Considerations in the Length of the Yellow Interval

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Most modern methods for setting the yellow interval at traffic signals start with the presumption that the yellow should be long enough so that a reasonable driver is never placed in a position of neither being able to enter on yellow nor stop before entering the intersection. If the yellow is too short, a dilemma zone [1] is created wherein a reasonable driver occasionally must either enter on red or stop beyond the stop line. The methods then go on to use a definition for a reasonable driver that is similar to the one in the ITE Handbook, [2] which uses reasonable limiting values of one second for the reaction time and 10 (or 15) ft/sec² for the deceleration rate. These values are assumed to be constant over all speeds. A kinematic model of vehicle behavior is then used to predict the minimum yellow time necessary to avoid a dilemma zone. Difference between procedures then center around the exact values that are appropriate for a reasonable driver.

The concept that a dilemma zone exists and that the avoidance of one should be used as a basis for setting the minimum length of the yellow interval is probably valid. It could be that a longer clearance interval is needed for safety, but then the usual procedure is to provide the excess time as an all-red interval. This paper concentrates on the manner in which a reasonable driver is defined and the dilemma zone determined. Its main departure is with the assumption that driver reaction time and declaration rate are constant over all speeds. It appears that existing data do not necessarily support the idea that reaction times and deceleration rates are constant over all speeds for a consistently defined reasonable driver.

The first problem is in defining just what is a reasonable driver. When setting speed limits, for example, the 85th percentile speed is usually used. [2] This implies that 15% of drivers are unreasonable. Researchers working in green extension of rural signals [3] usually define the “dilemma zone” in terms of the 10th and 90th percentile drivers on a stopping probability curve. This implies that 10% of drivers are unreasonable and will not stop if the light turns yellow at a point where the other 90% of drivers would stop. For the purpose of setting yellow
intervals, a similarly high percentile driver should probably be used as the design driver.

Olson [1] was probably the first to point out the philosophy of using stopping probability curves to help decide on the length of the yellow. The reasoning is this:
1. Reasonable drivers should not be forced to enter an intersection on red because of a too short yellow,
2. A reasonable driver is defined by a certain percentile behavior (85th or 90th, say),
3. The behavior in question is the decision of whether to stop or continue when the yellow light first comes on, and
4. The behavior is a function of how many seconds the driver’s vehicle is from the stop line when the light turns yellow.

The yellow light should not be shorter than the time corresponding to the distance away from the intersection at which 90% of drivers decide to stop and 10% decide to continue, otherwise the 90th percentile driver will be forced to enter on red.

This time (call it $\tau_0$) can be found by inspecting stopping probability curves an example of which is shown in Fig. 1. These curves show the percent of drivers deciding to stop plotted against the vehicle’s position at the moment the light turns yellow. Usually one curve is plotted for each speed, although for a single intersection the plot could be for all vehicles approaching the intersection. In the latter case, the approach speed is usually given. For the purposes of setting the yellow interval, the plot should be of percentage stopping vs time to the stop line. This plot, of course, can be derived from the usual stopping probability curve by converting distances to times using the known speeds. The minimum yellow interval can then be picked off the plot as the 90th percentile time, as shown in Fig. 1. Table 1 and Fig. 2 display the results of such a calculation on many published stopping probability curves. Also shown are the yellow intervals given by the formula in the ITE Handbook with decelerations of 10 and 15 ft/sec$^2$.

Sadly, the data are wildly inconsistent. Drivers in Kentucky appear to have almost a constant $\tau_0$ while drivers in Minnesota require times that increase with speed. Are drivers really this different from one location to another? Or do the measurement techniques account for the differences?

Notice that the slope of the line is related to the acceleration assum-
Figure 1. For this stopping probability curve, the minimum yellow interval for the 90th percentile driver is 5.3 seconds. Source: Ref. [4]

Table 1. Minimum Yellow Intervals for 90th Percentile Driver

<table>
<thead>
<tr>
<th>Source</th>
<th>10</th>
<th>15</th>
<th>20</th>
<th>25</th>
<th>30</th>
<th>35</th>
<th>40</th>
<th>45</th>
<th>50</th>
<th>55</th>
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</thead>
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<td>Olson &amp; Rothery [1]</td>
<td></td>
<td></td>
<td></td>
<td>4.1a</td>
<td>4.3</td>
<td>5.1</td>
<td></td>
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<tr>
<td>Williams [2]</td>
<td>6.3</td>
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<td>4.3</td>
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<td>4.5</td>
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<td></td>
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<td>4.4</td>
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<tr>
<td>Minnesota [5]</td>
<td></td>
<td></td>
<td></td>
<td>3.2</td>
<td>3.5</td>
<td>3.7</td>
<td>3.9</td>
<td>4.1</td>
<td>4.6</td>
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<td>Zegeer (Kentucky) [6]</td>
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<td>4.8</td>
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<td>Sheffi &amp; Mahmassani² [7]</td>
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<td>4.7</td>
<td>4.8</td>
<td>4.9</td>
<td>5.0</td>
<td>5.1</td>
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For comparison:

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<th>2.8</th>
<th>3.2</th>
<th>3.6</th>
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<th>4.3</th>
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<tr>
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<td>3.4</td>
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<td>Level grade, t = 1 sec, a = 15 ft/sec²</td>
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a. Average for two intersections
b. From a probit model calibrated with Zegeer data.

Note: No single study covered the whole speed range. All of these studies were done at intersections with straight, level approaches.
Figure 2. A display of minimum yellow intervals derived from several published stopping probability curves. For all, the minimum yellow interval is a function of the approach speed or the individual driver's speed. The sources are as follows: Olson & Rothery, [1] (average for 2 intersections) Williams, [5] Parsonson, [3] Herman, [6] Minnesota, [7] Zegeer (Kentucky), [8] Sheffi & Mahmassani [4] (from a probit model calibrated with Zegeer data), and the ITE Handbook [2] (level grade, t = 1 sec, a = 10 and 15 ft/sec²). Note: No single study covered the whole speed range. All of these studies were done at intersections with straight, level approaches.
ed and the y intercept is the reaction time. If a similar line were to be drawn through the Kentucky data, for example, the slope would give a deceleration of about 35 ft/sec² (more than the acceleration of gravity!) and a reaction time of about 4 sec. These are totally unrealistic results. They suggest, in fact, that the kinematic procedure is not supported by observation. On the other hand, note that the Minnesota data would give quite reasonable values for deceleration and reaction time.

Unfortunately, no firm conclusion can be drawn. The questions asked above are still unanswered. Perhaps, however, the situation is not bleak. In reviewing Table 1 and Fig. 2, it appears that the maximum $\tau_0$ found for slow approach speeds (20-30 mph) is about four sec, while for fast approach speeds it is about five sec. In the interim, these times might be used until the questions about the stopping probability curves are resolved.

Of help might be the data that have been collected to try to find values of constant reaction times and deceleration rates. Often, these studies only include data for vehicles that stop, since deceleration rates cannot be measured for vehicles that do not stop. Nevertheless, stopping probability curves cannot be found without observing vehicles that do not stop. Fortunately, a study now being conducted for the FHWA by the Texas Transportation Institute will involve observation of both vehicles that do and do not stop. Using the original observations from this and other studies, stopping probability curves can be constructed, and deductions can be made about $\tau_0$.

Note that the methodology outlined in this paper could be used to get around the problem of assuming a constant reaction time and deceleration rate, and the consequent problems in separating them out. Instead, driver behavior is investigated directly. What is lost is a simple kinematic model of vehicle motion. The data, however, do not necessarily support a simple kinematic model. Instead, most of the data seem to support the idea that drivers around the 90th percentile tend to base their decision on whether to stop on their time to the intersection, not on whether they can stop at a particular deceleration rate. If one thinks about this, time, rather than deceleration, seems more reasonable from a psychological viewpoint anyway.

REFERENCES


3. Parsonson, P. S., Roseveare, R. W., and Thomas, J. M., "Small Area Detec-


