Caught in an Option Zone

$PC(v_2) = 6 \text{ sec with Acceleration}$

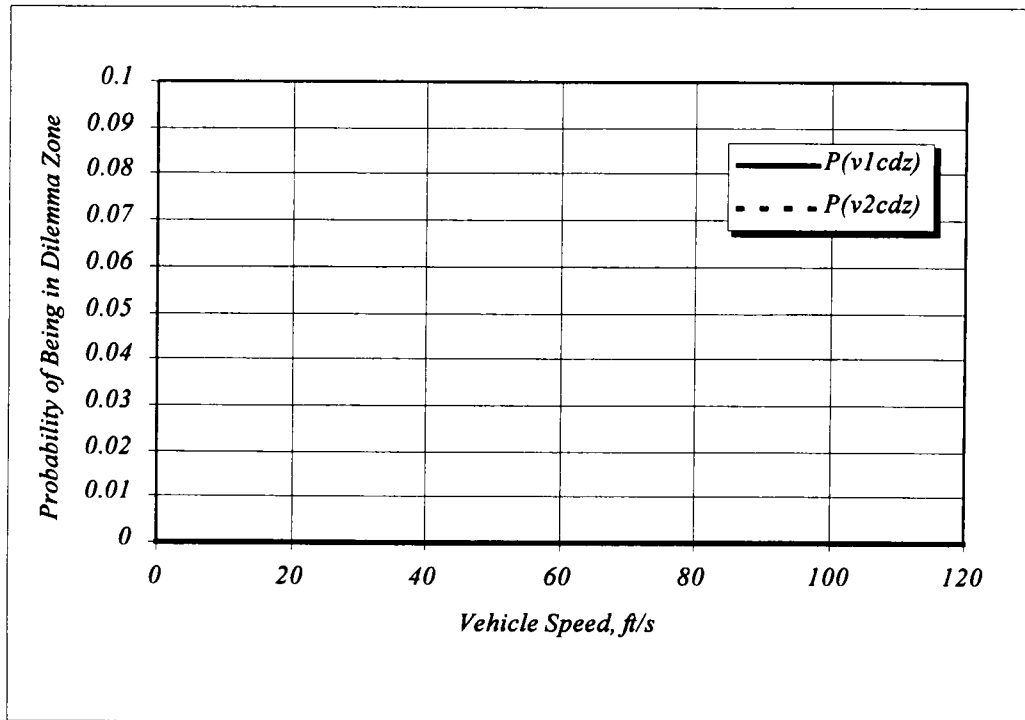


FIGURE B-23

Probability of Being in a Dilemma Zone

Given Case 1 and Case 2 Speed Distributions

$PC(v_2) = 6$ sec with Acceleration

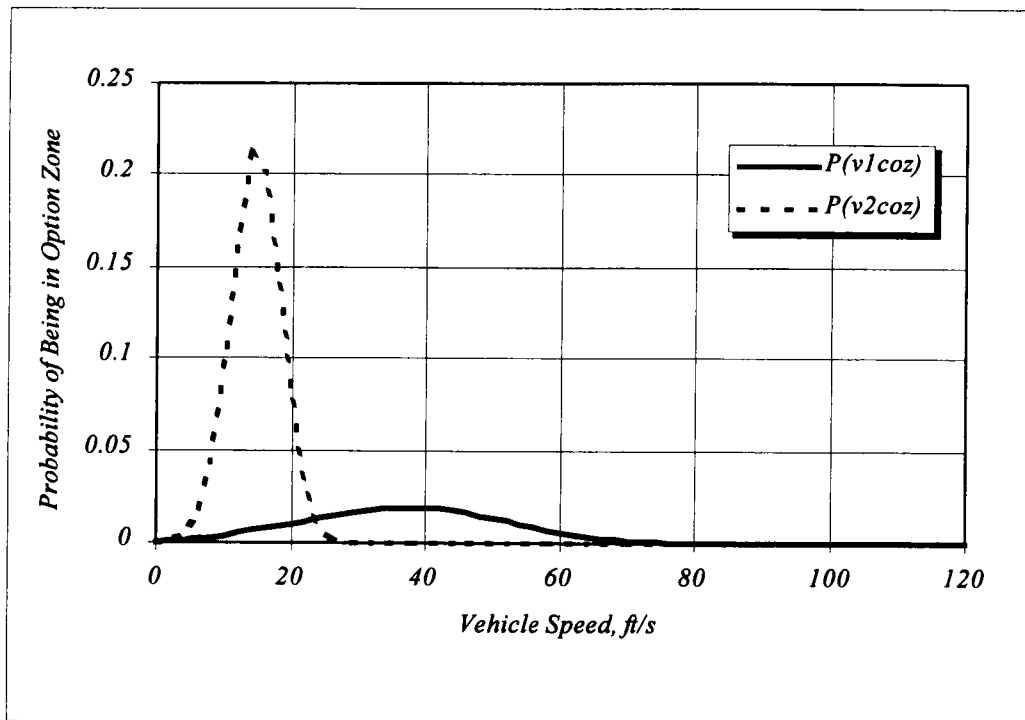


FIGURE B-24

Probability of Being in an Option Zone

Given Case 1 and Case 2 Speed Distributions

$PC(v_2) = 6$ sec with Acceleration