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ENGINEERING FACTORS AFFECTING TRAFFIC SIGNAL YELLOW TIME

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Final Report

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FHWA Contract Manager: Alberto J. Santiago (HSR-40)
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16. Abstract

Driver behavior to traffic signal change intervals (yellow plus all-red) was evaluated using the data collected from timelapse cameras at seven sites. Particularly, signal change interval timing was examined as a function of drivers' response characteristics involving their speed, distance, and time to reach the stop line.

Drivers' selected yellow response times and deceleration rates were analyzed. New perception and brake reaction time of 1.2 seconds with a 10.5 ft/s/s of deceleration rate for level grade appeared to be good estimators.

Four alternative methods to design signal change interval were discussed. These included (1) a continued use of the current formula using one perception-brake reaction time and one deceleration rate for all speeds, (2) a continued use of the current formula using different perception-brake reaction times and deceleration rates for different speeds, (3) the use of time to reach the stop line by clearing vehicles, and (4) the use of probability of a vehicle stopping. A new method to determine an all-red interval was presented.

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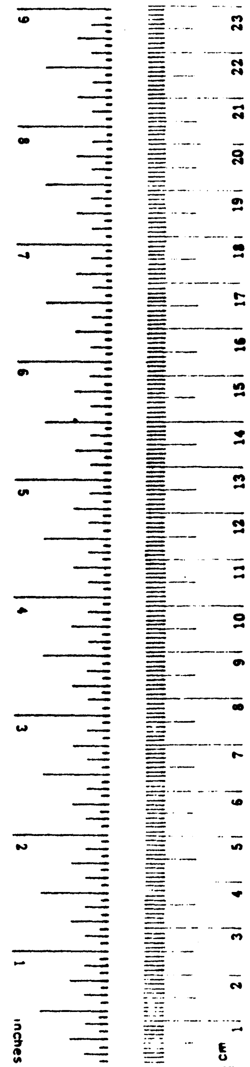
METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

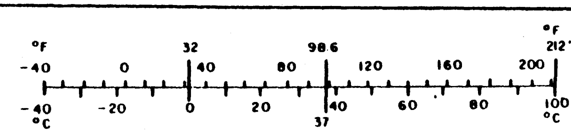
Symbol	When You Know	Multiply by	To Find	Symbol
	LENGTH			
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
	AREA			
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
	MASS (weight)			
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
	VOLUME			
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
	TEMPERATURE (exact)			
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
	LENGTH			
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
	AREA			
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
	MASS (weight)			
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
	VOLUME			
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
	TEMPERATURE (exact)			
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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I. INTRODUCTION

To stop or not to stop? That is the question asked about one-half billion times a day in the United States alone as drivers see the green light at a traffic signal ahead change to yellow and then to red. If the driver fails to respond safely, a major right-angle collision at the intersection is possible from cross street traffic. On the other hand, if the driver is startled or overreacts, a hazardous rear-end collision is possible with following vehicles. Due to the complexity of the driver-vehicle-environment control system involved and the potential severe consequences of system failure (a fatal accident), the design of the traffic signal yellow time and any following all-red interval should be optimized based on the best understanding of the engineering factors involved. The magnitude of the problem requires that traffic engineers do no less.

PROBLEM STATEMENT

The yellow signal indication is a warning of the impending loss of right-of-way to the traffic receiving the previous green phase. Upon seeing the yellow onset, drivers decide to stop or go on through the intersection based on judgment. An analysis of the physical laws, empirical evidence, and personal driving experience suggests that drivers' behavior in this situation appears to be affected by vehicle speed, position, and physical characteristics together with other geometric, environmental, and possibly traffic control factors.

The traffic signal change intervals (yellow plus all-red) are currently designed by most agencies responsible for traffic control based on the following equation:

$$Y + AR = t + \frac{v}{2d} + \frac{l + w}{v} \qquad (1)$$

where

- $Y + AR$ = duration of change intervals (yellow plus all-red), seconds.
- t = perception and brake reaction time of driver, seconds.
- v = approach speed, feet per second (ft/s).
- d = deceleration rate, feet per second per second (ft/s^2).

w = width of intersection, feet.

l = length of vehicle, feet.

Many researchers studied the magnitude of perception and brake reaction times (t) and deceleration rates (d) used in equation 1. However, the complete understanding of these two variables along with their variability associated with vehicle position and speed has not been well studied in the past. In addition, safety concern surrounding right-angle accidents that may be partially attributable to the signal change interval raises a possibility of an alternative change intervals design approach based on vehicles going through the intersection. Further, the driver's decision to stop or go through needs to be understood in order to critically examine the change intervals design as a function of driver behavior.

OBJECTIVES OF STUDY

The objectives of this study are to develop a comprehensive understanding of drivers' responses to the change interval, to examine change interval design as a function of driver behavior, and to quantify the values of the variables associated with drivers' perception and brake reaction times, and deceleration rates as applied in equation 1.

SCOPE OF WORK

To achieve the objectives of this study, field studies were conducted by which drivers' responses to change interval along with the characteristics of reaction time and deceleration rate could be analyzed.

To assist state and local transportation agencies to collect and analyze driver behavior data relating to change interval, a data collection manual has been prepared as a companion project report and documented separately.(1)

RESEARCH PLAN

To enhance reader understanding, the state-of-the-art on traffic signal change interval design is first presented. A research plan was structured to perform the sequence of data collection, reduction, and analysis procedures. The detailed activities performed in the study will be presented in this report.

Data collection was performed at seven sites using timelapse cameras. The collected data were reduced using timelapse data analyzer projectors. The reduced data were input to the computer for analysis. The data analysis was performed using statistical analysis techniques. The study findings and their implications along with applications were provided.

II. STATE-OF-THE-ART-REVIEW

The purpose of the yellow signal indication is to alert drivers of the imminent change of traffic right-of-way at signalized intersections. The legal meaning of the yellow signal indication has evolved through the Uniform Vehicle Code. However, different States provide different laws concerning the legal driver response to the yellow signal indication. Further, several methods and ranges have been proposed to design the signal change intervals (yellow plus all-red). Discussions on these two subjects will follow. Engineering factors directly and indirectly affecting the signal change intervals design will also be presented.

LEGAL MEANING OF YELLOW SIGNAL INDICATION

In 1962, the Uniform Vehicle Code(2) was modified to allow a vehicle to legally enter the intersection on the yellow and to legally clear the intersection when the red signal is displayed. This can be labelled as a "permissive rule" in contrast to a "restrictive rule" that required vehicles to clear the intersection before the end of the yellow signal.

Although all States have not adopted the modified Uniform Vehicle Code meaning for the yellow signal and there is a mixture of "restrictive" and "permissive" rules across the nation(3), Bissell and Warren(4) contend that operationally all States allow the intersection clearance to occur during the beginning of the red. Further, a recent survey by Benioff(3) also indicated that the procedure used for selecting the change interval was statistically independent of the State law regarding the meaning of the yellow indication.

SIGNAL CHANGE INTERVALS DESIGN

Upon observing the yellow onset, drivers approaching an intersection are faced with the choice of either stopping the vehicle prior to entering the intersection or continuing through the intersection. Although several methods and ranges of change intervals have been suggested,(5) more frequently signal change intervals design is based on equation 1, using the following modeling formulation developed by Gazis and others:(6)

A car approaching an intersection is at distance 'x' from the intersection at the yellow onset. If the driver is to stop before entering the intersection, we must have

$$(x - vt) \geq v^2 /2d \tag{2}$$

If the driver is to clear the intersection completely without acceleration before the green on the other street, we must have:

$$(x + w + 1) \leq v(Y + AR) \tag{3}$$

Assuming the equality, equation 2 defines a stopping distance (x_s),

$$x_s = vt + v^2 /2d \tag{4}$$

and equation 3 defines the clearing distance (x_c),

$$x_c = v (Y + AR) - (w + 1) \tag{5}$$

If $x_c > x_s$ and a driver is positioned between x_s and x_c such that $x_s < x < x_c$, then he can either stop or clear the intersection (called the nondilemma zone). However, if $x_c < x_s$ and a driver is positioned between x_c and x_s such that $x_c < x < x_s$ then he will be in a position where he neither can stop safely nor proceed through the intersection completely (called the dilemma zone). Therefore, the minimum change interval satisfying the safe execution of either one of the alternatives, stopping or going through the intersection without acceleration, corresponds to $x_c = x_s$. Then,

$$(Y + AR) v - (w + 1) = vt + v^2/2d$$

dividing both sides by v, then $Y + AR = t + v/2d + (w + 1)/v$.

Figures 1 and 2 show the conceptual layout of the dilemma and nondilemma zones. It is seen, in general, that when the change interval (that is, the slope of the linear line) decreases, the dilemma zone is increased while the nondilemma zone is reduced. Conversely, as the change interval increases, the nondilemma zone is increased and the dilemma zone is decreased. It is noted

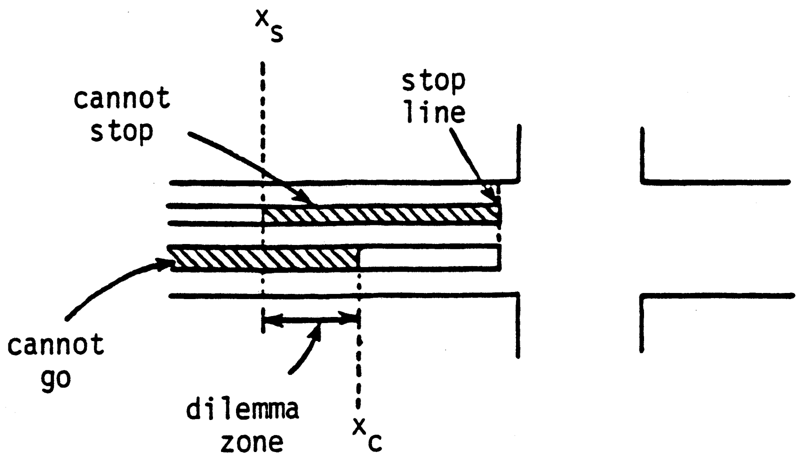


Figure 1. Schematic diagram of dilemma zone.

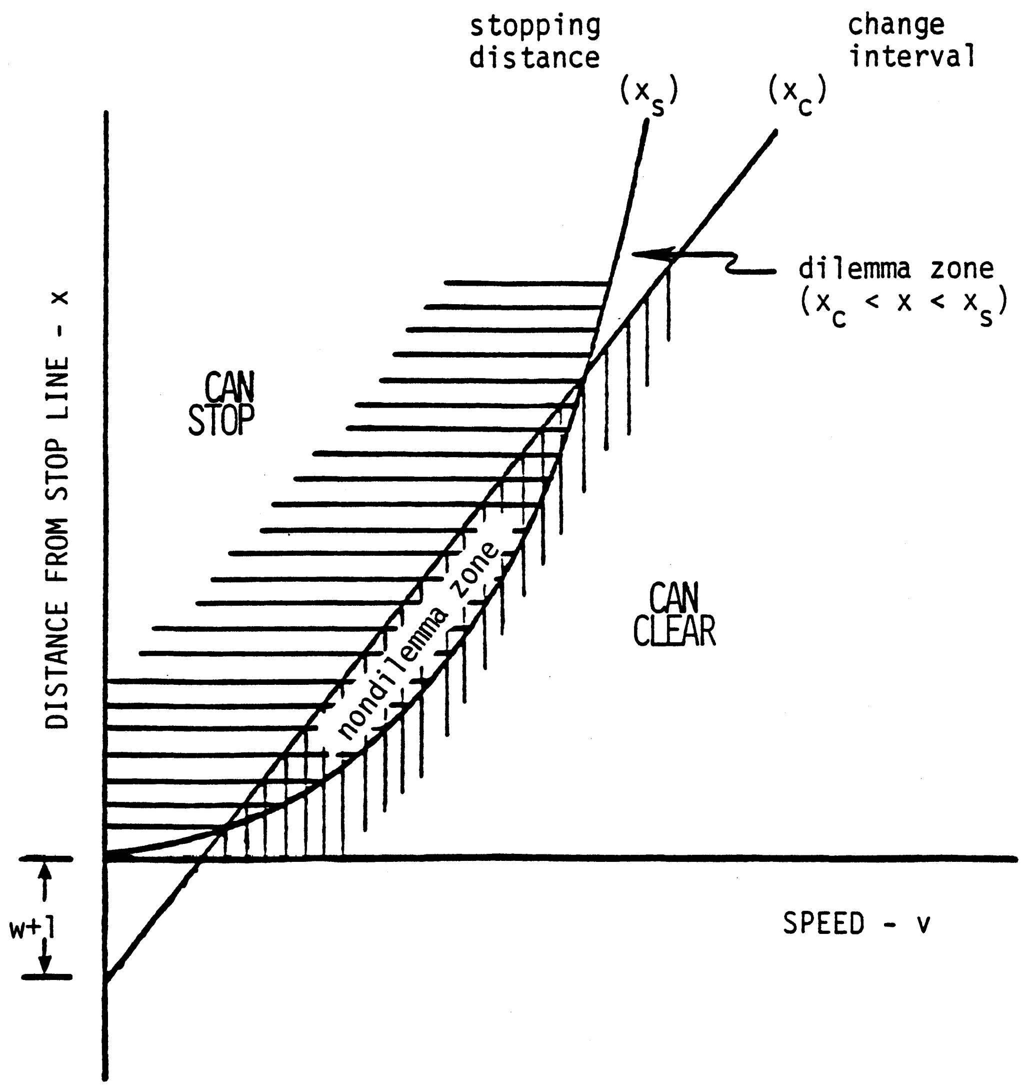


Figure 2. Dilemma and nondilemma zones as a function of vehicle speed and position.

that these zones depend on speed and position of the approaching vehicle.

FACTORS AFFECTING CHANGE INTERVALS DESIGN

As seen in equation 1, driver's perception and brake reaction time, deceleration rate, and approach speed are involved in the signal change interval design. Further, there is an obvious relationship between perception and brake reaction time and deceleration rate because these two variables are associated by sequence within a constrained distance. In addition, physical laws suggest that there is a grade effect on driver's deceleration rate. These subjects will be discussed in more detail.

Relationship Between Perception and Brake Reaction Time and Deceleration Rate

Note that the sum of the first and the second term in equation 1 is equal to the time elapsed to travel the distance from yellow onset to a complete stop, assuming a constant approach speed. In other words,

$$t + \frac{v}{2d} = (vt + v^2/2d)/\ v$$

$$= \text{(distance from the yellow onset to a complete stop)}/v$$

Note that for a fixed distance away from the intersection at yellow onset, there is a dependent relationship between perception and brake reaction time and deceleration rate when a driver decides to stop. As Parsonson[7] pointed out, an increase in perception and brake reaction time is accompanied by an increase in deceleration rate. The empirical results shown in figure 3 obtained from the current study further illustrates the dependent relationship between perception and brake reaction time and deceleration rate. The relationship further implies that an upper limit exists for drivers' perception and brake reaction time because drivers may not want to experience excessive deceleration rates beyond their preferred and tolerable ones.

Perception and Brake Reaction Time (t)

Perception time is the time required to come to the realization that the brakes must be applied. Perception time may vary considerably, depending upon the position and speed of the vehicle, and the driver's degree of attention and distraction due to the type and condition of the roadway and its environ-

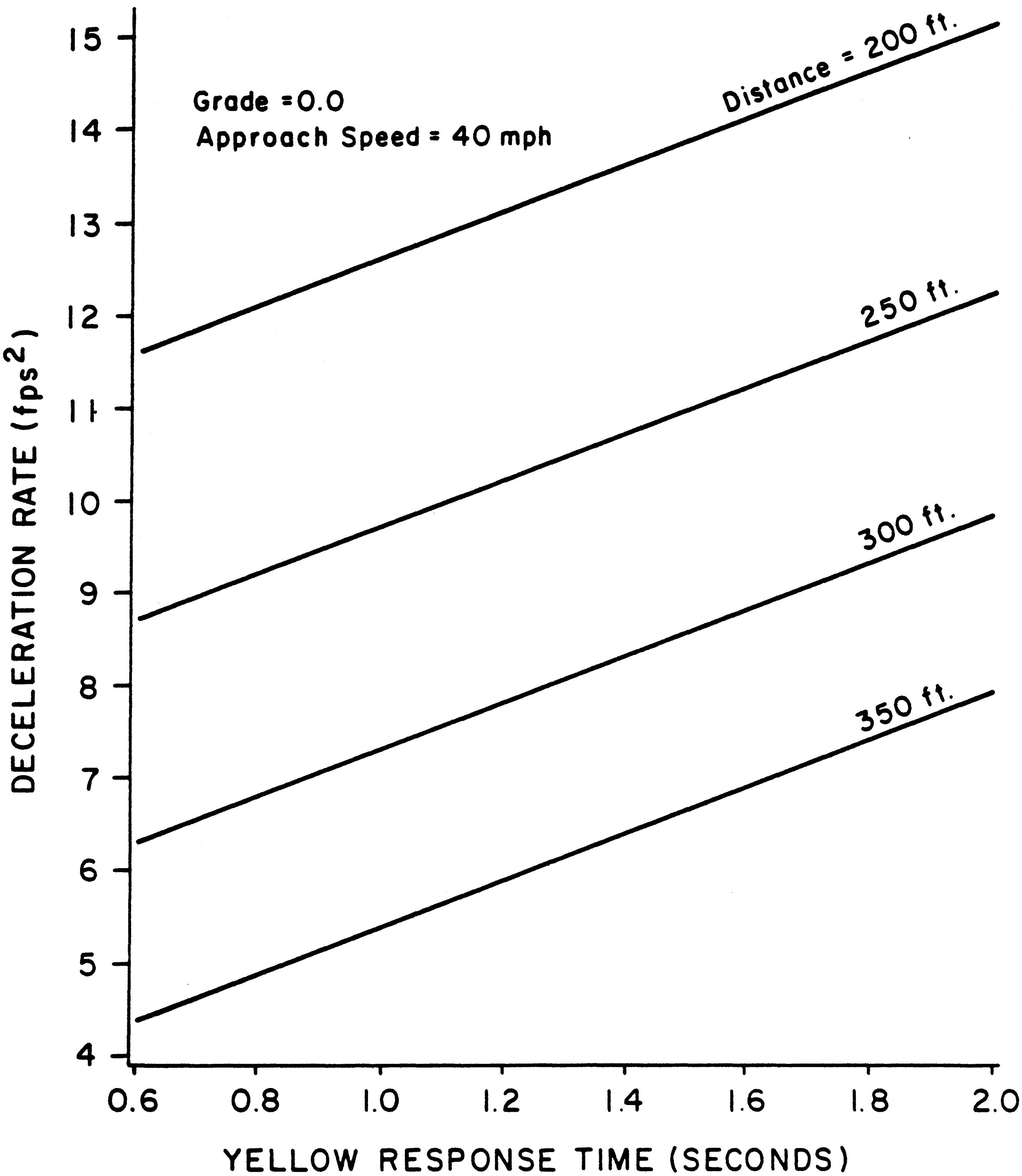


Figure 3. Dependent relationship between yellow response time and deceleration rate.

ment. Brake reaction time is the time required to apply brakes immediately after perception. American Association of State Highway Officials (AASHO)[8] used a total of 2.5 seconds for perception and brake reaction time in determining stopping sight distance.

The Institute of Transportation Engineers (ITE) Handbook[9] assumed a perception brake reaction time to a yellow signal of one second. One second is also suggested by Bissell and Warren,[4] and Parsonson and Santiago.[10]

Numerous studies were performed to develop data on perception and brake reaction time. A summary of most of these studies is found in a recent report by Taoka,[11] which is cited by Hooper and McGee.[12] Two general trends can be noticed in these studies in which one class presented a relatively higher 't' and another class reported a relatively lower 't'. The common drawback in those studies reporting a relatively higher 't' is that they failed to delineate response lag time from observed time. It is further noted that none of these studies measured responses to a yellow traffic signal. In the 1960's, two studies reported on the response time to a yellow traffic signal.[6,13] The combined data from these two studies revealed that the median response time was 1.1 seconds and the 85 percentile was 1.5 seconds. Recently, Wortman and Matthias[14] reported on yellow response times observed in Arizona. The mean time was 1.3 seconds and the 85 percentile was 1.8 seconds. Note that the response time to a yellow signal is not necessarily equivalent to drivers' perception and brake reaction time. In other words, yellow response time may also contain drivers' response lag time which may vary between 0 and a few seconds. Specifically, yellow response time is defined herein as perception and brake reaction time plus lag time.

One aspect missing in the yellow response time studies is that no study examined yellow response time as a function of speed, position, and the time to reach the stop line. An analysis of actual drivers' stopping distance data reported by Williams[15] and by Sheffi and Mahmassani[16] indicate that yellow response time for high speed drivers may be less than that of low speed drivers. It also appears that, as drivers' positions are farther away from the intersection, their yellow response time may be longer than that of drivers closer to the intersection. In addition, at a given speed less variation in response time is presumed to exist for drivers closer to the intersection than for drivers at farther distances away. These developments are based on drivers' physical limitations and the lag time that appears to frequently exist between perception and brake reaction. These hypotheses, graphically shown in figures 4 and 5, were tested and confirmed by Chang and others.[17]

Deceleration Rate

The ITE Handbook in 1976[18] used 15 feet per second per second (ft/s^2) as the deceleration rate. AASHO[8] recommended 18.0 ft/s^2 at a design speed of 60 miles per hour (mi/h) and 20.0 ft/s^2 at a design speed of 30 mi/h for dry pavements. For wet pavements, it recomended 9.7 ft/s^2 at 60 mi/h and 11.6 ft/s^2 at 30 mi/h, respectively. Analyses of stopping distance data from Williams[15], and Sheffi and Mahmassani[16] (assuming a perception brake reaction time of 1 second) indicated that the majority of drivers who stopped experienced deceleration rates of 5 to 11 ft/s^2 for approach speeds from 25 to 55 mi/h. Thus, it appears that the deceleration rates used by the ITE Handbook in 1976 and by AASHO for dry pavements are too high for representative values for driver response to the signal change interval.

The ITE Handbook in 1982[9] assumed 10 ft/s^2 as the deceleration rate. Bissell and Warren,[4] and Parsonson and Santiago[10] suggested 10 ft/s^2 for the deceleration rate. Olson and Rothery[19] reported that virtually all vehicles stop when the required deceleration was 8 ft/s^2 or less, whereas most vehicles do not stop when the required deceleration exceeds 12 ft/s^2.

Grades

Due to the gravitational effect, the stopping distance may be expected to increase on downgrades and decrease on upgrades. As suggested by Parsonson and Santiago,[10] using a modified coefficient friction on grade, equation 1 will be changed to:

$$Y + AR = t + \frac{v}{2d \pm 0.644g} + \frac{w + l}{v} \qquad (6)$$

where

g = percent of grade (use positive for upgrade and negative for downgrade).

It is noted, however, that equation 6 is based on the laws of physics and not on driver behavior. Drivers may compensate for some of the grade effect by braking harder (higher d) on downgrades.

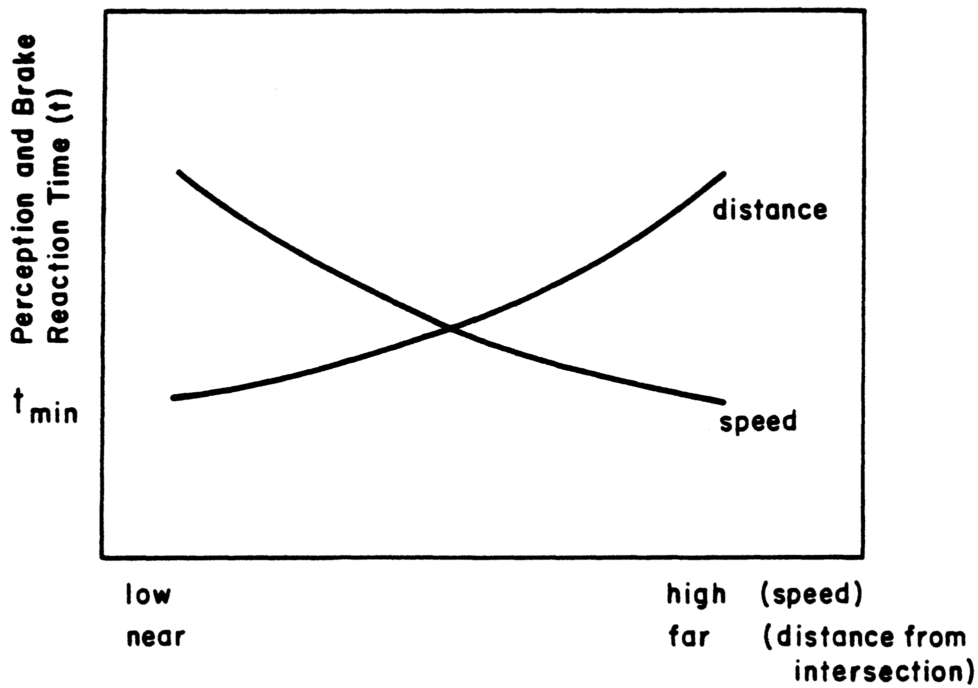


Figure 4. Perception and brake reaction time vs. vehicle speed/position.

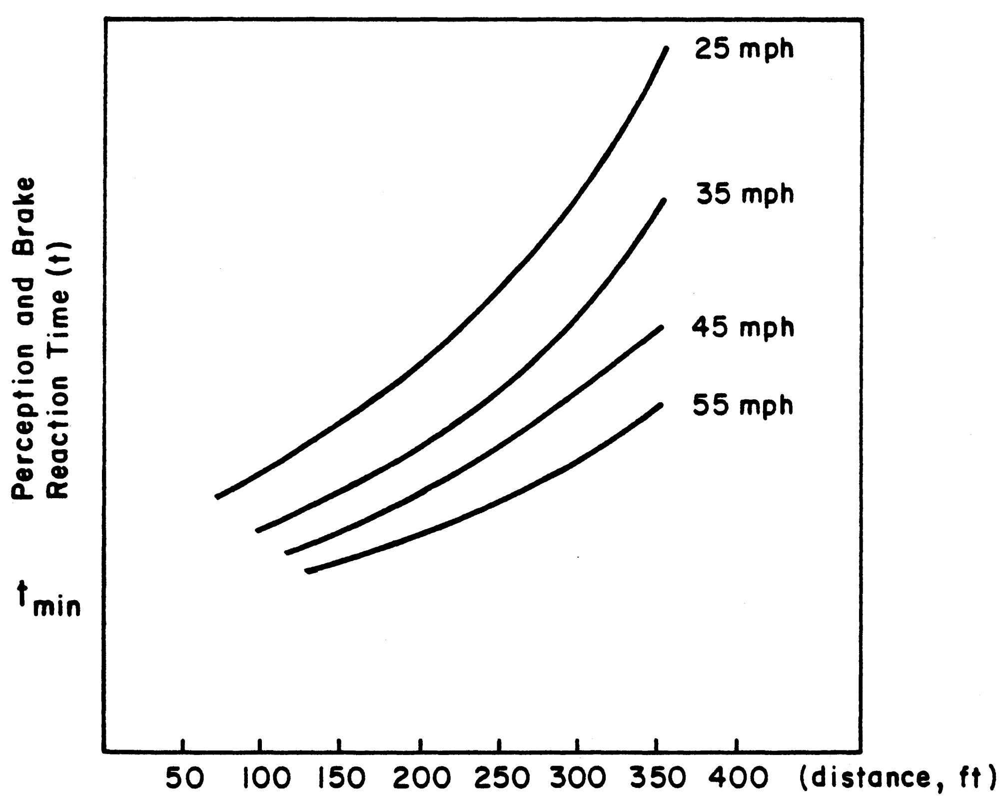


Figure 5. Perception and brake reaction time as a function of vehicle speed and position.

Approach Speed

A study of "spot speed" (speed measured at a specific location) on an approach to an intersection will produce a range (or distribution) of speeds. Typically, the 85th percentile approach speed or the posted speed limit has been used to determine the yellow change and all-red clearance intervals. Parsonson and Santiago[10] raised the question of "Should the reasonable and prudent traffic engineer consider also the heavily loaded truck lumbering through the intersection at the 15th percentile speed?" Suggesting that low speeds and wide intersections are a combination that may require longer signal change intervals (yellow plus all-red), they recommended calculating signal change intervals using both the 85th and 15th percentile speeds and adopting the longer of the two calculations as the signal change intervals (yellow plus all-red). This philosophy is also reflected in a recent traffic control devices handbook published by Federal Highway Administration (FHWA).[20]

Similar concerns on lower approach speed vehicles were shared by Butler.[21] He suggested a way to divide the yellow change interval and the all-red clearance interval within change intervals (yellow plus all-red) using both the 85th and 15th percentile approach speeds. He recommended computing the total change intervals (yellow plus all-red) using both the 85th and 15th percentile speed. If the result for the 15th percentile speed is greater than that for the 85th percentile, he suggested adding the difference to the all-red interval obtained using the 85th percentile speed.

Two terms in equation 1 are related to approach speeds. The second term is $v/2d$ and the third term is $(l+w)/v$. Note that the lower the approach speed the lower the yellow interval and the higher the all-red interval. The consequence of using a lower speed will surprise many drivers due to a shorter yellow interval and will also increase traffic inefficiency due to a longer all-red interval. One of the remedies is to combine the change intervals (yellow and all-red) such that the all-red interval is increased while the yellow interval is maintained at the 85th percentile approach speed. However, the problem of traffic operational inefficiency will still remain.

It should be clear that no standard practice can accommodate all drivers on the street. Further, it should be clear that all drivers are expected to control their vehicles knowing their driving capabilities and vehicles' operating characteristics. Consequently, no vehicle or driver should be immune to the consequences resulting from the negligence of their driving capabilities and vehicles' operating characteristics. All drivers have a duty to stop if they can and an obligation to clear as soon as possible. No lower speed can be justified in view of these obligations. It should be remembered that no traffic control device is a guarantee for safety without driver caution and care.

Start-Up Time and Light Jumping

Start-up time is defined as the interval between the signal's change to green and the movement into the intersection of the first waiting vehicle. Light jumping is defined as those situations when drivers advance into the intersection before the signal changes to green.

Williams(15) has noted the concept of a cross-flow time deduction that was first published in the 1950 edition of the Traffic Engineering Handbook. The concept deducts from equation 6 the time for traffic stopped on the cross street to react to the green and accelerate into the intersection. Williams warns that the cross-flow time deduction be applied with caution and a value of zero should be used if light jumping is possible. Parsonson and Santiago(10) noted that the first waiting vehicle may not respect the stop line. Further, they argued that the concept pertains to stopped traffic starting up on the green, and not to vehicles approaching the intersection at some speed when their signal turns green. Thus, they recommended omitting the cross-flow time deduction to provide a safety factor in signal change interval design. It is noted, however, that the light jumping is an illegal violation of traffic signal display.

Length of Yellow Change Interval

Traffic engineers appear to be reluctant to increase the yellow time for various reasons such as safety and operational concerns. One concern is the conviction that drivers are inclined to treat a long yellow interval as merely a continuation of the green phase. These engineers appear to believe that more drivers will run the red light when the yellow is too long.

Olson and Rothery(22) observed two pairs of intersections to determine if the behavior of drivers does change with significantly different yellow durations. One pair of intersections at a posted speed of 25 mi/h had 3 seconds and 4.75 seconds yellow intervals. Another pair of intersections at a posted speed of 40 mi/h had 2.9 seconds and 4.15 seconds yellow intervals. Subject to the two different intersections in each pair, they concluded that no significant behavioral change in drivers was associated with different yellow durations.

In another similar study, May[23] observed the stopping and going through characteristics during the yellow phase. An urban location using a 3 seconds yellow interval was compared with a 5 seconds yellow at the same location to ascertain the effect of increased yellow duration. He reported that stopping behavior remained essentially unchanged when the yellow duration was increased. He observed that traffic appears to be unaffected by increasing the yellow duration either because drivers are unaware of the increase, or drivers are aware but are not affected by it. Similarly, Stimpson and others[5] reported that no significant change was associated with the extension of the yellow interval up to 6 seconds.

Length of All-Red Clearance Interval

An all-red clearance interval is often provided to let vehicles clear the intersection during the protected time. Current signal change interval design provides all-red clearance time in the form of $(l + w)/v$.

Conventionally, vehicle length (l) is selected as 20 ft.[18,20] However, it is noted that this 20 ft is considerably less than the average truck length. If the intent of all-red clearance interval is to completely clear any vehicle on the passage of vehicles approaching from the opposite direction, using the average truck length rather than 20 ft should be encouraged where frequent truck traffic is operating and clearing during the change intervals.

The width of intersection (w) has conventionally been measured from the stop line to the far side of the intersection. It has been suggested that the intersection width should cover from the stop line to the far side intersection crosswalk. It appears that as long as intersection width is reflected in the all-red clearance interval, the difference between these two approaches are usually insignificant. The discretion of traffic engineers familiar with local conditions appears to be sufficient.

Few studies have been done on the effect of all-red clerance intervals on driver behavior. McGill[24] in Australia found that driver behavior in stopping or entering the intersection was unaffected by changes in yellow time from 3 seconds to 5 seconds. However, when the signal change interval

(Y + AR) were increased from 5.5 seconds to 8 seconds, the number of vehicles entering the intersection during the change interval was observed to increase about one vehicle per cycle. From these observations, she concluded that the number of vehicles entering the intersection after the green is independent of the yellow time but dependent on the total change interval. She further noted that there was no tendency for through vehicles to take advantage of longer yellow durations, while left turning vehicles appeared to take advantage of the extra time.

Benioff and others[3] observed the rate of entering vehicles with and without all-red intervals at 47 sites. All-red intervals ranged between 0.5 second to 3.0 seconds. They concluded that drivers do not have a significantly greater tendency to enter the intersection on a red signal when an all-red interval is added.

Ryan and Davis[25] examined the hypothesis that drivers use the red signal interval more frequently at intersections with all-red intervals. Subject to their determination of the decision zone and limitations in data collection and analysis, they found that more drivers ran the red light at intersections with all-red intervals than at intersections without all-red intervals.

A common drawback in previous studies examining the effect of change interval on drivers' responses is that they failed to consider the effect on a common background of traffic, highway geometric, and vehicular conditions. In other words, the use of average number of vehicles entering during the change interval was compared independent of given conditions. Rather, the comparison should be performed after all other effects are accounted for. For example, it is improper to compare the effect of change interval from the response of two vehicles having different approach speeds and distances. Rather, the question to be answered is whether different change intervals have an effect on drivers' responses if they are in the same conditions of approaching speed, distance, and other geometric and traffic flow characteristics.

EFFECT OF THE CHANGE INTERVALS ON SAFETY

There are basically three questions that should be examined regarding the effect of the change intervals on safety:

- How does the length of yellow affect safety?
- How does the length of the all-red affect safety?

- How does the length of the change intervals (Y + AR) affect safety?

Yellow Interval and Safety

Some researchers found that accidents increased with the increase of the yellow interval.[26] Others found that there is no effect of the yellow interval on drivers behavior.[3,5,22,24] These contradictory findings are primarily due to:

- The limitations for comprehensive accident analysis.
- Inadequate or improper exposure measures.
- Failure to reflect site specific characteristics.
- Use of aggregated accident data.
- Inadequate experimental design and model formulation.

All-Red Interval and Safety

Many studies including Benioff and others,[3] Newby,[27] City of Portland,[28] Michigan State,[29] City of Los Angeles,[30] and Hoppe[31] examined the effect of the all-red phase on accidents. Their general finding is that the number of accidents, particularly right-angle accidents, were significantly reduced by the introduction of an all-red phase (for example, see table 1.)

Change Intervals and Accident Patterns

Accidents associated with traffic signal change intervals are primarily caused by two violations. They are moving violations and expectancy violations. Moving violations refer to the cases when a vehicle is moving at a time when it is not supposed to, or a vehicle is at a location where it should not be. Expectancy violations refer to the cases when a driver's behavior to a cue is unexpected by other drivers.

Two major accident patterns are associated with traffic signal change intervals. They are right-angle and rear-end accidents. The right-angle accident involves vehicles that unsafely decide to go on through the intersection. The rear-end accidents involve vehicles that unexpectedly decide to stop.

To alleviate right-angle accidents, all-red clearance time can be newly provided or the current all-red clearance interval can be increased. It is noted, however, that an increase in the all-red clearance interval will cause

Table 1. Number of accidents before and after the all-red phase in the city of Los Angeles.

Accident Type	Before	After	Significant?
Right Angle	271	167	Yes
Left Turn	293	263	No
Pedestrian	33	24	No
Rear End	260	281	No
Other	126	65	Yes
TOTAL	983	800	

Source: Ref[30]

Note: Significant column indicates that whether a change of accidents before and after the all-red phase is statistically significant. That is, if the change is significant or neglible.

Note: Total number of intersections evaluated are 36 high-accident locations in the city of Los Angeles.

a decrease in traffic operational efficiency due to the increase of lost time associated with signals.

For the drivers' part, they should recognize that the green indication is not a safe and unconditional guarantee of right of way. Rather, it is just a conditional legal guarantee. Regarding the green indication, the Uniform Vehicle Code[2] stated that:

> "Vehicular traffic facing a 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." (Emphasis added by authors.)

For traffic engineers and agencies responsible for signalized intersection safety, they should strive to eliminate any obstructions (such as trees and parked cars) to intersection visibility such that those approaching vehicles receiving the green indication can see if any vehicles or pedestrians are within the intersection.

Traffic engineers and agencies are very limited in what they can do to alleviate rear-end accidents because these accidents are caused by driver expectancy violations and by following too closely under the conditions. Drivers should not assume the front car will go through. Rather, they should assume the car ahead will stop and should respond accordingly.

III. DATA COLLECTION

This chapter contains the data collection plan and describes field procedures employed for collecting data on drivers' responses to the signal change interval. Field data collection is performed to develop a comprehensive understanding of driver responses to the change interval, to examine change interval design as a function of driver behavior, and to quantify the values of the variables associated with drivers' perception and brake reaction times, and deceleration rates. The detailed approach taken for site selection and field data collection will be presented. The data to be collected along with the equipment used is also discussed.

SITE SELECTION: GENERAL CRITERIA

Seven study sites were selected in two States. Three sites in Virginia and four sites in Texas were selected for field study. Priority was given to intersections having no side streets or driveways within the observation distance to eliminate those occasional traffic interruptions that may mask the routine driver behavior. Further, those intersections having exceptionally skewed geometrics or whose sight distances were obstructed such that they may affect driver behavior were excluded.

Site selection guidelines provided by the Federal Highway Administration (FHWA) called for a variety of intersection geometric and traffic control conditions, as follows:

- Different change interval lengths from 3 to 8 seconds.
- Different all-red intervals.
- Different signal controllers (pretimed and actuated control).
- Different grades (level and downgrades).
- Different intersection geometrics (T and 4-legged intersection).
- Different left-turn controls.
- Different approach speeds between 25 and 55 mi/h.
- Different intersection locations (urban, suburban, and rural).

EXPERIMENTAL DESIGN FOR SITE SELECTION

Since it was not possible to provide all combinations of factors in seven sites, the sensitivity of 't' and 'd' on the change intervals was identified to determine the order of importance among the factors to be considered. The sensitivity testing was performed by relating the percentage of change interval to the specific percentage change of 'd' and 't' using case studies for given intersection characteristics. The test revealed that the change interval is much more sensitive to percentage changes in 'd' than in 't'. This finding in turn suggests that those factors affecting deceleration rates should be emphasized in the experimental design for site selection.

Figure 6 shows the potential factors affecting 'd' and 't' in terms of order of sensitivity based on the discussions presented in the state-of-the-art. Speed was found to be the most influential factor in changes of both 'd' and 't'. Thus, a comparison of approach speed, which is reflected in the site area (urban, suburban, and rural) and street type (main or cross street) was given primary consideration in the development of the experimental design for site selection.

SELECTED SITE CHARACTERISTICS

Site selection was constrained by the fact that seven sites cannot encompass the complete interaction of factors and their levels, and that there may not be intersections existing in the field satisfying desirable combinations of factors and their levels.

A survey form soliciting candidate intersections was mailed to several counties and cities in Virginia and Texas. The responses were arranged by intersection locations and speed categories. The research team and FHWA visited sites and decided to study the sites shown in table 2. Intersections observed for this study included a variety of combinations in intersection geometrics covering different intersection widths and grades. Further, traffic control conditions involved different speed limits, controller types, and change intervals.

DATA COLLECTION SCHEDULE

To examine driver responses during a variety of operating conditions, the data collection schedule specifically covered the following day-to-day operating conditions:

- Daytime and nighttime.
- Peak and off-peak periods.
- Dry and wet pavements.

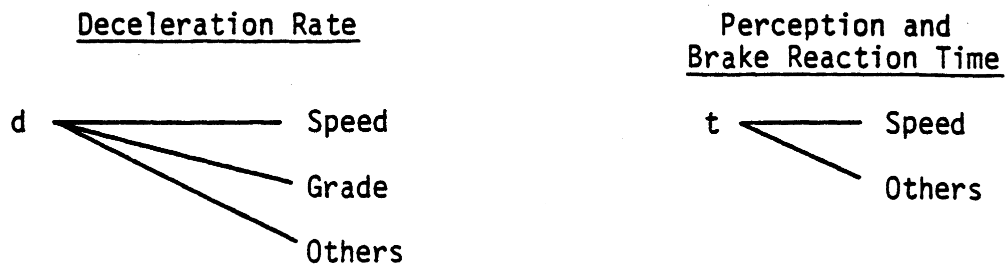


Figure 6. Order of sensitivity of variables affecting "d" and "t".

Table 2. Geometric and traffic control characteristics of study sites.

Location Area	Intersection and Approaches	Approach Grade (%)	Type * of Signal	Speed Limit (mi/h)	Yellow Time (s)	All Red Time (s)	Inter-sectior Width (ft.)
Urban	1. Commerce, TX	-2.5	A	30	4.5	1.5	125
	2. Industrial, TX	0.5	A	35	4.5	1.5	125
	3. US 29, VA	-4.5	P	35	3.0	1.5	220
	4. US 50, VA	-5.0	P	35	3.0	1.5	200
	5. Texas Ave., TX	0.8	P	45	4.0	1.5	100
	6. Univ. Dr., TX	-0.8	P	40	4.5	1.5	100
Suburban	7. S. Lamar, TX	-3.5	P	45	4.0	0.0	60
	8. Old Keene 1, VA	-3.5	A	45	5.0	0.0	60
	9. Old Keene 2, VA	-6.0	A	45	5.0	0.0	60
Rural	10. US 1 NB, VA	1.0	A	50	5.0	1.0	80
	11. US 1 SB, VA	-6.5	A	50	5.0	1.0	80
	12. SH 1, TX	0.0	A	55	4.0	1.0	80
	13. SH 2, TX	0.0	A	50	3.0	1.0	80

*: P = Pretimed traffic controller.
A = Actuated traffic controller.

The required sample size was determined considering the confidence level and an acceptable error of a variable. A standard deviation of deceleration rate, 4.43 ft/s^2, estimated from Williams(15), and a reported standard deviation of 0.58 seconds perception-reaction time(32) were used to determine sample size. The required sample size based on the perception and brake reaction time at a confidence level of 90 percent and 0.1 second of allowable error was higher and yielded about 100 stopping vehicles. Since the study involved not only the stopping vehicles but also the clearing vehicles, a similar number of clearing vehicles is required.

The number of hours required to collect 100 stopping vehicles and 100 going vehicles was estimated based on the peak and off-peak traffic volume, cycle length, change interval, and the assumption of Poisson arrivals. The following data collection period was used throughout the study:

- Two hours of peak and 2 hours of off-peak.
- Four hours of nighttime extending 2 days.
- Two hours of wet weather daytime.

PRELIMINARY DATA COLLECTION PLANNING

Before actual field observation, the research team conducted field visits to familiarize itself with the study sites and to develop a detailed field data collection plan. Preliminary data collection planning involves equipment aquisition and checking, field crew training, and establishment of observation distances.

Timelapse Camera Modification

Through camera experimentation before field study, it was found that the timelapse cameras did not display time in the remotely controlled continuous mode as required for inconspicuous data collection. From the information supplied by the manufacturer, the research team was able to electronically modify the timelapse cameras to display time in the remotely controlled continuous mode.

Equipment Check for Field Study

Before going out to a study site, a field equipment list was prepared and necessary equipment acquired. Batteries for timelapse cameras were fully charged the day before each study. The timelapse cameras were synchronized to display identical time.

Field Crew Training

The research team conducted a training exercise for the field crew before actual field data collection to insure their familiarity with equipment and to provide accurate knowledge on the use of equipment and data collection procedures involved. This training was conducted in two ways: laboratory training and field training. The laboratory training involved such topics as:

- How to setup the equipment.
- How to operate the equipment.
- How to date and slate each film.
- How to collect data.
- How to maintain equipment.

An intersection was selected for training of the data collection technicians. The field training involved the same procedures as the actual field study. During and after this field training, questions and answers were exchanged between the technicians and a supervisor.

Establishment of Observation Distance

The observation distance limited by upstream and downstream boundaries describes the roadway zone wherein vehicle operations are filmed.

The upstream boundary of the study area on an approach to the intersection is estimated from drivers' stopping characteristics. It is derived from observations(16) which reveal that more than 99 percent of drivers would stop at this distance and beyond. The upstream boundary distance specifies the roadway section within which drivers' responses to the signal change interval are expected. The following observation distances as related to speed limit (or approach speed, if higher) were established.

Approach Speed (mi/h)	25	30	35	40	45	50	55
Upstream Boundary (ft)	180	220	260	300	350	400	450

This upstream boundary was measured from the stop line longitudinally along the street to the outer observation boundary.

The downstream boundary is one vehicle length beyond the far side of the intersection for the approach studied. Figure 7 illustrates the study limits of an approach.

DATA COLLECTION PROCEDURES EMPLOYED

The field data collection procedures using two timelapse cameras on an approach will be presented below. The procedures used for distance reference, camera set up and mounting, and operation will be described in detail.

Establishment of Film/Roadway Reference Points

To convert film distance to roadway distance in later data reduction, four roadway reference points (RRPs), among which three were not on a straight line, were established. Small safety cones were used as reference objects. They did not appear to attract driver attention particularly due to their small size. Median and curb side locations were adequate for placing the reference objects, and an example is illustrated in figure 8.

Camera Set-Up and Mounting

Figure 9 presents a typical intersection situation including the general location of a timelapse camera on each approach studied. No more than two approaches were studied at one time. The camera located further upstream from the signal on an approach is denoted as camera 1, or the upstream camera. The camera located closer to the intersection is designated as camera 2, or the downstream camera. The basic reason for using two cameras is to reduce the potential reading error in distance near the intersection when employing only one camera on an approach.

The cameras generally were mounted on utility poles about 13 feet above the right pavement edge. The visibility requirements of traffic signals and vehicle tail lights along with the practical limitation of climbing heights using a ladder led to the target mounting height of 13 feet.

All timelapse cameras were connected to remote control units which could be operated from inconspicuous locations, such as in a parked car, in order not to disturb driver behavior.

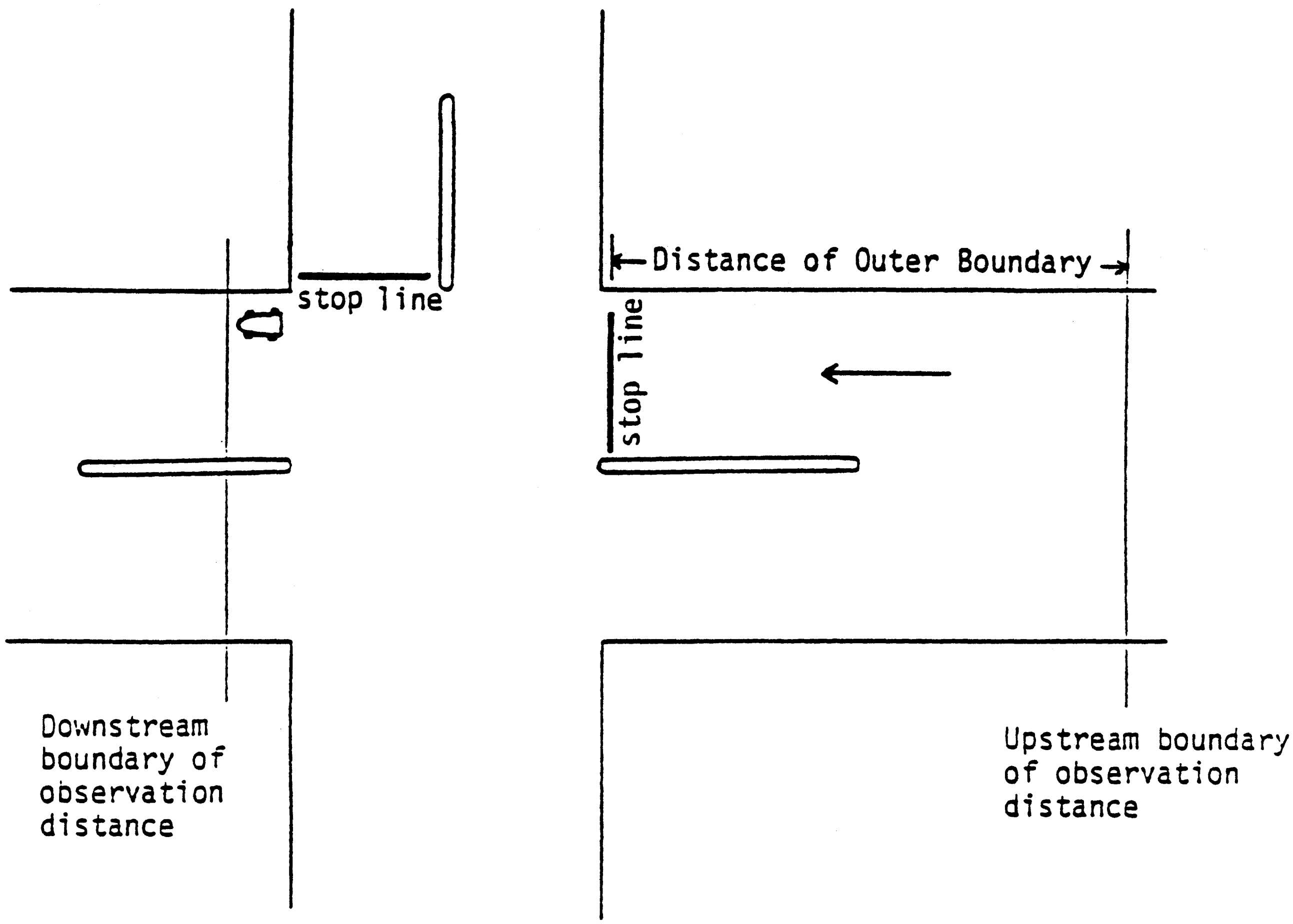


Figure 7. Observation distance schematics.

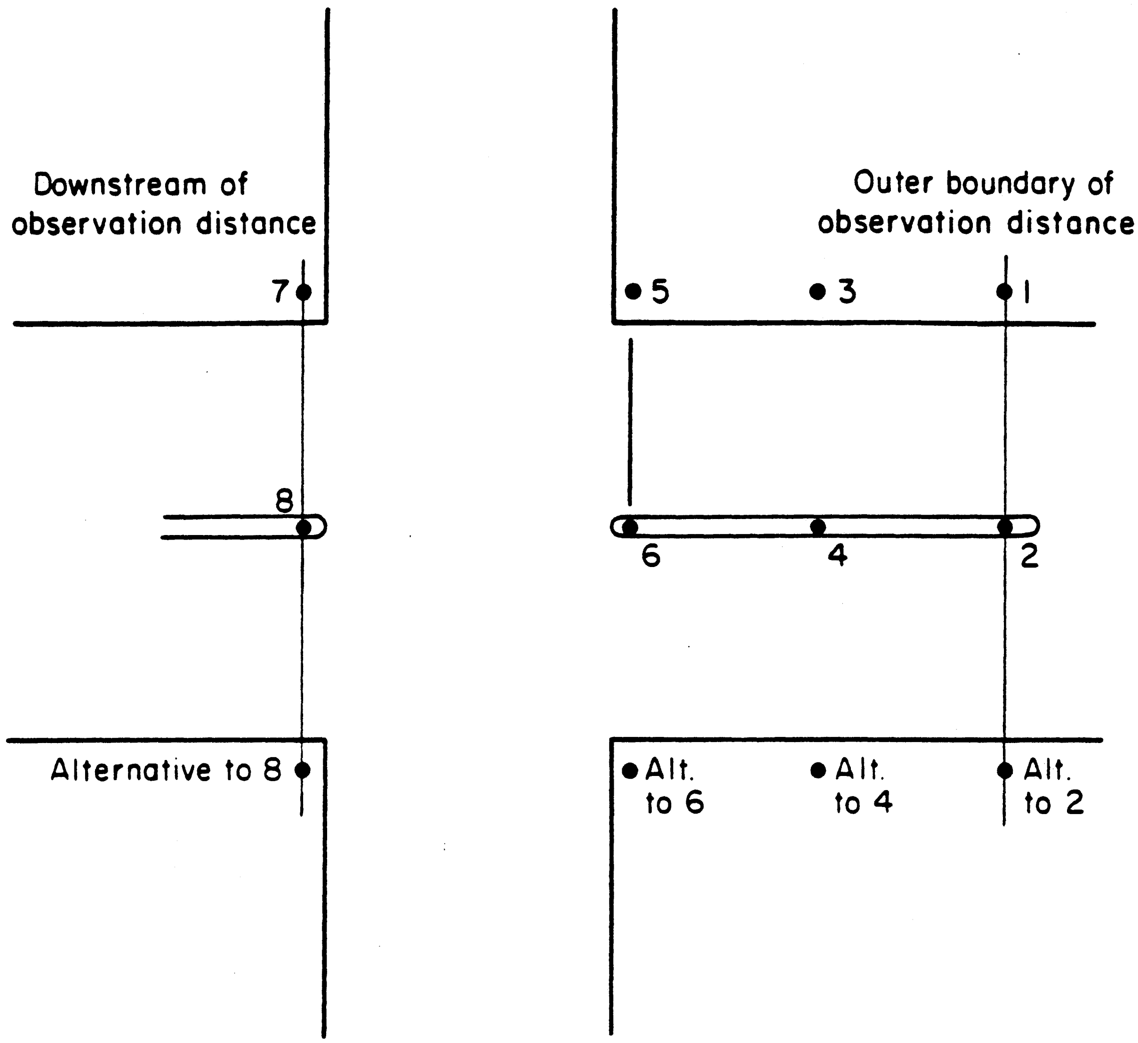


Figure 8. Schematics for roadway reference points placement on an approach.

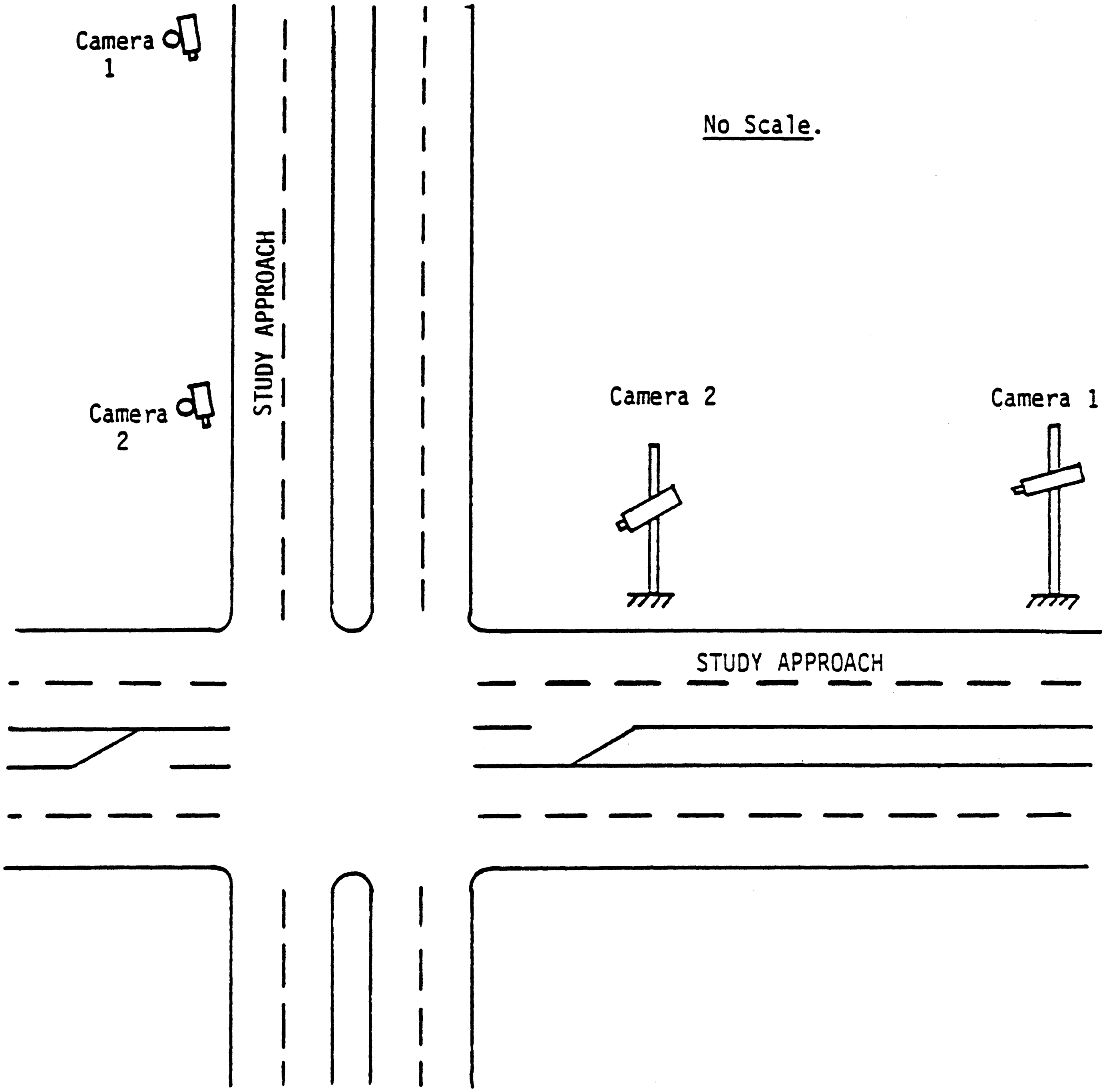


Figure 9. Typical intersection situation with simultaneous observations on two approaches.

Camera Operations

The purpose of the upstream camera is to collect approach speed of a vehicle, drivers response to the yellow onset, and vehicle stopping characteristics. The purpose of the downstream camera is to collect vehicle operations closer to the intersection including light jumping at the start of green, queue start-up delay, and driver responses during the signal change interval.

A person was assigned to operate each timelapse camera. Both cameras 1 and 2 were operated approximately 3 seconds before yellow and continued until the first car in each lane had made a complete stop. In addition, camera 2 was operated three seconds before green and continued until the first car in the queue had started, to collect light jumping and starting delay characteristics. A summary of camera 1 and 2 operation is shown in figure 10.

The film was shot at a setting of nine frames per second. During the daytime, Type A film was used. During the nighttime, Type G film was used.

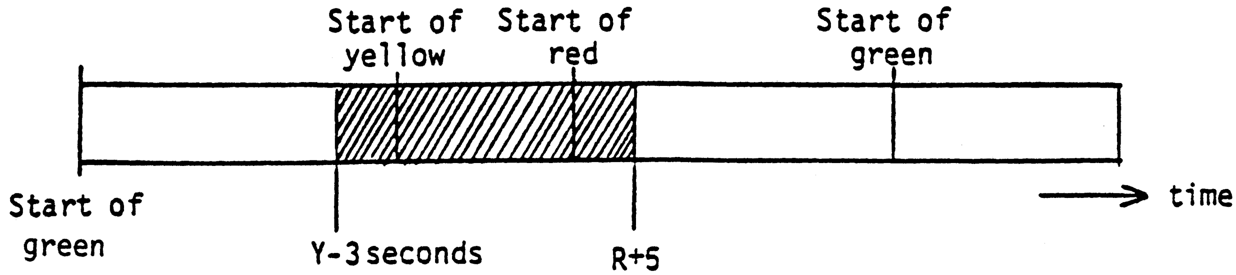
Data Collected

The list of data collected to evaluate the traffic signal change interval design and driver response is as follows:

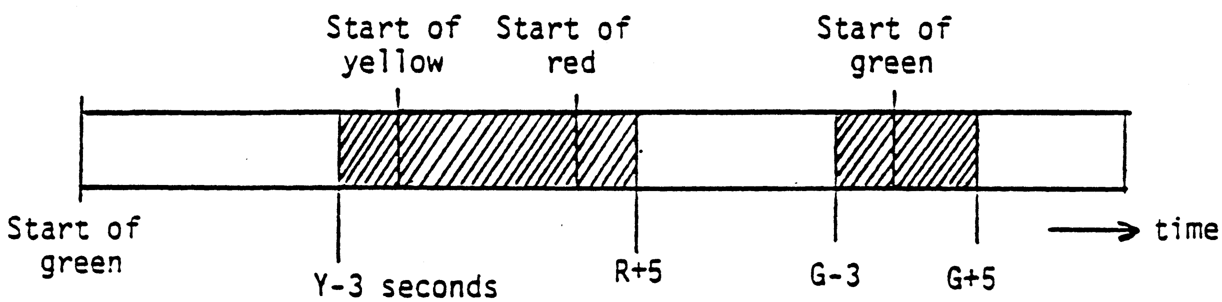
- The distance from intersection and speed of approach vehicle at the onset of yellow and the driver's decision to continue or to stop.
- The time and distance at which brakes were applied.
- The time and distance when a vehicle stopped.
- The time when a vehicle entered and cleared an intersection.
- The time when a vehicle in queue started.
- The type, directional movement, and spacing of a vehicle.

In addition, other related intersection geometric and traffic control characteristics were collected in the field, and they are as follows:

- Approach grades.
- Intersection width.
- Intersection geometric sketches.
- Traffic controller type.
- Traffic signal phasing plans and timing settings.
- Intersection approach volumes.
- Pavement and weather conditions at the time of the study.



A) Camera 1 Operation Cycle



B) Camera 2 Operation Cycle

Note: Shaded portion is camera filming time.

Figure 10. The first and second camera operation times.

IV. DATA REDUCTION

Data reduction process involves data reading from film, recording them on the data form, and inputing them to the computer for later analysis.

ESTABLISHMENT OF FILM/ROADWAY RELATIONSHIP

Exposed film was loaded into a Timelapse Model 3420 Data Analyzer Projector. Then, x and y coordinates of those field reference objects could be read with a convenient scale. The corresponding roadway coordinates of these four film reference points were already measured in the field and were available.

Using the basic characteristics of photography,[33] the relationship between film and roadway plane can be developed. A computer program modified from that of Bleyl[34] was developed to convert roadway coordinates to film coordinates. This computer program is presented in Appendix A.

DRAWING OF ROADWAY DISTANCE CONTOUR ON FILM PLANE

Given four corresponding coordinates of roadway reference points (RRPs) for each film and roadway, any roadway distance or points can be converted to that of the film plane. A 10 ft distance contour map was drawn from a computer output. Then, the film RRPs were superimposed onto the graph RRPs. After this superimposition was finished, data reduction to read time, distance, and other characteristics of a particular vehicle on the film could be performed.

DEFINITIONS OF BOUNDARY AND POPULATION VEHICLES

Before proceeding with the film analysis, it should be understood which vehicles are of interest. The following defines the boundary and population vehicles.

The vehicles within the observation distance from the stop line to the upstream outer boundary at the onset of yellow are the "boundary vehicles." All vehicles beyond the stop line and preceding the upstream outer boundary at the onset of yellow were ignored. Further, vehicles turning right were excluded because their response is anticipatory and not a random response.

Among boundary vehicles, those vehicles that go through the intersection and those first vehicles stopping in each lane are the "population vehicles." Vehicles stopping second in each lane are excluded because their response is constrained by the first vehicle stopped in each lane (that is, their probability of stopping = 1). Further, within the first vehicles stopped in each lane, vehicles braking during green through yellow, or braking coincident with the yellow onset, were also excluded because their response is not a response to the yellow signal but an anticipatory or coincidental response.

The schematics for boundary and population vehicles are shown in figure 11, assuming vehicles are positioned as in figure 11 at the onset of yellow.

For light jumping and starting delay at an approach, the population vehicles are the first vehicle in each lane waiting (or approaching) on red and departing before (that is, light jumping) and after (that is, starting delay) the onset of green.

DATA REDUCTION FROM FILM TO FORM

Data Form

Figure 12 shows the data form used to record and evaluate driver responses to the change interval. The data form is used to record time, distance, and action of vehicles during yellow and red. Other associated factors such as approach speed, yellow response time, deceleration and acceleration rates, and final speed could be derived from recorded data. Data form shown in figure 12 is self-explanatory except that subscript 1 refers to camera 1 while subscript 2 links to the camera 2.

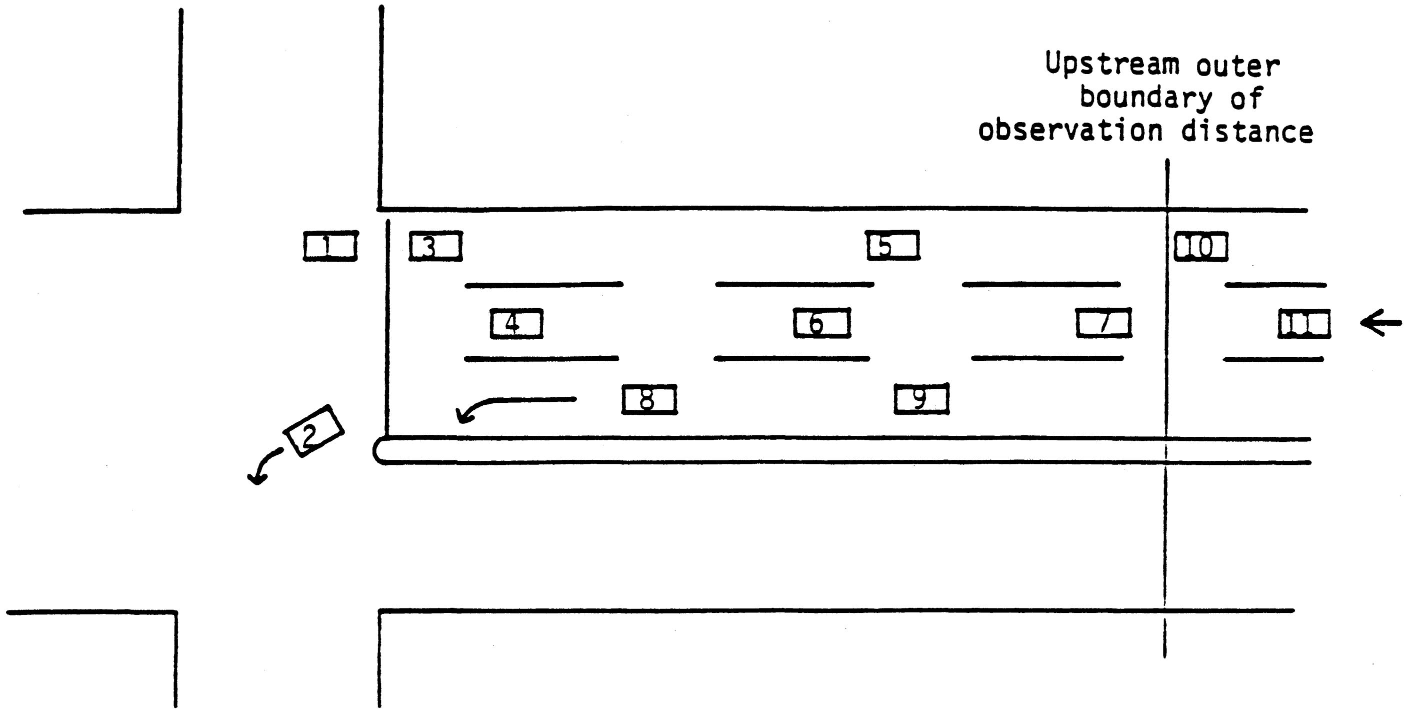
Data Coding

One of the popular statistical analysis packages, SAS[35] was used to input data for later analysis. There are three forms of variables needed for later analysis: (1) input variables, (2) assigned variables, and (3) derived variables. Input variables are those variables recorded on the data form characterizing each sample. Assigned variables are those variables describing intersection conditions and roadway environments. Derived variables are those variables derived from mathematical operation on input variables and will be explained subsequently.

DERIVED VARIABLES USED FOR DATA ANALYSIS

Approach Speed

Approach speed is derived from the differences between the starting frame and distance of a vehicle at green, and the frame and distance at yellow onset.



Boundary vehicles: Through vehicles 4, 5, 6, and 7.
Left turn vehicles 8 and 9.

- Assume vehicle 3 is right turning and is excluded.

Population vehicles: Through vehicles 4, 5, and 6.
Left turn vehicles 8 and 9.

- Assume vehicles 4 and 8 cleared and 5, 6, 7, and 9 stopped such that vehicle 7 stopped behind vehicle 6.

Figure 11. Example of boundary and population vehicles for responses to change interval at yellow onset.

TEXAS TRANSPORTATION INSTITUTE, TEXAS A&M UNIVERSITY STUDY,
"ENGINEERING FACTORS AFFECTING TRAFFIC SIGNAL YELLOW TIME".

Intersection Location: ____________________   Approach: ____________________

Intersection Width: ____________________   Posted Speed Limit: ____________________

Date of Film Taken: ____________________   Time of Film Taken: Peak, Off-peak, Night

Pavement Condition: Dry Wet   Recorder: ____________________

Legend
N Lane Number
VT Vehicle Type
Fr Frame
D Distance
Ent Entering
Cl Clearing

Time	Camera 1		Camera 2		VT	N	At the Start of Yellow			At the Start of Brakelight			Vehicle Action During Yellow	Frame of Go Vehicle				Frame of Stopping Vehicle			Vehicle Action During Red	At the Start of Red			Traffic Movement
														Ent		Cl									
	Fr_1	D_1	Fr_2	D_2			Fr_1	Fr_2	D	Fr_1	Fr_2	D		Fr_1	Fr_2	Fr_1	Fr_2	Fr_1	Fr_2	D		Fr_1	Fr_2	D	

Figure 12. Data coding form.

Final Speed

Final speed is defined as the average speed of going vehicles from the yellow onset to the complete clearance of the intersection.

Yellow Response Time

Yellow response time is measured in this study as the time elapsed from the onset of yellow until the brake light of a stopping vehicle is observed to come on.

Deceleration Rates

Deceleration rates can be derived from either one of the following equations:

$$\text{(a)}\quad d_t = \frac{2L}{T^2} \qquad \text{or} \qquad \text{(b)}\quad d_v = \frac{v^2}{2L} \tag{7}$$

where

L = braking distance, feet.

T = time elapsed from brake actuation to a complete stop, seconds.

v = speed at the time of brake actuation, ft/s.

d_t = deceleration rate derived from using L and T, ft/s^2.

d_v = deceleration rate derived from using L and v, ft/s^2.

DATA PROCESSING AND ERROR CHECKING

Since the accurate reading and recording of distance and time involved is critical to the study, the processed data should be checked for potential error involved.

Error Checking in Speed

The approach speed and the average speed of stopping vehicles from the yellow onset to brake actuation were compared. If the absolute difference between these two speeds was more than 5 ft/s, the film reading was checked. The criterion of 5 ft/s was based on both speed reduction from coasting and the effect of immediate acceleration near the yellow onset.

In addition, approach speed and average speed of going vehicles from yellow onset to the complete clearance of the intersection was checked. If the acceleration rate experienced by a going vehicle was more than 10 ft/s^2, the film reading was checked for possible error.

Error Checking in Deceleration Rates

Note that between the two deceleration rates in equation 7, vT = 2L must hold. If any measurement error is involved in one of v, T, and L, the two deceleration rates will not be identical. If (vT - 2L) had an error of more than $\pm$ 5 ft, then the film reading was checked. Further, if any one of the deceleration rates derived was beyond 0.5g (that is, 16 ft/s^2), the film reading was checked for possible error.

Error Checking in Other Variables

All variables were checked with respect to their algebraic sign. If any variable displayed a negative sign during the data processing, the data input, data recording form, and film was checked.

DATA CHARACTERISTICS

It is noted that the time variables were recorded in 0.1 second intervals. Further, the distance variables were recorded in 5 feet intervals for most of the data. Due to the accuracy limitations in time and distance, the derived variables such as deceleration rate could not, in some data, satisfy a physical relationship. As mentioned previously, there is a relationship of vT = 2L between two deceleration rates that can be derived. The data used for analysis has an error of $\pm$ 5 ft in vT - 2L. The deceleration rate used for analysis is an average of d_t and d_v shown in equation 7.

It is further noted that it was not entirely possible to read all vehicles that went through during the change interval. This is particularly applicable to those vehicles very close to the intersection at the yellow onset. The difficulty was due to the accurate reading of distance for those vehicles very close to intersections. Therefore, few of the going vehicles which were very close to the intersections at the yellow onset were inevitably omitted.

V. ANALYSIS METHODS

The reduced data is analyzed using the satistical analysis techniques involving regression models and probability modeling. The detailed analysis techniques and variables employed to understand the phenomena involved in drivers' responses to the yellow onset is introduced.

ANALYSIS TECHNIQUES EMPLOYED

Basic Summary and Descriptive Statistics

The following descriptive statistics are presented in quantitative or graphical form as appropriate.

- Measures of central tendency such as mean and median.
- Measures of dispersion such as variance, range, and percentiles.
- Frequency distribution.

Regression Models

Regression models were used to evaluate factors affecting driver's selected yellow response time and deceleration rate. Regression models are useful for the following reasons:

- To detect the significance of independent variables including their interactions affecting signal change interval design such as yellow response time and deceleration rates.
- To obtain the expected value of variables affecting signal change interval design.
- To examine the functional effects of driver and vehicle attributes affecting signal change interval design.

Probability Modeling

Probability modeling using probit(36) and logit(37,38) model was performed to evaluate drivers' decisions to stop or go through the intersection as a function of their attributes. Probability modeling is useful for the following reasons:

- To detect the significance of independent variables including their interactions affecting drivers' decisions to stop or go through.

- To evaluate the signal change interval design in terms of the observed driver's decision to stop or go through.

Variables Evaluated

The primary variables representing vehicle attributes at yellow onset are the approach speed and distance. The time to reach the stop line is an interaction variable of speed and distance. Throughout the models evaluated, these three variables (speed, distance, and time) were always included as primary variables. The additional variables characterizing geometric and traffic control characteristics of intersections, such as approach grade and controller type, were also evaluated. These additional variables will be further explained subsequently as appropriate.

VI. STUDY FINDINGS

The observed relative frequency of stopping and going characteristics introduce the findings. Basic descriptive statistics on yellow response time and deceleration rates observed from field studies are presented. The potential perception and brake reaction time is deduced from the yellow response time with consideration of speed influence. Subsequently, other relevant information, such as the time taken for the clearing vehicle to reach the stop line, follows. Further, other geometric, traffic control, and environment factors such as grade, controller type, and weather that may influence drivers' selected characteristics of yellow response time, deceleration rate, and their decision to stop or go through were analyzed.

DRIVER RESPONSE CHARACTERISTICS TO CHANGE INTERVAL

Table 3 shows the observed relative frequency of driver response characteristics with respect to signal change interval. It shows that the overall relative ratio of stopping to going is 1 to 2. Fifty-seven percent of the total vehicles entered intersections during yellow among which two-thirds cleared during yellow but the other one-third cleared during red. The high number and percentage of yellow entering and red clearing vehicle at the intersection of U.S. 29 and U.S. 50 (Approaches 3 and 4) is attributed mainly to its extremely wide intersection width.

Table 3 also shows that 7 percent of the total number of vehicles entered during red. A substantial portion of those vehicles were observed at the sites of U.S. 29 and U.S. 50 in Virginia and State highways in Texas. These two sites were operated at long cycle lengths and were observed to experience frequent long queues. The traffic operation appeared to contribute the impetus for drivers taking high risk by entering during red.

From observed site geometric and traffic operational conditions, two suggestions can be made to reduce the frequency of vehicles entering and clearing during red. The first is that a sufficient all-red interval is to be used for those wide intersections. The second is that traffic operations should be improved at those sites experiencing a high proportion of vehicles entering during red.

TABLE 3. Observed relative frequency of driver response to signal change interval.

Vehicle Action	Intersection Approach Numbers 1	2	3	4	5	6	7	8	9	10	11	12	13	Total	%
S	20	27	64	126	37	43	59	38	14	81	37	16	17	579	36
YEC	10	6	0	0	24	34	46	73	40	253	131	2	1	620	38
YERC	2	22	75	105	10	3	20	9	4	25	4	6	14	299	19
RE	1	1	18	64	8	0	1	2	0	2	2	9	8	116	7
Total	33	56	157	295	79	80	126	122	58	361	174	33	40	1614	100

Legend

S: Stopping
YEC: Enter and clear during yellow
YERC: Enter during yellow and clear during red
RE: Enter during red

YELLOW RESPONSE TIME

Table 4 summarizes the values observed for yellow response time at each intersection approach and the total vehicles observed. It shows that the mean yellow response time of all drivers in the subject population was 1.3 seconds and the median was 1.1 seconds. Eighty-five percent of stopping vehicles applied their brakes within 1.9 seconds while 95 percent did it within 2.5 seconds. The cumulative distribution of yellow response time for 579 stopping vehicles is shown in figure 13.

It is noted that yellow response time usually also includes some lag time because it is not necessarily from a situation requiring immediate reaction following a decision. To derive a perception and brake reaction time from yellow response time, speed influence is introduced. The hypothesis is that drivers' yellow response time at high speed (For example, 50 to 55 mi/h) may be closely equivalent to their perception and brake reaction time because their high speeds require immediate reactions to avoid excessive deceleration or even collison with other vehicles.

Figure 14 presents the yellow response time classified by speeds. The observed speeds were classified into seven categories from 25 to 55 mi/h. The speed shown is the middle point of 10-mi/h intervals. It is shown that the median yellow response time is stabilized at 0.9 seconds at speeds over 45 mi/h. The current value of one second assumed by the ITE handbook[9] corresponds to 70 percent of the total vehicles observed in the 55 mi/h speed category in this study.

To further validate the conceptual appropriateness of the derivation of perception and brake reaction time from yellow response time at high speed categories, another similar case requiring immediate reaction is considered in terms of distance for vehicles traveling over 40 mi/h. It is reminded that as vehicles are closer to the intersection, drivers tend to react immediately; whereas, when they are further away, their yellow response time will involve a substantial amount of response lag time. Figure 15 presents the yellow response time by distance for vehicles traveling over 40 mi/h. The distances shown are the middle points of 100-ft intervals. It is shown that when vehicles are relatively closer to the intersection, their 85 percent yellow response time was 1.1 seconds at 200 ft and 1.3 seconds at 250 ft. It is also noted that the median yellow response time for vehicles approaching over 40 mi/h is 0.9 seconds.

TABLE 4. Yellow response time characteristics of stopping vehicles.

	Intersection Approach Numbers													
Statistics	1	2	3	4	5	6	7	8	9	10	11	12	13	Total
Mean	1.2	1.5	1.4	1.4	1.3	1.3	1.4	1.1	0.7	1.0	1.2	1.0	1.0	1.3
Median	1.0	1.4	1.2	1.3	1.1	1.0	1.3	1.0	0.7	0.9	1.1	1.0	1.0	1.1
85%	1.7	2.1	1.9	2.2	1.9	1.8	2.0	1.6	1.0	1.5	1.6	1.3	1.1	1.9
95%	2.0	2.4	3.2	2.9	3.4	2.7	2.8	1.9	1.2	1.9	1.9	1.5	1.4	2.5

Note: Entry is in seconds.

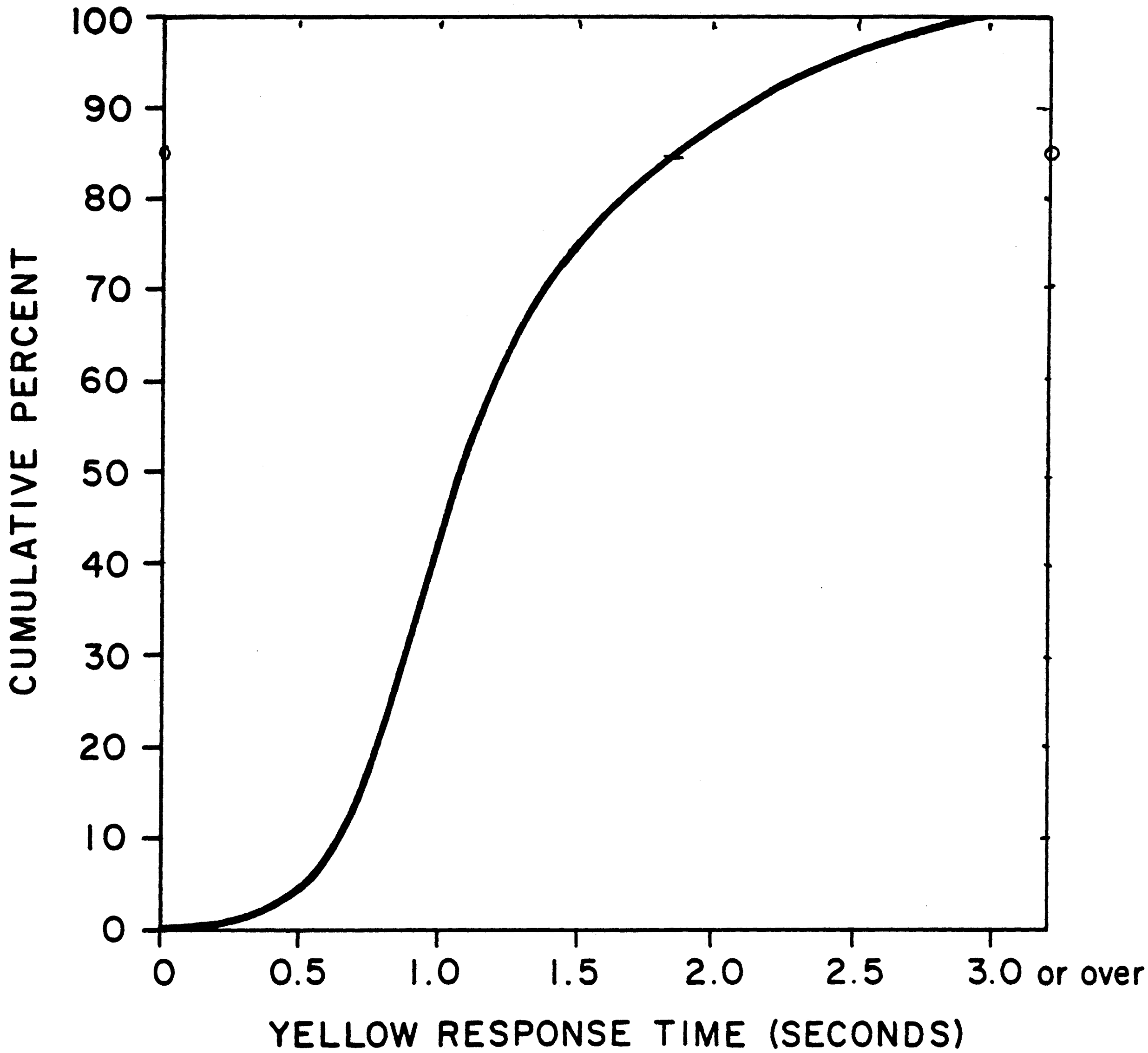


Figure 13. Cumulative distribution of yellow response time.

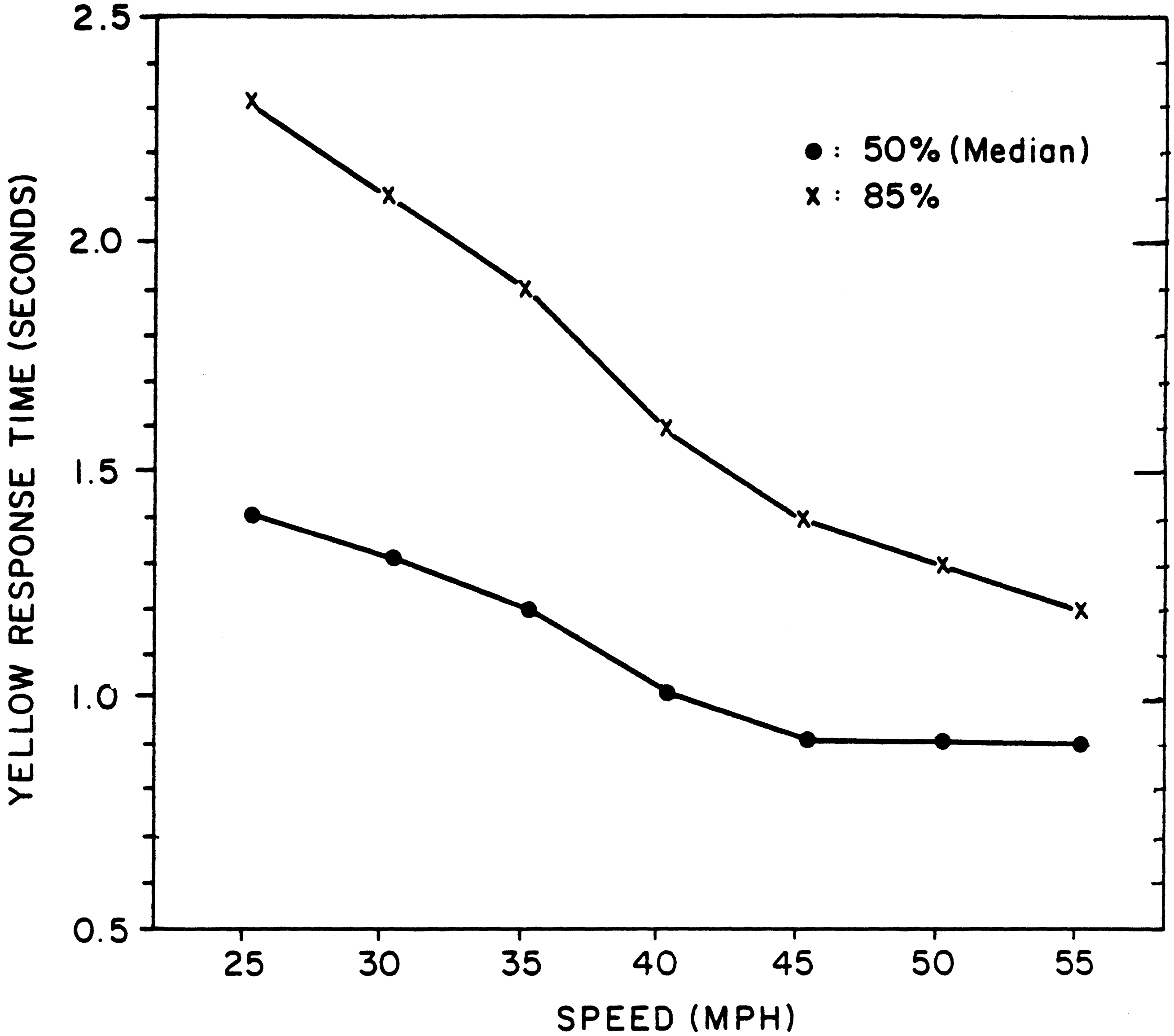


Figure 14. Yellow response time by speed categories.

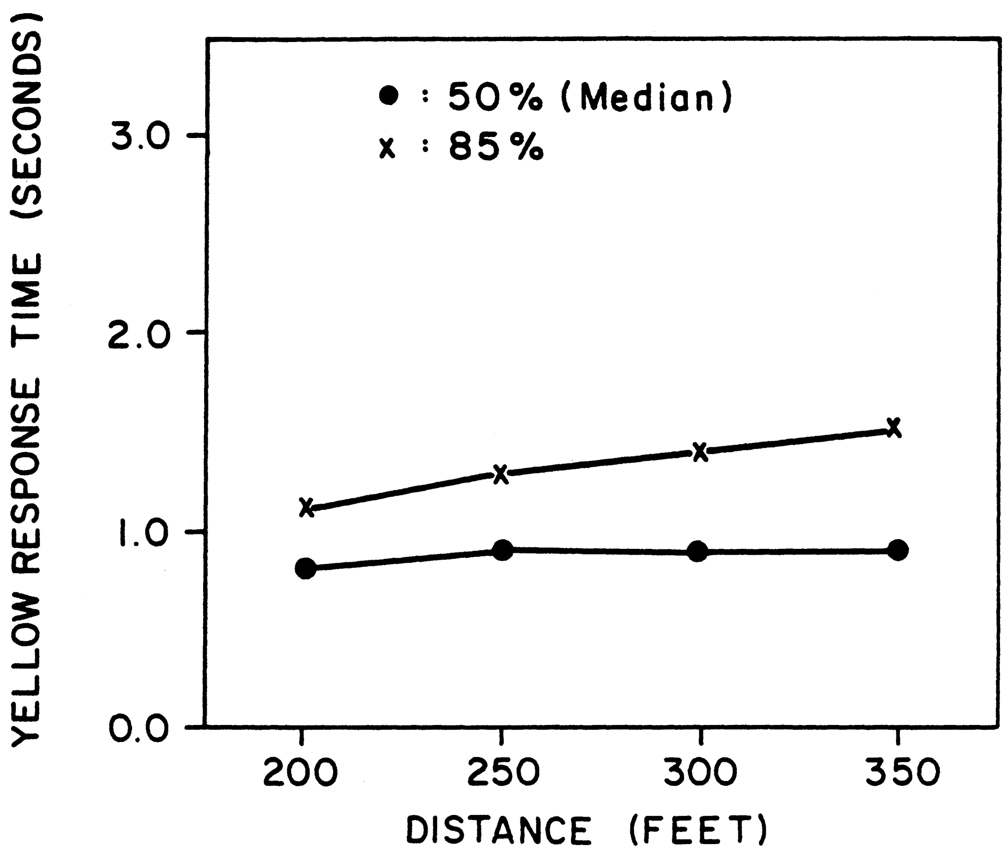


Figure 15. Yellow response time by distance for speeds over 40 mi/h.

Combined results from figures 14 and 15 indicate that median perception and brake reaction time of drivers at high speeds is 0.9 seconds. The response lag time for the median drivers are not expected to be significant and the probable value may be around 0.1 second.

If the practice of setting speed limit as 85 percent of approach speed is adopted, the combined results from figures 14 and 15 indicate that 1.2 seconds of yellow response time observed from both higher speed categories and the closer distances to the intersection appears to be a good estimator of perception and brake reaction time. It is also noted that the 1.2 seconds will also include an unidentified amount of response lag time. Therefore, the 85 percentile value taken from yellow response time may be close to 90 to 95 percent value in the perception and brake reaction time distribution.

Factors Affecting Yellow Response Time

The general characteristics of yellow response time affected by drivers' approach speeds, the distance to the intersection, and the interaction of these two are reported in a previous pilot study.[17]

This expanded analysis revealed similar characteristics. The effect of speed on yellow response time previously shown on figure 14 illustrated that the yellow response time decreases as speed increases, while it increases as speed decreases. This relationship is apparently attributed to driver response lag time which usually occurs between perception and brake reaction.

To examine the effect of distance on yellow response time, the observed distances were classified into six categories from 150 to 350 ft. The distance shown is the middle point of 100-ft intervals. Figure 16 presents the effect of distance on yellow response time over all speed categories. It shows, in general, that yellow response time increases as the distance to the intersection increases while it decreases as the distance to the intersection decreases. The driver response lag time is also applicable to this phenomenon.

To understand the combined effects of speed and distance, and its interaction on yellow response time, stepwise multiple regression is used and the model obtained at $\alpha = 0.03$ is as follows:

$$\begin{aligned} YRT = {} & 0.507 - 0.712\,(DONSETY/100) + 0.423\,(DONSETY/ASPEED) \\ & + 0.091\,(DONSETY/100)^2 \qquad R^2 = 0.39 \end{aligned}$$

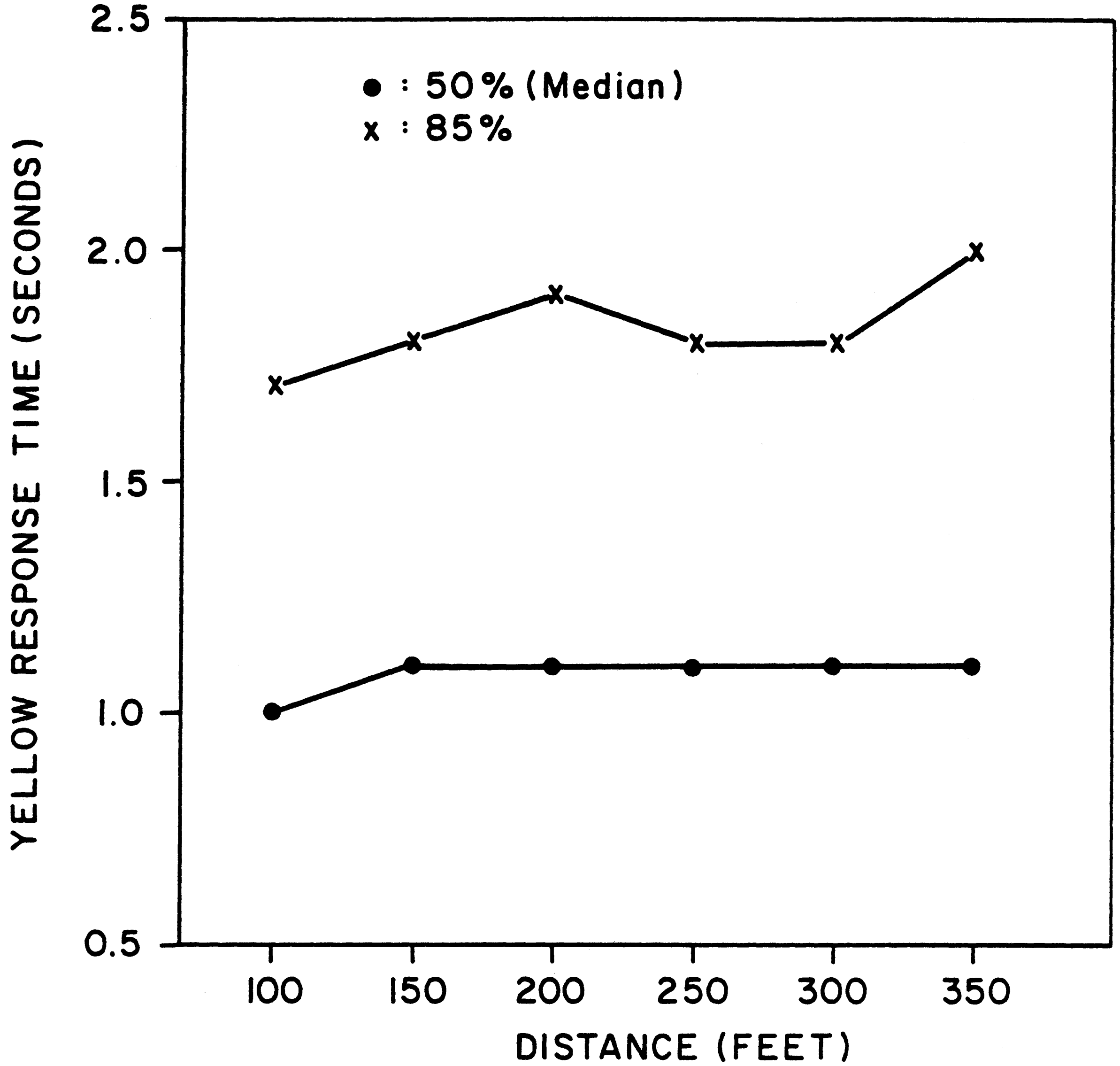


Figure 16. Yellow response time by distance categories.

where

YRT = Yellow response time, seconds.

DONSETY = Distance to intersection at yellow onset, feet.

ASPEED = Approach speed of vehicle, feet per second.

Graphical presentation of the model in figure 17 shows that driver's yellow response time decreases as approach speed increases and it decreases as distance to the intersection at yellow onset decreases. Further, the model revealed that driver's yellow response time decreases as time available to reach the stop line decreases.

Effect of Signal Controller Type on Yellow Response Time

A test was performed to see if there is any effect of signal controller type on yellow response time. The test revealed that there is a statistically significant effect, at $\alpha = 0.05$, due to different controller type on yellow response time. Specifically, drivers tended to react quicker to the actuated controller than to a pretimed controller. However, the magnitude of difference is found to be less than 0.1 seconds. Thus, the effect of controller type on yellow response time is practically negligible.

Effect of Light on Yellow Response Time

Three intersections having sufficient samples encompassing day and night time samples were tested to see if there is any difference on yellow response time due to light conditions. The test revealed that there is no difference in yellow response time, at $\alpha = 0.05$, due to the light condition. This suggests that drivers tend to react consistently during both the daytime and nighttime.

Effect of Weather on Yellow Response Time

One intersection having sufficient samples covering dry and wet pavement conditions was tested to see if there is any difference on yellow response time due to weather. The test revealed that there is no difference in yellow response time, at $\alpha = 0.05$, due to weather condition. This suggests that drivers do not appear to adjust their response particularly due to wet pavement conditions.

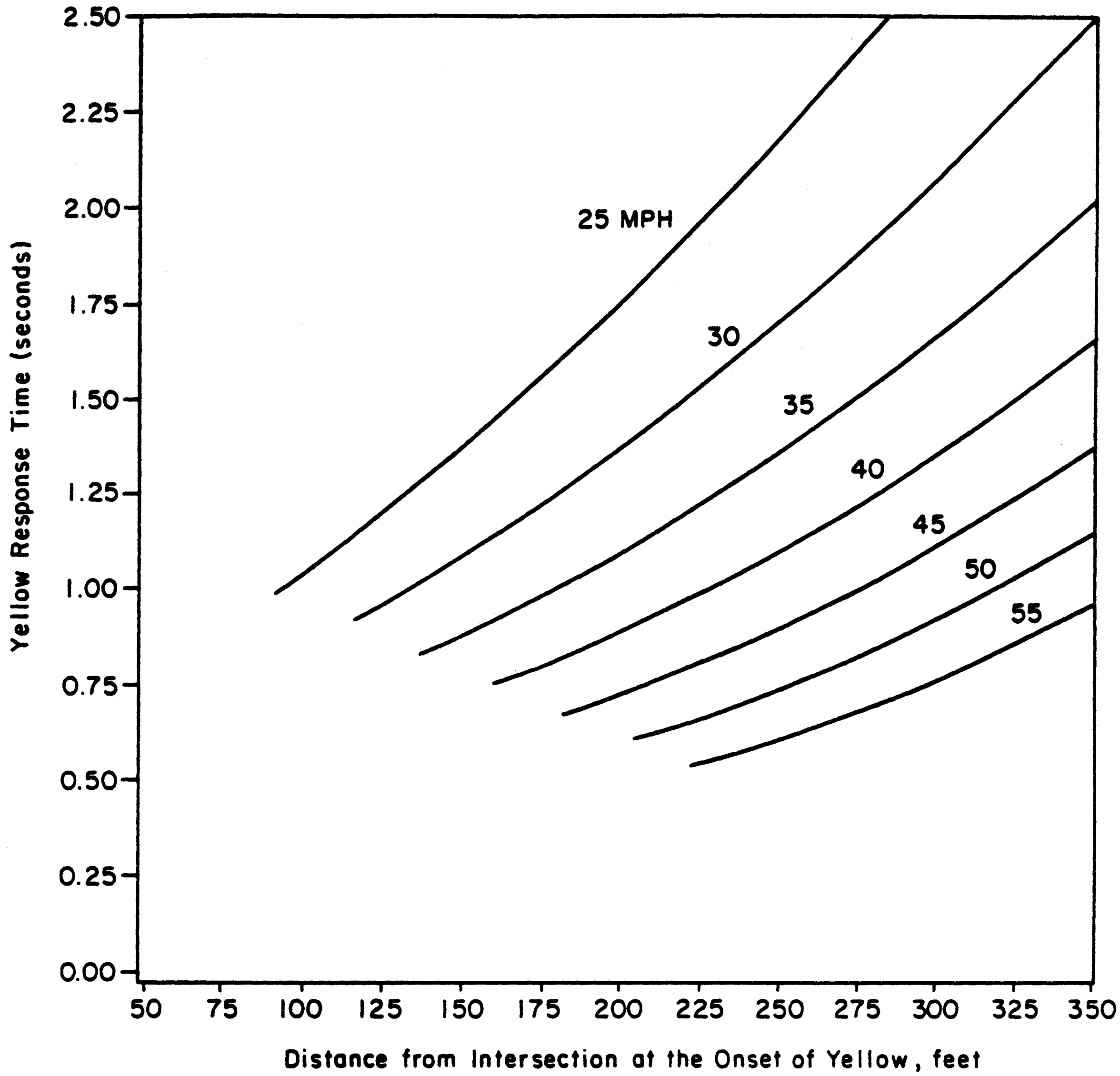


Figure 17. Yellow response time as a function of distance, speed, and time.

Discussion of the Various Effects on Yellow Response Time

The effects of various traffic control and environment conditions were tested on yellow response time. Note that these effects were not tested on perception and brake reaction time. The test results are to be interpreted as that the majority of drivers do not appear to react differently due to different conditions. The tests were not applied to identical drivers facing different conditions. It may be expected that the same driver may react quicker during nighttime and/or wet pavement conditions. However, the test based on identical drivers could not be performed for this study. It is also noted that even if there is a difference in perception and brake reaction time for a different condition by a driver, the practical difference may be small due to the limitations on the extent of mental and physical reactions.

DECELERATION RATE

Table 5 shows the deceleration performance observed from field studies. It shows that the mean and the median deceleration performance for the total stopping vehicle population was 9.5 and 9.2 ft/s^2, respectively. The cumulative distribution of deceleration rates observed for 579 stopping vehicles is shown in figure 18. Eighty-five percent of vehicles selected a deceleration rate of 5.6 ft/s^2 or more, and 95 percent used 4.3 ft/s^2 or more. It is noted that the deceleration rate observed from field studies is primarily the result of driver's selection of comfort. It is not an indication of whether they can perform certain deceleration rates.

To derive a deceleration rate that drivers can perform, the speed influence approach used in deriving perception and brake reaction time is adopted. Figure 19 shows the deceleration performance categorized by speed. It shows that 85 percent of vehicles at 55 mi/h speed categories can perform deceleration rates of 10.6 ft/s^2 or more. The deceleration rate of 10 ft/s^2 assumed by the ITE Handbook(9) corresponds to 90 percent performance in this speed category. It is further noted that this 10 ft/s^2 deceleration rate can also be performed by more than 85% of trucks.(39) A deceleration rate of 10.5 ft/s^2 appears to be a good estimator for level grade.

Factors Affecting Deceleration Rate

The model to evaluate the factors affecting deceleration rate must be guided by the "laws of motion" from physics. These indicate that the decele-

Table 5. Deceleration characteristics of stopping vehicles.

	Intersection Approach Numbers													
Statistics	1	2	3	4	5	6	7	8	9	10	11	12	13	Total
Mean	9.3	8.6	8.6	7.8	10.6	9.8	7.8	10.7	13.4	11.5	10.8	8.3	8.9	9.5
Median	7.7	8.6	8.1	7.6	10.9	9.4	7.9	10.2	12.9	10.8	11.3	8.1	9.1	9.2
85%	4.2	5.9	5.3	5.0	6.5	6.9	4.8	5.6	9.7	8.4	6.4	6.2	5.4	5.6
95%	3.1	3.6	3.8	4.2	5.5	5.5	3.7	4.6	7.4	6.8	4.2	4.0	4.9	4.3

Note: Entry is deceleration rate in feet per second per second (ft/s^2).

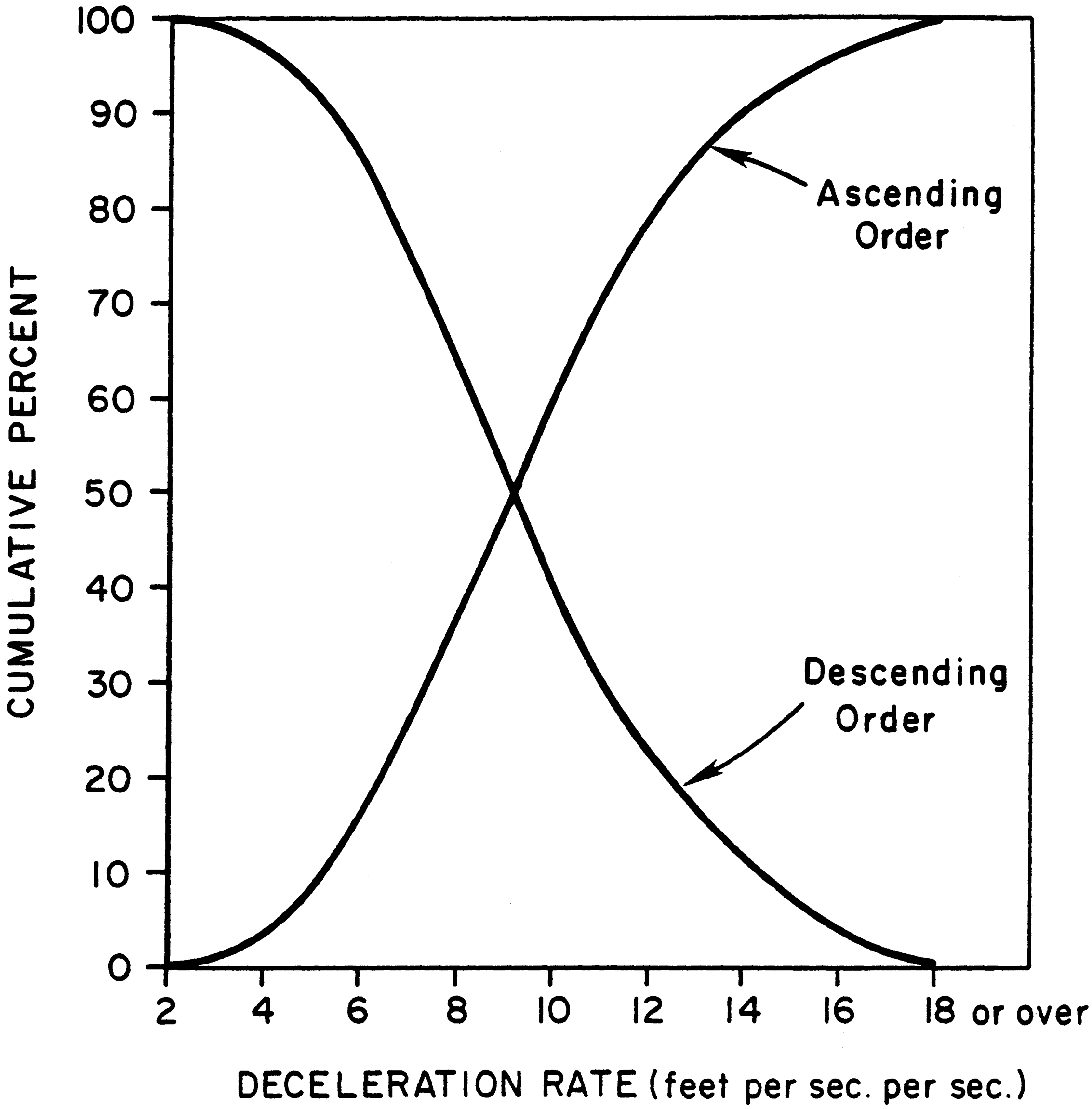


Figure 18. Cumulative distribution of deceleration rate.

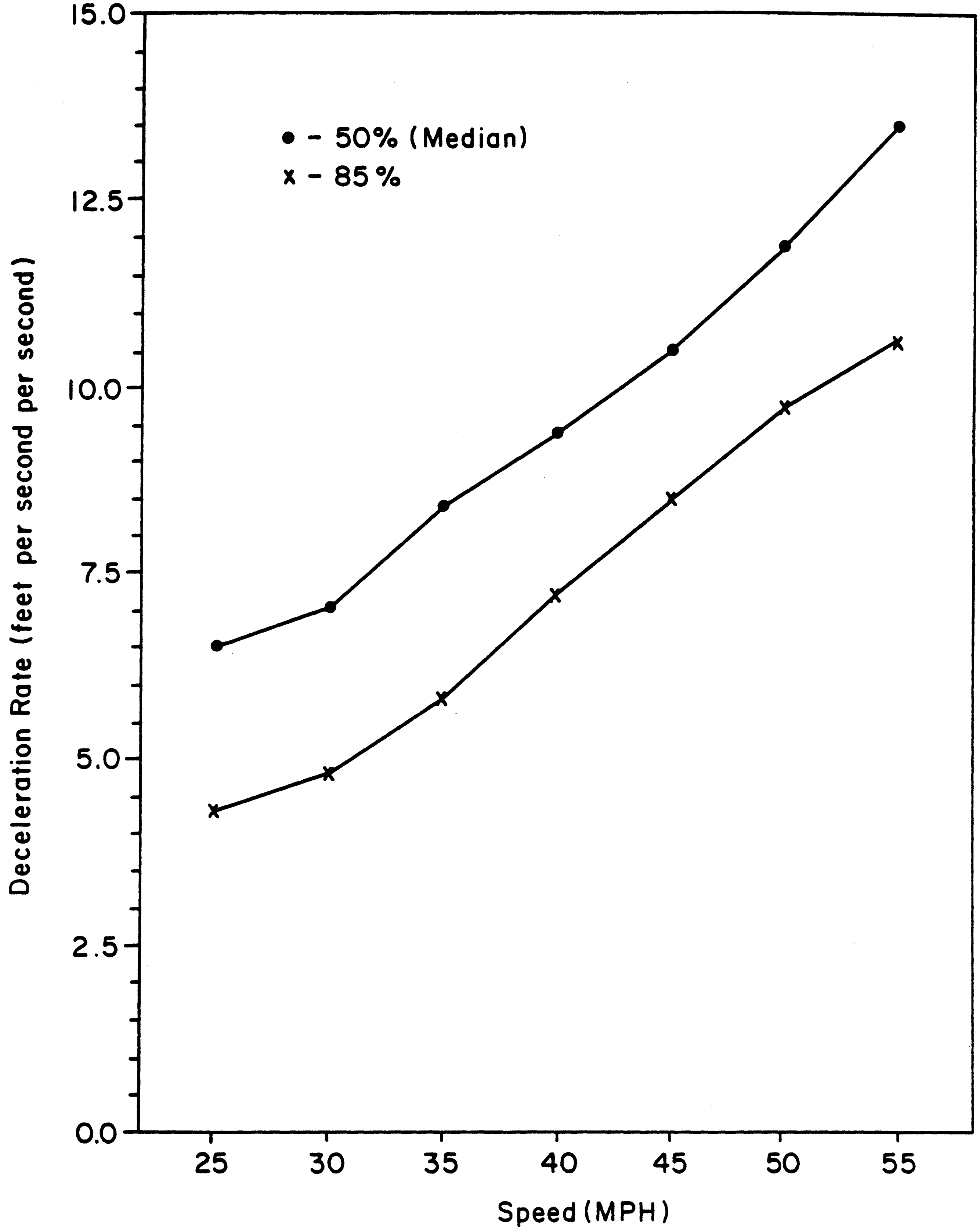


Figure 19. Deceleration rate by speed categories.

ration rate is affected by speed, distance, time, and grade. Since the braking distance available depends on the distance traveled during yellow response time, the additional interaction variable between speed and yellow response time is introduced. The deceleration rate (DR) model obtained from stepwise regression at $\alpha = 0.01$ is as follows:

$$DR = 13.365 + 0.176\ (ASPEED/10)^2 - 2.933\ (DONSETY/100) + 0.085\ Grade - 1.110\ (DONSETY/ASPEED) + 0.044\ (ASPEED \times YRT) \qquad R^2 = 0.86$$

The model shows that deceleration rate increases as speed, grade, and distance traveled during yellow response time increase. It increases as distance to the intersection at yellow onset and available time to reach the stop line decrease. The square distance term ($DONSETY^2$) is added for graphical presentation for the case of grade = 0 and YRT = 1 second, shown in figure 20, to increase its predictability.

Grade Effect on Deceleration Rate

The adjustment of grade effect on deceleration performance has been advocated by some researchers.[10] Consequently, the effect of grade on deceleration rate was tested. Several multiple linear regression models were evaluated using the general linear test method. Throughout the models tested, the coefficients of grade were significant at $\alpha = 0.05$ and remained relatively stable at around 0.065 to 0.085 at an average of 0.075.

When the exact adjustment of grade effect on deceleration rate is desired, the following equation may be used:

$$d = 10.5 \pm 0.075g$$

where

g = percent of grade (use positive for upgrade and negative for downgrade)

The following deceleration rate appears to be a good estimator for safety and practical purposes:

For level and upgrade	$d = 10.5\ ft/s^2$
For downgrade	$d = 10.0\ ft/s^2$

Effect of Other Factors on Deceleration Rate

A test was performed for three intersections having sufficient samples covering day and nighttime samples to see if there is any difference on

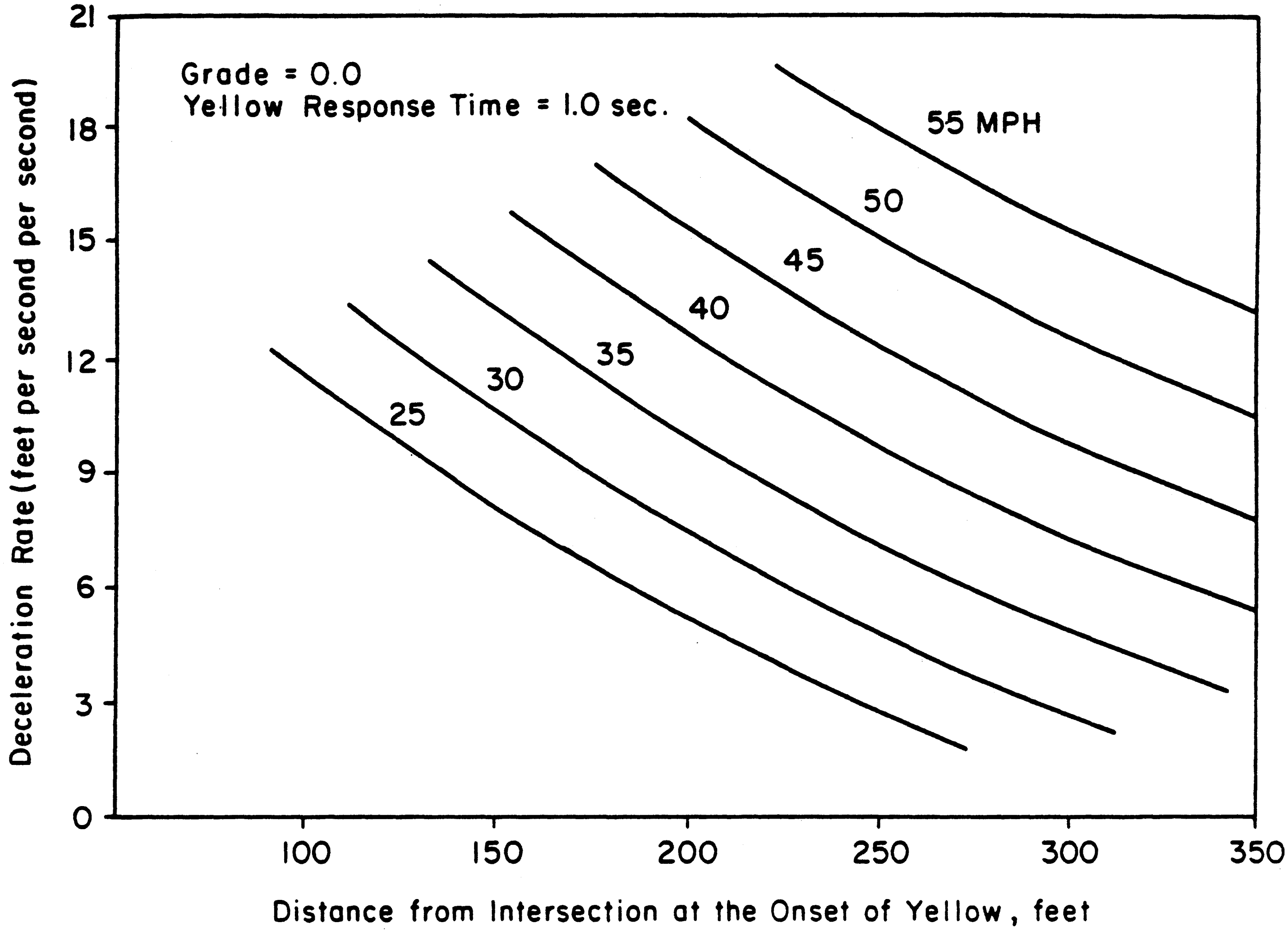


Figure 20. Deceleration rate as a function of distance, speed, and time.

deceleration rate due to light condition. The test revealed that there is no difference in deceleration rate, at $\alpha = 0.05$, due to light condition. This suggests that drivers do not appear to select significantly different deceleration rates during nighttime compared to daytime.

Further, a test was performed for an intersection having sufficient samples covering dry and wet pavement conditions to see if there is any difference on deceleration rate due to weather. The test revealed that there is no difference in deceleration rate, at $\alpha = 0.05$, due to weather condition.

It is noted that the limitations of test inapplicable to an identical driver discussed in yellow response time also holds for the test on deceleration rate.

DISTANCE AT BRAKE ACTUATION

The cumulative distribution of distance at brake actuation observed for 579 stopping vehicles is shown on figure 21. It is observed that the mean and the median distance at brake actuation was 180 ft. Eighty-five percent of vehicles actuated brakes at 235 ft and 95 percent applied brakes at 275 ft to the intersection.

Figure 22 shows the distance at brake actuation categorized by speed. The speed shown is the middle point of 10-mi/h intervals. It shows that 85 percent of drivers applied brakes at 170 ft to 265 ft at speed catagories ranging from 25 to 55 mi/h.

TIME EFFECT ON DRIVERS' DECISION TO STOP OR TO GO

It is hypothesized that drivers' perceived times to reach the stop line may influence their decision to stop or to go. The time to reach the stop line for stopping vehicles is obtained assuming constant approach speed. The time to reach the stop line for going vehicles is the actual time elapsed from the yellow onset to reach the stop line.

Figure 23 presents the time effect illustrated by speed categories. It shows that practically no vehicles stopped when they are 2 seconds or less away from the intersection. It reveals that 85 percent of stopped vehicles did stop because they were about 3 seconds or more away from the intersection. It also shows that 85 percent of going vehicles continued through because they could actually enter the intersection within about 3.7 seconds or less travel time. Further, it shows that ninety-five percent of going vehicles took less

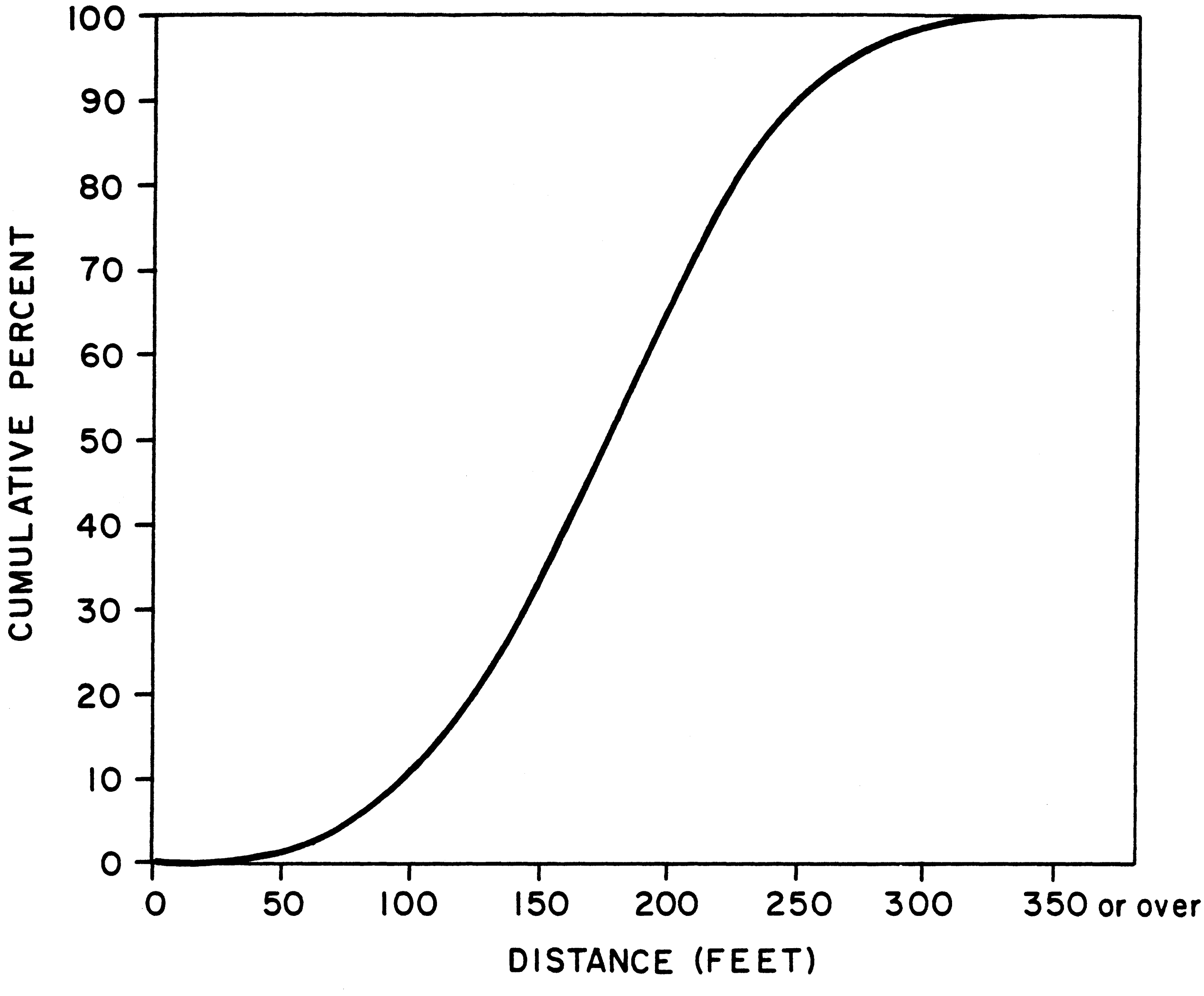


Figure 21. Cumulative distribution of distance at brake actuation.

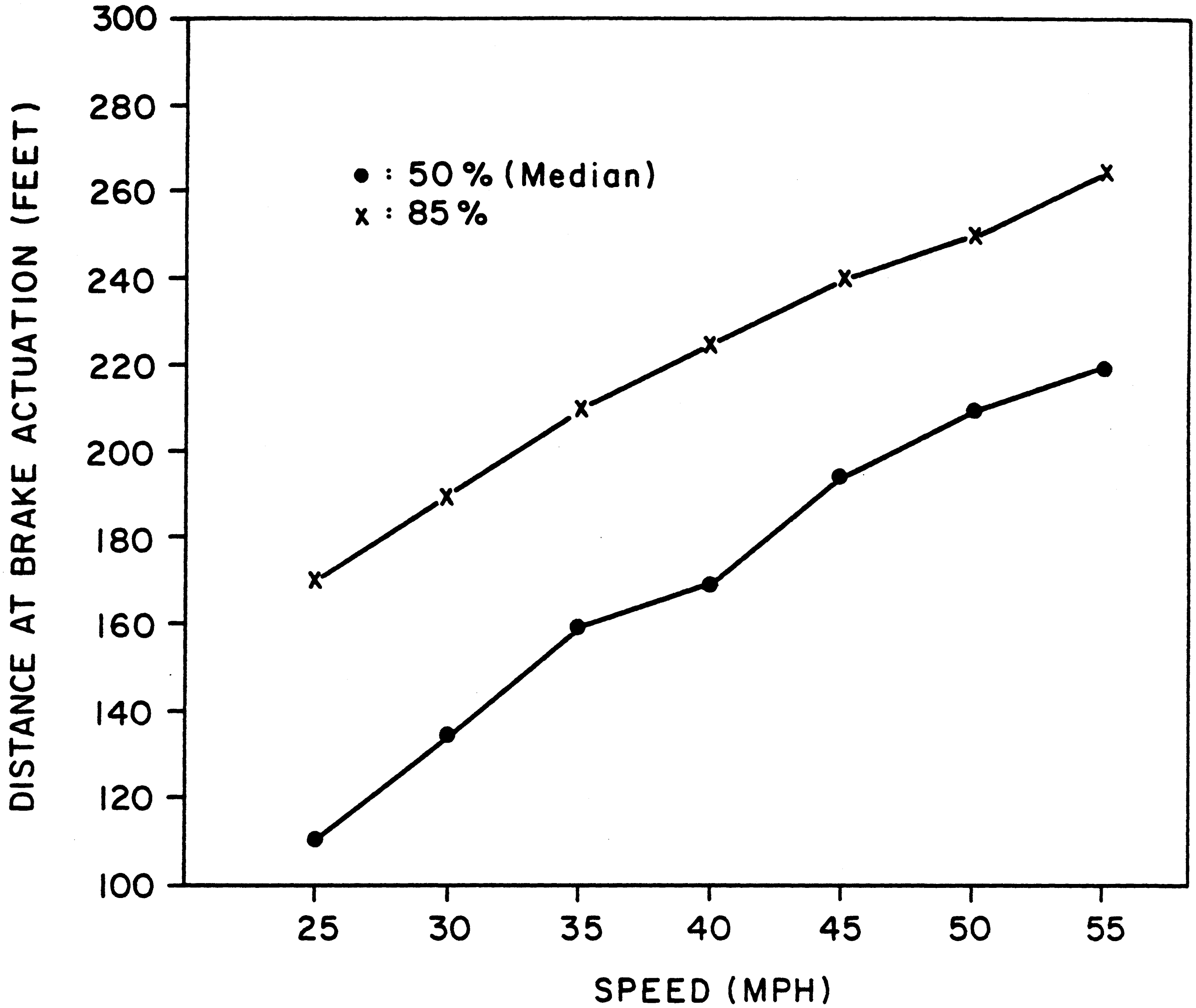


Figure 22. Distance at brake actuation by speed categories.

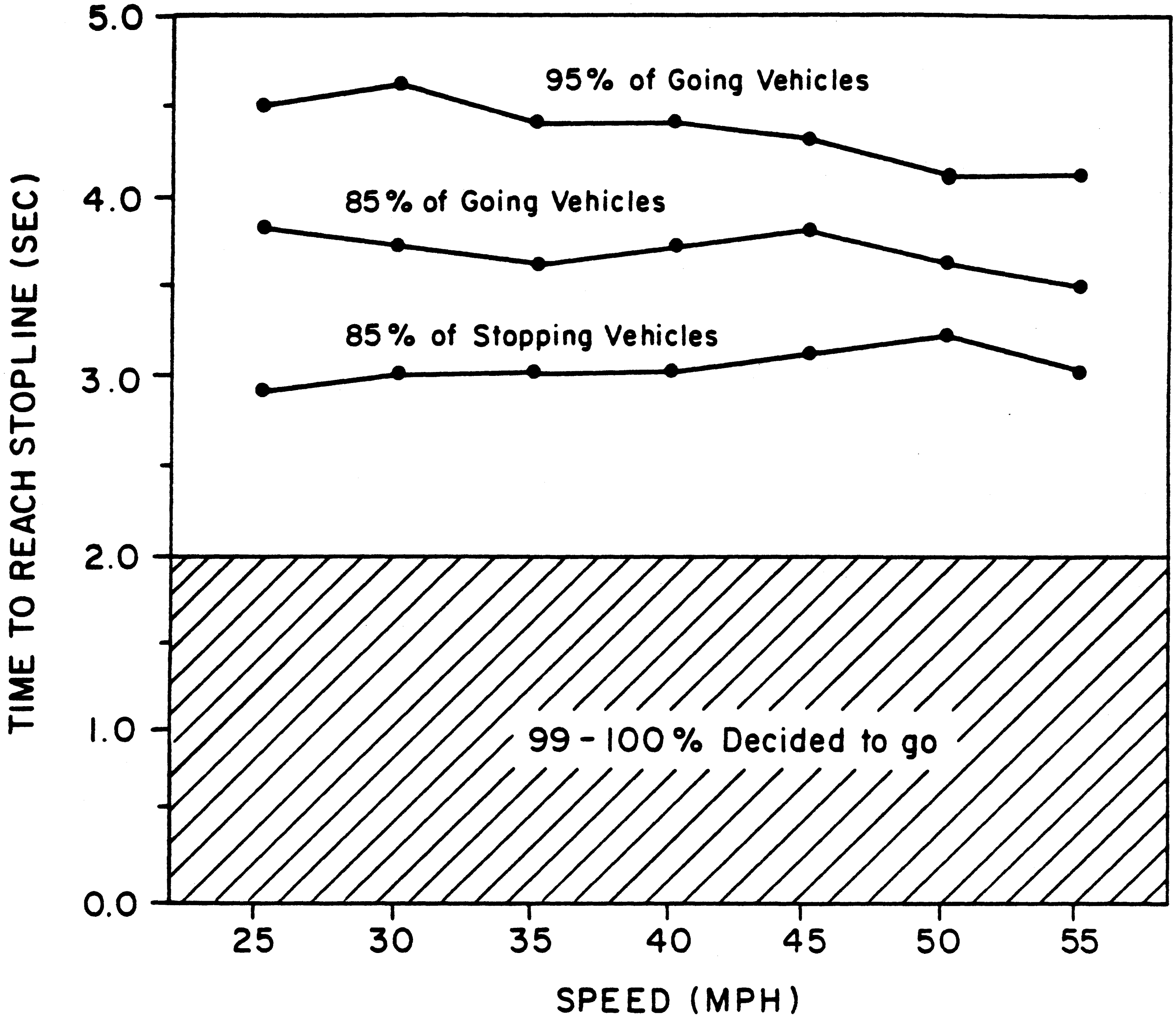


Figure 23. Driver's decision to stop or to go by time.

than 4.5 seconds to enter the intersection. It is also noted that the time effect is relatively stable across the speed categories. However, it is for each stopping or going vehicles.

Figure 23 also presents the dilemma of continued use of the current change interval design formula. The current formula provides increased time for higher speeds while practice provides minimum yellow interval for lower speeds. Figure 23 shows that the real danger may lie in lower speed categories below 40 mi/h.

Figure 23 also provides a good opportunity to consider the use of a constant yellow interval across the speed ranges. Olson and Rothery(13) suggested a constant yellow interval of 5.5 seconds proclaiming that such yellow duration will provide all or nearly all drivers time to clear an intersection. While their justification does not appear to be sound due to different dimensions of intersection width, figure 23 appears to show a warranting condition for a constant yellow interval of 4.5 seconds from the fact that 95 percent of going vehicle did go through when they took less than about 4.5 seconds regardless of their speeds. The determination of yellow interval based on going vehicles is warranted because the fundamental problem of the yellow interval lies in the clearing vehicle rather than the stopping vehicle. The basic reason is that the first car stopped has no vehicles with which to collide. The following vehicles may collide with the first vehicle stopped. However, the rear-end collision in this case is not due to the yellow interval but simply due to driver expectancy violation along with following too closely.

PROBABILITY MODELING OF DRIVER DECISION TO STOP OR TO GO

Past studies reported that drivers' decisions to stop or to go were affected by approach speed, distance from the intersection at the yellow onset, and the time to reach the stop line.(15,16,40) These three different decision affecting factors may be illustrated by figure 24. Figure 24a shows the case of speed dominance decision in which the slope of the same time is downward to the lower probability of stopping as distance is increased. Figure 24b shows the distance dominance decision in which the slope of the same time is upward to the higher probability of stopping as distance is increased. Figure 24c shows the time dominance decision in which the slope of the same time has approximately same probability of stopping. The model by

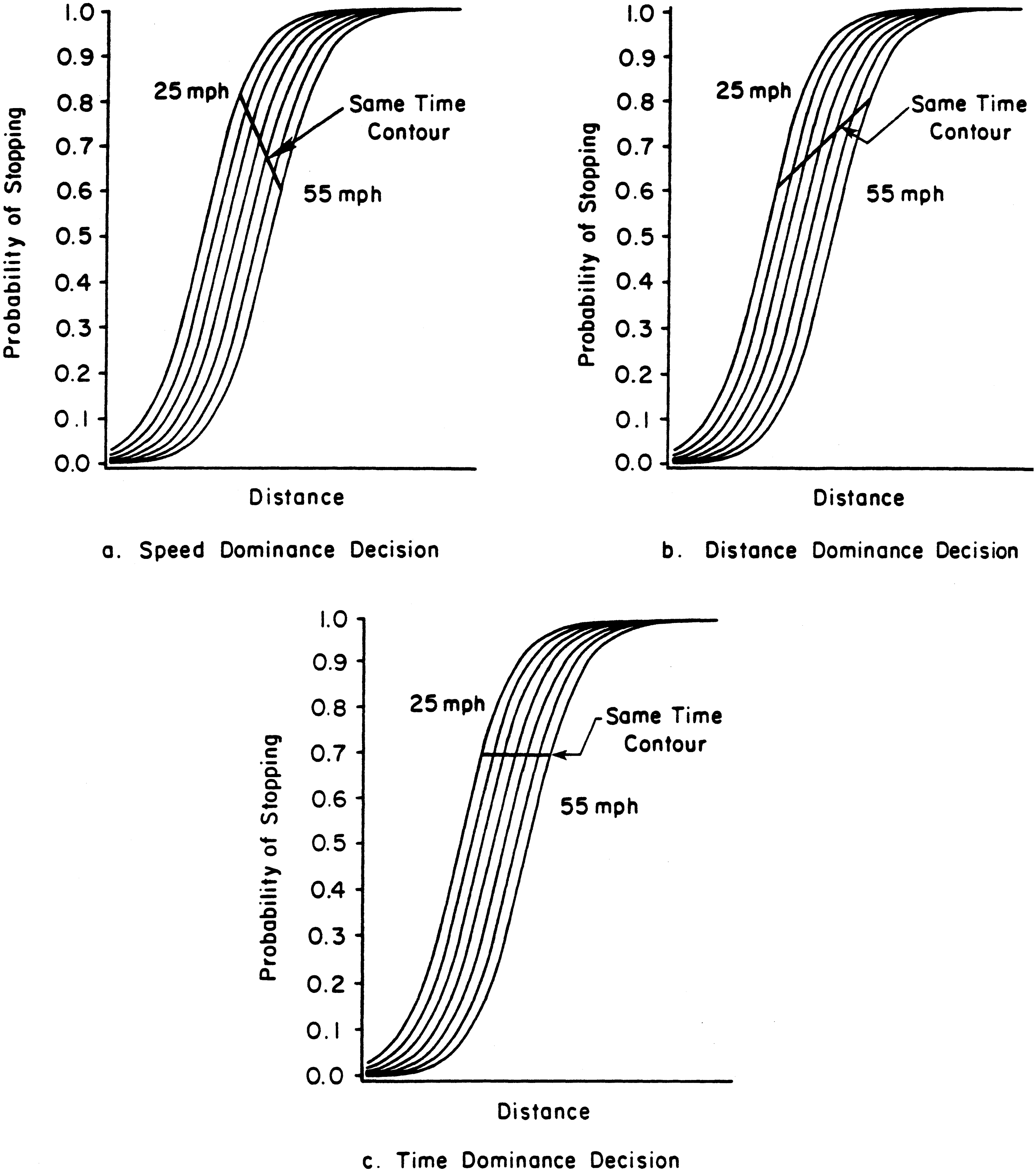


Figure 24. Relationship between probability of stopping and driver's decision pattern.

Williams[15] has a time characteristic of figure 24b while the model by Sheffi and Mahmassani[16] has a time characteristic of figure 24c.

Logistic regression (or logit model) was used to derive the probability of stopping or going as a function of speed, distance, and time. For the stopping vehicle, time is derived assuming constant speed as mentioned previously. The detailed model forms and interpretations will be subsequently presented.

Time Model

Probability of stopping as a function of time is analyzed and the model obtained is as follows:

Probability of stopping = 1/[1+exp(5.332-1.32 Time)]

The model revealed that the probability of stopping increases as the time to reach the stop line increases. The graphical presentation shown in figure 25 illustrates the time model. The predicted performance of the time model compared to the observed frequencies of stopping and going is shown in table 6a.

Time and Distance Model

The probability of stopping as a function of time and distance is analyzed and the model obtained is as follows:

Probability of stopping = 1/[1+exp(5.704-0.904 Time-0.948(DONSETY/100))]

The model revealed that the probability of stopping increases as time to reach the stop line and distance to the intersection increase. The graphical presentation shown in figure 26 illustrates the time and distance model. The predicted performance of the time and distance model compared to the observed frequencies of stopping and going is shown in table 6b.

Time and Speed Model

Probability of stopping as a function of time and speed is analyzed and the model obtained is as follows:

Probability of stopping = 1/[1+exp(7.285-1.384 Time-0.031 ASPEED)]

The graphical presentation shown in figure 27 illustrates the time and speed model. It revealed that the probability of stopping increases as time to reach the stop line increases and it decreases as approach speed increases. The predicted performance of the time and distance model compared to the

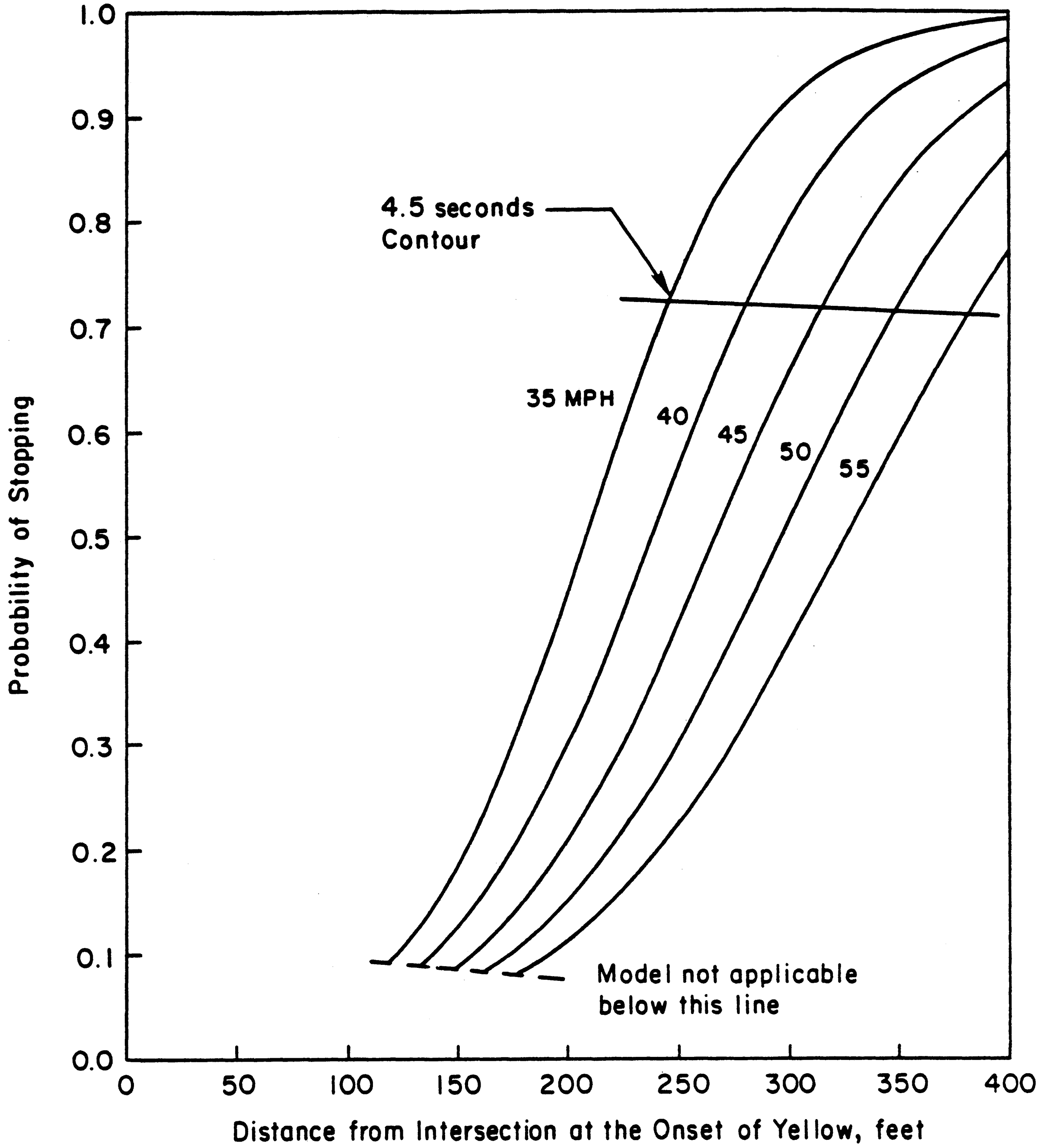


Figure 25. Probability of stopping for time model.

Table 6. Predicted performances of probability models.

A. Time Model		Predicted		
		Going	Stopping	Total
Observed	Going	882	153	1035
	Stopping	196	383	579
	Total	1098	536	1614

Correct Rate = (882 + 383)/1614 = 78.4%
False Stopping Rate = 153/536 = 28.5%
False Going Rate = 196/1078 = 18.2%

B. Time and Distance Model		Predicted		
		Going	Stopping	Total
Observed	Going	884	151	1035
	Stopping	182	397	579
	Total	1066	548	1614

Correct Rate = (884+397)/1614 = 79.4%
False Stopping Rate = 151/548 = 27.6%
False Going Rate = 182/1066 = 17.1%

C. Time and Speed Model		Predicted		
		Going	Stopping	Total
Observed	Going	883	152	1035
	Stopping	187	392	579
	Total	1070	544	1614

Correct Rate = (833+392)/1614 = 79.0%
False Stopping Rate = 152/544 = 27.9 %
False Going Rate = 187/1070 = 17.5%

D. Distance and Speed Model		Predicted		
		Going	Stopping	Total
Observed	Going	887	148	1035
	Stopping	182	397	579
	Total	1069	545	1614

Correct Rate = (887+397)/1614 = 79.6%
False Stopping Rate = 148/545 = 27.2%
False Going Rate = 182/1069 = 17.0%

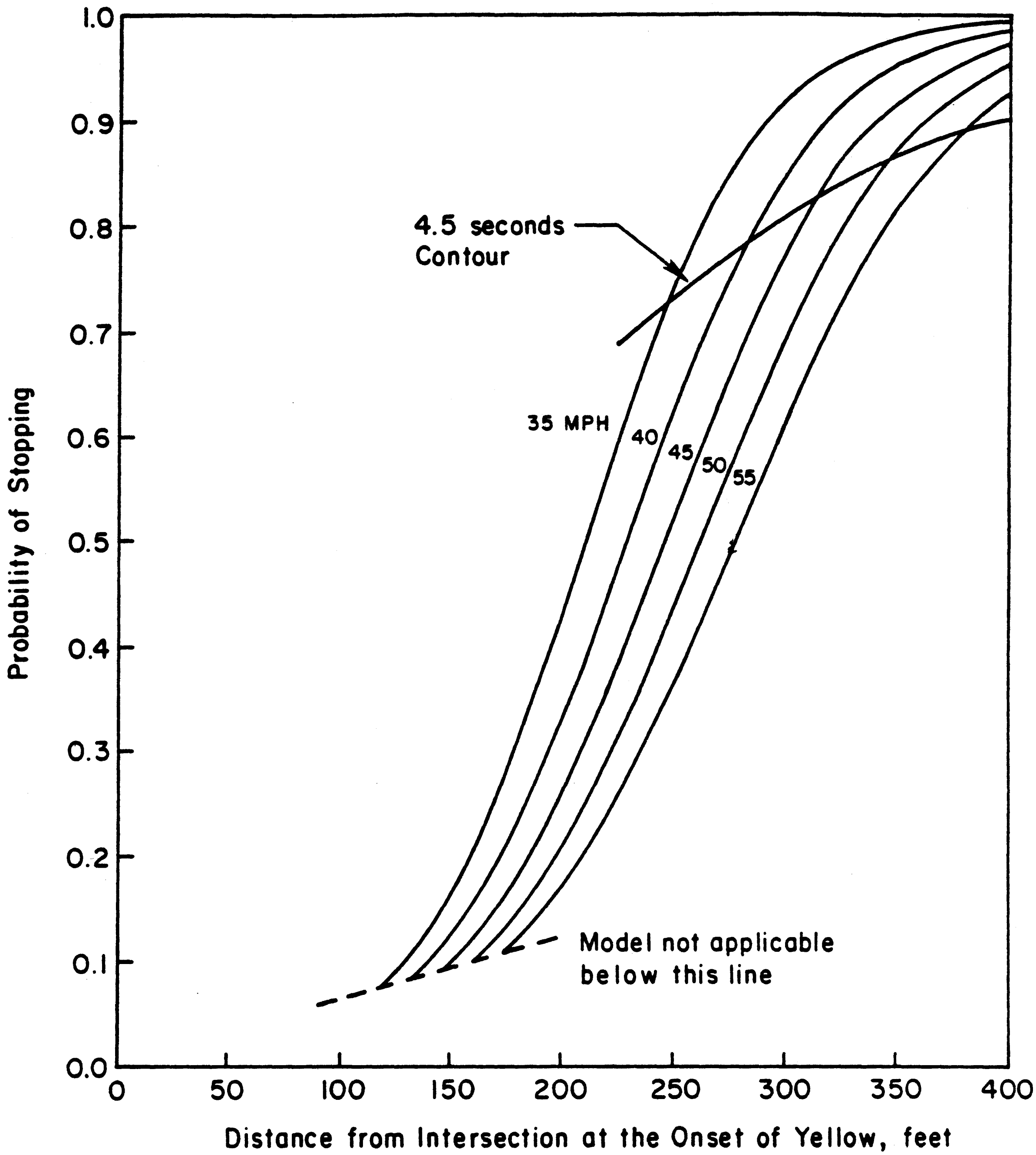


Figure 26. Probability of stopping for time and distance model.

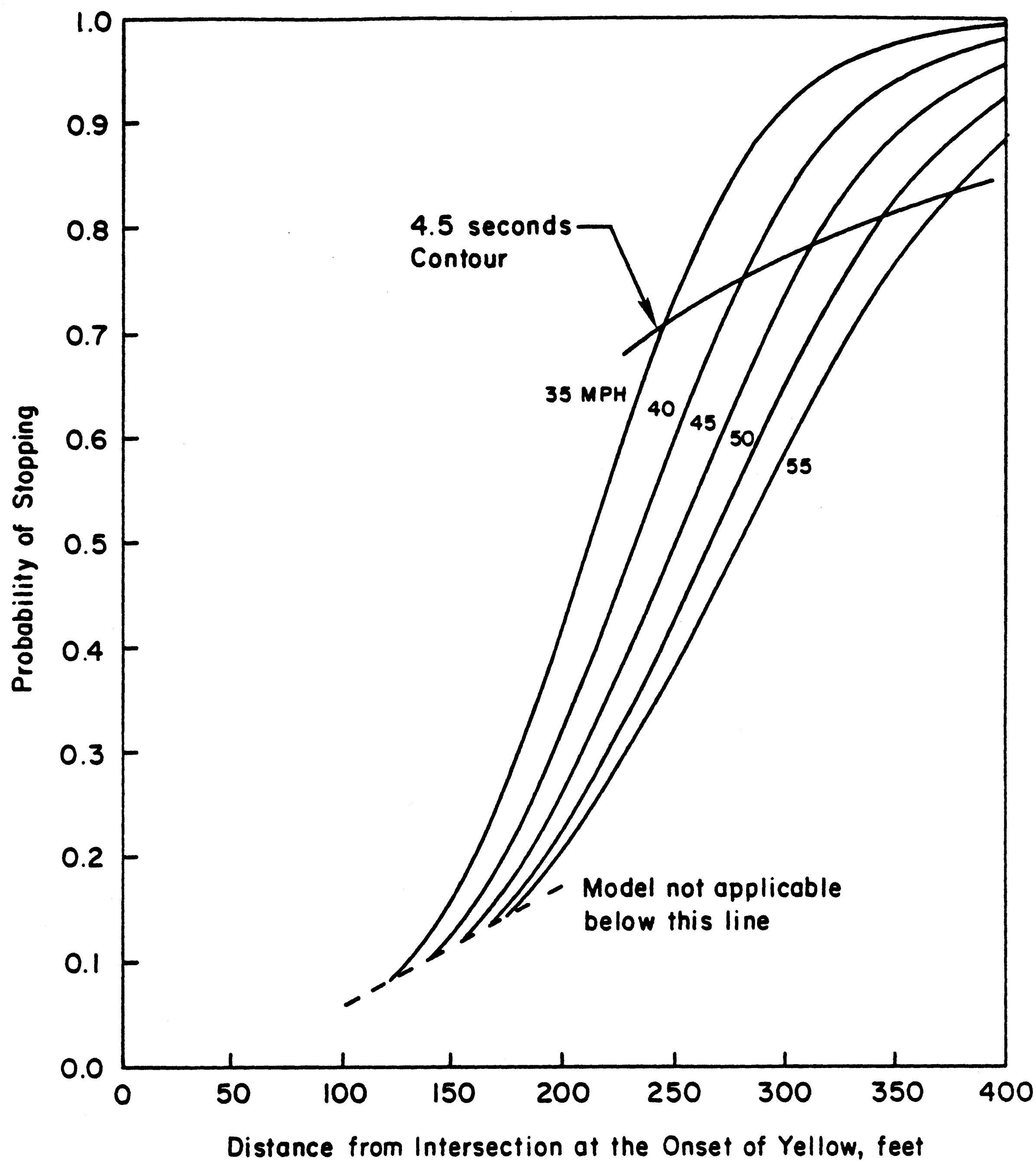


Figure 27. Probability of stopping for time and speed model.

frequencies of stopping and going is shown in table 6c.

Time, Distance, and Speed Model

The stepwise logistic regression model revealed that the first important variable entered was time, the next distance and the next speed in sequence. However, when the distance and speed are entered, time became insignificant at the chi-square value of zero. Thus, the model obtained at $\alpha = 0.05$ is as follows:

$$\text{Probability of stopping} = 1/[1+\exp\,(2.083 - 2.755(\text{DONSETY}/100) + 0.071\ \text{ASPEED})]$$

The model revealed that the probability of stopping decreases as distance decreases and it decreases as approach speed increases. The graphical presentation shown in figure 28 illustrates these characteristics. Figure 28 also reveals that the driver decision is a distance dominance pattern previously illustrated in figure 23b. The predicted performance of the probability model compared to the observed frequencies of stopping and going is shown in table 6d. The probability model predicted with eighty percent accuracy (the most among four models evaluated) the responses of stopping and going.

Effect of Grade on Driver Decision to Stop or to Go

It is postulated that grade may have an effect on drivers' decision to stop or to go. It is expected that more drivers may decide to go through rather than to stop, given same approach speed and distance to the intersection, at downgrades than upgrades. The effect of grade on drivers' decision to stop or to go was tested using the logistic model. The model obtained at $\alpha = 0.01$ is as follows:

$$\text{Probability of stopping} = 1/[1+\exp\,(1.870 - 2.790\,(\text{DONSETY}/100) + 0.069\ \text{ASPEED} - 0.115\ \text{GRADE})]$$

where

Grade = percent of grade (use positive for upgrade and negative for downgrade

The model revealed that the probability of stopping increases as grade increases and it decreases as grade decreases. In other words, more drivers tend to stop on upgrades but tend to go through on downgrades. The provision

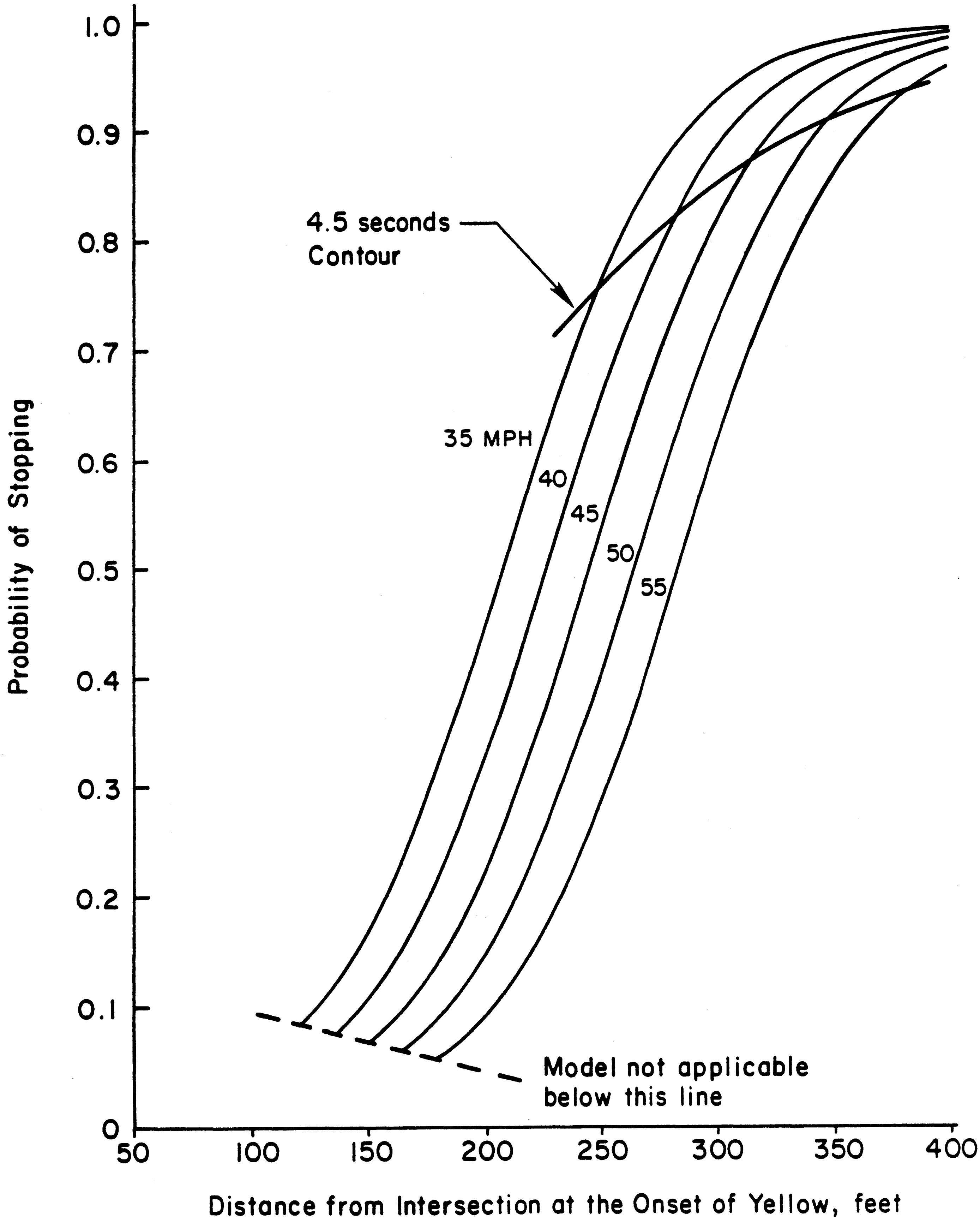


Figure 28. Probability of stopping as a function of distance and speed.

of an all-red interval on downgrades would be helpful to counterbalance the potential accidents due to the greater tendency of drivers going through on downgrade approaches.

Effect of Intersection Width on Driver Decision to Stop or to Go

It is also postulated that intersection width may also have an effect on driver's decision to stop or to go. It is expected that more drivers may decide to go through rather than to stop, given same approach speed, distance, and grade to the intersection, at narrower intersections than at wider intersections. The effect of intersection width on drivers decision to stop or to go was tested using the logisitic model. The model obtained at $\alpha = 0.01$ is as follows:

$$\text{Probability of Stopping} = 1/[1+\exp(5.038 - 3.013\ (\text{DONSETY}/100) + 0.044\ \text{ASPEED} - 0.198\ \text{GRADE} - 0.014\ \text{INTW})]$$

where

INTW = Intersection width, feet

The model revealed that the probability of stopping increases as intersection width increases and it decreases as intersection width decreases. In other words, more drivers tend to stop at wider intersections but tend to go through at narrower intersections.

LINKAGE BETWEEN PROBABILITY OF STOPPING AND YELLOW INTERVAL

Analysis was performed to link the probability of stopping and yellow change interval involving yellow response time, deceleration rate, and approach speed, using distance and speed model.

Using the probability of stopping model by distance and speed, probability of stopping was varied from 0.15 to 0.85 and approach speed was varied from 35 mi/h to 55 mi/h by 5 mi/h interval. Given a probability of stopping and approach speed, the corresponding distance was obtained. Then, given distance and speed, the expected yellow response time is obtained using the equation previously presented. Similarly, given the distance, approach speed, and yellow response time, the expected deceleration rate is obtained. Further, given yellow response time, approach speed, and deceleration rate, the yellow change interval is obtained. The SAS program illustrating these detailed steps to link the probability of stopping and yellow change interval is presented in table 7.

Table 7. SAS program to link probability of stopping and yellow change interval.

```
DATA PROB;
DO P=0.15 TO 0.85 BY 0.05;
DO AMPH=35 TO 55 BY 5;
ASPEED=AMPH*528/360;
GRADE=0;
DONSETY=(100/2.755)*(2.083+0.071*ASPEED-LOG(1/P -1);
YRT=0.507-0.712*(DONSETY/100)+0.423*(DONSETY/ASPEED)
    +0.091*(DONSETY/100)**2;
DR=4.256+0.383*ASPEED-0.119*DONSETY+0.999*(DONESTY/100)**2
   +0.079*GRADE +0.949*(DONSETY/ASPEED) +0.043*ASPEED*YRT;
YELLOW=YRT+(ASPEED/(2*DR));
OUTPUT; END; END;
```

Figure 29 presents the relationships among the probability of stopping, deceleration rate, and yellow response time for approach speeds from 35 to 55 mi/h. The figure 29 comprehensively illustrates the following relationships.

- As probability of stopping increases deceleration rate decreases and yellow response time increases; this means that when the majority of drivers decided to stop, they use both comfortable deceleration rate and yellow response time.
- As speed increases, deceleration rate increases and yellow response time decreases; this means that when majority of drivers decided to stop at higher approach speed, they react quicker and brake harder.

Figure 30 presents the relationship between probability of stopping and yellow change interval for speeds of 35, 45, and 55 mi/h. Figure 30 revealed that yellow change interval increases as the probability of stopping increases. It also showed that yellow change interval decreases as approach speed increases at a given probability of stopping. This relationship is a reverse characteristic of the conventional practice in which a longer yellow change interval is used for a higher approach speed. The reverse relationship is based on driver behavior and is apparent for following reasons presented throughout this report and specifically illustrated in previous page. That is, when drivers decided to stop at higher approach speed they react quicker and brake harder. Consequently, yellow change interval will be decreased as follows:

Yellow Change Interval = Yellow Response Time (or perception-reaction time)
+ Approach Speed/2(Deceleration Rate)

Note that as drivers react quicker, the first term (yellow response time) get smaller. Further, as drivers brake harder, the second term get also smaller due to higher deceleration rate. Thus, yellow change interval decreases as approach speed increases at a given probability of stopping. It is noted, however, that the yellow response time for lower speeds involves response lag time that should be discounted.

ALL-RED INTERVAL

All-red time is often provided at some intersections to let vehicles clear the intersection during the protected time. All-red time is particularly useful when the intersection is wide and when many vehicles tend to enter the intersection during the latter part of the yellow interval.

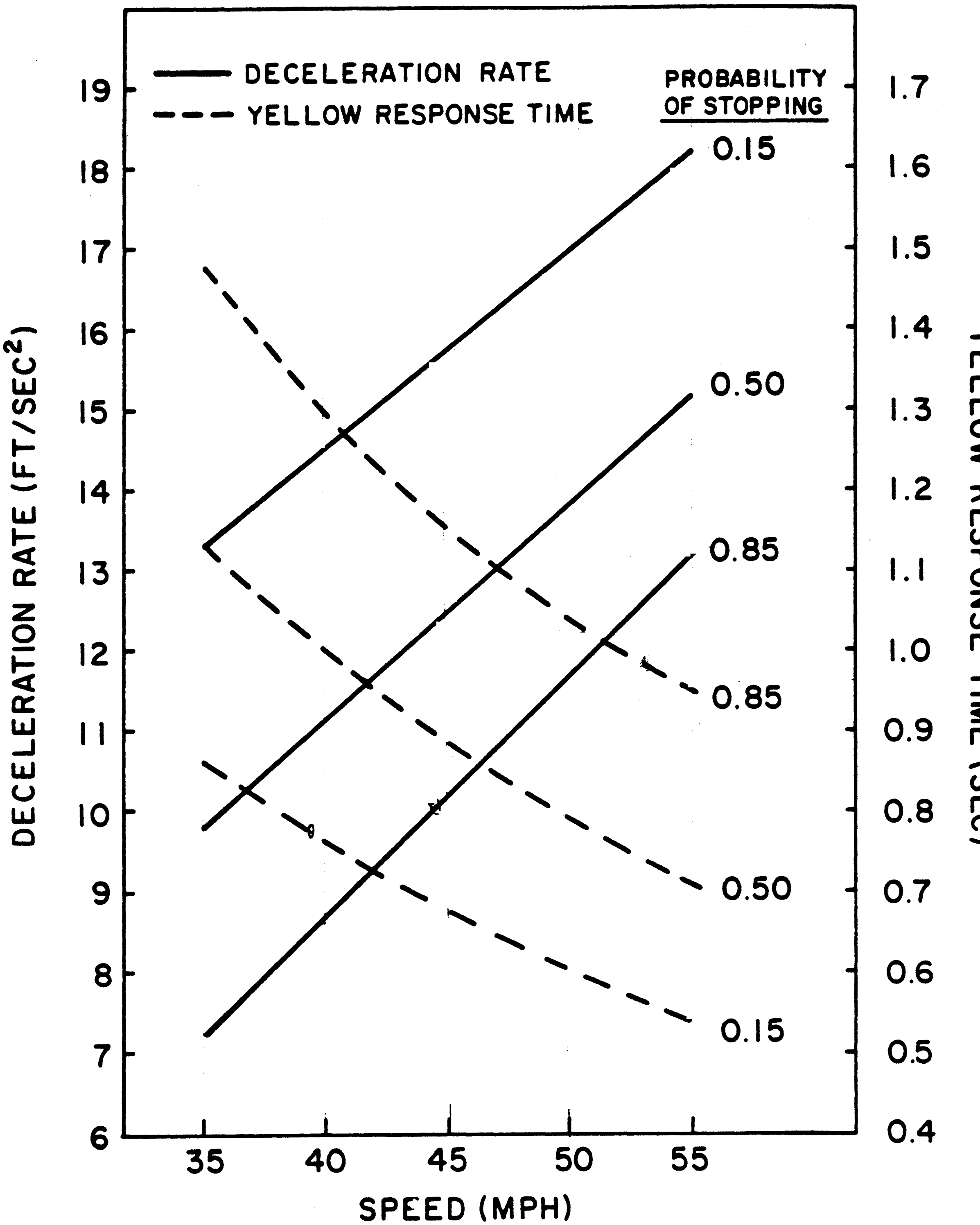


Figure 29. Relationship among probability of stopping, deceleration rate, and yellow response time by speeds.

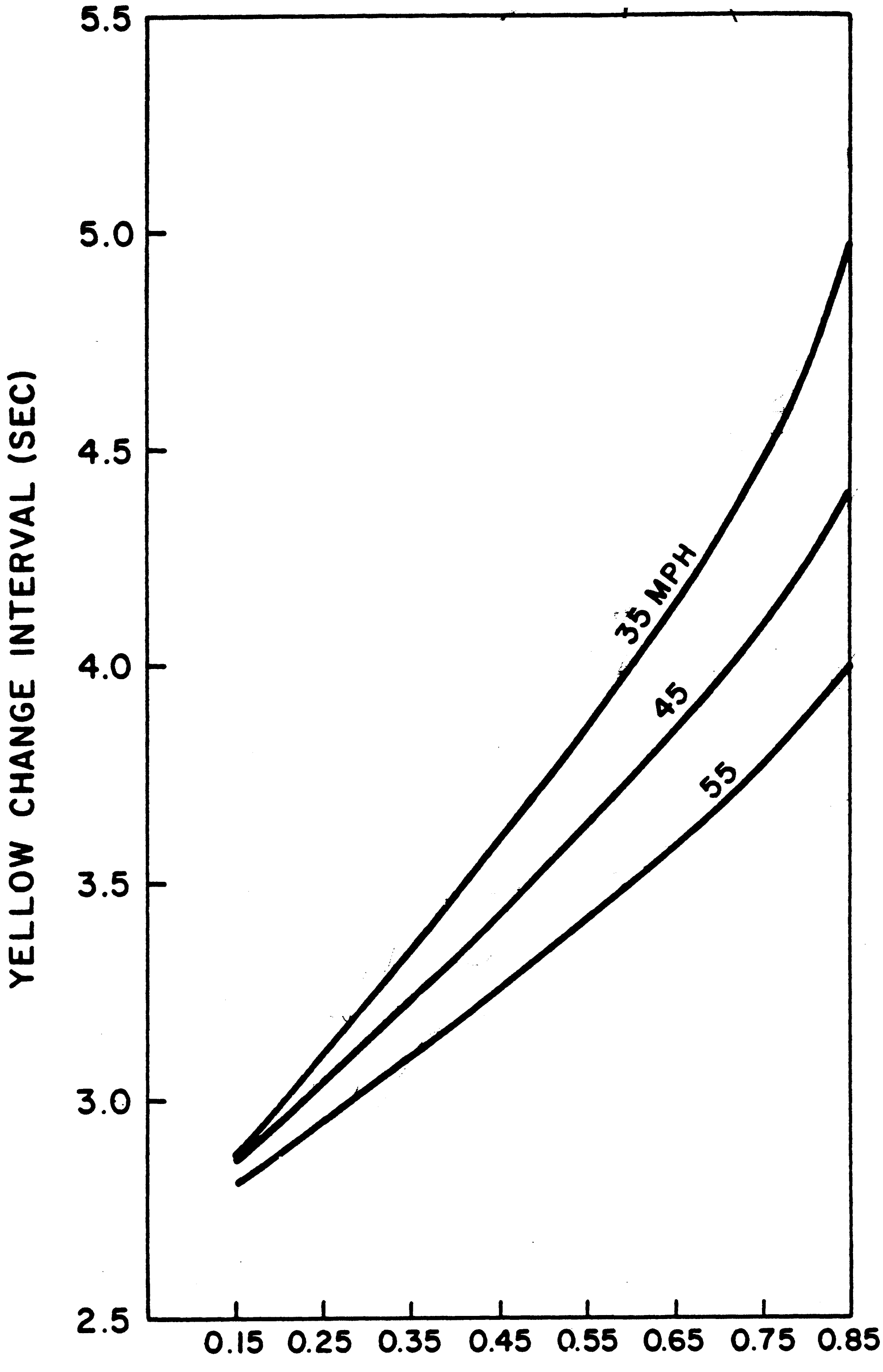


Figure 30. Relationship between probability of stopping and yellow change interval by speeds.

Speed Influence on All-Red Time

Current signal change interval design provides clearance time in the form of $(l+w)/v$. It is noted that a constant approach speed is used to derive clearance time. Observation of drivers revealed that the majority of drivers accelerate when they need to clear the intersection. It also appears to be a duty for those drivers who entered the latter part of yellow to clear the intersection as soon as possible within their vehicle's capabilities.

The speed difference between the approach speed before yellow (ASPEED) and the final average speed after yellow until the vehicle clears the intersection (FSPEED) was analyzed and the relationship obtained at $\alpha = 0.01$ is as follows:

$$\text{FSPEED} = 1.08 \text{ ASPEED}, \quad R^2 = 0.99$$

This suggests that use of constant speed may provide unnecessarily long all-red time particularly when the intersection is wide.

Starting Delay Influence on All-Red Time

When a driver first sees green onset, it takes time to start from a stopped position. Starting delay obtained from this study was analyzed.

The starting characteristics of a total of 3,527 vehicles being the first positioned in queue were observed at the onset of green. The cumulative distribution of starting delay observed for 3,527 vehicles is shown in figure 31. Twenty-seven vehicles started before green onset (this phenomenon is called "light jumping"), which was 0.8 percent of the total samples. It is noted, however, that light jumping is an illegal violation of traffic signal display. The mean starting delay was 1.8 seconds and the median was 1.7 seconds. Eighty-five percent of vehicles took more than 1 second to start. Ninety-five percent of vehicles took more than 0.8 seconds to start. Since the stop line is set back from the path of cross traffic, the use of one second starting delay may be applicable to 95 percent of vehicles.

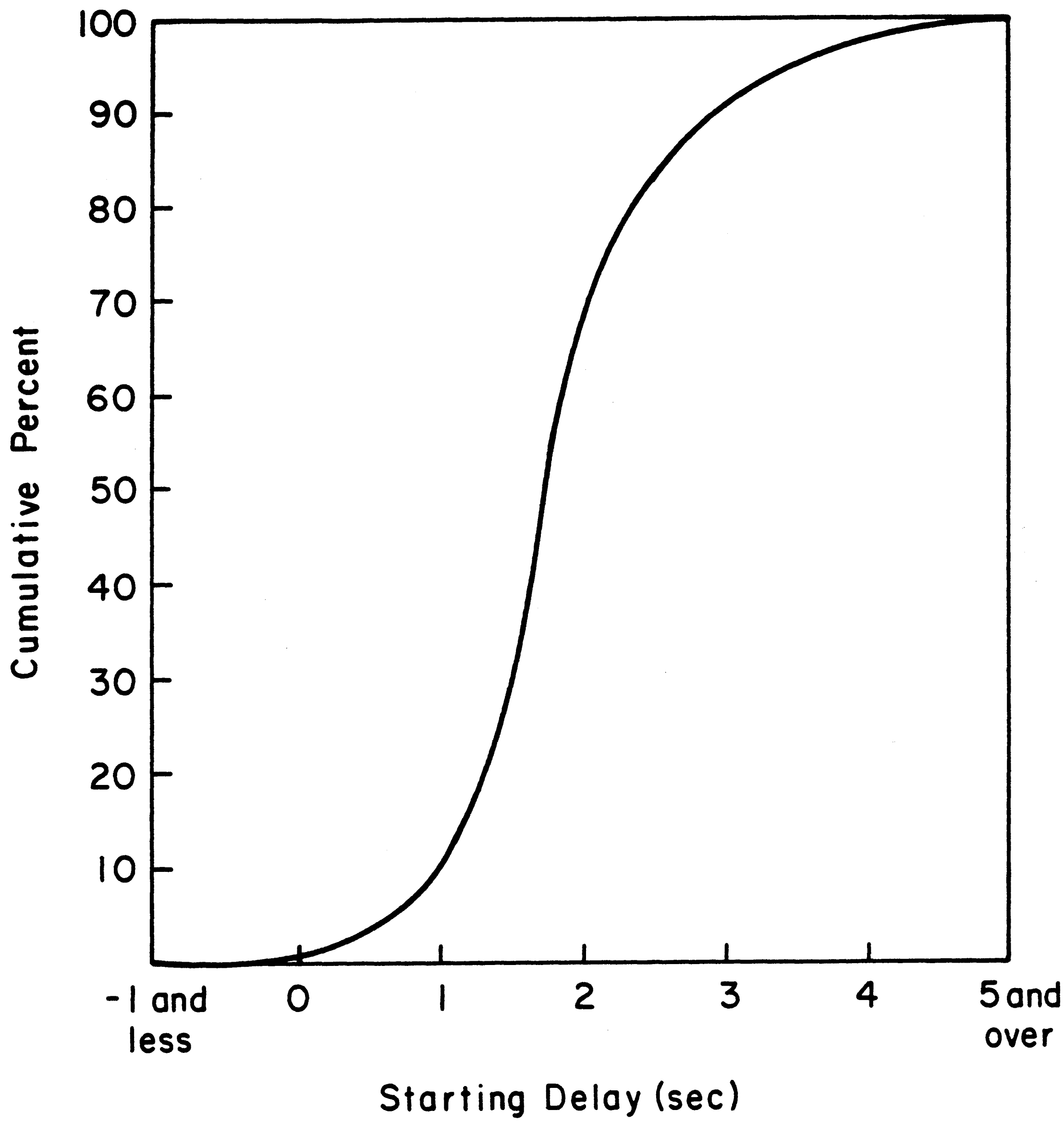


Figure 31. Cumulative distribution of starting delay.

VII. IMPLICATIONS AND APPLICATIONS

The previous analyses and findings revealed some implications on the establishment of change interval design practice. There appears to be several alternative concepts applicable for designing the change interval. These concepts will be illustrated and the corresponding results will be presented. Further, an example to determine all-red interval is presented. In addition, the application of findings to signal lost time is presented.

ALTERNATIVE METHODS FOR YELLOW INTERVAL DESIGN

The current formula shown as equation 1 uses one perception and brake reaction time and one deceleration rate for all approach speeds. However, driver behavior to the yellow signal revealed that this may not be the case. Further, the current formula is based on the stopping vehicles. However, the safety concern to the yellow signal appears to be more critical regarding the clearing vehicles rather than stopping vehicles. In addition, the probability of stopping or clearing may be used for establishing the change interval. These developments may be summarized as the following four alternative methods for designing change intervals.

- Method 1A: Continued use of current formula with one perception brake reaction time and one deceleration rate for all approach speeds.
- Method 1B: Continued use of current formula with different perception brake reaction times and deceleration rates for different approach speeds.
- Method 2: Design change interval based on clearing vehicles.
- Method 3: Design change interval based on the probability of stopping or clearing.

Method 1A

This method is the continued use of the current formula of equation 1 with one perception brake reaction time and one deceleration rate for all approach speeds. The current study reveals that 1.2 seconds of perception brake reaction time and 10.5 ft/s^2 deceleration rate at level grade appear to be good estimators. The advantage and the disadvantage of this method may be described as follows.

Advantage: The method is simple to use and straightforward. The method is based on driver populations independent of speed.

Disadvantage: The method does not explicitly reflect driver behavior which varies somewhat with approach speed.

Signal yellow interval based on this method using 1.2 seconds of perception brake reaction time and 10.5 ft/s^2 deceleration rate are presented below as compared to present ITE guidelines:[9]

	Approach Speeds (mi/h)						
	25	30	35	40	45	50	55
Signal Yellow Time t = 1.2 sec d = 10.5 ft/s^2	3.0	3.3	3.6	4.0	4.3	4.7	5.0
ITE Values t = 1.0 sec d = 10.0 ft/s^2	2.8[a]	3.2	3.6	3.9	4.3	4.7	5.0

a: Mimimum value considered safe by ITE guidelines is 3.0 seconds although the calculation value is 2.8 seconds.

Method 1B

This method is the continued use of the current formula but with different perception-brake reaction times and deceleration rates for different approach speeds. The current study revealed that drivers react quicker and decelerate harder (higher d) for higher approach speeds. The current study indicates that the upper bound of 1.5 seconds and the lower bound of 1.0 seconds appear to be good estimators for perception and brake reaction time. Further, the current study along with past studies indicates that the upper bound of 10.5 ft/s^2 and the lower bound of 8.0 ft/s^2 appear to be good estimators for deceleration rate. The advantage and the disadvantage of this method may be described as follows.

Advantage: This method reflects the driver response characteristics to the yellow signal in the real world.

Disadvantage: This method involves the proper determination of perception-brake reaction times and deceleration rates for different approach speeds, which may involve eventual engineering judgement and subsequent subjectiveness.

The signal yellow interval based on this method using the upper and lower boundary values mentioned above and the interpolation between them is presented below.

	Approach Speeds (mi/h)						
	25	30	35	40	45	50	55
Perception-Brake Reaction Time	1.5	1.4	1.3	1.2	1.1	1.0	1.0
Deceleration Rate	8.0	8.5	9.0	9.5	10.0	10.5	10.5
Signal Yellow Time	3.8	4.0	4.2	4.3	4.4	4.5	4.8

Method 2

This method is to design the yellow interval based on clearing vehicles. The current formula for signal yellow interval design is based on stopping vehicles. However, evaluation of the phenomenon surrounding the safety implications regarding the change interval suggests that the real safety concern is applicable to clearing vehicles instead of stopping vehicles. The advantage and the disadvantage of this method may be described as follows:

Advantage: The method is based on the driver response characteristics of clearing vehicles. Thus, the method may be more appealing to the objective consideration of safety concerns (particularly, angle accidents) surrounding change interval design.

Disadvantage: The method is a major change from the current formula in use for over 20 years.

The signal yellow interval based on this method using the current study results is presented below.

	Approach Speeds (mi/h)						
	25	30	35	40	45	50	55
85% of Clearing Vehicles	4.0	4.0	4.0	4.0	4.0	4.0	4.0
95% of Clearing Vehicles	4.5	4.5	4.5	4.5	4.5	4.5	4.5

This method is particulary noteworthy since it may favor a uniform yellow interval throughout the range of approach speeds.

Method 3

This method would design the yellow interval based on the probability of stopping of a vehicle facing the yellow signal. Three past studies[15,16,40] presented models or graphs to illustrate the probability of a vehicle stopping from a known distance and approach speed. The current study also developed models to explain the probability of a vehicle stopping given distance and approach speed. The advantage and the disadvantage of this method may be described as follows:

Advantage: The method is probabilistic which can be easily converted into a policy level of stopping probability (say 85 percent)

Disadvantage: Yellow response time is overestimated for lower speeds while it is underestimated for higher speeds. Similarly, reverse problem exists for deceleration rate estimation (See figure 29).

The signal yellow interval based on this method using the current study results is presented below:

Probability of Stopping	Approach Speeds (mi/h)				
	35	40	45	50	55
85%	5.0	4.7	4.4	4.2	4.0

SUMMARY OF ALTERNATIVE METHODS

Table 8 summarizes the yellow interval results obtained from alternative methods discussed. Several trends appear to be realized from table 8 as

TABLE 8. Summary of yellow interval from alternative methods.

Alternative Methods	Approach Speeds (mi/h)						
	25	30	35	40	45	50	55
1A	3.0	3.3	3.6	4.0	4.3	4.7	5.0
1B	3.8	4.0	4.2	4.3	4.4	4.5	4.8
2							
85% Clearing	4.0	4.0	4.0	4.0	4.0	4.0	4.0
95% Clearing	4.5	4.5	4.5	4.5	4.5	4.5	4.5
3 Probability of Stopping 85%	---	---	5.0	4.7	4.5	4.2	4.0
ITE Values	2.8[a]	3.2	3.6	3.9	4.3	4.7	5.0

a: Minimum value considered safe by ITE guideline is 3.0 seconds although the calculation value is 2.8 seconds.

follows:

- Method 1A appears to have insufficient yellow interval in lower speed categories compared to method 2 (based on clearing vehicles) and method 3 (based on the probability of stopping of a vehicle.)
- Method 2 appears to favor a uniform yellow interval across speed ranges which is a change from current practice.
- Method 3 appears to have problem in over or underestimation of yellow response time and deceleration rate for different speeds.
- Method 1B appears to reinforce lower speed categories than method 2 and can be a reasonable compromise between current practice and drivers' responses. The small difference of 1 second from 25 mi/h to 55 mi/h may play a role toward potential uniform yellow interval in the future.

EXAMPLE TO DETERMINE ALL-RED INTERVAL

Consider the following example of determining the duration of the all-red interval. The following intersection characteristics will be assumed:

Speed limit or 85 percentile speed = 40 mi/h = 58.7 ft/s
Intersection width = 100 ft
Passenger car length = 20 ft
Yellow time = 4 seconds

Step 1. Calculate distance traveled during the yellow interval.
Distance = 58.7 x 4 = 235 ft

Step 2. Add intersection width and the passenger car length.
Distance = 235 + 100 + 20 = 355 ft

Step 3. Divide step 2 distance by FSPEED.
Time = 355/(58.7 x 1.08) = 5.6 seconds

Step 4. Subtract the starting delay from step 3 time.
Time = 5.6 - 1.0 = 4.6 seconds

Step 5. Subtract the yellow time from Step 4.
All-red interval = 4.6 - 4.0 = 0.6 seconds

If the value obtained in Step 5 is negative, no all-red time is necessary. Therefore, this intersection would need 0.6 seconds of all-red time.

It is warned that the starting delay of 1 second should be applied with extreme caution and a value of zero should be used under the following conditions:

- Driver's view to the intersection is obstructed due to either intersection geometrics or adjacent large vehicles such as trucks.
- The crossing signal is visible or progression is provided such that approaching vehicles either do not tend to stop completely or do not take significant time to start.

Further, legal implications based on local laws and ordinances should be investegated before the engineer decides to apply the cross-flow reduction value.

CHANGE INTERVAL AND SIGNAL LOST TIME

Signal lost time is a parameter used for the calculation of cycle length and effective green time and consequently applied to signal timing and level of service at signalized intersections. The signal lost time is defined as

$$T_l = (Y+AR) - U_y + T_s$$

where

T_l = signal lost time, seconds
U_y = utilized yellow time, seconds
T_s = starting delay, seconds.

The current study revealed that the mean travel time for clearing vehicles was 2.6 seconds and the mean starting delay was 1.8 seconds. Therefore, mean signal lost time will be

$$T_l = (Y+AR) - 2.6 + 1.8 = Y+AR - 0.8$$

Taking a conservative value, the signal lost time corresponds to (Y+AR) -1 second.

VIII. CONCLUSIONS

The following conclusions were drawn from the data collected and field observations made within this study. They apply within the seven intersections studied and operational environments observed.

(1) The observed mean yellow response time selected by drivers was 1.3 seconds and the median was 1.1 seconds. Eighty-five percent of stopping drivers applied their brakes within 1.9 seconds after yellow onset while 95 percent of drivers did it within 2.5 seconds.

(2) The derived 85 percent perception and brake reaction time excluding driver's response lag time from yellow response time was 1.2 seconds.

(3) Driver's yellow response time was affected by distance to the intersection at yellow onset, approach speed, and the time available to reach the stop line after yellow onset.

(4) The observed mean deceleration rate selected by drivers was 9.5 ft/s^2, and the median was 9.2 ft/s^2.

(5) Grade affects deceleration rate about 0.075 ft/s^2 for each percentage of grade. For safety and practical purposes, a deceleration rate of 10.5 ft/s^2 is suggested for level and upgrades, and 10.0 ft/s^2 is suggested for downgrade.

(6) Driver's selected deceleration rate was affected by approach speed, the distance to intersection at yellow onset, the time available to reach the stop line after yellow onset, and the distance traveled during the yellow response time.

(7) Eighty-five percent of stopping vehicles stopped when they were more than 3 seconds away from the intersection.

(8) Eighty-five percent of going vehicles went through the intersection when they were less than 3.7 seconds away. Ninety-five percent of going vehicles continued when their actual travel times to the intersection were less than 4.5 seconds.

(9) The safety implication of going vehicles and the stability of going vehicles with respect to time suggest the potential use of a constant yellow interval of 4.5 seconds across all speeds.

(10) Driver's probability of stopping or going was affected by approach

speed and the distance to the intersection at yellow onset.

(11) A higher risk of accidents due to the yellow interval appears to exist for the lower speed categories below 40 mi/h.

(12) The average speed for going vehicles from the yellow onset to clearing the intersection is 8 percent higher on the average than their approach speeds before the yellow onset.

(13) The mean starting delay to green onset was 1.8 seconds and the median was 1.7 seconds. Eighty-five percent of the vehicles took more than 1 second to start. One second of starting delay may be applicable to determine all-red time.

IX. REFERENCES

1. Messer, C. J. and Chang, M. S., "Engineering Factors Affecting Traffic Signal Yellow Time: Data Collection Manual." Report No. FHWA-RD-85-055, Federal Highway Administration, Washington, D.C., 1985.
2. "Uniform Vehicle Code and Model Traffic Ordinances Revised 1968." National Committee on Uniform Traffic Laws and Ordinances, The Michie Company, Charlottesville, Virginia, 1968.
3. Benioff, B., Dock, F. C., and Carson C., "A Study of Clearance Intervals, Flashing Operation, and Left-Turn Phasing at Traffic Signals, Vol. II, Clearance Intervals." Report No. FHWA-RD-78-47, Federal Highway Administration, Washington, D.C., 1978.
4. Bissell, H. H. and Warren, D. L., "The Yellow Signal is Not a Clearance Signal." ITE Journal, Feb. 1981, pp. 14-17.
5. 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, Nov. 1980, pp. 22-29.
6. Gazis, D., Herman, R., and Maradudin, A., "The Problem of the Amber Signal in Traffic Flow." Traffic Engineering, Jul. 1960, pp. 19-26 and 53.
7. Parsonson, P. S., "Discussion of Paper on Evaluation of Driver Behavior at Signalized Intersections by Wortman, R. H. and Matthias, J. S." Transportation Reseach Record 904, 1983, pp. 16-18.
8. "A Policy on Geometric Design of Rural Highways." American Association of State Highway Officials (AASHO), Washington, D.C., 1965.
9. "Traffic and Transportation Handbook." Institute of Traffic and Transportation Engineers. Washington, D. C., 1982.
10. Parsonson, P. S., and Santiago, A., "Design Standards for Timing the Traffic-Signal Clearance Period Must be Improved to Avoid Liability." In Compendium of Technical Papers, Institute of Traffic Engineers, 1980, pp. 67-71.
11. Taoka, G. T., "System Identification of Safe Stopping Distance Parameters." Department of Civil Engineering, University of Hawaii at Manoa, Sept. 1980.
12. Hooper K. G. and McGee, H. W., "Driver Perception Reaction Time: Are

Revisions to Current Specification Values in Order?" Transportation Research Record 904, 1983, pp. 21-30.

13. Jenkins, R. J., "A Study of Selection of Yellow Clearance Intervals for Traffic Signals." Report TSD-TR-104-69, Traffic and Safety Division, Michigan Department of State Highways, 1969.
14. Wortman, R. H. and Matthias, J. S., "Evaluation of Driver Behavior at Signalized Intersections." Transportation Research Record 904, 1983, pp. 10-20.
15. Williams, W. L., "Driver Behavior During the Yellow Interval." Transportation Research Record 644, 1977, pp. 75-78.
16. Sheffi, Y. and Mahmassani, H., "A Model of Driver Behavior at High Speed Signalized Intersections." Transportation Science, Vol. 15, No. 1, 1981, pp. 50-61.
17. Chang, M. S., Messer, C. J., and Santiago, A., "Evaluation of Engineering Factors Affecting Traffic Signal Change Interval." Transportation Research Record 956, 1984, pp. 18-21.
18. "Traffic and Transportation Handbook." Institute of Traffic and Transportation Engineers, Arlington, VA, 1976.
19. Olson, P. L. and Rothery, R. W., "Deceleration Levels and Clearance Times Associated with Amber Phase of Traffic Signals." Traffic Engineering, Apr. 1972, pp. 16-19 and pp. 62-63.
20. "Traffic Control Devices Handbook." Federal Highway Administration, 1983.
21. Butler, J. A., "Another View on Vehicle Change Intervals." ITE Journal, Mar. 1983, pp. 44-48.
22. Olson, P. L. and Rothery, R. W., "Driver Response to Amber Phase of Traffic Signals." Traffic Engineering, Feb. 1962, pp. 17-21 and p. 29.
23. May, A. D. Jr., "Clearance Interval at Traffic Signals." Highway Research Record 221, 1968, pp. 41-71.
24. McGill, W. A., "Optimizing the Use of Amber in Traffic Signals." Proceedings of the Australian Road Research Board, Vol. 5, Part 3, 1970, pp. 95-110.
25. Ryan, T. A. and Davis, C. F., "Driver Use of All-Red Signal Interval." Transportation Research Record 881, pp. 9-16.

REFERENCESS (Continued)

26. Knoflacher, H., "The Relationship Between Amber Phase Duration and Accident Frequency at Signal Controlled Intersections." Feitschrift Fur Verkenhrsrecht 16, Sept. 1971, pp. 252-256.
27. Newby, R. F., "Accident Frequency at Signal-Controlled Crossroads with All-Red Period." Traffic Engineering and Control, 1961, pp. 102-103.
28. "Red Overlap Study." Bureau of Traffic Engineering, City of Portland, Sept. 1965.
29. Conradson, B. and Bunker, L., "Evaluation of an Operational Change at 17 Locations: Addition of an All-Red Clearance Interval to the Traffic Signal Timing Sequence." Report TSD-G-208-72, Traffic and Safety Division, Michigan Department of State Highways, 1972.
30. "All-Red." Staff Report, Department of Traffic, City of Los Angeles, 1973.
31. Hoppe, G. H., "The Effect of All-Red Signal Phasing on Traffic Accidents." 1977.
32. Sivak, M., Post, D. V., Olson, P. L., and Donohue, R. J., "Driver Responses to High-Mounted Brake Lights in Actual Traffic." Human Factors, Apr. 1981. pp. 231-235.
33. Hallert, B., "Photogrammetry, Basic Principles and General Survey." McGraw-Hill, New York, 1960.
34. Bleyl, R. L., "Traffic Analysis of Time Lapse Photographs Without Employing a Perspective Grid." Traffic Engineering, Aug. 1972, pp. 29-31.
35. "SAS User's Guide: Basics and Statistics." SAS Institute, Inc., Cary, North Carolina, 1982.
36. Finney, D. J., "Probit Analysis." Cambridge University Press, 1971.
37. Neter, J. and Wasserman, W., "Applied Linear Statistical Models." R. D. Iywin Inc., Homewood, Illinois, 1974.
38. "SUGI Supplemental Library User's Guide." 1983 Edition. SAS Institute, Inc., Cary, North Carolina, 1983.
39. Winter, P. A., "1974 Brake Performance Levels for Truck and Passenger Cars." Public Roads, Vol. 40, No. 3, 1976, pp. 108-115.
40. Zegeer, C. V., "Effectiveness of Green-Extension Systems at High Speed Intersections." Research Report No. 472, Kentucky Department of Transportation, May 1977.

Appendix A. Computer program to convert roadway coordinates to film coordinates.

```
//CORDNATE JOB (R653.001A.S02.005.MS),'M S CHANG'
//*MAIN        USER=R653$MS,ORG=XEROX
//*TAMU        HOLDOUT
//*PASSWORD***************************************************************
//*MAIN ORG=XEROX
//*XBM WATFIV
C*********************************************************
C
C   PURPOSE:
C         TO CONVERT ROADWAY COORDINATES INTO FILM COORDINATES
C         SO THAT ROADWAY DISTANCE MAP CAN BE DRAWN AND THE PROJECTED
C         FILM DATA CAN BE DIRECTLY CONVERTED INTO ROADWAY DATA.
C         THE PROGRAM IS USED TO DRAW FIVE FOOT INTERVAL MAP OF
C         ROADWAY DISTANCE.
C
C   USAGE:
C       USING 4 REFERENCE POINTS ON ROADWAY PLANE AND THEIR
C       CORRESPONDING 4 REFERENCE POINTS ON THE FILM PLANE,
C       THE PROGRAM FINDS 8 COEFFICIENTS FOR THE NUMERICAL
C       EQUATIONS BETWEEN THE TWO PLANES.
C       USING THE EQUATIONS, THE PROGRAM CONVERTS ANY DATA POINTS
C       OR DISTANCES IN THE ROADWAY PLANE INTO THAT OF FILM PLANE.
C
C   DEFINITION OF VARIABLES:
C       B(I):ARRAY RELATED WITH ITH COEFFICIENT IN CONVERSION
C            EQUATION.
C       XR(I);X COORDINATE OF ITH REFERENCE POINT IN ROADWAY.
C       YR(I);Y COORDINATE OF ITH REFERENCE POINT IN ROADWAY.
C       XF(I);X COORDINATE OF THE CORRESPONDING POINT IN FILM PLANE.
C       YF(I);Y COORDINATE OF THE CORRESPONDING POINT IN FILM PLANE.
C       XROAD;X COORDINATE OF EACH DATA POINT ON ROADWAY.
C       YROAD;Y COORDINATE OF EACH DATA POINT ON ROADWAY.
C       XFILM;X COORDINATE OF EACH CORRESPONDING POINT IN FILM.
C       YFILM;Y COORDINATE OF EACH CORRESPONDING POINT IN FILM.
C
C  DEFINITION OF SUBROUTINE;
C       SUBROUTINE CALIBR; SUBROUTINE FOR CALCULATING B(I),
C                          COEFFICIENTS FOR THE CONVERSION
C                          EQUATION USING REFERENCE POINTS.
C       SUBROUTINE COORD; SUBROUTINE TO CONVERT ROADWAY DATA INTO
C                         CORRESPONDING FILM DATA USING THE COMPLETED
C                         EQUATIONS.
C************************************************************
C
C
      DIMENSION A(8,8),B(8)
      DIMENSION XR(4),YR(4),XF(4),YF(4)
C
C   READ 4 REFERENCE POINTS IN BOTH ROADWAY AND FILM.
       DO 10 I=1,4
       READ (5,20) XR(I), YR(I), XF(I), YF(I)
 10    CONTINUE
 20    FORMAT(4(F10.2))
C
C   PRINT OUT REFERENCE POINTS IN BOTH PLANES
       WRITE(6,17)
 17    FORMAT(////,53X,'*4 REFERENCE POINTS*',//50X,'ROAD',16X,'FILM
     *',//,47X,'XR',8X,'YR',8X,'XF',8X,'YF',//)
       DO 30 J=1,4
```

```
          WRITE(6,40) XR(J),YR(J),XF(J),YF(J)
 30       CONTINUE
 40       FORMAT(41X,4(F10.2))
C
C    CALCULATE THE COEFFICIENTS FOR CONVERSION EQUATION.
        CALL CALIBR(XR,YR,XF,YF)
        WRITE(6,41)
   41   FORMAT(/////,42X,'*ROADWAY DATA AND THE CONVERTED FILM DATA*')
C
C    READ  X AND Y COORDINATES OF ROADWAY DATA POINTS
 11     READ(5,90,END=999) X,Y
 90     FORMAT(2(F10.4))
C
C    STORE X,Y COORDINATES OF ROADWAY DATA
        XROAD = X
        YROAD = Y
C
C    CONVERT ROADWAY DATA INTO FILM DATA
        CALL COORD(X,Y)
        XFILM=X
        YFILM=Y
C
C    PRINT OUT COORDINATES OF BOTH ROADWAY AND FILM
        WRITE(6,200) XROAD,YROAD,XFILM,YFILM
 200    FORMAT(//,17X,'XROAD = ',F8.3,5X,'YROAD = ',F8.3,10X
      *,'XFILM = ',F8.3,5X,'YFILM =',F8.3)
        GO TO 11
 999    CONTINUE
        STOP
        END
C
C


        SUBROUTINE CALIBR(XR,YR,XF,YF)
C
C    ESTABLISHES RELATIONSHIP BETWEEN TWO RECTANGULAR
C    COORDINATE SYSTEMS IN DIFFERENT PLANES
C
        DIMENSION A(8,8),B(8)
        DIMENSION XR(4),YR(4),XF(4),YF(4)
C
C    SET UP 8X8 MATRIX A AND 1X8 MATRIX B
C
        DO 10 I = 1,4
        A(I,1) = 1.0
        A(I,2) = XR(I)
        A(I,3) = YR(I)
        A(I,4) = -XR(I) * XF(I)
        A(I,5) = -YR(I) * XF(I)
        A(I,6) = 0.0
        A(I,7) = 0.0
        A(I,8) = 0.0
        B(I) = XF(I)
 10     CONTINUE
C
C
        DO 20 I = 5,8
        J = I-4
        A(I,1) = 0.0
        A(I,2) = 0.0
```

```
      A(I,3) = 0.0
      A(I,4) = -XR(J)*YF(J)
      A(I,5) = -YR(J) * YF(J)
      A(I,6) = 1.0
      A(I,7) = XR(J)
      A(I,8) = YR(J)
      B(I) = YF(J)
20    CONTINUE
C
C   SOLVE EIGHT SIMULTANEOUS EQUATIONS AX=B
       DO 120 I=1,8
30    F = A(I,I)
      IF (F .NE. 0.0) GO TO 80
      J=I
40    J=J+1
      IF (J .GE. 9) GOTO 60
      IF (A(J,I) .EQ. 0.0) GOTO 40
C
C    ALTERNATE FOUND.  SWITCH ROWS
      DO 50 L=1,8
      TEMP = A(I,L)
      A(I,L) = A(J,L)
      A(J,L) = TEMP
50    CONTINUE
C
      TEMP = B(I)
      B(I) = B(J)
      B(J) = TEMP
      GO TO 30
C
C    NO ALTERNATE CAN BE FOUND
60    WRITE(6,70)
70    FORMAT('1 MATRIX HAS NO SOLUTION ')
      STOP
C.
C    DIVIDE ROW TO GET UNITY COEFFICIENT
80    DO 90 J=1,8
      A(I,J) = A(I,J)/F
90    CONTINUE
      B(I) = B(I)/F
C
C    ZERO REMAINDER OFCOLUMN
      DO 110 J=1,8
      IF (J .EQ. I) GO TO 110
      F = A(J,I)
      IF (F .EQ. 0.0) GO TO 110
      DO 100 K=I,8
      A(J,K) = A(J,K) - F*A(I,K)
100   CONTINUE
      B(J) = B(J) - F*B(I)
110   CONTINUE
120   CONTINUE
C
C   SOLUTION OF 8 SIMULATANEOUS EQUATIONS COMPLETED AND
C   PRINT OUT THE COEFFICIENTS OF THE CONVERSION EQUATIONS
      WRITE(6,38)
  38  FORMAT(//,42X,'*COEFFICIENTS OF THE CONVERSION EQUATION*',/)
      DO 55 I=1,8
      WRITE(6,39)I,B(I)
  39  FORMAT(/,55X,'B(',I1,')',1X,'=',F10.5)
```

```
   55 CONTINUE
      RETURN
C
C   CALCULATE FILM COORDINATES GIVEN ROADWAY COORDINATES
      ENTRY COORD(X,Y)
      XX = (B(1) + B(2)*X + B(3)*Y)/(B(4)*X+B(5)*Y+1.0)
      Y = (B(6)+B(7)*X+B(8)*Y) / (B(4)*X+B(5)*Y+1.0)
      X = XX
      RETURN
      END
```

//$DATA

4 REFERENCE POINTS

ROAD		FILM	
XR	YR	XF	YF
37.00	375.00	20.00	30.00
0.00	375.00	260.00	14.00
14.00	215.00	256.00	70.00
63.00	215.00	121.00	77.00

COEFFICIENTS OF THE CONVERSION EQUATION

B(1) = 307.89810

B(2) = -1.50978

B(3) = -0.65876

B(4) = 0.00043

B(5) = -0.00204

B(6) = 84.85852

B(7) = 0.11414

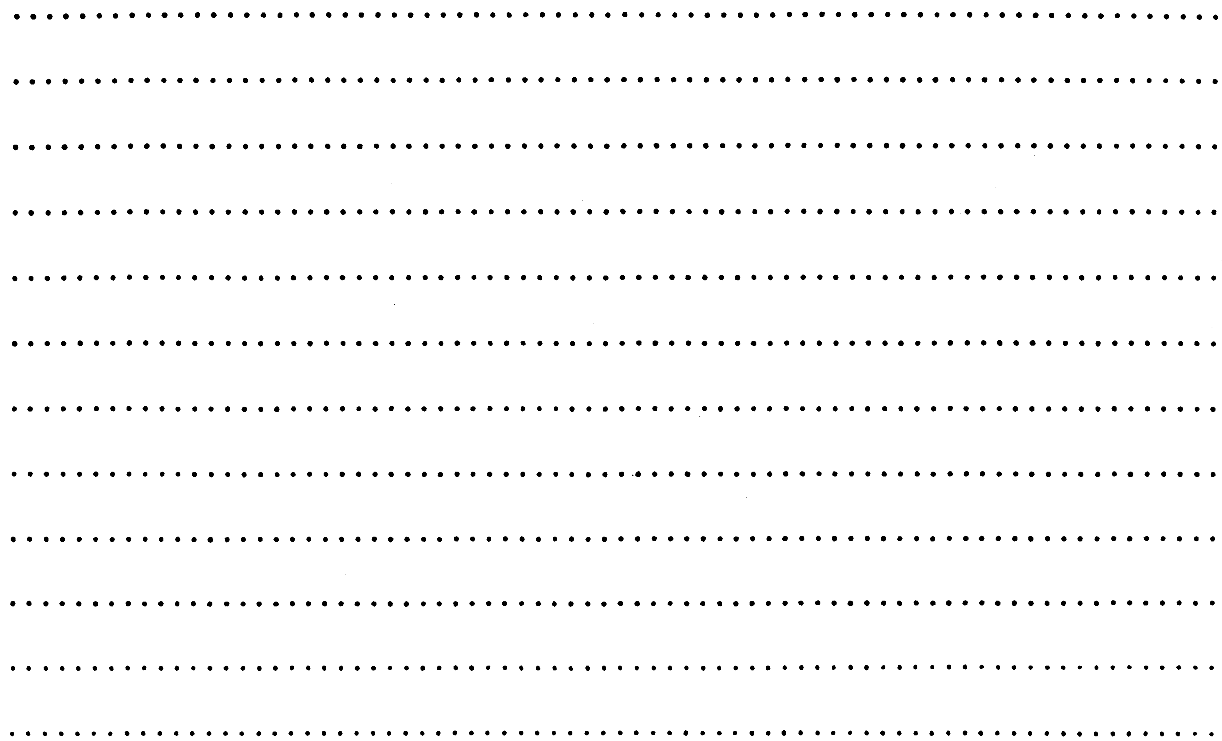
B(8) = -0.21755

ROADWAY DATA AND THE CONVERTED FILM DATA

XROAD = 0.000 YROAD = -110.000 XFILM = 310.584 YFILM = 88.831

XROAD = 22.000 YROAD = -110.000 XFILM = 281.286 YFILM = 90.184

XROAD =	42.000	YROAD =	-110.000	XFILM =	255.040	YFILM =	91.396
XROAD =	0.000	YROAD =	-90.000	XFILM =	310.171	YFILM =	88.221
XROAD =	22.000	YROAD =	-90.000	XFILM =	279.874	YFILM =	89.625
XROAD =	42.000	YROAD =	-90.000	XFILM =	252.746	YFILM =	90.883
XROAD =	0.000	YROAD =	0.000	XFILM =	307.898	YFILM =	84.859
XROAD =	22.000	YROAD =	0.000	XFILM =	272.105	YFILM =	86.550
XROAD =	42.000	YROAD =	0.000	XFILM =	240.144	YFILM =	88.060
XROAD =	0.000	YROAD =	5.000	XFILM =	307.747	YFILM =	84.635
XROAD =	22.000	YROAD =	5.000	XFILM =	271.590	YFILM =	86.346
XROAD =	42.000	YROAD =	5.000	XFILM =	239.310	YFILM =	87.873
XROAD =	0.000	YROAD =	10.000	XFILM =	307.593	YFILM =	84.407
XROAD =	22.000	YROAD =	10.000	XFILM =	271.064	YFILM =	86.137
XROAD =	42.000	YROAD =	10.000	XFILM =	238.458	YFILM =	87.682
XROAD =	0.000	YROAD =	15.000	XFILM =	307.435	YFILM =	84.174
XROAD =	22.000	YROAD =	15.000	XFILM =	270.527	YFILM =	85.925
XROAD =	42.000	YROAD =	15.000	XFILM =	237.588	YFILM =	87.487
XROAD =	0.000	YROAD =	20.000	XFILM =	307.275	YFILM =	83.936
XROAD =	22.000	YROAD =	20.000	XFILM =	269.978	YFILM =	85.708
XROAD =	42.000	YROAD =	20.000	XFILM =	236.700	YFILM =	87.289
XROAD =	0.000	YROAD =	25.000	XFILM =	307.110	YFILM =	83.693



XROAD = 42.000	YROAD = 390.000	XFILM = -56.109	YFILM = 21.703
XROAD = 0.000	YROAD = 395.000	XFILM = 246.780	YFILM = -5.557
XROAD = 22.000	YROAD = 395.000	XFILM = 71.391	YFILM = 7.091
XROAD = 42.000	YROAD = 395.000	XFILM = -74.407	YFILM = 17.604
XROAD = 0.000	YROAD = 400.000	XFILM = 242.553	YFILM = -11.810
XROAD = 22.000	YROAD = 400.000	XFILM = 58.067	YFILM = 1.816
XROAD = 42.000	YROAD = 400.000	XFILM = -94.564	YFILM = 13.090