NORTH CAROLINA

WAKE COUNTY

IN THE GENERAL COURT OF JUSTICE
SUPERIOR COURT DIVISION
10-CVS-19930

BRIAN CECCARELLI and LORI MILLETTE individually and as class representative, Plaintiff, v. TOWN OF CARY, Defendant.	PLAINTIFFS' RULE 59 MOTION FOR NEW TRIAL

Pursuant to Rule 59, "[a] new trial may be granted to all or any of the parties and on all or part of the issues for any of the following causes or grounds:

> (7) Insufficiency of the evidence to justify the verdict or that the verdict is contrary the law;
> (8) Error in law occurring at the trial and objected to by the party making the motion, or
> (9) Any other reason heretofore recognized as grounds for new trial."

The Court's judgment, entered March 4, 2013, is internally inconsistent with itself and/or with evidence at trial that was uncontradicted. Plaintiffs move for a new trial on the following grounds.

I. **A new trial should be granted to Plaintiff Ceccarelli because the judgment is contrary to the law.**

1. The Court found that "[b]oth Plaintiff Ceccarelli and other drivers traveling through the intersection of Cary Towne Boulevard and Convention Drive during the relevant time period could reasonably assume that the legal speed limit on that section of roadway was 45 mph." (Order, Finding of Fact ¶ 11.)

2. The Court found that: "While the duration of the yellow light change interval at the intersection of Cary Towne Boulevard and Convention Drive during the relevant time period ideally would have reflected 45 mph, the Court cannot conclude based upon the evidence submitted during the trial of this matter what the posted statutory speed limit was in 1991 – the time the applicable official signal plan of record was prepared." (See Order, finding of fact ¶ 12.)

3. However, a signal plan was prepared, signed and sealed by the North Carolina Department of Transportation on November 4, 2009. (See Plaintiff's Exhibit #30, Sheet 4.) This document had a plan date of September 2009. NCDOT actually knew the speed limit used for the "signal plan of record" was wrong, no later than September 2009. The Town of Cary also knew the "signal plan of record" was wrong before Ceccarelli and his class were cited.[1]

4. The November 4, 2009 Traffic Signal Plan of record (1) corrected the speed limit to 45 mph and (2) changed the yellow change interval from 4.0 seconds to 4.5 seconds. (See Plaintiff's Exhibit #30, Sheet 4.)

5. This November 4, 2009 plan was the considered judgment of NCDOT until March 19, 2010, when a new plan that "supersedes the plan signed and sealed on 11/4/09" was signed, sealed and actually placed in operation on the ground. (See Plaintiff's Exhibit #30, Sheet 5.) (Revisions to the November 4, 2009 plan did not pertain to the timing of the yellow change interval. (Spencer Dep., 18:18 – 19:13.)

[1] The Town of Cary received the updated signal plan dated November 4, 2009. Additionally, on November 30, 2009, David Spencer, Town of Cary engineer, sent an email to Laura Cove, Town of Cary Associate Director of Engineering, stating "I confirmed this with Ron Garrett so it's official that the signal plan was done with the incorrect speed limit." (See Spencer Dep., 22:5 – 19.) As of November 30, 2009, NCDOT authorized the Town of Cary to change the existing yellow timings in the controller based on the new November 4, 2009 plan. (See Plaintiffs' Exhibit #6, Page 3.)

6. The Town of Cary conceded, at closing argument, that if the speed limit at that intersection was 45 mph, then Plaintiff Ceccarelli was shorted 0.50 seconds on the yellow change interval as well as those class members who were cited between December 2, 2009 and March 19, 2010.
7. All of the evidence is that the Plaintiff Ceccarelli entered the intersection 0.38 seconds after the red clearance interval began. (See Plaintiffs' Exh. #1, Ceccarelli Aff., attached Exh. A.)
8. All of the evidence is that 50% of all the drivers at all the intersections in question entered a half second or less into the red clearance interval time. Because data was supplied to the Plaintiff by the Town of Cary on the last business day before trial, it is now possible to identify those drivers who entered each intersection 0.50 seconds or less after the red interval began. (See Ceccarelli Post-Trial Aff., ¶ 2.) At the intersection of Cary Towne Boulevard and Convention Drive between December 2, 2009 and March 19, 2010 40% of the red light violators entered 0.50 seconds or less after the red clearance interval began: 139 drivers ran the light in 0.50 seconds or less. (See Ceccarelli Post-Trial Aff., ¶ 3.) Out of the 139 drivers, 117 citations were paid in full. (See Ceccarelli Post-Trial Aff., ¶ 3.) The names and address of these 117 class members are available.
9. The Town's position is that the statutes have progressively changed to be more lenient to the Town. At least for those drivers, including Plaintiff Ceccarelli, who were cited before March 20, 2010, the issue is whether the Red Light Camera program was authorized under the law then prevailing. The legal issue is not whether Ceccarelli or any other driver could have stopped under some other law. The Town of Cary was under a statutory duty to ensure the yellow light duration was (1) equal to or greater than the

signed and sealed signal plan of record and (2) in full conformance with MUTCD. (N.C.S.L. Ch. 2004-141, Section 3.) Considering the signed and sealed November 4, 2009 plan, the answer is no. For the period through January 15, 2010 MUTCD read "[t]he duration of a yellow change interval shall be predetermined." (See Defendant's Exhibit F15, Page 4D-8, Section 4D.10.) It was certainly "predetermined," but that is tautological since all signal plans predetermine the yellow change interval. For the period after January 15, 2010 MUTCD read "[t]he duration of the yellow change interval shall be determined <u>using engineering practices</u> (emphasis added)." (See Plaintiff's Exhibit #14, Page 485, Section 4D.26.) No engineer would (or did) testify that engineers can practice their profession by using data that is known to be false.

II. <u>A new trial should be granted to Plaintiff Ceccarelli because calculations of whether one could stop were not going to be considered by the Court.</u>

1. During the hearing on Defendant's motion to exclude Plaintiffs' witnesses (particularly Hennings) the Court announced that it was not going to receive Marceau's calculation that Ceccarelli could have stopped, and therefore Hennings' rebuttal of that calculation was not used.
2. But the court, on its own initiative, elicited testimony at the very end of trial and found that: "Drivers at the intersection of Cary Towne Boulevard and Convention Drive during the relevant period of time could, even with the length of the yellow time being determined by using a 35 mph velocity versus a 45 mph velocity, nonetheless safely bring their vehicle to a stop as shown by the application of the laws of motion within the time and distance provided with only a slightly greater but still reasonably comfortable braking force." (See Order, finding of fact ¶ 16.)

3. However, the enabling statute required:

> The duration of the yellow light change interval at intersections where traffic control photographic systems are in use shall be no less than the yellow light change interval duration on the traffic signal plan of record signed and sealed by a licensed North Carolina Professional Engineer in accordance with Chapter 89C of the General Statues, and shall be in full conformance with the requirements of the Manual on Uniform Traffic Control Devices ["MUTCD"].

(N.C.S.L. Ch. 2004-141, Section 3.)

4. The MUTCD sets the following standards: The duration of the yellow change interval shall be determined using engineering practices and the yellow light duration is to be consistent with this determined value.[2] (MUTCD 2009 Rev 2, May 2012, Section 4D.26, ¶¶ 3, 8: page 485.) The yellow light change interval can only be determined using the actual speed limit, known to be 45 mph no later than September 2009 (if one is even thinking of using engineering practices or standards). The yellow change interval failed to comply with the MUTCD until it was corrected on March 19, 2010.

5. The Court found that Plaintiff Ceccarelli himself could have stopped his car on November 6, 2009. But this is irrelevant to the question whether the red light camera program was authorized at that intersection during that time period when NCDOT and the Town had actual knowledge that its signal plan of record was in error, and the facts on the ground did not comply with the last signed and sealed signal plan prepared by NCDOT.

[2] "Engineering practices for determining the duration of yellow change . . . intervals can be found in ITE's 'Traffic Control Devices Handbook' and in ITE's 'Manual on Traffic Signal Design…' (MUTCD, Section 4D.26, ¶7.)

6. Plaintiff did not introduce new evidence whether drivers could stop, nor whether 13.2 feet/second squared was a comfortable stopping distance, because such a determination was not relevant to the statutory requirement.

7. If Plaintiff had been informed by the Court, contrary to its earlier ruling, that the issue was whether 13.2 feet/second squared was a "comfortable stopping distance" then the Plaintiffs could have introduced the following additional evidence. Plaintiffs could have introduced a 1986 article about safe and comfortable deceleration rates – which makes clear that 12 -15 feet per second squared was considered too aggressive by traffic engineers:

> Considerable research has been conducted to assess driver behavior during the signal change intervals, and all of it has concluded that the majority of drivers will continue to enter the intersection if, based on their speed and distance, they would be forced to decelerate at a rate of 12-15 ft/sec^2 or faster. For most drivers, this would be an abrupt stop. The vast majority of drivers who can stop at a deceleration rate of 10 ft/sec^2 or less will do so. The current edition of the ITE Handbook reduced the recommended deceleration value from 15 ft/sec^2 to 10 ft/sec^2 in response to the evidence supporting the slower rate.

(See Transportation Quarterly, Vol. 40, No. 3, July 1986, *Traffic Signal Change Intervals: Policies, Practices, and Safety* written by Howard S. Stein, pages 440 – 441, attached as Exhibit A.) Plaintiffs could have quoted The Federal Highway Administration (FHWA) on comfortable deceleration rates.

> The deceleration rate of 10 ft./sec.2 suggested by ITE is based on a comfortable deceleration rate that has been supported by research. The 2001 American Association of State Highway and Transportation Official's A Policy on *Geometric Design of Highways and Streets*, otherwise known as the "Green Book," (46) recommends 11.2 ft./sec.2 for determining stopping sight distance. They note that this is a comfortable deceleration for most drivers. The deceleration rate suggested by ITE is a more conservative

deceleration rate for purposes of calculating the yellow interval and will result in longer intervals.

(See U.S. Department of Transportation, Federal Highway Administration, FHWA Safety, *Making Intersections Safer: A Toolbox of Engineering Countermeasures to Reduce Red-Light Running, An Informational Report*, Chapter 3 - Engineering Countermeasures, excerpts attached as Exhibit B [Page 17 of 22] and available at http://safety.fhwa.dot.gov/intersection/redlight/cameras/rlr_report/chap3.cfm.)

8. Plaintiffs could have reminded the Court of the following evidence already admitted: (a) the MUTCD itself uses 11.2 ft/s/s for placement of warning signs[3] (See Plaintiff's Exhibit # 14, Section 2C.05., ¶ 03); (b) "a 'comfortable' deceleration of 1/3 *g*" (approximately 10.7 feet/seconds^2) was cited by The Problem of the Amber Signal Light in Traffic Flow, by Denos Gazis et al. (Plaintiff's Exhibit #28, page 130, attached as Exhibit C); and (c) "[i]ncreased perception reaction time is needed to allow time for drivers to make the proper decision when information conflicts and driver expectancy may be in error" as addressed in Stopping Sight Distance and Decision Sight Distance prepared for Oregon Department of Transportation (Plaintiff's Exhibit #11, page 11, attached as Exhibit D).

III. A new trial should be granted to Plaintiff Millette because MUTCD requires the duration of a yellow change interval shall not vary on a cycle-by-cycle basis within the same signal timing plan.

1. "The duration of a yellow change interval shall not vary on a cycle-by-cycle basis within the same signal timing plan." (MUTCD 2009, 4D.26, ¶ 09 Standard.)

[3] "Typical condition is the warning of a potential stop situation. Typical signs are Stop Ahead, Yield Ahead, Signal Ahead, and Intersection Warning signs. The distances are based on the 2005 AASHTO Policy, Exhibit 3-1, Stopping Sight Distance, providing a PRT of 2.5 seconds, a deceleration rate of 11.2 feet/second/second, minus the sign legibility distance of 180 feet."

2. The length of the steady 'circular yellow' or 'yellow arrow' for the vehicular traffic facing the indication, cannot vary from cycle-to-cycle. (See Ceccarelli Post-Trial Aff., ¶ 4.)

3. MUTCD associates a yellow change interval to a specific traffic movement, not to a phase. From the point of view of the left-turning driver who goes through an intersection more than once, there is unpredictability as to the length of the yellow change interval for left turns. Sometimes the driver gets 3.0 seconds and sometimes 4.5 seconds for a yellow change interval. (See Ceccarelli Post-Trial Aff., ¶¶ 4 & 5.)

Pursuant to Rule 59, the Plaintiffs move for a new trial on these questions raised to "take additional testimony and amend the findings of fact and conclusions of law or make new findings or conclusions, and direct entry of a new judgment."

This the 4th day of March, 2013.

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Traffic Signal Change Intervals: Policies, Practices, and Safety

HOWARD S. STEIN

Howard Stein is Transportation Engineer for the Insurance Institute for Highway Safety, an independent scientific organization devoted to the cause of highway loss reduction. His work at the Institute has covered a wide range of research including examination of the role of signal timing and roadway geometric design features in crashes, evaluation of anti-lacerative windshields, and analysis of the role of large trucks in crashes. He received his BSCE and MSCE from Georgia Institute of Technology. He has been involved in several Institute of Transportation Engineers committees including ITE policies, effects of geometric highway standards on highway accidents, timing of signal change intervals, and large truck crashes.

THE recommended procedures for timing traffic signal change intervals (the yellow and all-red phase after a green light) are not applied consistently throughout the United States. The duration of signal change intervals should be related to individual intersection characteristics such as vehicle speeds and cross-street width; however, signal timing practices do not often account for these factors. If signal change intervals are too short, drivers may be forced to emergency brake or to be in the intersection when cross-street traffic is given the right-of-way with a green light. This article reviews the basic policies for traffic signal change interval timing, the current timing practices based on these policies, and their implications for the safety of motorists and pedestrians.

Policies for timing traffic signal change intervals are based on laws regarding the purpose of the yellow light of the traffic signal. Most states have laws that regard the yellow light as a warning that the green light has ended and that the red light will appear next.[1] Consequently, vehicles may enter the intersection throughout the yellow phase. Some states with this policy add the restriction that, although vehicles are allowed to enter on yellow, they are prohibited from being in the intersection when the signal turns red. In contrast,

1. E. Kearney, "Are Your Traffic Laws Modern and Uniform?" *ITE Journal* 52, no. 3 (March 1982): 18–19.

TRANSPORTATION QUARTERLY, Vol. 40, No. 3, July 1986, 433–445

the laws of 10 other states require that drivers in traffic facing a yellow light following a green light stop before the intersection unless they cannot stop safely.

RECOMMENDED CHANGE INTERVAL TIMING PRACTICES

Guidance for the timing of traffic signal change intervals is provided by three basic sources: *Manual of Uniform Traffic Control Devices (MUTCD); Traffic and Transportation Engineering Handbook (ITE Handbook); and Traffic Control Devices Handbook (TCDH)*. The *MUTCD,* which serves as a national standard for signs, signals and roadway markings, simply states that the yellow light is a warning that the green light allowing traffic movement is being terminated.[2] It also recommends that the yellow phase should generally range from 3 to 6 seconds. The yellow phase may be followed by an all-red clearance phase to permit the intersection to clear before cross-traffic begins. No specific details are given as to how to determine the appropriate duration for the yellow phase or all-red clearance phase.

The current *ITE Handbook* states that the minimum length for the change interval should accommodate both drivers who can safely stop at the stop line and those who choose to go through the intersection.[3] Table I shows two formulas that are presented in the *ITE Handbook.*[4] These formulas are derived by determining the critical point upstream from the intersections at which the majority of drivers, given the deceleration required, would decide *not* to attempt to stop, and by computing the time required for *continuing* drivers to enter (and clear) the intersection. The first formula, for intersections in jurisdictions where vehicles are prohibited from entering the intersection during the yellow phase, allows drivers of vehicles who *do not choose* to stop to have just enough time to enter the intersection before the startup of cross-street traffic. Its variables include driver perception-reaction time, vehicle speed, and vehicle deceleration rate. The deceleration rate used in this formula refers not to actual vehicle performance but to a more gradual constant rate that drivers tend to

2. Federal Highway Administration, *Manual on Uniform Traffic Control Devices* (Washington, D.C.: U.S. Department of Transportation, 1978), p. 4B-15.

3. L. Rach, "Traffic Signals," *Transportation and Traffic Engineering Handbook*, 2nd ed. (Washington, D.C.: Institute of Transportation Engineers, 1982), p. 756.

4. These two formulas are based on work conducted by D. Gazis, R. Herman, and A. Maradudin. See "The Problem of the Amber Signal Light in Traffic Flow," *Traffic Engineering* 30, no. 7 (July 1960): 19–26.

TABLE I—SIGNAL CHANGE INTERVAL TIMING EQUATIONS

A. Minimum signal change interval time required for vehicles not stopping to *enter* intersection

$$Y + AR = t + \frac{V}{2a}$$

Y = Yellow time, sec
AR = All-red time, sec
t = Perception-reaction time, sec
V = Vehicle speed, ft/sec
a = Deceleration rate, ft/sec^2

B. Minimum signal change interval time required for vehicles not stopping to *enter and clear* the intersection

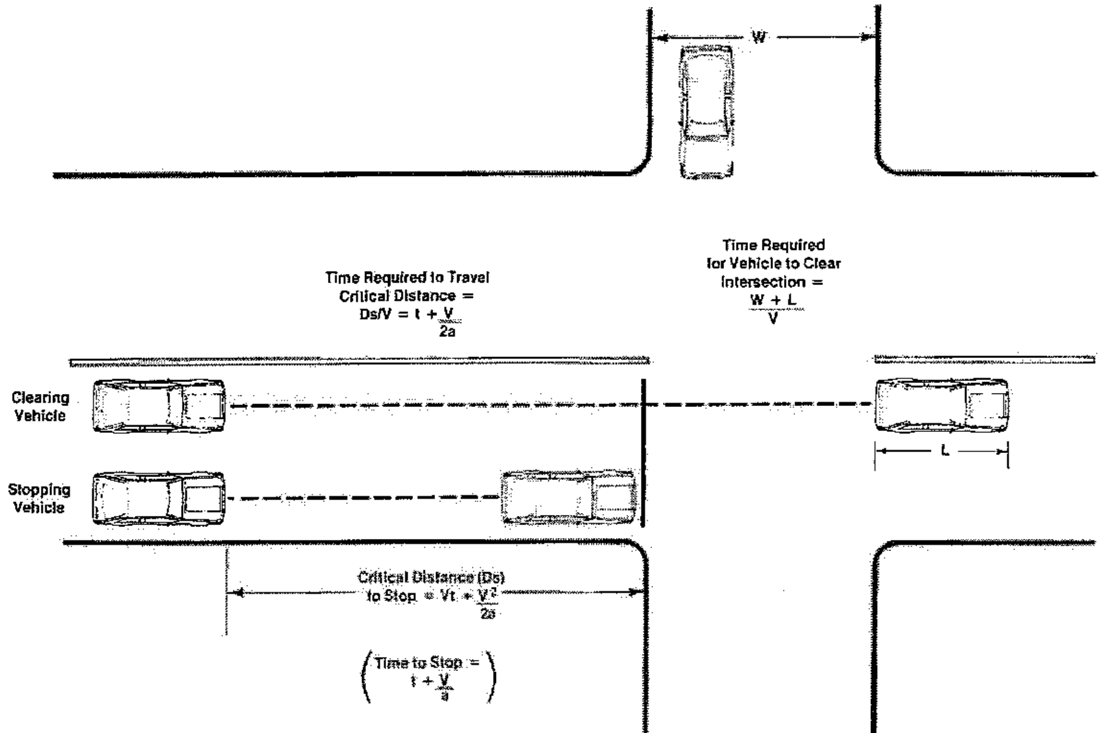
$$Y + AR = t + \frac{V}{2a} + \frac{W + L}{V}$$

Y = Yellow time, sec
AR = All-red time, sec
t = Perception-reaction time, sec
V = Vehicle speed, ft/sec
a = Deceleration rate, ft/sec^2
W = Width of cross street, ft
L = Length of vehicle, ft

select and feel comfortable with in slowing to stop at an intersection. In this formula, the duration of the signal change interval increases with driving speed.

There is a common misconception that this formula, which is often referred to as the "stopping" formula, gives the time required to stop a vehicle. However, an examination of the basic equations of kinematics shows that the time required to stop (t + V/a) is longer than the time required for a vehicle to travel the same distance at a constant speed (t + V/(2a)). This relationship is illustrated in Figure 1, which compares a stopping vehicle with one attempting to clear the intersection.

A second, longer formula for intersections in jurisdictions where vehicles can enter on yellow adds to the previous formula a variable equal to the time required for a vehicle to clear the intersection (see Table I). The relationship of the length of the signal change interval to vehicle speed and intersection width is illustrated in Figure 2. As can be seen from this figure, the duration of the change interval differs for these two formulas, and its values are sensitive to specific intersection characteristics. The *ITE Handbook* is careful to point out that these formulas produce *minimum* values. It also warns that excessively long yellow lights (5 seconds or greater) may lead to loss of drivers' respect



Note: For Definition of Variables See Table I.

Figure 1. Comparison of parameters for vehicles responding to signal change intervals

for the yellow light and suggests that an all-red clearance phase can be substituted for some portion of the calculated yellow time to provide adequate change intervals.

The *ITE Handbook* also mentions the possibility that a "dilemma zone" may exist in which a driver can neither stop safely nor proceed safely through the intersection if the change intervals are not adequate. For example, if the cross street is 84 feet wide and the change interval duration is based on a posted speed limit of 35 mph, the total duration of the change interval (yellow plus all-red phases) based on the longer ITE formula would be 5.6 seconds. For vehicles traveling at higher speeds, e.g., 45 mph, a dilemma zone exists at approximately 268–284 feet upstream from the intersection. The vehicles can neither stop (285 feet required) nor travel completely through the intersection (5.9 seconds required) in the time allowed. A similar hazard exists for slower vehicles, especially at wide intersections (see Figure 2). The warning that change interval values obtained from the ITE formulas provide *minimum* values should be emphasized.

The *TCDH* follows the *ITE Handbook* except that it modifies the

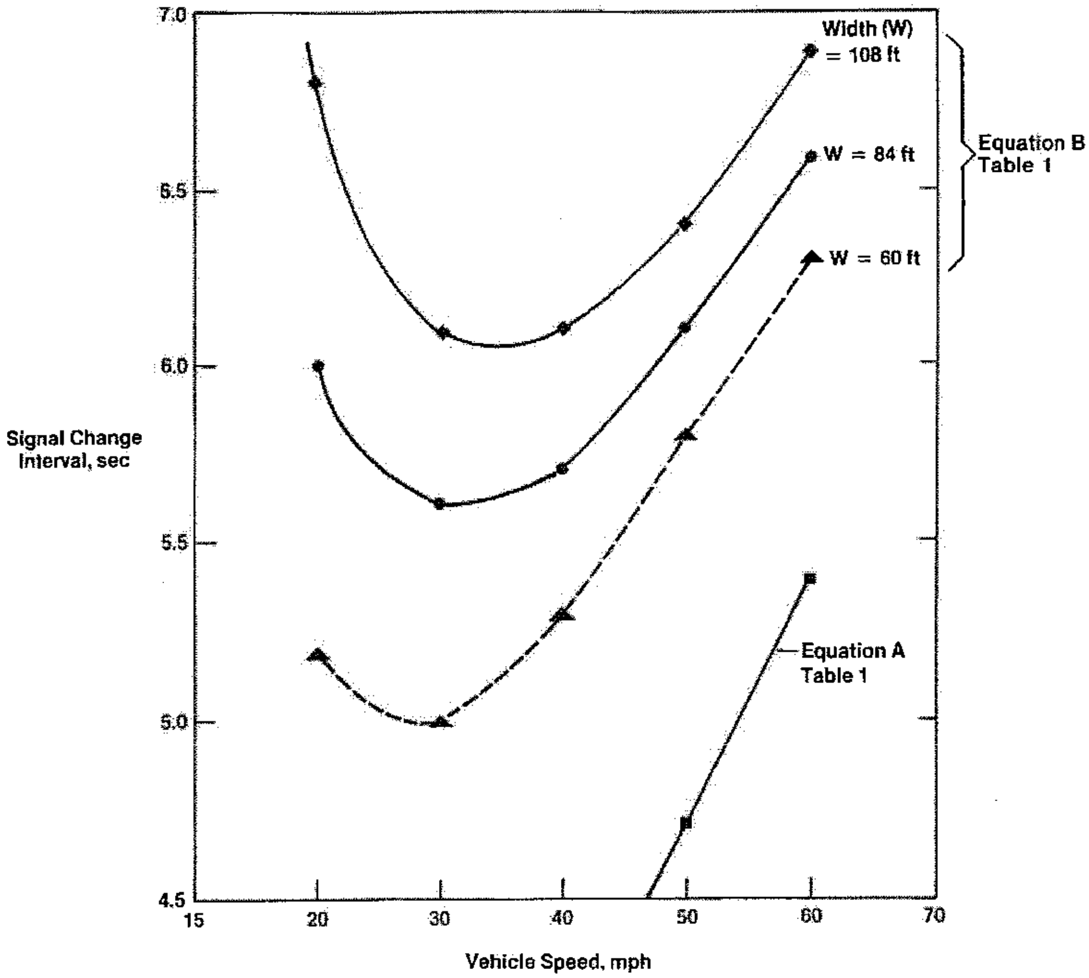


Figure 2. Duration of signal change interval as a function of vehicle speed and intersection width

timing formulas to account for the grade of the roadway.[5] Vehicles on downhill approaches require greater physical force to decelerate as quickly as they would on a level road. If drivers responded as though they were on a level road, they would require a greater distance in which to stop their vehicles. Consequently, vehicles that attempt to clear the intersection need additional time. The *TCDH* further recommends that both the 15th and 85th percentile speeds be used to compute the change interval because slow traffic on wide approaches may require longer intervals (see Figure 2). The *TCDH* points out that the current edition of the *ITE Handbook* has lowered the acceptable driver deceleration rate from 15 to 10 ft/sec^2. It further recommends rounding up to the next half second of the first two terms

5. Federal Highway Administration, *Traffic Control Devices Handbook* (Washington, D.C.: U.S. Department of Transportation, 1983), pp. 4:102–103.

of the ITE equation, maintaining a yellow phase of between 3 and 5 seconds, and including any additional change interval time in an all-red phase.

In contrast to these recommendations, some traffic engineers have argued that the yellow phase should not include clearance time and that an additional all-red phase should be *selectively* used to provide clearance time.[6] It has been suggested that the posted speed limit be modified to justify short signal change intervals.[7] Some traffic engineers assume that up to two late left-turning vehicles will complete their manuever during the change interval. Timing change intervals without providing adequate clearance time places the responsibility on individual motorists to be sure that there is sufficient time left to completely clear the intersection. In addition, it places the responsibility on cross-street motorists to be sure the intersection is clear before they enter it. These timing practices assume that, if a collision occurs, the motorist is at fault for not yielding the right-of-way or making a wrong choice between entering or stopping.

The role of the traffic engineer is to provide a margin of safety so that motorists can pass safely through the intersection during either the yellow or all-red phase. Any philosophy that accepts crashes that could be prevented merely to save 1 or 2 seconds of signal timing is contrary to traffic safety principles. Most states allow vehicles to enter the intersection when the signal is yellow, and common driving experience and research indicate that many drivers will enter at the end of the yellow phase. An appropriate *total signal change interval* (yellow and all-red phases) will protect most motorists who continue through the intersection from the cross-street traffic. It has been recognized for some time that "any clearance interval (i.e., signal change interval) or interpretation . . . which requires drivers to brake harder appears to be contrary to normal driver behavior and is therefore unrealistic."[8]

Special modifications of change interval timing should also be considered to accommodate large trucks and buses with typical vehicle lengths of up to 65 feet and poor deceleration characteristics compared to passenger vehicles. Intersections with unusual geometry where the

6. H.H. Bissell and D.L. Warren, "The Yellow Signal is Not a Clearance Interval," *ITE Journal* 51, no. 2 (February 1981): 14–17.

7. Federal Highway Administration, *Traffic Control Devices Handbook*.

8. P. Olson and R.W. Rothery, "Deceleration Levels and Clearance Times Associated with the Amber Phase of Traffic Signals," *Traffic Engineering* 42, no. 4 (April 1972): 16–19; 62–63.

path the vehicle travels is longer than the perpendicular intersection width may also require special timing procedures.[9]

Pedestrian safety is also affected by signal timing. The pedestrian walk signal, which is displayed as soon as the conflicting traffic is shown a red light, is usually interpreted to mean pedestrians may proceed safely across the roadway.[10] It is not assumed that pedestrians should wait for potential clearing traffic. If signal change intervals are timed using the ITE formula for jurisdictions that prohibit vehicles from entering the intersection during the yellow light, pedestrians could enter the street 2 or 3 seconds before late clearing vehicles reach the crosswalk.

It should be noted that the ITE equations have some inherently conservative assumptions. The timing formulas assume that drivers decelerate in a uniform manner and that they attempt to clear an intersection at a uniform speed. However, some drivers exert additional pressure on the brake pedal or even panic brake to ensure that their vehicles will come to rest before the intersection. Also, late-arriving drivers may speed up to make the light. Some drivers may expect late-coming vehicles and wait for vehicles on the cross-street to stop; this may be particularly true at intersections where late-clearing vehicles are common. In addition, drivers may tend to enter the intersection when they are facing only light traffic on the cross street, and they may tend to stop when facing heavy traffic on the cross street; both of these behaviors contribute to collision avoidence.

IMPLICATIONS OF TRAFFIC SIGNAL CHANGE INTERVAL TIMING

In actual practice, many jurisdictions do not correctly apply the recommended procedures for timing signal change intervals. A survey of the state-of-the-art found that procedures for determining the length of the yellow phase were inconsistent with the laws regarding its purpose. Of the approximately 230 traffic agencies responding, more than half used the ITE formulas.[11] Some jurisdictions use uniform change intervals (all or most intervals timed typically at 3 or 4 seconds) regardless of relevant individual intersection characteristics.

9. J.M. Frantzeskakis, "Signal Change Intervals and Intersection Geometry," *Transportation Quarterly* 38, no. 1 (January 1984): 47–58.

10. Federal Highway Administration, *Manual on Uniform Traffic Control Devices* (Washington, D.C.: U.S. Department of Transportation, 1978), p. 4D-1.

11. B. Benioff and T. Rorabaugh, *A Study of Clearance Intervals, Flashing Operation, and Left-Turn Phasing at Traffic Signals* (Washington, D.C.: Federal Highway Administration, 1980), FHWA-RD-78-46.

A survey of intersections in the Southeast found that about half of the approaches were deficient relative to the previous *ITE Handbook* formula ($a = 15$ ft/sec^2). Almost all of these intersections were deficient in terms of the current formula.[12] In another nationwide study, only about half of the approaches studied had a total signal change interval sufficient to meet the current ITE formula.[13]

Lack of appropriate signal change interval timing procedures can have serious legal implications. The city of Flint, Michigan was held responsible for the death of a driver in a car-truck collision at an intersection because the signal change interval failed to account for the deceleration characteristics of trucks. The court stated that this was a defect in the intersection design.[14]

Surveys of drivers' knowledge about change intervals have reported that, generally, drivers are unaware of the specifics of the law where they reside.[15] Although the jurisdictions' laws allowed vehicles to enter the intersection on yellow, more than 50 percent of those questioned responded that they should stop before the intersection if it was safe to do so. Although it is not known what choices these drivers make in actual driving, rear-end crashes may result if a driver chooses to stop and the driver behind chooses to legally attempt to enter the intersection.

Considerable research has been conducted to assess driver behavior during the signal change intervals, and all of it has concluded that the majority of drivers will continue to enter the intersection if, based on their speed and distance, they would be forced to decelerate at a rate 12–15 ft/sec^2 or faster.[16] For most drivers, this would be an abrupt

12. P.S. Parsonson and A. Santiago, "Design Standards for Timing the Traffic-Signal Clearance Period Must be Improved to Avoid Liability," *ITE Compendium of Technical Papers* (Washington, D.C.: ITE, 1980), pp. 67–71.

13. P. Zador, H. Stein, S. Shapiro, and P. Tarnoff, "The Effect of Signal Timing on Traffic Flow and Crashes at Signalized Intersections," *Transportation Research Record 1010*, Transportation Research Board, Washington, D.C., 1985, pp. 1–8.

14. Parsonson and Santiago, "Design Standards for Timing the Traffic-Signal Clearance Period Must be Improved to Avoid Liability."

15. Benioff and Rorabaugh, *A Study of Clearance Intervals, Flashing Operation, and Left-Turn Phasing at Traffic Signals.*

16. Gazis, Herman, and Maradudin, "The Problem of the Amber Signal Light in Traffic Flow;" Olson and Rothery, "Deceleration Levels and Clearance Times Associated with the Amber Phase of Traffic Signals;" Zador, Stein, Shapiro, and Tarnoff, "The Effect of Signal Timing on Traffic Flow and Crashes at Signalized Intersections;" M. Chang, C.J. Messer, and A. Santiago, "Timing Signal Change Intervals Based on Driver Behavior," *Transportation Research Record 1027*, Transportation Research Board, Washington, D.C., 1985, in press; M. Chang, C.J. Messer, and A. Santiago, "Evaluation of Engineering Factors Affecting Traffic Signal Change Intervals," *Transportation Research Record 956*, Transportation Research Board, Washington, D.C., 1984, pp. 18–21; W.A. Stimpson, P.L. Zador, and

stop. The vast majority of drivers who can stop at a deceleration rate of 10 ft/sec^2 or less will do so. The current edition of the *ITE Handbook* reduced the recommended deceleration value from 15 ft/sec^2 to 10 ft/sec^2 in response to the evidence supporting the slower rate.

Research has also shown that the choice to continue through the intersection is largely independent of intersection characteristics and the actual duration of the change interval. In the several studies where drivers were observed responding to the onset of the yellow light, few significant differences were noted in the deceleration rates or response times of drivers. In addition, several studies have evaluated changes in the behavior of drivers with extended signal change intervals, and all of these studies found little difference in driver response. However, the longer signal change intervals allowed significantly more vehicles to clear the intersection before the startup of cross-street traffic.[17]

A major evaluation of the effects of longer signal change intervals was conducted in Australia in 1980.[18] The change intervals of 58 intersections in Newcastle were retimed using a formula similar to the longer ITE formula. Previously, the yellow phase was 3 seconds at all the intersections and the duration of any all-red phase varied. On the average, the yellow phase was increased to 4 or 4.5 seconds. Detailed observations of drivers entering the intersection were made at 15 of these intersections before the timing changes and 3 months after they were made. Overall, vehicles entering the intersections on red decreased 63 percent, from 9 per 1,000 vehicles entering to 3.4 per 1,000 vehicles entering. In Sydney, the change intervals of four

P.J. Tarnoff, "The Influence of the Time Duration of Yellow Traffic Signals on Driver Response," *ITE Journal* (November 1980): 22–29; W.L. Williams, "Driver Behavior During the Yellow Signal Interval," *Transportation Research Record 644*, Transportation Research Board, Washington, D.C., 1977, pp. 75–78; R.H. Wortman and J.S. Mathias, "An Evaluation of Driver Behavior at Signalized Intersections," *Transportation Research Record 904*, Transportation Research Board, Washington, D.C., 1983; and R. Wortman, J. Witkowski, and T. Fox, "Traffic Characteristics During Signal Change Intervals," *Transportation Research Record 1027*, Transportation Research Board, Washington, D.C., 1985, in press; and P.L. Zador, *Driver Behavior at Signalized Intersections* in Relation to Yellow Intervals (Washington, D.C.: Insurance Institute for Highway Safety, 1980).

17. Benioff and Rorabaugh, "A Study of Clearance Intervals, Flashing Operation, and Left-Turn Phasing at Traffic Signals;" Zador, Stein, Shapario, and Tarnoff, "The Effect of Signal Timing on Traffic Flow and Crashes at Signalized Intersections;" Chang, Messer, and Santiago, "Timing Signal Change Intervals Based on Driver Behavior;" and Wortman, Witkowski, and Fox, "Traffic Characteristics During Signal Change Intervals."

18. R.D. Munro and L. Marshall, *Analysis of the Newcastle Survey of Driver Observance of Traffic Signals*, report to Department of Main Roads, Sydney, Australia, 1982.

intersections were increased based on the Newcastle experience, and an evaluation was performed to determine if this increase had any effect on the capacity of these intersections. This study concluded that an increase in change interval time would have a negligible effect on intersection capacity.[19]

Recent research has found that traffic signal change interval timing can significantly affect intersection crash rates. A national study found that, on average, crash rates were significantly higher at intersections with less adequate change intervals.[20] Traffic and signal operations were observed at 91 intersections in eight different U.S. cities. Despite differences in intersection characteristics, the flow of traffic through the intersections during the change intervals was similar. However, intersections with less adequate timing relative to the ITE formula had significantly higher rear-end and right-angle crash rates. Figure 3 shows the relationship between the clearance ratio (ratio of actual signal change interval to signal change interval computed using longer ITE formula) and the daytime crash rate for the observed street. This study also reported that the intersections with the least adequate change intervals had, on the average, slower approach street traffic and intersected with wider cross streets than the intersections with more adequate change intervals. This finding confirms the earlier observations about the sensitivity of the ITE longer formula to intersection characteristics (Figure 2). The methods used in this study were repeated in another evaluation of intersections, and the results were similar.[21]

Several other studies have examined the effects of modifying signal change intervals on crashes, but they have not compared these effects to the adequacy of the change intervals. The Australian study did examine changes in crashes at the intersections with modified timing and found that, overall, there was little change in the number and severity of crashes.[22] However, because there were differences in the

19. A.B. Finlay, *Evaluation of Increased Intergreen Time at Signal Sites Operating Close to Capacity*, report to Department of Main Roads, Sydney, Australia, 1984.

20. Zador, Stein, Shapario, and Tarnoff, "The Effect of Signal Timing on Traffic Flow and Crashes at Signalized Intersections."

21. A. Taghipour-Z, *Relationship Between Accident Experience and Timing of the Clearance Interval at Signalized Intersections* (unpublished thesis Atlanta, GA: Georgia Institute of Technology, School of Civil Engineering, March 1985).

22. P.C. Croft and B.C. Traudinger, *Crashes at Signalized Intersections-Effects of a Trial of Signal Timings Adjustments,* Traffic Authority of New South Wales, Australia, 1983.

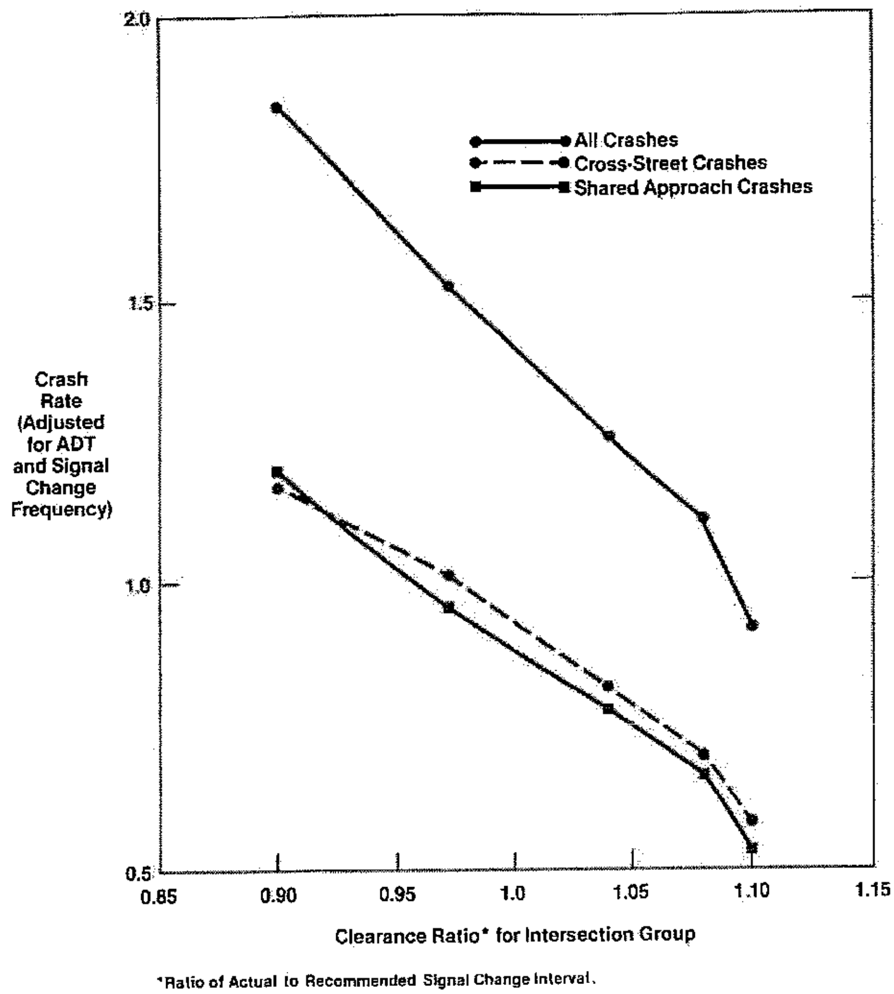


Figure 3. Crash rates by crash type and intersection group

pattern of changes in crashes (day/night) and traffic volumes (after period volumes increased 17 percent during the day, 8 percent at night) that were not fully accounted for in the crash analyses, these results are inconclusive.

Another major study investigated changes in traffic operations and crashes following implementation of a uniform yellow phase in Fresno, California.[23] In this study, the yellow phase was adjusted to between 3.6 to 4.0 seconds; the existing yellow duration was increased at some intersections and decreased at others. An analysis of crashes at 80 selected intersections concluded that, overall, the crash rate did not change but the severity of crashes was reduced. The most affected crashes were not multiple vehicle crashes as would be expected but

23. Benioff and Rorabaugh, "A Study of Clearance Intervals, Flashing Operation, and Left-Turn Phasing at Traffic Signals."

those classified as "other," such as single vehicle crashes. The study also noted that these changes to the yellow phase had a negligible effect on intersection capacity.

Two studies have examined the effect of adding an all-red phase to the existing change interval and have found that this reduces right-angle crashes.[24] However, these studies did not examine whether the effect was due to increasing the total change interval length or the specific addition of red versus yellow time.

DISCUSSION

This article has reviewed the policies and procedures commonly used to determine the duration of traffic signal change intervals. The main conclusions that can be drawn from the research that has been performed are:

1. Driver behavior at yellow lights (e.g., stopping decisions, perception-reaction times, and deceleration rates) is largely independent of differences in state laws concerning the purpose of yellow lights, individual intersection characteristics, and traffic conditions.

2. Not all traffic signals have change interval timing that is specific to the intersection's characteristics, and many do not provide sufficient time for vehicles to clear the intersection before the start-up of cross-street traffic.

3. Inadequate traffic signal change interval timing relative to the ITE recommended longer formula is associated with higher crash rates for both rear-end and right-angle crashes.

None of the official sources of guidance for timing traffic signal change intervals systematically provide for the safety of a late-arriving vehicle. They do not require clearance time even though research has repeatedly indicated that drivers do enter the intersection during the yellow phase and are unfamiliar with local laws requiring them to stop on yellow. Recently, the Institute of Transportation Engineers proposed new standards for the timing of traffic signal change intervals that failed to require clearance time at all intersections.[25] The provision of clearance time to accomodate late-entering drivers

24. Benioff and Rorabaugh, "A Study of Clearance Intervals, Flashing Operation, and Left-Turn Phasing at Traffic Signals;" and T.A. Ryan and C.F. Davis, "Driver Use of the All-Red Signal," *Transportation Research Record 881,* Transportation Research Board Washington, D.C. 1982, pp. 9–16.

25. ITE Technical Committee 4A-16, "Proposed Recommended Practice for Determining Vehicle Change Intervals," *ITE Journal* 55, no. 5 (May 1985): 61–64.

would simply reflect the common engineering practice of providing a margin of safety. It is similar to providing the automatic retracting mechanisms in elevator doors that prevent the doors from closing on (and potentially crushing) a late-entering passenger. The available research documents that adequate traffic signal change interval timing, which in most cases would only add 1 or 2 seconds, will reduce traffic conflicts without significantly affecting traffic operations.

Making Intersections Safer: A Toolbox of Engineering Countermeasures to Reduce Red-Light Running

EXHIBIT

B

An Informational Report

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Chapter 3 - Engineering Countermeasures

INTRODUCTION

As Chapter 2 described, there are a number of intentional and unintentional factors that cause drivers to run red-lights. With this information, several engineering measures can now be developed that reduce the occurrence of this behavior. From an engineering perspective, red-light running may be reduced if, in general, any one of these actions is taken:

- Ensure that the traffic signal, and specifically the red display, is visible from a sufficient distance and captures the motorists' attention (i.e., it is conspicuous);
- Increase the likelihood of stopping for the red signal, once seen;
- Address intentional violations; and
- Eliminate the need to stop.

If a traffic signal is the most appropriate choice of traffic control for the intersection, it is important to ensure that the motorist can see the traffic signal far enough away from the intersection so that he/she can stop safely upon viewing the yellow and red display. Then, upon viewing the yellow, and certainly the red, ensure that signal operations and conditions do not entice the motorist to intentionally or unintentionally enter on red and ensure that a driver who tries to stop his/her vehicle can successfully do so before entering the intersection. Recognizing that there are some motorists that will intentionally violate the red signal at certain times and situations, those conditions that encourage this behavior must be minimized. Engineers should also examine whether or not the traffic signal is the most appropriate choice of control for an intersection and if it can be replaced with another form of control or design that eliminates the signal and therefore the problem.

This chapter identifies various engineering measures that can be grouped under these general solutions. For each, the measure is described, applicable design standards or guidelines in the Manual on *Uniform Traffic Control Devices* (MUTCD) *(22)* are provided, and where known, its effectiveness in reducing red-light violations and resulting crashes is presented. Other considerations in implementation and use are noted where appropriate.

IMPROVE SIGNAL VISIBILITY

Motorists who violate the red traffic signal frequently claim, "I did not see the signal." As reported in Chapter 2, 40 percent of those surveyed claim they did not see the signal and another 12 percent apparently mistook the signal indication and claimed there was a greensignal indication. While there is no doubt that many of these claims are false, there probably are situations where a more visible signal would not have been violated. For whatever reason-motorist inattention, poor vision, poor signal visibility-the motorist did not see the signal, and specifically, the red signal in time to come to a stop safely. The countermeasure for this problem is to ensure that the signal is visible from a sufficient distance upstream.

Improve Signal Visibility

- Placement and number of signal heads
- Size of signal display
- Line of sight

Signal heads placed in accordance with the MUTCD should be visible to all motorists approaching the intersection. Although the MUTCD requires a minimum of two signal faces be provided for the major movement on an approach, through the signalized intersection or suffer the perceived long delay associated with sitting for the red signal. However, many traffic engineers use longer cycle lengths to move significant volumes on the mainline of arterial roadways. By providing a sustainable progression along a corridor, the saturated roadway can move higher volumes and reduce queue lengths. Delays associated with numerous start-up times are also diminished if progression is maintained.

When comparing cycle lengths, it should be noted that with longer cycle lengths, there are actually fewer numbers of times per hour when drivers are confronted with the yellow and red signal intervals. For example, when comparing a cycle length of 1 min. to 2 min., in an hours time in the 1-min. cycle, there will be twice as many opportunities for drivers to be confronted with the changing signal from green to red. Consequently, the longer cycle length does reduce the number of opportunities for traffic-signal violations.

After consideration of the pros and cons, one of the best tools to utilize in determining signal-cycle length is computer simulation and optimization. The computer generated optimized cycle length combined with the traffic engineers' knowledge and experience should result in the most efficient traffic-signal timing practical. As part of signal-timing strategies, the need to address specific times of day should be included. For example, typical timing plans would include multiple plans to accommodate the morning or afternoon peak periods, midday, late night, weekends, etc.

Yellow-Change Interval

The MUTCD (22) requires that a yellow-signal indication be displayed immediately following every circular green or green-arrow signal indication. It is used to warn vehicle traffic that the green-signal indication is being terminated and that a red indication will be exhibited immediately thereafter.

A properly timed yellow interval is essential to reduce signal violations. An improperly timed yellow interval may cause vehicles to violate the signal. If the yellow interval is not long enough for the conditions at the intersection, the motorist may violate the signal. Motorists have some expectancy of what the yellow interval should be and base their decisions to proceed or stop based on their past experiences. In order to reduce signal violations, the engineer should ensure that the yellow interval is adequate for the conditions at the intersection and the expectations of the motorists.

In many jurisdictions, the yellow-change interval is followed by an all-red interval. During this all-red "clearance' interval, the red-signal indication is displayed to all traffic. The yellow interval and all-red interval are often referred to collectively as the change period or change interval. The all-red interval is addressed separately in a subsequent section.

There is currently no nationally recognized recommended practice for determining the change interval length, although numerous publications provide guidance including the MUTCD (22), *Traffic Engineering Handbook (44), and the Manual of Traffic Signal Design (45).* The MUTCD provides guidance that a yellow-change interval should have a duration of approximately 3 to 6 sec., with the longer intervals reserved for use on approaches with higher speeds.

In the current edition of ITE's *Traffic Engineering Handbook (44)*, a standard kinematic equation is provided as a method to calculate the change interval length. The equation for calculating the change period, CP, is as follows:

$$CP = t + \frac{V}{2a + 64.4g} + \frac{W + L}{V} \qquad [1]$$

The principal factors that are taken into account in the development of the change period are:

- Perception-reaction time of the motorist, *t*, typically 1 sec.;
- Speed of the approaching vehicle, V, expressed in ft./sec.;
- Comfortable deceleration rate of the vehicle, a, typically 10 ft./sec.2;
- Width of the intersection, W;
- Length of vehicle, *L*, typically 20 ft.; and
- Grade of the intersection approach, g, in percent divided by 100 (downhill is negative).

The equation allows time for the motorist to see the yellow signal indication and decide whether to stop or to enter the intersection. This time is the motorist's perception-reaction time, generally 1 sec. It then provides time for motorists further away from the signal to decelerate comfortably and motorists closer to the signal to continue through to the far side of the intersection. These times are dependent on the characteristics of the traffic and the roadway environment. If there is a grade on the approach to the intersection, the equation adjusts the time needed for the vehicle to decelerate.

If available, the 85th percentile speed should be used as the approach speed in this equation. In the absence of 85th percentile speed, some jurisdictions use posted speed as the approach speed. In most cases, using the 85th percentile speed will produce intervals that are more conservative (i.e., longer). In no case should the approach speed used in the calculation be less than the posted speed limit.

The deceleration rate of 10 ft./sec.2 suggested by ITE is based on a comfortable deceleration rate that has been supported by research. The 2001 American Association of State Highway and Transportation Official's A Policy on *Geometric Design of Highways and Streets*, otherwise known as the "Green Book," *(46)* recommends 11.2 ft./sec.2 for determining stoppingsight distance. They note that this is a comfortable deceleration for most drivers. The deceleration rate suggested by ITE is a more conservative deceleration rate for purposes of calculating the yellow interval and will result in longer intervals.

Effectiveness of Decreasing Violations

Various studies have evaluated the relationship between the length of the change interval and the occurrence of signal violations. Retting and Green *(47)* examined redsignal violations in New York where the yellow or allred intervals were shorter than a practice proposed by ITE in 1985 *(48)* that is similar in calculation to Equation 1. They conducted a before-and-after study with a control group at 20 approaches. For the afterperiod, the researchers retimed the yellow interval at four sites, the all-red interval at five sites, and both the all-red and the yellow at four sites. Seven sites were used as the control group. The yellow retiming increased the yellow change interval by 0.5 to 1.6 sec., depending on the intersection. The all-red retiming increased the red-clearance interval by 0.8 to 3.6 sec. The researchers recorded the number of cycles with red-signal violations and the number of cycles with late exits at the intersections. Red-signal violation cycles were defined as cycles where at least one of the vehicles on the approach entered the intersection on red. Lateexit cycles were defined as cycles where at least one vehicle from the approach was still in the intersection at the release of conflicting traffic.

Logistic regression was used to analyze the data. The researchers concluded that increasing the length of the yellow signal towards the ITE proposed practice significantly decreased the chance of red-signal violations. They also found that late exits decreased as the all-red interval increased. It appeared that sites with shorter yellow signals had more late exits. Increasing the yellow to ITE-suggested values was as effective as increasing the all-red clearance interval at decreasing the occurrence of late exits.

Wortman et al. *(49)* conducted a similar before-andafter study at two intersections in Arizona. In the afterperiod, the yellow interval was extended from 3 sec. to 4 sec. The researchers observed a statistically significant reduction in the percentage of vehicles entering during the red-signal indication. These results should be viewed cautiously, however, since the treatment sites only included two intersections and since there was no indication of comparison or control sites.

R. A. van der Horst *(50)* found that increases in the yellow interval decreased the amount of red-signal violations. He conducted a behavioral before-and-after study at 23 urban and rural intersections in the Netherlands. One year after the yellow intervals were lengthened by 1 sec., the number of red-signal violations at the intersections lowered by one half. Bonneson's research indicates *(39)* that yellow increases in the range of 0.5 to 1.5 sec., that do not yield durations above 5.5 sec. can potentially reduce red-light running by about 50 percent.

Drawbacks of Lengthened Yellow Intervals

Although lengthening the yellow interval may reduce signal violations, an interval that is too long could decrease the capacity of the intersection and increase the delay to motorists and pedestrians. Present thought is that longer intervals will cause drivers to enter the intersection later and it will breed disrespect for the traffic signal. The tendency for motorists to adjust to the longer interval and enter the intersection later is referred to as *habituation*.

The before-and-after study by Retting and Greene *(47)* evaluated the presence of habituation to the longer yellow interval by using a second after-period. The same after-period measurements (cycles with signal violation and late exits) were collected in a second after-period approximately six months after the first after-period. They were compared to the first afterperiod. The authors concluded that habituation to the longer yellow did exist although it may have been only largely present at one site for signal violations. No significant habituation was observed for late exits. In the before-and-after study at the two intersections in Arizona, Wortman et al. *(49)* compared plots of the time of entry of vehicles into the intersection. The researchers observed an increase in the number of drivers entering towards the end of the interval, possibly due to the lengthened yellow interval.

Additional research is needed to further understand the effect of lengthening the yellow interval on driver behavior.

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THE PROBLEM OF THE AMBER SIGNAL LIGHT IN TRAFFIC FLOW

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A theoretical analysis and observations of the behavior of motorists confronted by an amber signal light are presented. A discussion is given of the following problem: when confronted with an improperly timed amber light phase a motorist may find himself, at the moment the amber phase commences, in the predicament of being too close to the intersection to stop safely or comfortably and yet too far from it to pass completely through the intersection before the red signal commences. The influence on this problem of the speed of approach to the intersection is analyzed. Criteria are presented for the design of amber signal light phases through whose use such 'dilemma zones' can be avoided, in the interest of over-all safety at intersections.

WE LIVE in a difficult and increasingly complex world where man-made systems, man-made laws and human behavior are not always compatible. This paper deals with a problem peculiar to our present civilization, for which a satisfactory solution based on existing information and analysis is not available. The problem in question is that of the amber signal light in traffic flow.

Undoubtedly everyone has observed at some time or other the occurrence of a driver crossing an intersection partly during the red phase of the signal cycle. There are few of us who have not frequently been faced with such a decision-making situation when the amber signal light first appears, namely, whether to stop too quickly (and perhaps come to rest partly within the intersection) or to chance going through the intersection, possibly during the red light phase. In view of this situation we were led to consider the following problem: can criteria presently employed in setting the duration of the amber signal light at intersections lead to a situation wherein a motorist driving along a road within the legal speed limit finds himself, when the green signal turns to amber, in the predicament of being too close to the intersection to stop safely and comfortably and yet too far from it to pass through, before the signal changes to red, without exceeding the speed limit? From experience we feel that a problem exists, and we ask if it is feasible to construct a signal light system such that the characteristics of a driver and his car, the geometry

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of the road and intersection, and the law are all compatible with one another.

Some thought has already been devoted to this question[1,2] but it is our opinion that the problem at hand does not appear to have been thought through deeply enough as a problem in operations research nor does it appear to have been supported adequately by published observational and experimental data. It is our intention in this paper to contribute toward the understanding of this situation. First, we derive and discuss some simple relations between car speed, driver decision and reaction time, the parameters of the road and intersection, and the duration of the amber signal light. The results of measurements of the duration of amber signal lights, driver decision plus reaction time, and other parameters entering into the theoretical discussion are next presented. Finally, we discuss the experimental results in the light of theory and the traffic codes of cities and towns throughout the country.

We are well aware that there may be practical difficulties involved in incorporating the results and conclusions of an analysis such as ours into the practical planning of traffic systems, and we do not consider such problems here. It is our hope, rather, that in pointing out the existence and nature of the amber-signal-light problem we may stimulate others to pursue it further and make certain that the driver is confronted with a solvable decision problem. We are, of course, also motivated by the desire to contribute effectively toward the improvement of over-all driver safety and, in this case specifically, safety at intersections. 1

ANALYTICAL CONSIDERATIONS

We consider the traffic situation depicted in Fig. 1, in which a car traveling at a constant speed v_0 toward an intersection is at a distance x from the intersection when the amber phase commences. The driver is then faced with two alternatives. He must either decelerate and bring his car to a stop before entering the intersection or go through the intersection, accelerating if necessary, and complete his crossing before the signal turns red. In these cases his acceleration or deceleration will begin at a time δ_1 or δ_2 after the initiation of the amber phase, respectively. These time intervals δ_i measure the reaction time-lag of the driver-car complex as well as the decision-making time of the driver. 2

In order to carry out a mathematical investigation of the problem we assume a constant acceleration a_1 in the case of crossing the intersection, or a constant deceleration a_2 in the case of stopping before entering the intersection. If, furthermore, the effective width of the intersection is denoted by w, the length of the car by L and the duration of the amber phase by τ, the following relations can be derived:

1. If the driver is to come to a complete stop before entering the intersection, we find that

$$(x - v_0\,\delta_2) \geqq v_0^2/2a_2. \tag{1}$$

2. If the driver is to clear the intersection completely before the light turns red, we must have

$$(x + w + L - v_0\,\delta_1) \leqq v_0\,(\tau - \delta_1) + \tfrac{1}{2}\,a_1\,(\tau - \delta_1)^2. \tag{2}$$

It is to be noted that the effective width, w, used in the preceding equation is meant to denote the approximate distance between a painted stopping line or a building line and a 'clearing line' whose position is necessarily somewhat indefinite because of the geometry of real intersections.

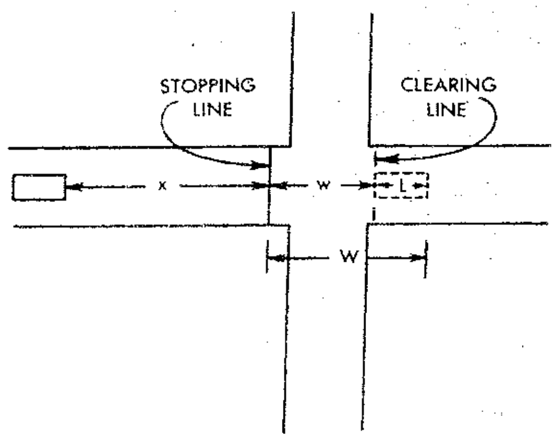

Fig. 1. Geometry of an intersection showing distances to be covered by a car of length L in the two alternative cases of going through and stopping before the intersection.

Equations (1) and (2) can be used for the discussion of the two alternatives and their ramifications. Thus, solving equation (1) for a_2 we obtain, assuming the equality sign,

$$a_2 = \tfrac{1}{2}\,v_0^2/(x - v_0\,\delta_2). \tag{3}$$

Equation (3) gives the (constant) deceleration needed in order to bring the car to a stop just before the intersection as a function of the distance of the car from the intersection at the initiation of the amber phase. We see that a_2 becomes infinite for $x = v_0\,\delta_2$, as it must. However, even for values of x greater than $v_0\,\delta_2$, the deceleration given by (3), while finite, may be so large as to be uncomfortable to the driver and his passengers, or may be unsafe under the prevailing road conditions, or even physically impossible. Therefore, assuming the existence of a maximum deceleration a_2^* by which the car can be brought to a stop before the intersection safely and comfortably, equation (1) defines a 'critical distance', namely,

$$x_c = v_0\,\delta_2 + v_0^2/2a_2^*. \tag{4}$$

If $x > x_c$ the car can be stopped before the intersection, but if $x < x_c$ it will be uncomfortable, unsafe, or impossible to stop. We note that this critical distance is independent of the duration of the amber phase, τ, and depends only on the characteristics of the driver-car complex. The required deceleration is plotted versus distance in Fig. 2.

Turning, now, to the second alternative, namely, going through the

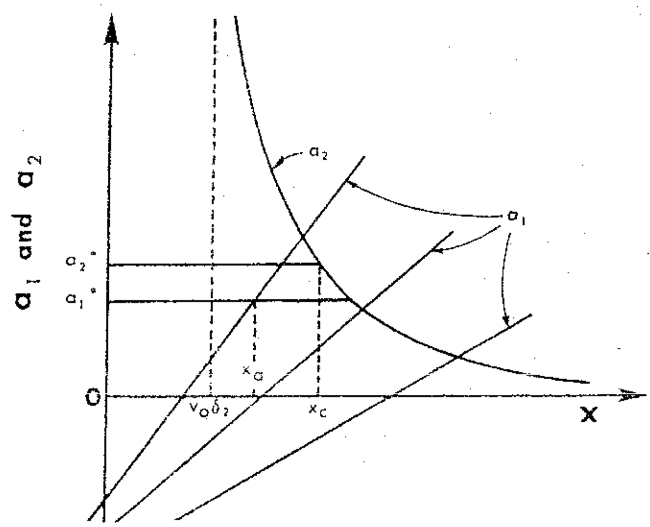

Fig. 2. Variation of the deceleration required in order to stop before the intersection, a_2, or the acceleration required to clear the intersection, a_1, versus the distance from the intersection, x. The x-intercept of the a_1 versus x lines defines a distance x_0 which is the maximum distance, apart from the width of the intersection and the length of car, which can be covered without acceleration during the amber phase.

intersection, we solve equation (2) for a_1, assuming the equality sign, and obtain

$$a_1 = 2\,x/(\tau-\delta_1)^2 + 2\,(w+L-v_0\,\tau)/(\tau-\delta_1)^2. \tag{5}$$

Equation (5) gives the (constant) acceleration needed in order that the car may clear the intersection just as the signal turns red, as a function of the distance x of the car from the intersection at the start of the amber phase. For various values of the parameters involved, equation (5) represents a family of straight lines in the x,a_1-plane with slope

$$da_1/dx = 2/(\tau-\delta_1)^2, \tag{6}$$

and intercept on the x-axis,

$$x_0 = v_0\tau - (w+L). \tag{7}$$

The quantity x_0 is the maximum distance the car can be from the intersection at the start of the amber phase and still clear the intersection without acceleration during the amber phase. The position of x_0 with respect to x_c, and the character of the line represented by equation (5), determine whether or not the duration of the amber phase has been adequately designed, taking into account the requirements of the law and the physical 'boundary conditions' of the problem. Thus, if $x_0 > x_c$, the driver,

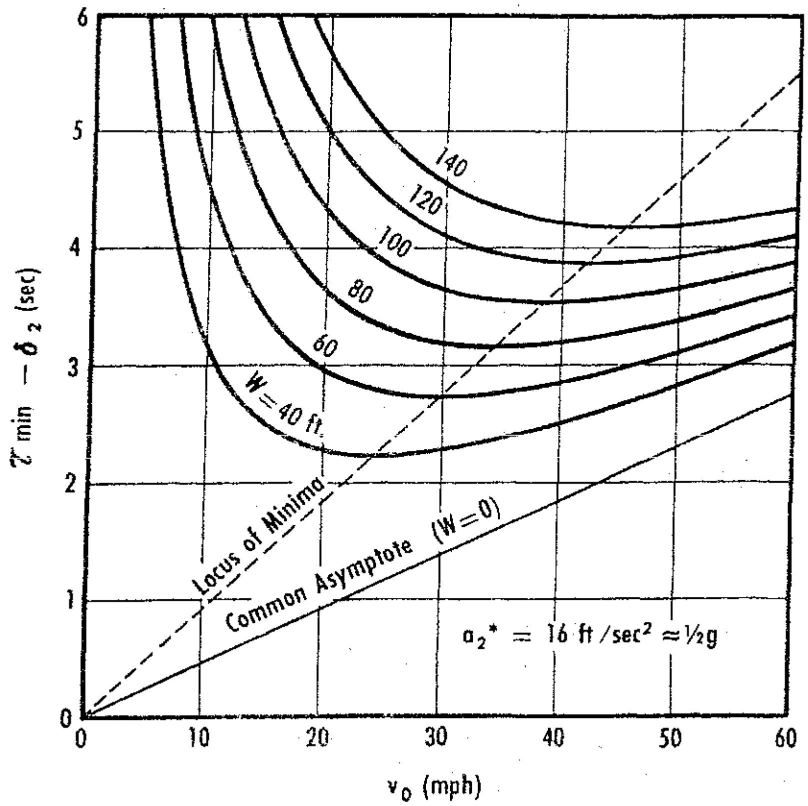

Fig. 3. Variation of the minimum amber-phase duration, τ_{min}, which is required in order that there be no dilemma zone, versus constant approach speed, v_0, for various intersection widths plus car length, W. (The constant deceleration is assumed to be 16 ft/sec^2.)

once past the critical distance x_c, can clear the intersection before the signal turns red. If, however, $x_0 < x_c$, a driver at a distance x from the intersection such that $x_0 < x < x_c$ will find himself in a very awkward position if the amber phase begins at that moment. He cannot stop safely and hence he has to attempt to go through the intersection. From Fig. 2 we see that he can achieve this only by accelerating. If, however, v_0 happens to be the maximum allowable speed, the driver will find himself in the following predicament. He can neither bring his car to a stop safely nor can he go through the intersection before the signal turns red without violating the speed limit.

There is an even worse possibility, which is realized for even shorter values of τ. This is the case where $x_0 < x_c$ and the slope da_1/dx is sufficiently large that the line represented by equation (5) intersects a line $a_1 = a_1^*$, where a_1^* is a maximum possible acceleration, at a point which has an abscissa x_a smaller than x_c. Then, for $x_a < x < x_c$, a driver cannot stop safely and he cannot clear the intersection before the initiation of the red light phase even if he is willing to utilize all the power resources of his car while violating the speed limit.

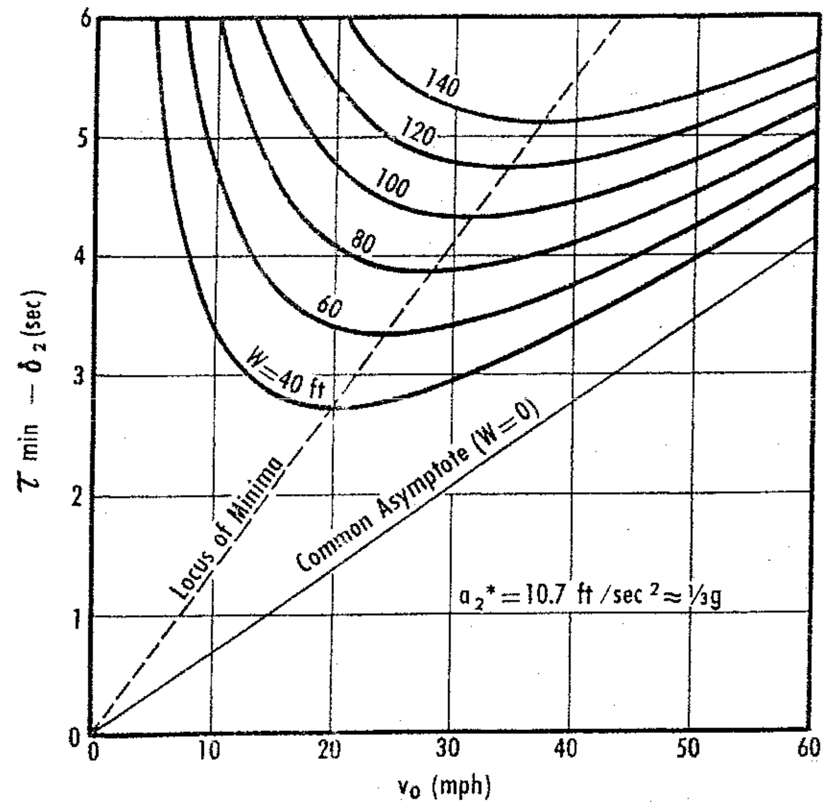

Fig. 4. Variation of the minimum amber-phase duration, τ_{min}, which is required in order that there be no dilemma zone, versus constant approach speed, v_0, for various intersection widths plus car length, W. (The constant deceleration is assumed to be 10.7 ft/sec^2.)

It may be pointed out that this maximum possible acceleration depends on the approach velocity v_0. It is well known that the higher the velocity of a car the lower its accelerating capability. Thus an average good car can have an acceleration of as much as $\frac{1}{2}\,g$ starting from rest, but only about 0.08 g when traveling at 65 mi/hr.† (Note that g is the earth's gravitational acceleration.)

Let us now discuss the design of the duration of the amber phase. From the graphical representation of Fig. 2, we see that the minimum

† We are indebted to Mr. Joseph Bidwell for furnishing us with the experimental data on the accelerating capability of a car as a function of its speed.

TABLE I

COMPARISON OF OBSERVED AND CALCULATED AMBER-PHASE DURATIONS

Street	Cross street	Speed limit (mi/hr)	Approximate effective width of intersection	Duration of amber phase	Theoretical τ_{min}: eq. (5)[a] $a_2^* = 10.7$ ft/sec², $\delta_2 = 1.14$ sec	$a_2^* = 10.7$ ft/sec², $\delta_2 = 0.75$ sec	$a_2^* = 16$ ft/sec², $\delta_2 = 1.14$ sec	$a_2^* = 16$ ft/sec², $\delta_2 = 0.75$ sec
South of Main	Catalpa	25	60	2.7[b]	4.91	4.52	4.33	3.94
North on Mound	Chicago	30	75	3.4	5.25	4.86	4.56	4.17
East on Chicago	Van Dyke	30	80	4.0	5.36	4.97	4.67	4.28
North on Woodward	Calvert	30	—	3.6	—	—	—	—
East on 11 Mile	Van Dyke	35	55	3.4	4.90	4.51	4.10	3.71
West on 14 Mile	Southfield	35	60	6.8	5.00	4.61	4.20	3.81
South on Woodward	9 Mile	35	80 to	4.5	5.39	5.00	4.59	4.20
			120		6.16	5.77	5.36	4.97
North on Woodward	Savannah	35	65	3.85	5.10	4.71	4.30	3.91
North on Mound	13 Mile	40	50	3.6	5.00	4.61	4.09	3.70
West on Chicago	Van Dyke	40	80	4.0	5.51	5.12	4.60	4.21
West on 8 Mile	Ryan	40	70	3.9	5.34	4.95	4.43	4.04
North on Van Dyke	12 Mile	40	80	4.1	5.51	5.12	4.60	4.21
East on 12 Mile	Van Dyke	45	65	4.0	5.44	5.05	4.41	4.02
North on Woodward	11 Mile	45	80	3.44	5.67	5.28	4.64	4.25
North on Woodward	Lincoln	45	75	3.75	5.59	5.20	4.56	4.17
South on Van Dyke	Chicago	50	70	3.8	5.74	5.35	4.60	4.21

[a] Two values of the time lag δ_2 were assumed. One of them is the observed average 1.14 sec and the other a lag of 0.75 sec frequency assumed as a minimum. A car length was taken as 15 ft to be conservative. Two values for the maximum deceleration a_2^* were assumed. One of them is equal to $\frac{1}{3}g$ which is feasible but is a fairly high deceleration not desirable in normal driving. The other one is equal to $\frac{1}{2}g$, which corresponds to a very hard stop. (Note that $0.6g$ is about the absolute maximum deceleration under ideal conditions.)

[b] The amber phase here was measured at about 2.1 sec prior to a modification in the signal cycle. We have been informed of an even shorter amber phase of only about 1.5-sec duration at an intersection in California where an individual received a ticket for being in this intersection on the red signal.

amber-light duration, denoted by τ_{min}, which guarantees the safe execution of either one of the alternatives of stopping or going through the intersection without accelerating, corresponds to $x_0 = x_c$. Hence

$$\tau_{min} = (x + w + L)/v_0, \tag{8}$$

and, using equation (4),

$$\tau_{min} = \delta_2 + \tfrac{1}{2} v_0/a_2^* + (w+L)/v_0. \tag{9}$$

2

A simple numerical example will show the magnitude of the quantities involved. Assuming $v_0=45$ mi/hr$=66$ ft/sec, $a_2^*=0.5$ $g\approx 16$ ft/sec^2, $\delta_2=1$ sec, $w=65$ ft, and $L=15$ ft, we find $x_c=202$ ft and $\tau_{\min}=4.28$ sec.

It may be noted that the length of the car, L, is added to the effective width of the intersection, w, in order to determine the length of travel through the intersection. The length of the car contributes the quantity

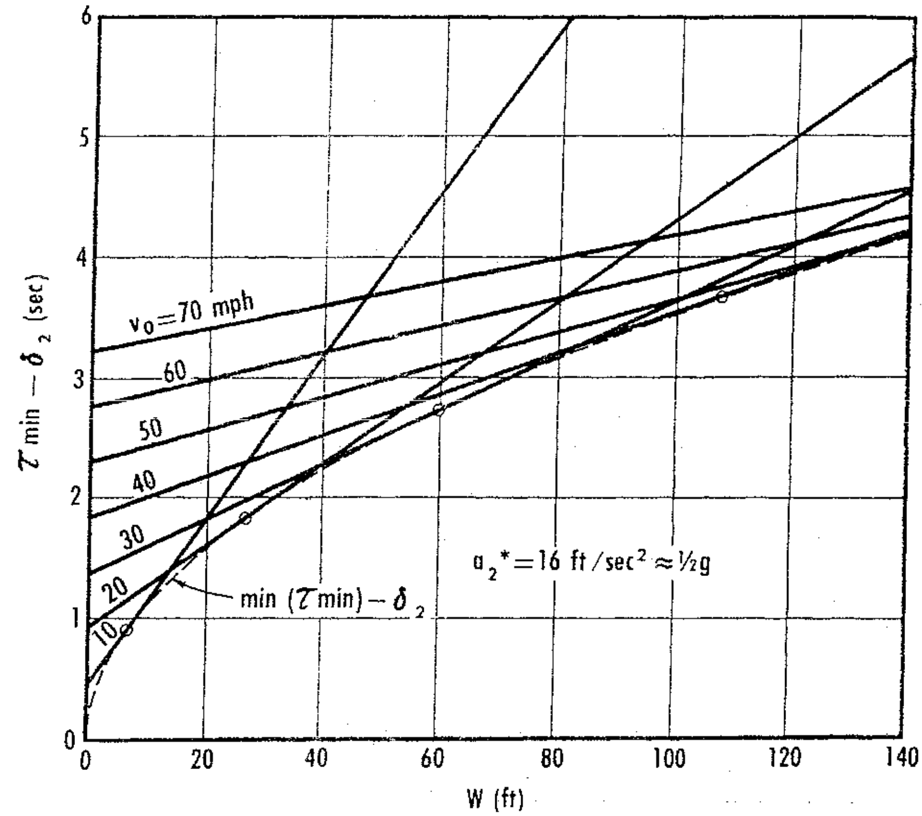

Fig. 5. Variation of the minimum amber-phase duration, $\tau_{\min}$, which is required in order that there be no dilemma zone, versus the intersection width plus car length, W, for various values of the constant approach speed, v_0. (The constant deceleration is assumed to be 16 ft/sec^2.)

L/v_0 in the computation of $\tau_{\min}$. This means that the required $\tau_{\min}$ is substantially longer for vehicles such as long trucks, buses, or vehicles with trailers, even assuming that these vehicles can stop with the same maximum deceleration a_2^* as shorter ones. One may retort that traffic signals should not be designed for these 'unusual' cases. However, these unusual vehicles *are* allowed on the highways, and if the design of the amber phase does not take them into account then the questions raised in the introduction regarding the compatibility of law and physical characteristics become even more acute.

Returning now to the expression for $\tau_{\min}$ given in equation (9), we use this result to plot τ_{min} versus v_0 in Figs. 3 and 4 for various values of the parameter

$$W = w + L \tag{10}$$

and for two values of the maximum deceleration a_2^*, namely, $\frac{1}{2}\,g$ and $\frac{1}{3}\,g$. (For comments on the magnitude of these decelerations, see the first

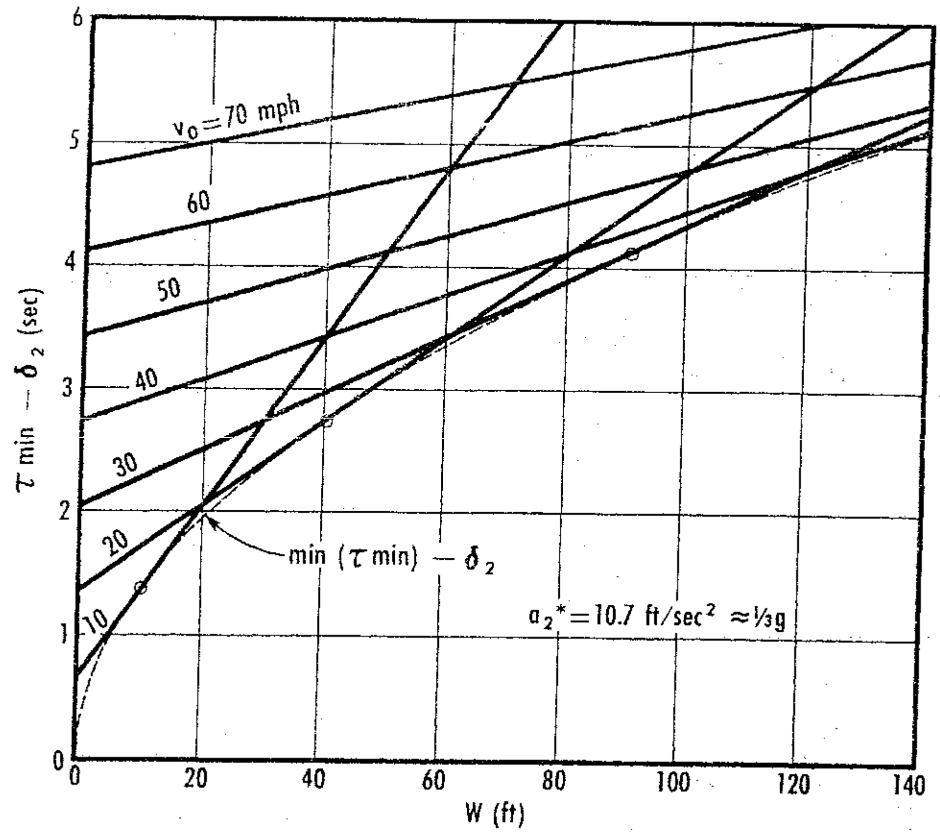

Fig. 6. Variation of the minimum amber-phase duration, τ_{min}, which is required in order that there be no dilemma zone, versus the intersection width plus car length, W, for various values of the constant approach speed, v_0. (The constant deceleration is assumed to be 10.7 ft/sec.)

footnote in Table I, as well as reference 2, p. 68.) The minima of the various curves correspond to values of the approach velocity v_0, assumed equal to the speed limit, which would minimize τ_{min} for a given value of W. From equation (9) we have

$$\partial\tau_{min}/\partial v_0 = 1/2a_2^* - W/v_0^2, \tag{11}$$

and $\partial\tau_{min}/\partial v_0 = 0$ for

$$v_0 = \sqrt{2\,a_2^*\,W}. \tag{12}$$

Hence the absolute minimum length of the amber phase is given by

$$\min(\tau_{min}) = \delta_2 + \sqrt{2\,W/a_2^*}. \tag{13}$$

Figures 5 and 6 contain plots of $(\tau_{\min}-\delta_2)$ versus W for different values of the approach velocity v_0, and for the same two values of a_2^* as in Figs. 3 and 4. Equation (9) yields a family of straight lines in the plane $(\tau_{\min}-\delta_2)$ versus W. The envelope of these lines corresponds to $\min(\tau_{\min})$ as given by equation (13).

The foregoing discussion is illustrated in Fig. 7, where each of the two shaded zones precludes one of the two alternatives of stopping or going through the intersection. Thus, a car at a distance from the intersection smaller than x_c cannot stop safely, whereas a car at a distance greater

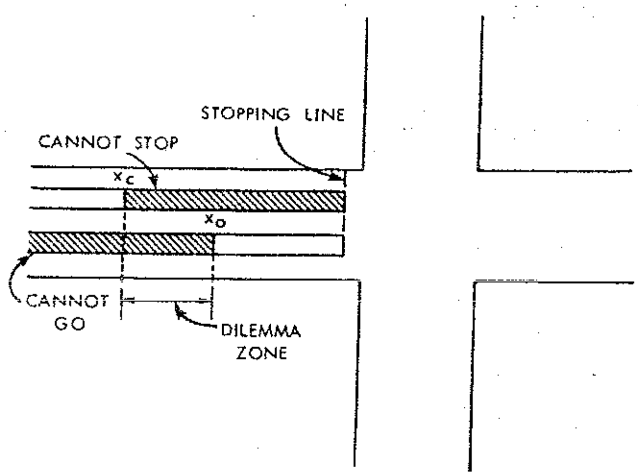

Fig. 7. Schematic diagram showing the 'dilemma zone' near an intersection.

than x_0 cannot go through the intersection without accelerating before the light turns red.

As mentioned already, when $x_0<x_c$ the driver is in trouble if he finds himself in the region $x_0<x<x_c$, which in the sequel will be referred to as the 'dilemma zone.'

The preceding arguments have been established on the assumption that the approach speed of the motorist is equal to the speed limit so that he cannot accelerate to clear the intersection without exceeding the speed limit. It is possible, however, that even if the amber phase is improperly set so that a dilemma zone exists for an approach speed equal to the speed limit, a motorist may, under certain circumstances, avoid encountering such a dilemma zone if his approach speed is smaller than the speed limit. This is so because the critical distance, x_c, decreases rapidly as the approach speed decreases. On the other hand, if the driver is at a distance from the intersection slightly larger than this reduced x_c when the amber-light phase begins he may be able, under certain circumstances, to clear the intersection within this phase by accelerating until he has reached the speed limit and then proceeding through the intersection at this speed. An example of this case is illustrated in Fig. 10, which is discussed a little later.

If we assume that the driver's acceleration from v_0 to v_l (the speed

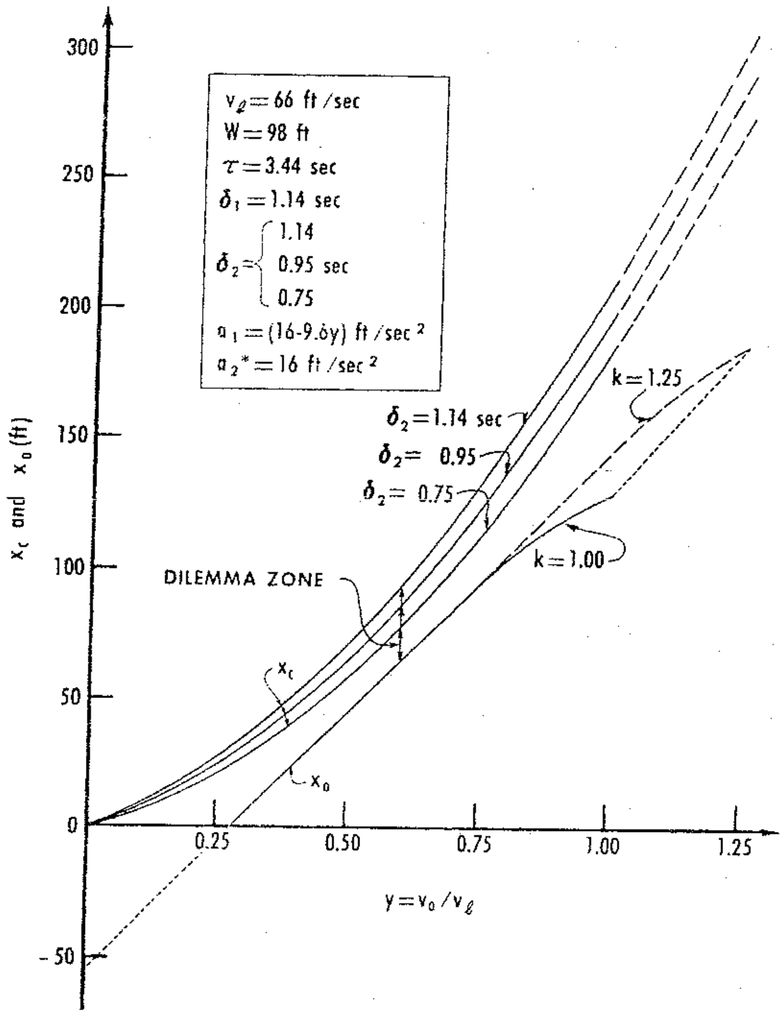

Fig. 8. Northbound on Woodward Avenue at 11 Mile Road. Variation of the critical distance, x_c, and the maximum distance which can be covered within the amber phase duration, x_0, versus the ratio of the approach speed to the speed limit, $y = v_0/v_l$. It is assumed that in crossing the intersection the car may accelerate up to a speed not in excess of kv_l.

limit) is constant and equal to a_1, the equation which replaces equation (2) is

$$x_0 = \begin{cases} v_0\,\delta_1 - W + (v_l^2 - v_0^2)/2a_1 + v_l\,[\tau - \delta_1 - (v_l - v_0)/a_1] \\ \qquad\qquad\qquad\qquad \text{for} \quad \tau \geqq \delta_1 + (v_l - v_0)/a_1, \\ v_0\,\delta_1 - W + v_0\,(\tau - \delta_1) + (\tfrac{1}{2}\,a_1)\,(\tau - \delta_1)^2 \\ \qquad\qquad\qquad\qquad \text{for} \quad \tau \leqq \delta_1 + (v_l - v_0)/a_1, \end{cases} \tag{14}$$

where W is given by (10) and x_0 is the distance of the car from the intersection at the moment the amber phase commences. It is assumed that the car just clears the intersection before the light turns red. Rewriting (14) to give x_0 as a function of $y=v_0/v_l$, where $0\leqq y\leqq 1$, we obtain

$$x_0=\begin{cases} -W+v_l\,\tau-v_l\,\delta_1\,(1-y)-(v_l^2/2a_1)(1-y)^2 & \\ \qquad\qquad\qquad\qquad\text{for}\quad \tau\geqq\delta_1-(v_l/a_1)(1-y), & \\ -W+\tfrac{1}{2}\,a_1(\tau-\delta_1)^2+v_l\,\tau y \qquad \text{for}\quad \tau\leqq\delta_1-(v_l/a_1)(1-y). & \end{cases} \tag{15}$$

Equation (1) remains unchanged, so that

$$x_c=\delta_2\,v_l\,y+(v_l^2/2a_2^*)\,y^2. \tag{16}$$

For simplicity we assume that $\delta_1=\delta_2=$ 1.14 sec (*see* the following section), while $a_2^*=\tfrac{1}{2}\,g=16$ ft/sec^2. The (constant) acceleration a_1 is, however, a function of the car speed at the moment when the car begins to accelerate. An analytic expression for this speed dependence of a_1, which fits the experimental data adequately enough for our purposes is

$$a_1(v_0)=\begin{cases}(16-0.145\,v_0)\ \text{ft/sec}^2 & \text{for}\quad 0\leqq v_0\leqq 110\ \text{ft/sec},\\ 0 & \text{for}\quad v_0\quad >110\ \text{ft/sec},\end{cases} \tag{17}$$

where v_0 is given in ft/sec. We assume, for simplicity, that a car traveling at an approach speed v_0 can maintain a constant acceleration a_1, as given by equation (17), for a length of time of the order of τ. It should be noted that there are marked differences in the dynamic characteristics of various cars with regard to acceleration. The preceding equation gives an acceleration which is on the high side and is applicable to the high-powered modern car. Low-powered cars develop considerably lower accelerations, particularly at high speeds. If one were to assume lower accelerations, the problem of the dilemma zone would be accentuated.

Using equations (15), (16), and (17), we have plotted x_0 and x_c as functions of y for three different intersections in Figs. 8, 9, and 10.

The curve for x_0 has a straight segment, corresponding to the second expression in (15), and a curved segment corresponding to the first expression. These two segments are tangent at the point y_t satisfying the equation

$$1-y_t-(\tau-\delta_1)(a_1/v_l)=0. \tag{18}$$

Hence, in view of (17), we have

$$y_t=[1-16\,(\tau-\delta_1)/v_l]/[1-0.145\,(\tau-\delta_1)], \tag{19}$$

where speeds are given in ft/sec and times in seconds.

From Fig. 7 we see that there is no dilemma zone if $x_0 > x_c$; of the situations depicted in Figs. 8–10 we see that in only one case, namely, that shown in Fig. 10, is there an absence of a dilemma zone, and this is so only for $0.15 < y < 0.57$. This means that at this particular intersection a car traveling at the speed limit of 65 mi/hr would encounter a dilemma zone of 106 ft, approximately six car-lengths, at a distance of 286 ft from the

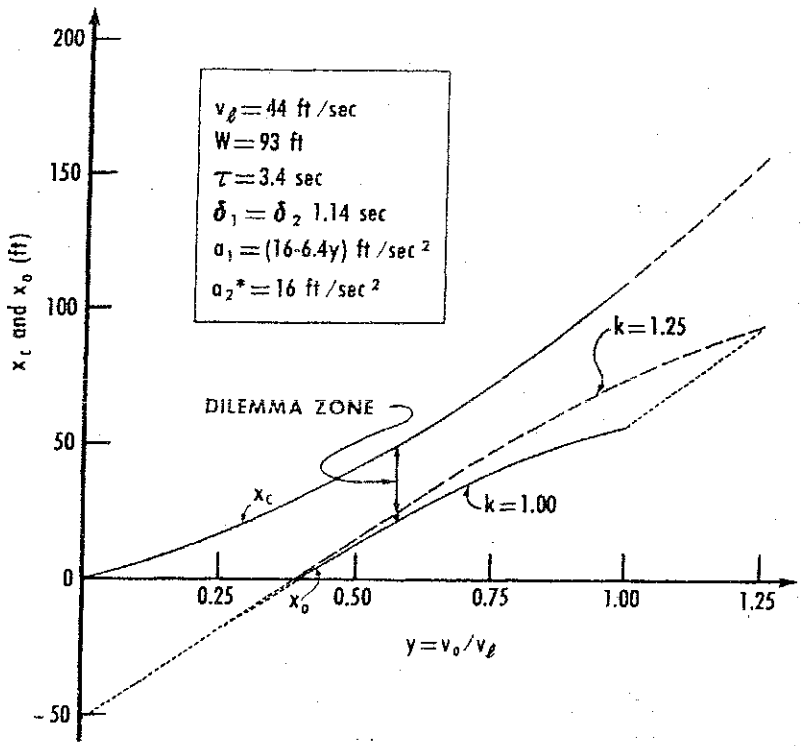

Fig. 9. Northbound on Mound Road at Chicago Road. Variation of the critical distance, x_c, and the maximum distance which can be covered within the amber-phase duration, x_0, versus the ratio of the approach speed to the speed limit, $y = v_0/v_l$. It is assumed that in crossing the intersection the car may accelerate up to a speed not in excess of kv_l.

intersection. On the other hand, if the speed of the car is 37 mi/hr or lower, no such zone exists. It need hardly be pointed out that under ordinary driving conditions a speed of 37 mi/hr on a highway with a 65 mi/hr-maximum is unrealistic, and quite possibly dangerous.

From the preceding discussion we ascertain that if one were to assume, for low-powered cars, accelerations lower than those given by (17), the values of x_0 would be reduced considerably and the dilemma zones increased in the entire range $0 \leqq y \leqq 1$.

Approaching an intersection at a speed lower than the speed limit is one facet of defensive driving. It is seen from the preceding discussion that this in itself is not always sufficient to obviate the dilemma-zone problem. Another facet of such defensive driving consists of the maneuver of coasting toward the signal light with one's foot readied on the brake. The advantage, in this case, which comes from shortening the reaction

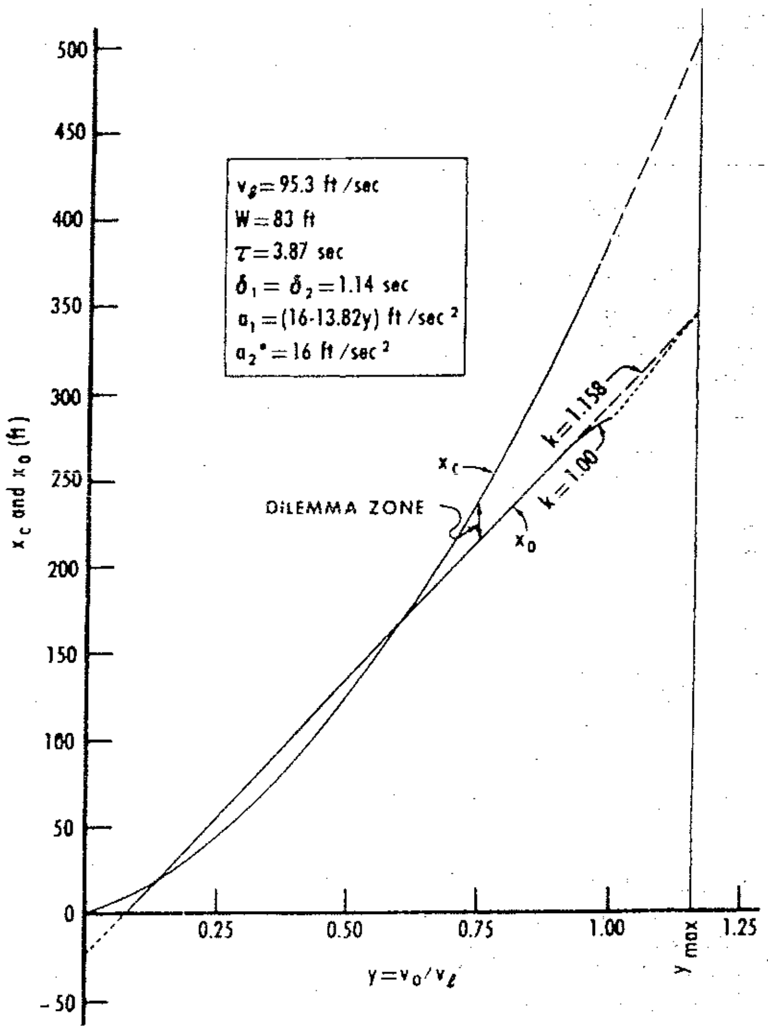

Fig. 10. Northbound on Stephenson Highway at 15 Mile Road. Variation of the critical distance, x_c, and the maximum distance which can be covered within the amber-phase duration, x_0, versus the ratio of the approach speed to the speed limit, $y = v_0/v_l$. It is assumed that in crossing the intersection the car may accelerate up to a speed not in excess of kv_l. The value of $k = 1.157$ corresponds to the maximum speed of 110 ft/sec according to equation (17).

time, is reflected in a decrease of the critical distance x_c. The improvement, which is by no means an absolute cure, can be seen from the curves plotted in Fig. 8 for two values of δ_2 other than the observed average. Such defensive driving, however, should be used with discrimination and great caution when approaching intersections in a high-density traffic pattern since it may induce a rear-end collision—a prominent type of accident in traffic today.

Many drivers take the attitude that there is nothing sacred about the speed limit! Suppose one, starting with an initial speed $v_0=yv_l$, where v_l is again the official speed limit, accelerates to a final speed equal to or less than v_l' given by

$$v_l'=kv_l. \qquad (k>1) \quad (20)$$

The analysis already carried out can be applied to this case on the assumption that the 'effective speed limit' is $v_l'=kv_l$ and the initial speed

$$v_0=y'v_l'=(y/k)\ v_l'. \qquad (0\leqq y'\leqq 1) \quad (21)$$

The x_c versus y curve obviously does not change. The ordinate of the x_0 versus y curve at $y'=1$, i.e., at $y=k$, is

$$x_0^*=-W+v_l\,\tau k. \qquad (22)$$

In Figs. 8 and 9 we have plotted with dashed lines the curves of x_0 corresponding to 'effective speed limits' equal to 1.25 v_l (i.e., $k=1.25$). Similarly in Fig. 10 we have plotted with a dashed line the curve of x_0 for $k=1.158$. This value of k corresponds to an 'effective speed limit' equal to the assumed maximum possible speed of 110 ft/sec (75 mi/hr), according to equations (17). Again, these curves are made up of two segments, one straight and one curved, which are tangent at the point

$$y_t'=[k-16\ (\tau-\delta_1)/v_l]/[1-0.145\ (\tau-\delta_1)]. \qquad (23)$$

The straight segment is an extension of the one already plotted on the basis of the second expression in (15), which is independent of the effective speed limit.

From these figures we see that even if the driver is willing to accelerate to speeds greatly in excess of the speed limit, he still cannot eliminate the dilemma zone.

With regard to the length of the dilemma zone, the following additional remark can be made on the basis of the preceding discussion. If a driver encounters a dilemma zone, the maximum possible distance of the rear bumper of his car from the clearing line of Fig. 1 at the moment the red phase commences is equal to the length of the dilemma zone. This maximum distance is realized if the driver is just past x_c when the amber phase commences. Now, if the indecision zone is greater than the effective width of the intersection plus the car length, W, the driver may even have to *enter* the intersection during the red phase. From Fig. 10 it is seen that this may happen, at the intersection under consideration, to a driver who approaches the intersection at the speed limit and does not want to exceed this limit, since in this case the dilemma zone of 106 ft is greater than $W = 83$ ft.

OBSERVATIONS

In order to compare the theoretical results of the preceding section with physical reality the following kinds of observations were carried out on

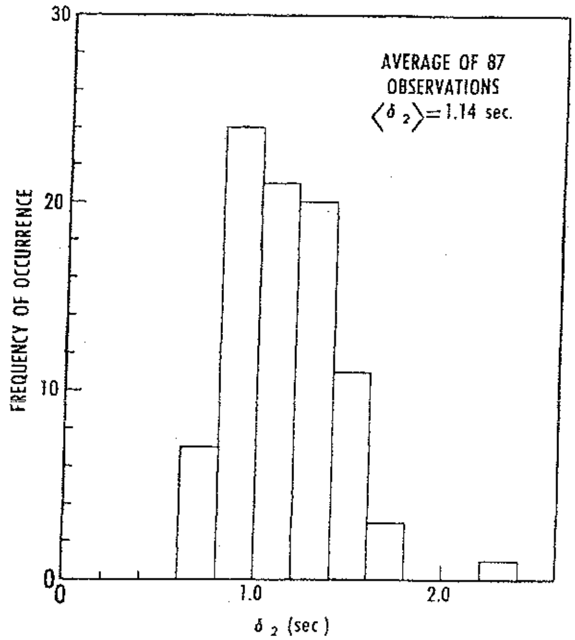

Fig. 11. Histogram showing the observed frequency of occurrence of various intervals of decision and reaction time in braking, δ_2, in a total of 87 measurements.

the manner in which people actually drive and the pattern in which amber signal light phases are in practice set:

1. Duration of amber-light phase.
2. Motorists' braking reaction time (including the decision time and the reaction time lag).
3. Average number of motorists per cycle who run through the red light.
4. The dimensions of the road and intersection together with the posted speed limit.
5. Traffic density.
6. The effective critical distance x_c.

Most of the observations were made at street intersections within about a fifteen-mile radius of the General Motors Technical Center. It was not our intention to make our data exhaustive, but we feel that enough measurements were made so that fairly definite conclusions based on them could be drawn.

We begin by presenting in Table I a sampling of the data obtained on amber-signal-light times, speed limits, and approximate intersection widths, at a number of intersections, together with theoretical values of the minimum amber-light phase, $\tau_{\min}$, calculated from equation (8) using two values of the maximum deceleration and two values of the braking reaction time.

TABLE II

OBSERVED AND CALCULATED CRITICAL DISTANCE, x_c

Street	Cross street	Speed limit	Effective x_c	Theoretical x_c ($a_2^* = 0.5\ g$)
North on Woodward Avenue	Lincoln	45 mi/hr	165 ft	211 ft
West on 8 Mile Road	Ryan	40	145	174
North on Woodward Avenue	11 Mile Road	45	185	211

In measuring the drivers' braking reaction time, an observer was stationed near a given intersection at a distance somewhat greater than the estimated x_c. The observer would then arbitrarily choose a car in the interval between himself and the intersection and would measure the time

TABLE III

TRAFFIC FLOW AND PER CENT TRAFFIC-LIGHT VIOLATIONS

Street	Cross street	Number cars in intersection per cycle	Average number cars running through red signal per cycle	Per cent of cars running through red signal	Amber phase (sec)
North on Woodward Avenue	Lincoln	62.3	1.2	1.93	3.75
		53.8	0.8	1.49	3.73
West on 8 Mile Road	Ryan	42.1	0.7	1.66	3.9
North on Woodward Avenue	11 Mile Road	54.5	1.2	2.20	3.49
North on Woodward Avenue	Woodland	91.6	0.5	0.55	4.23
North on Woodward Avenue	Sylvan	95.1	0.1	0.11	4.69
North on Woodward Avenue	Webster	46.1	0.4	0.87	3.67

interval between the moment the amber signal came on and the moment when the red brake tail light flashed. The distribution of such delay times is plotted in Fig. 11 on the basis of 87 observations. The mean delay time was found to be 1.14 seconds.

The determination of an average effective x_c was carried out using the following criterion: it is the closest distance at which a car can be from the intersection, when the amber signal commences, and still be capable of stopping before entering the intersection. Measurements of this quantity were made at several intersections and the results are shown in Table II together with the theoretical values calculated from equations (4). The observed x_c was in general a little smaller than the theoretical x_c corresponding to the speed limit of the observed intersections. This was probably due to the fact that the traffic was moving, on the average, a little slower than the posted speed limit, since our observations were made during the heavy traffic of the rush hour.

Finally, we measured at a few intersections the average number of cars that ran through the red signal per signal light cycle during rush hour traffic (4:30–6:00 P.M.), together with the average number of cars that pass through the intersection per signal light cycle. These results are shown in Table III.

The preceding pertains to a single traffic light. Analogous results may be obtained for two closely spaced traffic lights, as in the case of crossing of a divided highway. However, this case is rather complicated and will not be discussed here. There are other variations to the problem of the dilemma zone such as the case of a vehicle approaching an intersection at slow speed with the intention of making a turn. This is a case of known practical difficulty and some information can be obtained from the present analysis with w taken equal to the distance traversed while turning.

Some additional data regarding the amber-light phase were obtained from three other cities, namely, Washington, D. C., Silver Spring, Maryland, and Los Angeles, California. On the average, the amber-light phases were slightly shorter in Los Angeles and slightly longer in the Washington, D. C., area, relative to those in the Detroit area. There are no significant differences, and the conclusions of this paper will apply in those areas also.

DISCUSSIONS AND CONCLUSIONS

THE *Uniform Vehicle Code* of the National Committee on Uniform Traffic Laws and Ordinances[3] gives the following definition for the purpose of the amber signal light:

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

Most of the traffic ordinances throughout the United States that we have seen have followed this definition with slight variations such as the omission of the phrase "or be crossing (the intersection). . . ." Some ordinances make an attempt to provide an operational definition of the meaning of the amber signal with definite instructions to the driver on how to behave. A typical example of such an ordinance is the following:

Vehicular traffic facing the signal shall stop before entering the nearest crosswalk at the intersection, but if such stop cannot be made in safety, a vehicle may be driven cautiously through the intersection.

Both definitions, of course, assume that the signal has been designed properly so that the driver can behave as directed and in general can solve the decision problems he encounters. It is interesting to note that the *Manual on Uniform Traffic Control Devices for Streets and Highways*[4] makes the following statement:

Confusion has frequently arisen from the misuse of this yellow lens. When the length of yellow vehicle-clearance interval is correct, and the standard meaning above described† is generally observed, necessary functions of warning and clearing the intersection are performed by this interval.

This is a reasonable statement to which we, of course, subscribe. Our investigations show, however, that out of approximately 70 intersections studied, only one had an amber phase long enough to prevent an appreciable dilemma zone, i.e., a zone longer than about one car-length, if one assumes a 'comfortable' deceleration of $\frac{1}{3}$ g and a decision and reaction time-lag equal to our measured average of 1.14 sec. Even if one assumes the very large deceleration of $\frac{1}{2}$ g and a decision-reaction time lag of 0.75 sec, only four out of the 16 typical intersections of Table I yield a dilemma zone smaller than one car-length. Out of these four, one, namely the sixth zone in Table I, gives no dilemma zone at all and is the only such intersection observed in the area.

The fact that almost all the intersections have sizeable dilemma zones is reflected in the data of Table III, which indicate that at the intersections studied as many as two cars went through the red light per light cycle, with an average of close to one car per cycle. It is true that in none of the observed cases did there appear to be any distinct possibility of an accident. However, the fact remains that an average of eleven out of every thousand cars were very much in the middle of the intersection when the red signal started, in violation of the *Uniform Vehicle Code*. This leaves them open to the possibility of receiving a traffic citation from an assiduous police officer. We might mention here that we were rather surprised to discover a traffic ordinance that made no distinction whatsoever between the yellow and red lights. The instruction regarding both was that "Vehicular traffic facing the signal shall stop before entering the nearest crosswalk at the intersection," a requirement which is clearly impossible to obey under many circumstances. It is interesting to note that in a state-issued driver-instruction pamphlet we again find that the amber and red lights are inter-

† The standard meaning referred to is precisely that quoted above as due to the *Uniform Vehicle Code* of the National Committee on Uniform Traffic Laws and Ordinances.[4]

preted alike without regard to the operational problems considered here. The same pamphlet instructs the drivers to "... drive a reasonable speed which will allow me to stop when the amber light comes on." The analysis given in this paper clearly shows that even reduction of speed and defensive driving when approaching an intersection does not necessarily eliminate the dilemma zone problem if the amber phase is inadequate.

The problem of determining the proper duration of the amber phase of the light cycle is perhaps more difficult and complicated than may appear at first sight. In this connection we quote MATSON, SMITH, AND HURD:[1] "In urban areas where speeds are relatively low, yellow lights of about 3-sec duration are satisfactory at most locations. At rural, high-speed locations where stopping time may have a duration of 5 to 8 sec, road users tend to attempt to clear the intersection rather than stop. Five seconds is probably a practical maximum yellow duration in such location."

We are aware of the fact that traffic engineers are inclined to shorten the amber phase for various reasons. One of them, probably one of the most important ones, is their conviction, undoubtedly substantiated, that drivers are inclined to ignore a long amber phase and treat it as merely a continuation of the green phase. They believe that as many drivers, if not more, will go through the red light when the amber phase is too long, as will do so if it is too short. However, we believe that it is the duty of the traffic engineers and the drafters of traffic ordinances to present the average, honest, driver with a solvable decision problem. As it stands now, a driver who is in the middle of an intersection when the red light comes on may not be a deliberate violator, but may be the victim of an improperly designed light cycle. It is true that accidents are in general prevented because of some delay of approach of the cross traffic and also by the judicious use of overlapping red cycles. This fact, however, does not release the unwilling violator from the legal responsibility which may become alarming in the case of an accident. On the other hand, with an adequate amber phase it would be easier to separate the violators from the nonviolators, insofar as traffic is concerned.

We believe that a correct resolution of this problem may be found in one of the following alternatives:

1. Design the amber-light phase according to some realistic criteria in order to guarantee that a driver can always be in a position to obey the law.

2. If the amber-light phases are to be kept short relative to criteria such as determined herein, it may be desirable to state the vehicle code in such a way as to make it compatible with the driver, car, road, and signal characteristics.

In either case it would be very advisable to educate both the driving public and the law-enforcing agencies as to the exact operational definition of the amber light. Needless to say, the fewer the variations of traffic

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ordinances in this respect, from one locality to another, the fewer the chances of confusion. We wish to re-emphasize our hope that a well-thought-out and operationally sound traffic and enforcement system, together with the healthy driver attitudes of a properly educated public, will promote safer and more efficient driving conditions

ACKNOWLEDGMENTS

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EXHIBIT

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PLAINTIFF'S DEPOSITION EXHIBIT

PENGAD 800-631-6989

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Discussion Paper No. 8.A

STOPPING SIGHT DISTANCE AND DECISION SIGHT DISTANCE

prepared for

Oregon Department of Transportation
Salem, Oregon

by the

Transportation Research Institute
Oregon State University
Corvallis, Oregon 97331-4304

February 1997

Discussion Paper No. 8.A

STOPPING SIGHT DISTANCE AND DECISION SIGHT DISTANCE

PERCEPTION-REACTION TIMES

PIEV Process

The perception-reaction time for a driver is often broken down into the four components that are assumed to make up the perception reaction time. These are referred to as the PIEV time or process.

PIEV Process	
• Perception	the time to see or discern an object or event
• Intellection	the time to understand the implications of the object's presence or event
• Emotion	the time to decide how to react
• Volition	the time to initiate the action, for example, the time to engage the brakes

Current Design Perception-Reaction Time

Human factors research defined perception-reaction times for (1):

• design	2.5 sec
• operations/control	1.0 sec

These perception reaction times were based on observed behavior for the 85th percentile driver; that is, 85% of drivers could react in that time or less. More recent research has shown these times to be conservative for design.[2]

Wortman and Mathias (2) reported both the "surprise" and alerted 85th percentile perception reaction times. This was in an urban environment; the time was measured after the yellow indication until brake lights appeared.

The Wortman et al. research found:

• alerted 85% perception-reaction time	0.9 sec
• "surprise" 85% perception-reaction time	1.3 sec

(1) AASHTO, "Policy on Geometric Design of Streets and Highways," Washington, DC, 1984, 1990, and 1994.

(2) Wortman, R.H., and J.S. Matthaas, "Evaluation of Driver Behavior at Signalized Intersections," Transportation Research Record 904, T.R.B, Washington, D.C., 1983.

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STOPPING SIGHT DISTANCE AND DECISION SIGHT DISTANCE

PERCEPTION-REACTION TIMES (Continued)

Perception-Reaction Time Research

Recent studies have checked the validity of 2.5 seconds as the design perception reaction time. Four recent studies have shown maximums of 1.9 seconds as the perception-reaction time for an 85th percentile time and about 2.5 seconds as the 95th percentile time.

Brake Reaction Times Studies

	85th	95th
Gazis et al. (1)	1.48	1.75
Wortman et al. (2)	1.80	2.35
Chang et al. (3)	1.90	2.50
Sivak et al. (4)	1.78	2.40

Perception-Reaction Times by Road Type

Some researchers have suggested that the perception-reaction should reflect the complexity of traffic conditions, expectancy of drivers and the driver's state. They suggest that the perception reaction times may be altered accordingly (4).

Table 1. Perception-Reaction Times Considering Complexity and Driver State

	Driver's State	Complexity	Perception-Reaction Time
Low Volume Road	Alert	Low	1.5 s
Two-Lane Primary Rural Road	Fatigued	Moderate	3.0 s
Urban Arterial	Alert	High	2.5 s
Rural Freeway	Fatigued	Low	2.5 s
Urban Freeway	Fatigued	High	3.0 s

(1) Gazis, D.R, et al, "The Problem of the Amerber Signal in Traffic Flow," Operations Research 8, March-April 1960.

(2) Wortman, R.H., and J.S. Matthaas, "Evaluation of Driver Behavior at Signalized Intersections," Transportation Research Record 904, T.R.B, Washington, D.C., 1983.

(3) Chang, M.S, et al, "Timing Traffic Signal Change Intervals Based on Driver Behavior," T.R. Record 1027, T.R.B, Washington, D.C., 1985

(4) Sivak, M., et al, "Radar Measured Reaction Times of Unalerted Drivers to Brake Signals," Perceptual Motor Skills 55, 1982.

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HUMAN FACTORS

An appreciation and understanding of human factors, behavior and abilities are needed to determine the sight distance criteria. The physical abilities and psychological limitations impact these criteria, and should be reviewed here to obtain perspective.

Visual

AcuityThe primary stimulus for operation and safe control of vehicles is eye sight. The physical composition of the eye and its functioning constitute limits that must be considered when developing sight distance criteria.

Visual Acuity

3 -4 cone	best vision – can see texture, shape, size, color, etc.
10 cone	clear vision – critical traffic control devices must be in this cone
20 cone	satisfactory vision – regulatory and warning traffic control devices should be this cone of vision
~ 90 cone	peripheral vision – only movement can be seen with this vision

Drivers focus their attention down the roadway in the cone of clear vision at 3 to 4 times the stopping distance. They then shift their vision to the right and left to keep track of traffic conditions, pedestrians and local activities. The eye movement time includes the time required for a driver to shift their eyes and to focus on an object.

Eye Movement Time

Shift to New Position	0.15-0.33 sec
Fix or Focus on Object	0.1-0.3 sec

It takes roughly 0.5 second for a driver to shift his eyes and focus. Thus, a full cycle to right and back to the left takes about 1 second. If there is glare, it takes 3 seconds to recover full visual acuity and 6 seconds to recover from bright to dim conditions.

Human Mind is Single Channel

Humans are sequential processors; that is, drivers sample, select and process information one element at a time, though very quickly. Therefore, complex situations create unsafe or inefficient operations because it takes so long for drivers to sample, select and process the information. This means that as complexity increases a longer perception-reaction time should be available. The visual acuity limitations, visibility constraints of glare/dimness recovery and complexity of traffic conditions, when taken together, require much longer perception-reaction times or decision times.

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STOPPING SIGHT DISTANCE AND DECISION SIGHT DISTANCE

HUMAN FACTORS

Driver Expectancy

Drivers are led to expect a particular operation condition based on the information presented to them. They use both formal and informal information.

- Formal information – this includes the traffic-control devices and the geometric design features of the roadway, but does not include the roadside features such as ditch lines, guardrail, and other street furniture.
- Informal information – this includes roadside features and also land use features, such as brush lines, tree lines, fences and information signing.

Drivers develop expectations on how to drive a roadway through experience, training and habit. At times these expectations are in error because they use inappropriate informal information, or the formal information provided is not proper or gives mixed messages. Often, the information at a location is conflicting, and drivers who are familiar with the location will read traffic conditions differently than unfamiliar drivers. Traffic conditions vary dramatically on major facilities; consequently, the information that drivers receive from other vehicles is constantly changing.

Increased perception reaction time is needed to allow time for drivers to make the proper decision when information conflicts and driver expectancy may be in error.

Further, high volume and high speed conditions require longer decision times and compound any problems arising from driver expectancy.

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STOPPING SIGHT DISTANCE AND DECISION SIGHT DISTANCE

DECISION SIGHT DISTANCE

Decision Sight Distance Appropriate for Access Management

As indicated in the discussion of perception reaction time and stopping sight distance, there are many situations where stopping sight distance is not sufficient for safe and smooth operations. Complex conditions, problems of expectancy, high volumes and high speed require more time for the perception-reaction process. These conditions are present on arterial streets and highways, particularly in urban areas. The AASHTO Policy on Geometric Design has provided for such situations through the decision sight distance.

Distinction Between Stopping Sight Distance and Decision Sight Distance

The distinction between stopping sight distance and decision sight distance must be understood.

- Stopping sight distance is used when the vehicle is traveling at design speed on a poor wet pavement when one clearly discernable object or obstacle is presented in the roadway.
- Decision sight distance applies when conditions are complex, driver expectancies are different from the situation, or visibility to traffic control or design features is impaired.

Most situations presented on arterials for access management require stopping sight distance at a minimum; however, decision sight distance should be provided for safety and smoother operations.

AASHTO Decision Sight Distance

The decision sight distance as defined by the AASHTO Green Book is "the distance required for a driver to detect an unexpected or otherwise difficult-to-perceive information source or hazard in a roadway environment that may be visually cluttered, recognize the hazard or its threat potential, select an appropriate speed and path, and initiate and complete the required maneuver safely and efficiently." According to AASHTO, the decision sight distance requires about 6 to 10^s to detect and understand the situation and 4 to 4.5^s to perform the appropriate maneuver. The sight distance is typically measured from a 1070 mm (3.5 ft.) height of eye to 150 mm (6 in.) object; however, this should depend on the condition that requires the decision sight distance. A table showing the recommended decision sight distances for various maneuvers is given in Table 4.

STOPPING SIGHT DISTANCE AND DECISION SIGHT DISTANCE

DECISION SIGHT DISTANCE (Continued)

Table 4A. Decision Sight Distance (meters)

Design Speed (km/h)	Decision Sight Distance for Avoidance Maneuver, (meters) A	B	C	D	E
50	75	160	145	160	200
60	95	205	175	205	235
70	125	250	200	240	275
80	155	300	230	275	315
90	185	360	275	320	360
100	225	415	315	365	405
110	265	455	335	390	435
120	305	505	375	415	470

Table 4B. Decision Sight Distance (English units)

Design Speed (mph)	Decision Sight Distance for Avoidable Maneuver, (ft.) A	B	C	D	E
30	220	500	450	500	625
40	345	725	600	725	825
50	500	975	750	900	1025
60	680	1300	1000	1150	1275
70	900	1525	1100	1300	1450

*Note: Avoidance Maneuvers

1. Avoidance maneuver A: Stop on rural road
2. Avoidance maneuver B: Stop on urban road
3. Avoidance maneuver C:Speed/path/direction change on rural road
4. Avoidance maneuver D: Speed/path/direction change on suburban road
5. Avoidance maneuver E: Speed/path/direction change on urban road

Various operating conditions require different maneuvers in response to a situation. The perception-reaction times are shorter for the less complex rural conditions than for urban.

STATE OF NORTH CAROLINA

COUNTY OF WAKE

IN THE GENERAL COURT OF JUSTICE
SUPERIOR COURT DIVISION
10-CVS-019930

BRIAN CECCARELLI and LORI MILLETTE, individually and as class representatives,, **Plaintiffs,** **v.** **TOWN OF CARY** **Defendant.**	**CERTIFICATE OF SERVICE**

The undersigned hereby certifies that a copy of the foregoing RULE 59 MOTION FOR NEW TRIAL was served on the defendant's counsel in this action by depositing a copy of the same in the United States Mail, first-class postage prepaid, and addressed as follows:

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This the 4th day of March, 2013

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