The Influence of the Time Duration Of Yellow Traffic Signals On Driver Response

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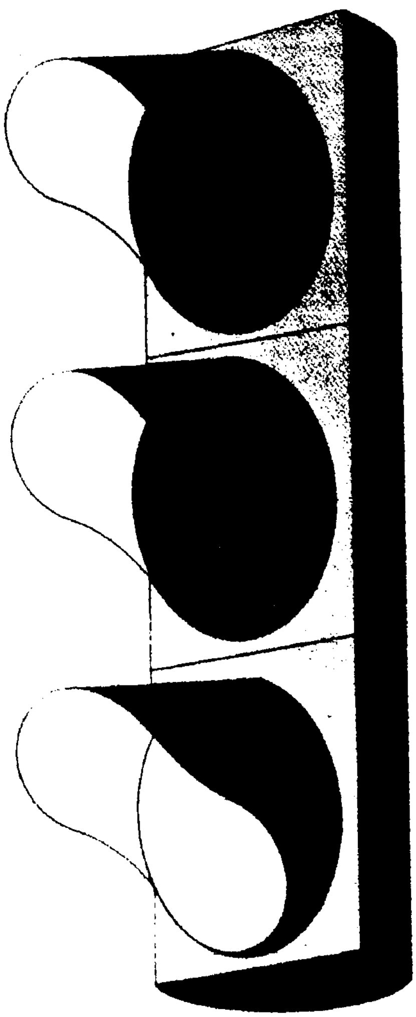
The yellow phase in traffic signal cycles at intersections is used to alert drivers of the imminent change in the direction of traffic flow. Upon observing the onset of yellow, the driver of a car approaching an intersection is faced with the choice of stopping or continuing through the intersection. Among drivers who continue, some will clear the intersection before the onset of the red phase in the cycle, but others will be unable to do so. Vehicles that have not cleared the intersection before red onset can be blocking the paths of cross-street traffic that has a green signal for varying periods of time. These vehicles are in potential conflict with cross-street vehicles and as a result may collide with them. This suggests that intersection crashes could be reduced by adjusting the traffic signal cycles to minimize the frequency of such conflicts. Specifically, changing the time duration of yellow signals (referred to as the change interval) should affect the frequency of potential conflicts.

To determine change intervals that keep the frequency of conflicts low, it is necessary to understand the response of drivers to the onset of yellow signals. The goal of the present study was to determine how intersection conflict frequency depended on yellow signal time duration at two suburban arterial intersections under different traffic and environmental conditions. The findings of this study apply to the common, pretimed traffic signals, not the more expensive traffic actuated signals.



REVIEW OF PREVIOUS RESEARCH

The concept of "dilemma zone" was introduced in a quantitative form by Gazis, et al.[1] in 1960. Gazis reported that if the yellow signal time duration is below a threshold value, some approaching drivers can neither stop nor clear the intersection before red signal onset. Such drivers are said to be in the dilemma zone. He calculated the dilemma zone boundary in terms of speed of travel and distance from intersection at yellow signal onset and calculated the yellow signal time duration threshold at which the dilemma zone disappears (Appendix A). Additionally, Gazis found that "out of approximately 70 intersections studied, only one had a yellow phase long enough to prevent an appreciable dilemma zone."

What drivers do when caught in the dilemma zone has been investigated in several research projects. Crawford and Taylor[2] observed driver decisions using eight subjects in repeated runs. In this experiment subjects faced onset of yellow at varying speeds (20–60 mph) and at varying distances from the traffic signal (50–350 ft.). There were no other vehicles interfering with driver decisions and yellow duration was fixed at 3 seconds. At given speeds, the percentage of drivers that stopped was found to increase linearly with the logarithms of distance from the intersection. Yellow intervals that would have produced greater percentages of drivers stopping were also estimated as a function of approach speed and intersection width (Appendix A).

In an observational study conducted in 1962, Olson and Rothery[3] determined the percentages of drivers stopping after yellow onset as a function of deceleration needed for stopping for several intersections. Comparing résults between pairs of intersections that had different yellow durations but were otherwise similar, these authors concluded that the percentages were not affected by length of change interval. A formula for determining duration of yellow in terms of approach speed, intersection width and "distance from intersection at which desired percentile cutoff (for stopping probability) occurs" was also derived (Appendix A). Olson and Rothery[4] repeated some of their observations in 1972 and found that the percentages of drivers stopping increased at one and decreased at the

Appendix A. Proposed timing formulae for yellow duration

Source	Date	Formula[2]	Comment
1. Gazis, et al.[1]	1960	$t + \frac{1}{2}\frac{V}{a} + \frac{W+L}{V}$	Velocity and deceleration assumed same for all drivers.
2. Crawford and Taylor	1961	$0.68\left[\frac{W}{V} + KV^{3/5}\right]$	Constant K depends on proportion of responses; value of K obtained from one experiment and was not further validated.
3. Olson and Rothery	1962	$(A + W + L)/V$	No explicit rule for determining A is given.
4. Olson and Rothery	1972	5.5 sec.	". . . [A]mber periods of about 5.5 seconds are realistic . . . [they] provide a clearing time that allows all or nearly all motorists to clear an intersection." The limits of the applicability of the recommendation are unclear.
5. MUTCD	1971	3 – 6 sec.	Too unspecific to be useful.
6. Williams	1977	$t + \frac{V}{2a^{-}_{.85}} + \frac{W+L}{V_{.85}} - \left[t^{*} + \sqrt{\frac{2d}{a^{+}}}\right]$	Cross street start-up and acceleration time is subtracted; traffic and pavement conditions are not accounted for.

(1) *Transportation and Traffic Engineering Handbook* 7 cites same formula.
(2) Definition of Symbols:

t, t^{*}	=	Driver reaction times for stopping and starting
$V, V_{.85}$	=	Mean approach speed, 85th percentile of approach speed
W	=	Intersection width
L	=	Vehicle length
A	=	Distance from stop line at which desired percentile for stopping occurs
$a, a^{-}_{.85}, a^{+}$	=	Deceleration rate, 85th percentile of deceleration accepted, maximum acceleration of cross-flow traffic.
d	=	Distance between vehicle and cross-flow stopline

other intersection during the intervening decade.

In another observational study conducted in 1968 by May[5], the percentages of drivers stopping as a function of approach speed and distance from intersection were determined for two yellow settings (initial and extended) at two intersections (1 urban, 1 rural) both with and without supplementary advance signing and additional pavement markings. One of the findings of this study was that the number of cars entering the intersection after red onset was reduced when yellow duration was extended. However, both the initial and the reduced conflict frequencies were very small.

Stop-go decisions of drivers free to cross an urban intersection (i.e., drivers whose paths were not blocked by other stopping vehicles) were studied by Williams in 1977[6]. Logarithmic relationships between approach speed and stopping distance for constant percentages of drivers stopping were found to describe the data adequately. A yellow duration formula was proposed in terms of deceleration rate accepted by preassigned driver percentage, maximum acceleration rate of cross-flow traffic, and other variables (Appendix A).

These various studies suggest that the percentage of drivers stopping depends on their approach speed and distance from the intersection when the signal changes. They also suggest that the percentage of drivers stopping after yellow onset does not depend on duration of yellow. It appears, therefore, that potential conflict rate, defined as the probability that a vehicle blocks cross street traffic for longer than a preassigned period, may be controlled by modifying the change interval.

REVIEW OF CURRENT PRACTICE

The *Manual on Uniform Traffic Control Devices for Streets and Highways* (MUTCD)[8] provides only general guidance for the selection of change intervals. It states that "yellow vehicle change intervals should have a range of 3 to 6 seconds." It also recommends that longer intervals be used with higher approach speeds. The MUTCD permits the use of a short all-way red clearance to allow traffic to clear the intersection. No guidance is provided either for the identification of locations where the use of all-way red is appropriate, or for the duration of the interval.

More definitive guidance is provided by the *Transportation and Traffic Engineering Handbook*[7] and repeated by the *Traffic Control Devices Handbook* published by the National Advisory Committee on Uniform Traffic Control Devices.[9] These suggest that the yellow change duration has two purposes: (1) to permit vehicles the time to come to a safe stop without entering the intersection before the red commences, and (2) to allow vehicles that have entered the intersection the time to clear it prior to the release of opposing traffic or pedestrians. The use of formula (1) in Appendix A is recommended in[7].

The only survey of actual field practice was conducted by May[5] in 1968. This survey was conducted by sending questionnaires to 50 state highway departments, 32 major cities outside of California, and 17 California cities. Of the responses received, 47 percent of the organizations employed a fixed change interval at all intersections; 20 percent of the organizations calculated the change interval using approach speed (V) in accordance with equation (1) (Appendix A) but without the last term; 22 percent of the organizations used both approach speed and intersection width (W) in the calculation in accordance with the complete equation (1); and 11 percent of the organizations used other factors besides V and W such as crash rate, traffic volume and engineering judgment.

From the variety of practices currently in use, it is apparent that uniform procedures for setting signal durations do not exist and it can be concluded that the existing body of research results has not achieved widespread acceptance.

The study presented here is a first step in the development of a formula enabling local traffic engineers to set change interval timing in a manner which should minimize potential crash-producing conflicts.

EXPERIMENTAL METHOD

Design. Driver response to yellow onset was observed at two suburban arterial street intersections under eight combinations of three dichotomous factors corresponding to traffic and pavement conditions and yellow signal duration;

Factor	*Level 1*	*Level 2*
Traffic Condition	Peak	Off-peak
Pavement Condition	Dry	Wet
Yellow Duration	Short (Initial)	Long (Extended)

Sites. The study sites were selected to satisfy each of the following criteria:

- Average approach speed near 30 mph.
- Current yellow duration relatively short; i.e. less than 5 seconds.
- The signal should not be activated by through traffic on the main roadway.
- No pedestrian signals visible to drivers on the main roadway.
- Reasonably isolated relative to other signalized intersections.
- Four-legged intersection with simple geometrics (e.g. relatively straight approaches intersecting at right angles) and good pavement surface.
- Essentially level approaches on main roadway.

At both sites, approaching traffic could be photographed from suitably localized building. These vantage points were hidden from the view of all motorists. Other site characteristics are summarized in Table 1. One of the sites was located in Bethesda, Maryland and the other in Atlanta, Georgia.

The two sites are diagrammed in Figs. 1 and 2.

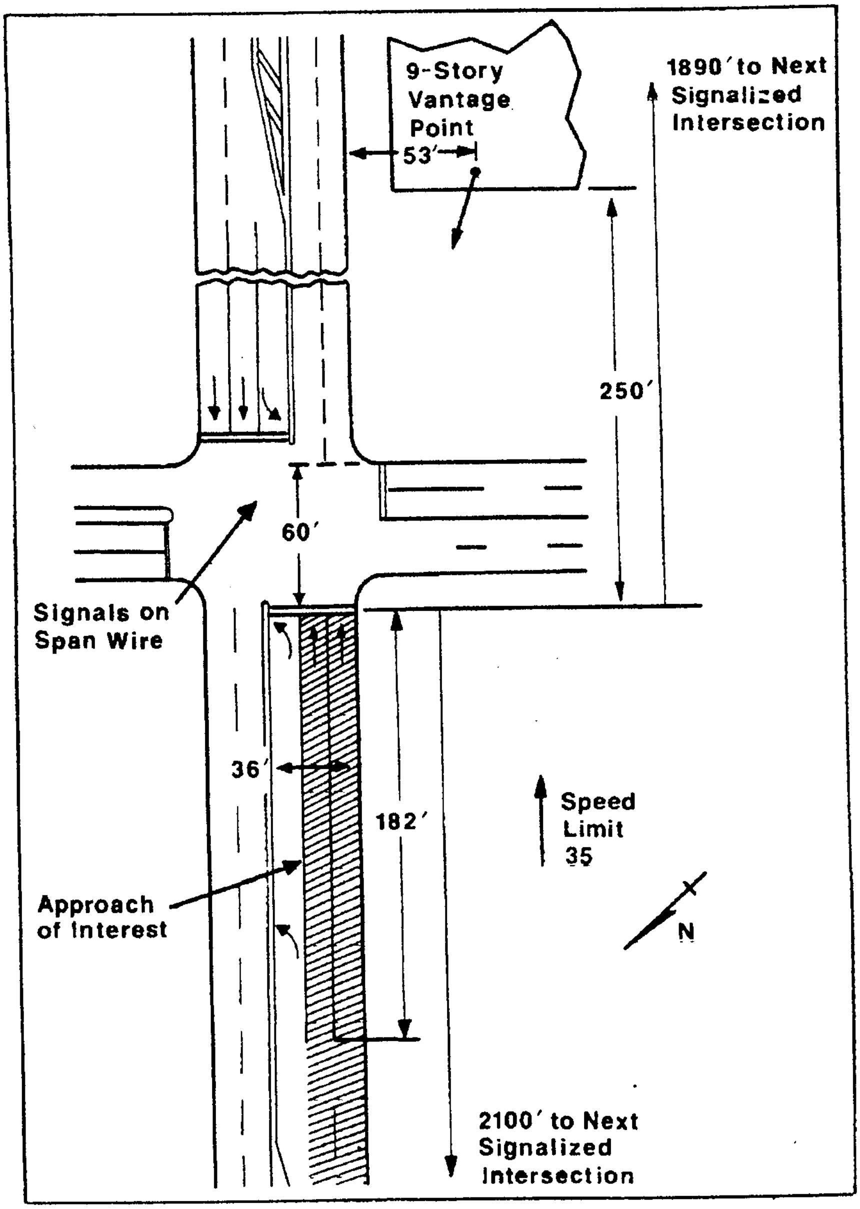


Figure 1.

DATA COLLECTION

A NIZO S560 Super 8 mm time lapse movie camera was used to record the motion of vehicles approaching the intersection. This camera insured filming rate accuracy of 0.16 ± seconds 95 percent of the time.

At each site a "catch zone" that included the dilemma zone at most approach speeds was defined and the motions of all vehicles in the catch zone at yellow onset were recorded. The upstream extremity of the catch zone was chosen as the point from which a car with an initial speed of 10 mph in excess of the local average speed could come to a full stop at the traffic signal using a uniform deceleration of 0.25g (8 ft/sec^2). The downstream extremity was chosen as the point from which a vehicle traveling 10 mph below the average at

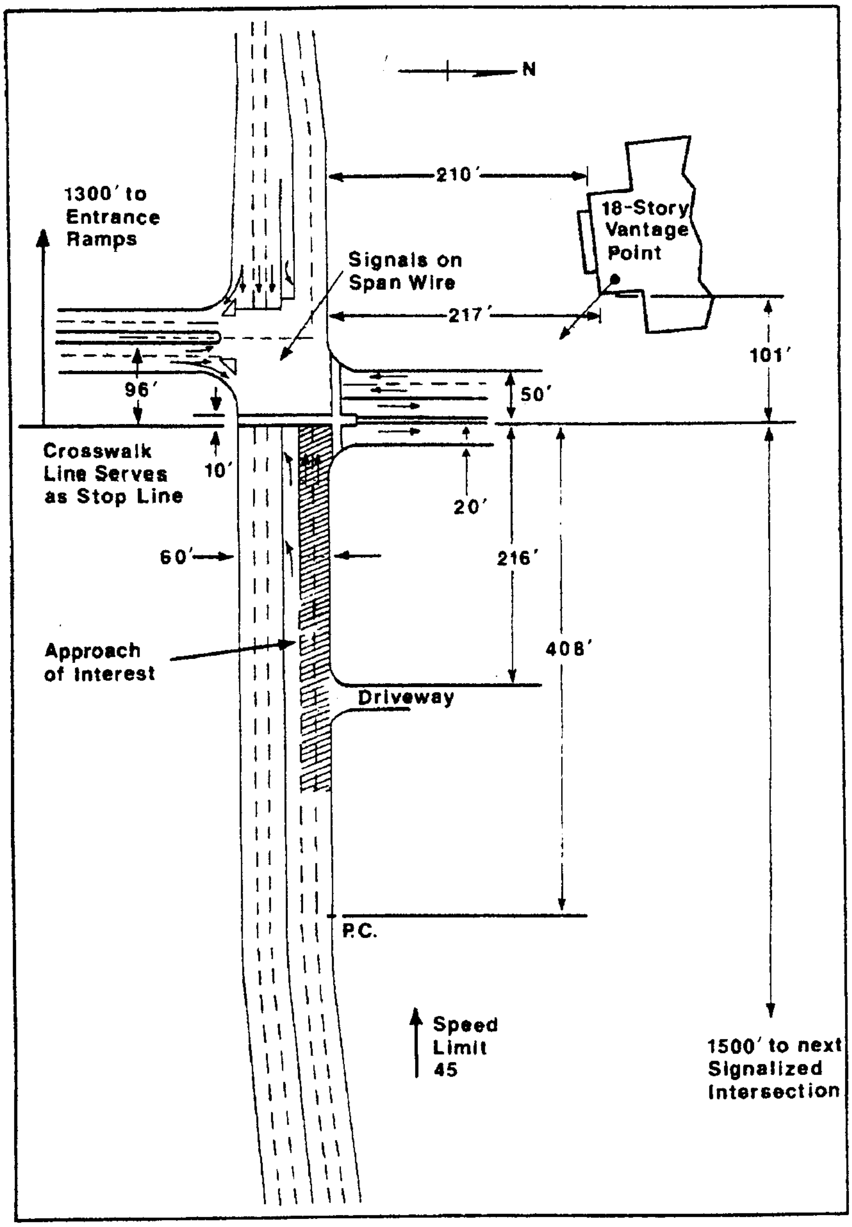


Figure 2.

yellow onset could just clear the cross street prior to red onset. At the Maryland site the catch zone extended from 65 feet to 320 feet and at the Georgia site from 25 feet to 320 feet.

At the Maryland site, the camera was located on a roof top 105 feet (including tripod height) above ground level in dry weather and behind a hallway window 84 feet above ground level in rain; at the Georgia site, all filming was done from a rooftop 158 feet above the roadway. The field of view of the camera ranged from the traffic signal to 100 feet upstream of the catch zone.

At the start of each film cartridge, an identification card and the face of a stopwatch with 0.01 sec. increments were filmed for 100 frames. The camera was then turned, zoomed, and focused on the prescribed field of view. Filming commenced at least two seconds prior to yellow signal onset and continued until all vehicles initially in the catch zone either stopped or cleared the intersection.

For the purpose of this study, a vehicle was called a decision vehicle if, in a particular approach lane, it was 1) the first vehicle to stop, or 2) the last vehicle to cross the intersection. Data collection continued until about 150 decision vehicles were obtained at each site under each experimental condition.

Yellow duration was extended after all data with the initial yellow had been collected. The duration of the extended yellow at the Maryland site was set equal to the maximum duration acceptable to the traffic engineer responsible for signal timing at that location. The duration of the extended yellow at the other site was chosen to produce a percentage increase of similar magnitude. Data collection started two weeks after yellow duration was extended and continued for several months at both locations.

DATA EXTRACTION AND CALIBRATION OF DISTANCE ESTIMATION

The longitudinal and lateral vehicle coordinates were calculated using a method developed by Huber and Tracy[10] and reduced to a FORTRAN program by Bleyl[11]. This method is based on identifying four roadway reference points with known coordinates on a screen. The screen coordinates of the point of interest (right headlight in this case) are related to the screen coordinates of the reference points and the distances of interest are calculated using equations derived from geometrical optics and the known distances between the reference points. Four 1-foot × 2-foot paddles painted bright orange on the side facing the camera but black on the side facing the traffic served as reference markers in this study. A rear-screen projection box containing a Kodak MFS-8 stop-action movie projector was used for the reading of coordinates.

The accuracy of the Huber-Tracy method for estimating distance was known to depend inversely on the angle subtended by the horizontal plane and the line between camera lens and object photographed. To determine the accuracy of the method in the present case with sharp acute angles (7.4° for worst case in Maryland), true roadway coordinates were compared to their estimates at both sites using linear regression. Results were satisfactory ($R^2 \geq 0.985$) and the regression relationships were incorporated in the computer program to further improve the accuracy of the distance estimates.

The method was validated by comparing known test-run velocities of 25, 30, 35, and 40 mph with their estimates under the worst case in Maryland viewing conditions. The estimated error was found to be less than 1 mph even at 625 feet from the camera, a distance some 50 feet beyond the upper end of the catch zone.

Films were read frame-by-frame for a subsample of decision vehicles. Polynomial regression analysis was employed to assess the validity of a constant deceleration model for these vehicles. It was found that vehicle distance from the stop line is adequately described by a quadratic polynomial in time past yellow onset for both stopping and non-stopping vehicles ($R^2 \geq 0.99$), and so the constant deceleration model was adopted for this study.

In reducing the total set of time and position data from the screen, frame numbers and coordinates for the following decision vehicle checkpoints were read and directly coded for computer processing:

- An approximation of the decision point, 0.5 second downstream of yellow onset.
- The stopping position or stop line, depending on whether or not the vehicle stopped.
- On a time-division basis, one-third and two-thirds points between the two checkpoints described above.

For a non-stopping decision vehicle, the speed computed by the program for the last one-third of the approach was extrapolated to estimate the time at which the vehicle cleared the cross street conflict area. For every decision vehicle, the frame number at red onset was used by the program to interpolate vehicle position at that time. The average speed over each of the intervals between checkpoints was calculated.

Last, average overall acceleration and deceleration were computed by assuming uniformly accelerated motion. In the case of stopping decision vehicles, this included both the deceleration actually experienced and the acceleration which would have been required to clear the cross street prior to red onset. Conversely, the computation for non-stopping vehicles included both the acceleration actually applied and the deceleration which would have been required to stop.

Table 1. Characteristics of Study Sites

	Maryland	Georgia
Yellow Duration (seconds):		
-Initial	4.7	4.3
-Extended	6.0	5.6
Signal Cycle Length (seconds):		
-Peak Period	120	100
-Off-peak	75	90
Average Approach Speed (mph):		
-Peak Period, Dry	37	27
-Off-peak, Dry	38	29
-Peak Period, Wet	32	—
-Off-peak, Wet	34	—
Typical Volume of Through Plus Right-Turn Vehicles on Approach of Interest (vph):		
-Peak Period	1,450	1,100
-Off-peak	700	900
Pavement Type	Portland Cement Concrete	
Approach Grade	Negligible	Negligible

Table 2. Number of first-to-stop and last-to-cross vehicles observed at intersections

Yellow Interval	Pavement Condition	Off-Peak Period (10 am-noon & 1–3 pm)	Peak Period (7:30–9:30 am)
		Maryland Intersection	
Initial	Dry	141[1]	144
(~ 4.7 sec.)	Wet	148	136
Lengthened	Dry	148	160
(~ 6.0 sec.)	Wet	156	132
		Georgia Intersection	
(~ 4.3 sec.)	Dry	150	156
(~ 5.6 sec.)	Dry	147	143

[1] Number of decision vehicles traced.

RESULTS

The total sample of 1,761 observed decision vehicles is displayed by site and experimental condition in Table 2. Only passenger vehicles, pickup trucks and vans were included in these samples.

Pickup trucks and vans accounted for less than 10 percent of the decision vehicles in all cases, and the percentage of stopping decision vehicles depended little on vehicle type. Consequently, vehicle type was not used as a factor in any of the analyses.

INFLUENCE OF YELLOW DURATION ON MOTORIST PERFORMANCE

The frequencies and percentages of potential intersection conflicts are shown in Table 3 by site, traffic condition, pavement condition, and yellow duration. In determining these frequencies, only those vehicles were included that spent at least 0.2 second* in the intersection past red onset and were, in fact, the last-to-cross the intersection in the cycle. The potential conflict percentages in the table were obtained by dividing

* 0.2 second has been used throughout this study as the minimum elapsed time for which various events were examined.

Table 3. Frequency and Percentage of Crossing Vehicles in Intersection when Signal Changes to Red by Site, Traffic Condition, Pavement Condition and Yellow Duration

Site	Traffic Condition	Pavement Condition	Yellow Duration[1]	Crossing Vehicles in Conflict		All Crossing Vehicles	% Reduction in Conflict Rate
				No.	%		
Maryland	Off-Peak	Dry	4.6	11	15	73	91
			6.1	1	1	78	
	Peak	Dry	4.7	15	19	77	89
			6.0	2	2	91	
	Off-Peak	Wet	4.7	10	12	83	90
			6.0	1	1	83	
	Peak	Wet	4.7	12	17	72	100
			5.9	0	0	54	
Georgia	Off-Peak	Dry	4.4	40	63	63	70
			5.6	14	19	74	
	Peak	Dry	4.2	71	90	79	77
			5.8	14	21	68	

[1] Minor variations in yellow duration were observed

Table 4. Frequency and Percentage of Last-to-Cross Vehicles that Cleared the Intersection (T + 0.2) or More Seconds Past Yellow Onset by Site, Traffic Condition, Pavement Condition and Yellow Time Duration[1]

Site	Traffic Condition	Pavement Condition	Yellow Duration[2]	Vehicles Crossing (T + 0.2) Sec. or More After Yellow Onset		All Crossing Vehicles
				No.	%	
Maryland	Off-Peak	Dry	4.6	0	0	73
			6.1	1	1	78
	Peak	Dry	4.7	1.5[3]	2	77
			6.0	2	2	91
	Off-Peak	Wet	4.7	1	1	83
			6.0	1	1	83
	Peak	Wet	4.7	0	0	72
			5.0	0	0	54
Georgia	Off-Peak	Dry	4.4	17	27	63
			5.6	14	19	74
	Peak	Dry	4.2	25	32	79
			5.8	14	21	68

[1] T = duration of yellow phase after extension
[2] Minor variation in yellow duration was observed
[3] There was one vehicle that cleared the intersection during the 5.9–6.1 second class interval

these frequencies by the corresponding frequency of all vehicles that were last to cross. *The percentage of potential conflicts is, therefore, the percentage of last-to-cross vehicles that spent at least 0.2 seconds in the intersection past red onset*

The results in Table 3 show that the extension of yellow duration reduced the frequency of potential conflicts in all cases studied. The frequency of potential conflicts at the Maryland site ranged from 12 percent to 19 percent with the initial yellow duration of 4.6 seconds. With the extended yellow (6.0 seconds), these were reduced to between 0 and 2 percent. Thus, an increase of 1.4 seconds or about 30 percent in yellow duration virtually eliminated all potential conflicts at the Maryland site.

At the Georgia site, the initial conflict percentages were between 63 and 90 percent. With the extended yellow, these were reduced to 19 to 21 percent. Thus, an increase of 32 percent or 1.6 seconds in yellow duration reduced frequency of potential conflicts at the Georgia site to about 25 percent of their original frequency.

The results in Table 3 also show that potential conflict percentages differed between the two sites both with the initial and the extended yellow durations. These differences undoubtedly reflect differences between the two sites in terms of geometry, approach speed and traffic volume (see next section) but there is not at present quantitative relationships that would predict potential conflict frequency in terms of these, and possible other, factors.

It has frequency been claimed that if the yellow is "too long," more drivers will use part of the yellow as green. More drivers—it was argued—would cross after yellow onset with long than with short yellow. This possibility can be explored by comparing the percentages of last-to-cross vehicles that are still in the intersection when the signal changes to red. Using the observations from the short yellow durations it is possible to compute the percentages of vehicles that would be still in the intersection when the signal changed even if it had had a longer yellow duration. These percentages can then be compared with the actual percentages observed with the longer yellow durations, and if driver behavior is unaffected by the yellow duration then the percentages should be the same.

Table 4 displays the frequency and percentage of last-to-cross vehicles that cleared the intersection (T + 0.2) or more seconds past yellow onset (T = duration of yellow phase after extension) by site, traffic condition, pavement condition, and yellow setting. The data show that the percentage of last-to-cross vehicles clearing the intersection (T + 0.2) seconds or more past yellow onset was not appreciably changed by the extension of the yellow phase. At the Maryland site, the percentage of such crossings was 0.8 percent with the initial yellow and 1.3 percent with the extended yellow. At the Georgia site, the comparable percentages were 29.6 percent and 19.7 percent. None of these differences are greater than what could be expected on the basis of statistical fluctuations ($p > 0.05$, $X^2 = 3.2$, d.f. = 1 for Georgia, $p > 0.05$, $X^2 = 0.01$, d.f. = 1 for Maryland).

Frequencies and percentages of last-to-cross vehicles clearing the intersection at least 0.2 seconds past red onset were compared at the two sites in peak and off-peak traffic and on dry and wet pavement between two different yellow settings (Table 3). The percentages of these vehicles, that is of vehicles that could have been involved in a conflict with cross-street traffic, were substantially smaller at both sites and under all conditions after the yellow duration was extended. No evidence was found at either site, under any of the conditions, that the vehicles that were in potential conflict with cross-street traffic with the extended yellow would have cleared the intersection earlier in the cycle if the yellow had not been extended. Thus, the extensions of yellow duration employed in this study substantially reduced the frequency of potential intersection conflicts.

While a number of different measures could have been used as possible surrogates for intersection crash rates, potential conflict frequency was selected. This measure was defined as the percentage of last-to-cross vehicles that have spent at least 0.2 seconds in the intersection past red onset. In preliminary analyses of the data, two other measures were also explored. One of these was the average distance traveled in the intersection by a crossing vehicle after red onset. The other was the average time past red onset that a crossing vehicle was in the intersection. The conclusions reached with these other measures were very similar to the conclusions stated in this paper.

INFLUENCE OF TRAFFIC AND PAVEMENT CONDITIONS ON MOTORIST PERFORMANCE

The rate of potential conflicts was found to be generally higher in peak than in off-peak traffic. In Maryland, the conflict rate with initial yellow duration in peak traffic exceeded the conflict rate in off-peak traffic by 29 percent on dry pavement and by 39 percent on wet pavement. A similar comparison between peak and off-peak conflict rates cannot be made for the extended yellow since after the extension virtually all conflicts disappeared. In Georgia, the conflict rate with the initial yellow duration in peak traffic exceeded the conflict rate in off-peak traffic by 42 percent. The comparable figure for the extended yellow is 6 percent. Although only one of the four comparisons indicated change that was larger than what could be explained on the basis of statistical fluctuations alone (the 42 percent increase in Georgia was significant at the 0.001 level, $X^2 = 11.99$, d.f. = 1), the evidence, when examined together, suggests that the influence of traffic conditions on potential conflict rates is real.

The rate of potential conflicts was found to be generally higher on dry than on wet pavements. Conflict rate was higher on dry pavement than on wet pavement by 26 percent in off-peak and by 17 percent in peak traffic. However, these differences were not larger than what could be explained on the basis of common statistical fluctuations alone.

These findings suggest that both traffic and pavement conditions influence the rate of potential intersection conflicts. It would be of obvious interest to relate these influences to changes in approach speed and traffic density, for these traffic characteristics are easy to determine from on-site observations. That changes in approach speed do not, by themselves, explain the findings may be seen by noting that approach speeds were consistently lower in peak than in off-peak traffic and lower on wet than on dry pavement (Table 1). However, potential conflict rates were higher in peak than in off-peak traffic and higher on dry than on wet pavement, thus indicating that conflict rates do not always change in the same direction as approach speeds.

CONCLUSIONS

The response of drivers to onset of yellow was observed at two signalized arterial street intersections. It was found that potential intersection conflicts could be virtually eliminated with small increases in the duration of the yellow phase. Since the numerical relationship between potential conflict (as defined and measured in this study) and the frequency of intersection crashes is currently unknown, it is impossible to estimate the magnitude of the crash loss reduction which may be achieved through the use of slightly longer yellow durations.

The frequency of potential conflicts at signalized intersections was found to be higher in peak than in off-peak traffic and higher on dry than on wet pavement when other factors were held constant. Thus, it can be concluded that conflicts are dependent not only on intersection geometry and travel speed, but on traffic density and possibly pavement condition. The precise nature of this dependence remains to be determined.

There exists no consensus among traffic engineers as to what constitutes an optimal yellow interval under everyday traffic and environmental conditions. Since the influence of change interval on intersection crash frequency has not been investigated, further work is needed to first determine and then to introduce change intervals that will insure the reduction of avoidable intersection crashes by reducing the frequency of potential intersection conflicts.

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