Relation of Signalized Intersection Level of Service to Failure Rate and Average Individual Delay

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The most widely used signalized intersection design procedure is that found in the 1965 Highway Capacity Manual. This manual utilizes vehicles per hour of green as an indicator of the level of service offered by an installation. An alternative intersection solution procedure utilizes the percentage of cycle failures as an indicator of fixed time intersection performance. The research reported here indicates that failure rate does not apparently correlate with level of service as defined by the Highway Capacity Manual. The 2 indexes have a varying relationship. Charts and tables are presented for use in conversion. For a constant service level, a low approach volume will allow higher failure rates than may be tolerated for high volumes.

In line with prior work in this field, an investigation was made into the feasibility of utilizing average individual delay as an index of the level of service offered by a signalized intersection. Delay was related to failure rate and charts prepared for various cycle lengths utilizing Webster's equation for average individual delay. These charts indicated that failure rate and delay also have a varying relationship. This relationship was oriented in a different manner from the preceding relationship. Using constant delay lines, higher failure rates were allowed for high volumes. A combined plot of level of service lines and delay lines indicates an apparent divergence of level of service and average individual delay. This divergence demonstrates that, although an intersection may satisfy the Highway Capacity Manual's criteria of a given vehicles per hour of green for a desired level of service, average individual delay may vary considerably depending on arrivals, cycle length, and length of green time.

THE 1965 HIGHWAY CAPACITY MANUAL (1) introduced the concept "level of service" for both uninterrupted flow conditions and street intersections with signalized control. For uninterrupted flow conditions, speed and the volume/capacity ratio were selected as measures of the level of service offered by a facility. Chapter 6 of the 1965 Highway Capacity Manual discusses the level of service concept as it relates to at-grade intersections or interrupted flow (1, p. 130):

Inasmuch as level of service is described in terms of driver satisfaction, the substitute measure should be some factor that the driver himself sees and interprets in terms of degree of congestion. Of the several factors that have been discussed in the previous section, probably load factor is the
most evident to the average driver. Hence it is the best measure of the level of service at individual
intersections with no or only average signal coordination.

The 1965 Highway Capacity Manual uses load factor as the determinant for the vari-
ous service levels. The computational procedure for determining the level of service
offered by a signalized intersection is basically the same as that presented by the 1950
Highway Capacity Manual (2), in that capacity at various levels is obtained from charts
in terms of a specific number of vehicles per hour of green after applying appropriate
correction factors. The 1950 Highway Capacity Manual used this technique for deter-
mining basic, possible, and practical capacities of an installation. Possible capacity
required a continual backlog of vehicles, hence a high load factor and a low level of
service. Practical capacity was defined as the volume where most vehicles would
clear the intersection without waiting for more than one complete signal cycle, hence
a lower load factor and higher level of service.

FAILURE RATE

Drew and Pinnell (3) have presented evidence that peak-period traffic flow approach-
ing a signalized intersection is accurately defined by the Poisson probability distribu-
tion. They have utilized this finding to develop a design procedure using the percent-
age of cycle failures for fixed time installations. A cycle failure is defined as any
cycle during which approach arrivals exceed the capacity for departures. Briefly this
procedure assumes that departures during a green phase of a cycle may be computed
by the equation

\[ X = \frac{G - (K - D)}{D} \]

where

- \( X \) = maximum departures per lane for one approach during a green phase;
- \( G \) = length of the green phase of the cycle in seconds including yellow time;
- \( K \) = sum of starting delay and time for last vehicle to cross intersection; and
- \( D \) = average minimum headway in seconds.

For design purposes, the \( K \) factor was determined to be 6 seconds for the average
intersection and \( D \), the average minimum headway, was established as 2 seconds. Using
this equation and constants with the cumulative Poisson probability distribution, a de-
sign chart was established (Fig. 1). This chart is entered with the average number of
arrivals per approach lane per cycle and the green time allocated to that approach
lane. Noting the intersection of the 2 variables, a determination may then be made of
the probability that more vehicles will arrive at the approach than may pass through
the installation during a green phase for that approach. Of course, this neglects the
carry-over of queues from one cycle to the next, inasmuch as only new arrivals are
being considered. For high traffic volume it overstates the probability that a particular
vehicle will clear the intersection during its first cycle at the signal.

In their development of the failure rate design procedure, Drew and Pinnell (3) have
derived a multiple regression equation for the determination of the peak-period flow
rate when the peak-hour flow rate is known. Their equation is as follows:

\[ Y' = 1.225 - 0.000135 X_1 \pm (0.1X_2 - 0.00003X_3) \]

where

- \( Y' \) = factor to be applied to approach peak-hour flow rate to determine peak-period
  flow rate for the approach;
- \( X_1 \) = population 1,000;
- \( X_2 \) = ratio distribution = \( \frac{\text{distance between intersection and CBD}}{\text{distance from CBD to city limits}} \); and
- \( X_3 \) = peak hourly volume per approach.
The plus or minus factor in this equation is positive for morning flow and negative for evening flow.

Drew and Pinnell (3) attempted to develop an equation for the duration of the peak period. However, this relationship proved to be statistically unreliable.

Drew and Pinnell (3) have utilized these concepts to develop a design method for high-type signalized intersections where every movement has a separate signal phase. Knowing the peak-hour flow rate, the peak period flow rate may be determined using this multiple regression equation. A trial design of the intersection layout is then made. From the number of lanes allocated to an approach, the average number of vehicles per cycle per approach lane for each phase may be determined. For a 4-leg high-type intersection, there may be 4 separate phases: 2 through and 2 left turn phases. After determining the average lane arrivals per cycle for each phase and choosing for each phase the lane with the highest average arrivals, which is referred to as the critical lane volume for that phase, a cycle length is chosen. Green times are then allocated to each phase such that when the design chart is entered with the green time and average arrivals, a desired failure rate for all phases is obtained. Of course, the summation of the green times for all phases must equal the cycle length. However, some determination must be made as to what percentage failure rate is acceptable for design practice. Drew and Pinnell state (3):

Although additional research is needed in deciding just what percentage of failures may reasonably be allowed, it seems that a level of 30 to 35 percent during the peak period represents a practical design level (remembering that this would be only about 10 to 15 percent of the peak hour).

The writers have solved intersection problems utilizing both the procedure in the 1965 Highway Capacity Manual and Drew and Pinnell’s failure rate procedure. In some instances, a low level of service at the 30 to 35 percent failure rate level has been obtained. Although the 2 concepts are different in approach, in that one is subjective and
based on thousands of actual observations and the other is mathematical, an investigation into their relationship appeared worthwhile. The relation presented will be for only high-type intersections such as might be solved by the failure rate method.

**RELATION OF LEVEL OF SERVICE TO FAILURE RATE**

In order to provide some common ground for relating the 2 procedures, nomograms developed by Leisch (4) were utilized for determining allowable vehicles per hour of green for various levels of service and approach lanes. Leisch's nomograms were developed from the Manual's procedures, but they utilize an MP correction factor that combines the Manual's peak period and population factors. To determine the MP factor, one needs only to know the population of the city where the installation is to be made. Of course, if the peak-period factor as defined by the Manual is known, it and the city population can be used to obtain a factor for use in the nomograms. This relationship is shown in Figures 6.5 through 6.9 in the 1965 Highway Capacity Manual. These nomograms were utilized to develop the allowable vehicles per hour of green for various intersection locations and approach lane arrangement. For comparison with Drew and Pinnell's method, 12-ft through lanes and 10-ft left turn lanes were assumed because this lane width would probably be used at the type of intersections considered by Drew and Pinnell. The allowable vehicles per hour of green for various numbers of approach lanes and locations within a city as obtained from Leisch (4) are given in Table 1. The charts for one-way operation (two-way charts used for single through lane approaches) and no parking were chosen because it was considered that these conditions best approximated approach operation at a high-type intersection where each movement has a separate signal phase. Leisch's turning lane charts were used directly. No truck, bus, or turn factors other than those built into the charts are considered for this general case. The definitions for central business district (CBD), fringe area, outlying

<table>
<thead>
<tr>
<th>Table 1</th>
<th>VEHICLES PER HOUR OF GREEN (VPHG) PER APPROACH LANE AND VEHICLES PER SECOND OF GREEN (VPSG) FOR VARIOUS LOCATIONS WITHIN A CITY FOR DIFFERENT LEVELS OF SERVICE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area</td>
<td>Level of Service</td>
</tr>
<tr>
<td>---------</td>
<td>-------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>CBD</td>
<td>A 810 vphg</td>
</tr>
<tr>
<td></td>
<td>B 810 vphg</td>
</tr>
<tr>
<td></td>
<td>C 900 vphg</td>
</tr>
<tr>
<td></td>
<td>D 1,080</td>
</tr>
<tr>
<td></td>
<td>E 1,170</td>
</tr>
</tbody>
</table>
|         | Note: These data are for a city population of 250,000. The factors in Table 2 should be used to adjust the values in this table for other populations. 
|         | aData from Leisch's turn lane charts (4). 
|         | bData from Leisch's two-way charts, no parking (5). 
|         | cData from Leisch's one-way charts, no parking (6). 
| OBD or fringe | A 810 vphg                                                                                     | 729 1,020 1,140 1,066 1,058 |
|         | B 810 vphg                                                                                       | 729 1,080 1,164 1,088 1,080 |
|         | C 900                                                                                           | 810 1,060 1,200 1,132 1,125 |
|         | D 1,080                                                                                         | 972 1,366 1,220 1,214 1,239 |
|         | E 1,170                                                                                         | 1,053 1,440 1,356 1,270 1,294 |
| Residential | A 810 vphg                                                                                        | 729 1,020 1,238 1,175 1,175 |
|         | B 810 vphg                                                                                        | 729 1,080 1,220 1,200 1,200 |
|         | C 900                                                                                           | 810 1,080 1,300 1,250 1,250 |
|         | D 1,080                                                                                         | 972 1,368 1,430 1,358 1,375 |
|         | E 1,170                                                                                         | 1,053 1,440 1,470 1,400 1,438 |
|         | Note: These data are for a city population of 250,000. The factors in Table 2 should be used to adjust the values in this table for other populations. 
|         | aData from Leisch's turn lane charts (4). 
|         | bData from Leisch's two-way charts, no parking (5). 
|         | cData from Leisch's one-way charts, no parking (6). 

Note: These data are for a city population of 250,000. The factors in Table 2 should be used to adjust the values in this table for other populations. 

aData from Leisch's turn lane charts (4). 
bData from Leisch's two-way charts, no parking (5). 
cData from Leisch's one-way charts, no parking (6).
business district (OBD), and residential area are found in the Manual and Leisch's paper, but these are basically self-explanatory. This table was developed assuming an MP of 1.00, which corresponds to a city of 250,000 population. Correction factors for use with cities of different populations will be discussed later.

In relating the level of service and failure rate procedures, the first step is to determine the maximum allowable average arrivals to an approach lane of an installation with a given cycle length and a given green time, while holding the vehicles per hour of green constant. This may be done by converting the vehicles per hour of green to vehicles per second of green per lane. The length of green time for the particular phase may be multiplied by the allowable vehicles per second of green time per approach lane to determine the maximum allowable average arrivals per cycle to an approach lane.

It is to be noted that this is in reality the basic operation performed while using the Manual's design procedure. Assume that an approach volume is given and it is desired to know the green requirements for a particular level of service. The ratio of approach volume to the given vehicles per hour of green determines the required signal split. The equations for this operation would be as follows:

\[ \frac{g}{c} = \frac{v_{ph}}{v_{phg}} \]

\[ v_{ph} = v_{phg} \left( \frac{g}{c} \right) \]

if \( m \) = maximum arrivals per cycle, then

\[ m = \frac{v_{ph}}{\text{cycles/hour}} = \frac{v_{ph}}{3600/\text{cycle length}} = \frac{v_{phg}(g/c)}{3600/\text{cycle length}} \]

Note that the cycle lengths cancel out, leaving

\[ m = \frac{\text{green}(v_{phg})}{3600} \]

where green is in seconds of green time.

This expression is the same as discussed earlier in that the product of green time and vehicles per second of green yields maximum average arrivals per cycle for a given vehicles per hour of green.

With this relationship, points may be plotted on the failure rate chart (Fig. 1) for various numbers of vehicles per hour of green. However, an adjustment must be made to the chart's green time because it contains yellow times and the Manual data do not include yellow times. The authors have assumed a 3-second yellow time for the purpose of this presentation. The Manual states that 2 or 3 seconds' yellow time may normally be expected. However, 3 seconds is generally accepted as a minimum yellow time. The actual suggested computational procedure presented later is such that one may use any yellow time in making level of service checks.

To illustrate the conversion procedure, assume that it is desired to determine the maximum average arrivals per cycle that may be accommodated for a level of service C at a 2-lane through approach. Neither left nor right turns are considered beyond those built into the Manual's charts (the assumption is made that left turns are handled by a separate phase). Also assume that the installation is at an outlying business district location. Data given in Table 1 indicate that no more than 1,200 vehicles per lane per hour of green should approach the intersection to maintain a level of service C. This converts to \( \frac{1}{2} \) vehicle per second of green. If the actual green time per cycle is 30 seconds, then, in order to plot a point on the failure rate graph, the green time must be multiplied by the vehicles per second of green, which in this example results in an allowable average arrival of 10 vehicles per cycle. A point may then be plotted by determining the intersection with the m arrivals of 10 vehicles per cycle and the green curve of 33 seconds, remembering that the curves include yellow time. This procedure has been followed for the cases given in Table 1, and the various level of
service lines have been plotted. Figure 2 shows a plot for a single left turn lane, and Figure 4 shows a plot for a 2-lane approach in a city of 250,000 population and an OBD or fringe area.

However, to this point it has been assumed that the peak-period flow rate equals the peak-hour flow rate, because Figures 2 and 4 are plots of failure rates for the whole peak hour and level of service in terms of vehicles per hour of green. In order to obtain a relationship between peak-period failure rates and levels of service, Drew and Pinnell's equation for determining peak-period factor must be used. As an illustration, each of the m average arrivals plotted in Figures 2 and 4 have been factored up using a peak-period factor obtained from Drew and Pinnell's equation. A city population of 250,000 and an OBD location have been assumed, which is consistent with the Table 1 assumptions. The result of this factoring is shown in Figures 3 and 5. As to be expected, the peak-period failure rate based on a given vehicles per hour of green is greater than the peak-hour failure rate, which according to Drew and Pinnell (3) is not accurately described by the Poisson distribution. As an illustration of the effect that population has on the relation of failure rate and level of service, Figure 6 has been developed to show the varying relationship of failure rate and the lines for the level of service C for populations of 100,000, 250,000, and 1,000,000. The vehicles per hour of green for the different populations were determined using Leisch's MP factor, which will be discussed later. The level of service C is considered significant inasmuch as the Highway Capacity Manual states that this is the level typically associated with urban design practice.

In making actual conversions from failure rate solutions to level of service, the 2 illustrated figures may be consulted if the approach under consideration satisfies the description contained within the figures presented. The procedures would be to enter the graph with the average arrivals per cycle for the peak hour (not peak period) and the computed green time. The area where the intersection of the 2 variables lie would

Figure 2. Failure rate and level of service for a single left-turn lane approach (no peak period correction applied).
Figure 3. Failure rate and level of service for a single left turn lane approach (peak period correction applied).

Figure 4. Failure rate and level of service for a two-lane approach, city of 250,000, and an OBD or fringe location (no peak period correction applied).
Figure 5. Failure rate and level of service for a two-lane approach, city of 250,000, and an OBD or fringe location (peak period correction applied).

Figure 6. Failure rate and level of service C for different city populations.
indicate the level of service. For example, if the approach under consideration satisfies the description of Figure 4, with an average peak-hour arrival rate of 10 vehicles per cycle and a green time of 30 seconds, a level of service E would be indicated. Note that the intersection is at the 30 percent failure level if a peak-period factor of 1.18 is used commensurate with a city population of 250,000.

Charts could have been prepared for all conditions, but this did not appear warranted. Therefore, an alternate solution procedure would be to take the arrivals per phase per cycle and the green time allotted to each phase and compute the arrival rate in vehicles per second of green (vpsg). Of course, yellow time would be deducted from the green time determined by the failure rate method to determine the arrival rate in vehicles per second of green. It may be noted that Table 1 also gives levels of service in terms of vehicles per second of green. Because Table 1 covers all general cases developed by the Manual, which are considered appropriate for use in solving high-type intersection problems, it may be consulted to determine the level of service for the installation. This table is for a city population of 250,000, and Table 2 should be consulted to adjust the Table 1 values of vpsg, if a city of population other than 250,000 is under consideration. As an illustration, the previous example of 10 arrivals and 30 seconds of green converts to 0.370 vpsg after deducting the assumed 3 seconds' yellow time, which, for use with Table 1, could actually be any reasonable length to be used.

As in the previous example, the description shown in Figure 2 will be assumed, and Table 1 will be used directly, because Figure 2 shows a population of 250,000. The OBD or fringe row and the 2-lane column will be consulted. Because 0.370 vpsg lies between 0.366 and 0.377, a level of service E is again indicated. As stated earlier, if the city population is other than 250,000, the Table 2 factors must be applied. These factors are from Leisch's paper (4). If the actual peak-period factor and population are known, they may be used as shown in Figures 6.5 through 6.9 of the Highway Capacity Manual to obtain an equivalent MP factor for use. To illustrate the adjustment for population, assume that the previous example is used, except that the city population is over 1,000,000. The Table 1 values for a 2-lane approach in a CBD area with appropriate correction factors are given in Table 3. As may be observed, a vpsg of 0.370 is less than the flow rate of 0.380 vpsg, the upper limit for level of service A. Therefore, this design would operate at an A level of service if it were in a city of over 1,000,000 population.

If the failure rate method is to be used by a city traffic engineering department, it is recommended that all data given in Table 1 be converted to the population of the city. Table 1 may then be used as a fast check of the level of service, as defined by the Manual, which would result by designing by the failure rate method. Of course, the average arrivals for the peak hour, not peak period, must be used with this table.

Some conclusions may be drawn from the slope of the level of service lines (Figs. 2 through 6). The slope of these lines indicates that failure rate and level of service, as defined by the Manual, apparently do not correlate. If they were well correlated, one would expect the level of service lines to be relatively level and consistent for different populations. Therefore, it appears that a practical failure rate design level may not be established as suggested by Drew and Pinnell. The acceptable failure rate decreases as the approach volume per cycle increases. Each failure rate solution should therefore be checked by Table 1 to determine the level of service for that solution.

**TABLE 2**

ADJUSTMENT FACTORS TO BE USED WITH TABLE 1 DATA FOR CITIES OF OTHER THAN 250,000 POPULATION

<table>
<thead>
<tr>
<th>Population</th>
<th>MP</th>
<th>Population</th>
<th>MP</th>
</tr>
</thead>
<tbody>
<tr>
<td>50,000</td>
<td>0.85</td>
<td>500,000</td>
<td>1.05</td>
</tr>
<tr>
<td>100,000</td>
<td>0.90</td>
<td>750,000</td>
<td>1.10</td>
</tr>
<tr>
<td>175,000</td>
<td>0.95</td>
<td>1,000,000</td>
<td>1.15</td>
</tr>
<tr>
<td>250,000</td>
<td>1.00</td>
<td>over 1,000,000</td>
<td>1.20</td>
</tr>
</tbody>
</table>

**TABLE 3**

EXAMPLE OF CONVERSION FOR POPULATION

<table>
<thead>
<tr>
<th>Level of Service</th>
<th>vpsg  (Table 1)</th>
<th>1,000,000 Population MP Factor</th>
<th>Adjusted vpsg</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>&lt;0.317</td>
<td>1.20</td>
<td>&lt;0.380</td>
</tr>
<tr>
<td>B</td>
<td>&lt;0.324</td>
<td>1.20</td>
<td>&lt;0.389</td>
</tr>
<tr>
<td>C</td>
<td>&lt;0.333</td>
<td>1.20</td>
<td>&lt;0.400</td>
</tr>
<tr>
<td>D</td>
<td>&lt;0.366</td>
<td>1.20</td>
<td>&lt;0.439</td>
</tr>
<tr>
<td>E</td>
<td>&lt;0.377</td>
<td>1.20</td>
<td>&lt;0.453</td>
</tr>
</tbody>
</table>
AVERAGE INDIVIDUAL DELAY AT A SIGNALIZED INTERSECTION

Some investigators in the field of signalized intersections have recommended that average individual delay be used as an indicator of the level of service offered by a signalized intersection. May and Pratt (5) have expressed some difficulty in correlating load factor with average delay at high load factors. They developed revised level of service load factor limits to obtain a more uniform divergence of average delay for the different levels of service. Their recommendation is given in Table 4.

Because there appeared to be some support for a delay approach to level of service, an investigation was made into the relation that failure rate bears to average individual delay.

To develop this relationship, the delay equation derived by Webster (6) was utilized. Webster's equation is as follows:

\[ \bar{d} = \frac{c(1 - \lambda)^2}{2(1 - \lambda x)} + \frac{x^2}{2Q(1 - x)} - 0.65 \left( \frac{c}{Q^2} \right)^{1/4} x (2 + 5\lambda) \]

where
- \( \bar{d} \) = average delay per vehicle on the particular lane passing through the intersection;
- \( c \) = cycle length;
- \( \lambda \) = proportion of the total that is effectively green for the phase under consideration, \( (\text{green} - \text{lost time}) \), where lost time is the green time not effectively utilized cycle each phase (Webster recommends a lost time of 2 seconds for the average installation);
- \( Q \) = lane flow = average number of vehicles passing a given point on the road in the same direction per unit of time;
- \( S \) = saturation flow = maximum rate of discharge of the queue during the green period; and
- \( x \) = degree of saturation = ratio of actual flow to maximum flow that can be passed through the intersection on a given lane = \( Q/\lambda S \).

An inspection of the variables in the equation indicates that the average individual delay may be computed for each point of intersection of the average arrivals and the green time with its associated maximum departures. A lost time per cycle of 2 seconds was assumed, as recommended by Webster (6). The computations have been made for 40-, 50-, 60-, 80-, and 100-second cycles for a representative number of intersection points.

Knowing the delay at the points where delay was computed, equi-delay lines, corresponding to May and Pratt's recommended delay break points for level of service determination, were plotted. These equi-delay lines were plotted in similar fashion to a contour map for delays of 15, 30, 45, and 60 seconds. Figures 7 and 8 show the equi-delay lines for a 60- and 100-second cycle. Here again some conclusions may be drawn from the slope of the equi-delay lines. Because they are not horizontal, one must conclude that average individual delay apparently does not correlate with failure rate. The acceptable failure rate for a given delay line increases with an increase in volume per cycle.

These 2 charts may be used for checking average individual delay for solutions based on both the failure rate and the level of service methods. Similar charts could easily be developed for a full range of cycle lengths. The delay range may be found by entering the charts with the average arrivals per cycle for the peak hour and finding the intersection with the computed green time and noting where the point of intersection lies with respect to the equi-delay lines.

<table>
<thead>
<tr>
<th>Level of Service</th>
<th>Revised Load Factor Limits</th>
<th>Average Individual Delay (seconds per vehicle)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>(&lt; 0.1)</td>
<td>(&lt; 15)</td>
</tr>
<tr>
<td>B</td>
<td>(0.1 - 0.66)</td>
<td>(&lt; 30)</td>
</tr>
<tr>
<td>C</td>
<td>(0.66 - 0.82)</td>
<td>(&lt; 45)</td>
</tr>
<tr>
<td>D</td>
<td>(0.72 - 0.91)</td>
<td>(&lt; 60)</td>
</tr>
<tr>
<td>E</td>
<td>(1.0)</td>
<td>(\geq 60)</td>
</tr>
</tbody>
</table>
Figure 7. Failure rate and average individual delay in seconds per vehicle for a 60-second cycle.

Figure 8. Failure rate and average individual delay in seconds per vehicle for a 100-second cycle.
RELATION OF LEVEL OF SERVICE TO AVERAGE INDIVIDUAL DELAY

A general inspection of the slopes of the lines for the 2 graphical relationships, developed to this point, raises some question as to the relationship that level of service, as defined by the Manual, actually bears to average individual delay.

Figure 9 shows delay lines for a 60-second cycle (Fig. 7) and the level of service lines for the 2-lane approach (Fig. 4). It is readily apparent that the level of service lines cut across all delay lines indicating delays from less than 15 seconds to over 60 seconds. This indicates that 2 signalized intersections may satisfy the Manual's criteria of vehicles per hour of green for a given level of service, but the intersections may have considerable difference in average individual delays, depending on the volume being accommodated per cycle. An inspection of the other delay and level of service charts indicates that similar relationships exist for other cycle lengths and approach configurations.

This deviation is due in part to the fact that the Manual's criteria of vehicles per hour of green does not take into account cycle length or g/c ratios. Cycle length is well known for having an effect on average delay; this subject has been investigated by several individuals including Webster (6). The g/c ratio has an effect on delay due to the different percentages of the green time that are not utilized. A design start up and clearance time is stated by Drew and Pinnell (3) to be 6 seconds per phase. The fact that the Manual does not include yellow time in its green designation does not offset the loss of 6 seconds per cycle. Therefore, as green times get shorter, there is an inequity in the relationship that green time less yellow time bears to usable green time (green time less start up and clearance time).

![Figure 9. Level of service for a two-lane approach related to average individual delay for a 60-second cycle (Figs. 4 and 7 combined).](image-url)
CONCLUSIONS AND RECOMMENDATIONS

When using the failure rate method of intersection design, one should check the level of service actually provided. Delay checks should also be made for each phase. The tables and charts presented here could be of some use in making these checks. For use in a particular city, Table 1 should be adjusted for that city's population.

It is recommended that consideration be given to including average individual delay as an index of the level of service offered by a signalized intersection. If speed is to be considered the criterion for uninterrupted flow conditions, then a delay index appears commensurate for intersection design. Perhaps one of the objections to using delay is the difficulty in obtaining field data for average individual delay. No doubt load factor is considered an easier field measurement. However, Sagi and Campbell (7) have developed an equation for determining average individual delay that does not require any more field work than a load factor determination. Perhaps this may remove some objections to the use of average individual delay as an index of the level of service at an intersection. If delay is to be accepted as an index, the Manual's present term of vehicles per hour of green will have to be modified to account for cycle length and g/c ratios. An inspection of Figure 9 makes this need apparent.

If average individual delay is to be used extensively for design, it is recommended that a nomogram be developed to facilitate the use of Webster's equation. Using such a nomogram, actual saturation flow rates and start up and clearance times could be used for a particular intersection and average individual delays easily obtained.

ACKNOWLEDGMENT

This paper is based on Mr. Tidwell's thesis, The Relation of Failure Rate at a Fixed Time Signalized Highway Intersection to Level of Service as Defined by the 1965 Highway Capacity Manual, presented to the University of Tennessee, September 1969. The assistance of W. H. Wilson and members of the Research and Planning Division of the Tennessee State Highway Department in the preparation of the illustrations is greatly appreciated.

REFERENCES


Discussion

G. W. SKILES, Los Angeles Department of Traffic—One view of the research reported on here is that 2 alternative approaches to the capacity analysis of signalized intersections yield results that are inconsistent and that neither approach produces solutions that correlate with average delay, a quantity thought by some to be a desirable figure of merit.

Taking this view, the results of the research are disappointing. There is not much advantage in having alternative approaches if they will not give the same answer. It is worse if neither answer is correct.
However, the purpose of the research was not to evaluate the 2 (or 3) alternate methods of capacity analysis. It was to develop a methodology whereby a solution obtained by using the failure rate method of design could be expressed in terms of expected level of service and delay. This was done. The results could be of significant value to those who wish to use the failure rate technique.

In addition to allowing a solution to be expressed in terms that may be more meaningful, the authors' results could provide a very useful aid in the use of the failure rate design procedure. As Drew and Pinnell point out, in using their procedure one has an infinite combination of phase lengths from which to choose. The same failure rate need not be used for all phases.

The authors' methodology provides the designer with a tool for limiting his appropriate field of choice. If, for example, he wishes to provide the same level of service for opposing phases, the failure rate-level of service comparison allows him to do so quite easily. Similarly, the chart showing both failure rate and delay facilitates a selection of phase length combinations for equal average delay or for any desired ratio of delays.

The unfortunate part of the authors' results is that one is still faced with an apparently conflicting choice. His selection of appropriate phase lengths will differ, depending on whether he wishes to use delay or level of service as a criterion. This may or may not be logical or desirable.

The authors' implication is that the element in error is the level of service criterion. I am less certain that this is the case. The final figure shows failure rate, level of service, and average delay superimposed. An examination of the chart for logical relationships does not lead to firm conclusions. However, there are some such relationships.

Comparing failure rate and level of service, it is found that, for a given approach volume, as failure rate increases, the level of service decreases—a result one should expect. Why the level of service curves should not be more nearly parallel to the failure rate lines, though, I do not understand. If I do understand the procedure followed, the level of service curves (expressed in terms of service volume) are derived basically from the load factor curves in the Highway Capacity Manual. By definition, load factor and failure rate are very similar. Miller (8) indicates that they are related approximately by the ratio $e^{\frac{1}{560}}/e^{\frac{1}{300}}$, where $\phi$ is a function of flow rate, saturation flow rate, and degree of saturation.

I note, incidentally, that the level of service curves as plotted on the failure rate charts show, for a given level of service, a nearly constant ratio of $m/(x + 1)$, where $m = \text{average volume per cycle and } x + 1 = \text{the lowest volume constituting a cycle failure}$. I wonder if this has a pertinent meaning.

A rationale that could be developed from the failure rate-level of service comparison goes like this:

At lower volumes, a higher failure rate can be accepted for a given level of service because, upon failure, there are fewer vehicles in queue and green intervals occur frequently. The queue left over can be expected to clear the next cycle. A cycle failure will occur only 20 percent of the time, at most, if one limits his choice to the A through E level of service area.

At higher volumes, a cycle failure is more serious. The number left in queue is likely to be higher than for the previous case. Hence, for a given level of service, one should use a lower failure rate for higher volume levels.

The failure rate-average delay comparisons indicate an opposite conclusion. After all, the reason one would be concerned about the number of times a queue remains, and the number left in queue, is because of the effect on delay. As the authors point out, though, the results indicate that a higher failure rate can be accepted at higher volume levels for constant delay. To me, this is not a logical relationship. I question that it should be so, although my main objection may be that it casts a shadow on my earlier rationalization.

The failure rate-delay comparisons do show some logical consistencies. For a given cycle length and green interval, delay and failure rate increase as average
approach volume increases. For a constant cycle length and volume, delay and failure rate increase with decreasing green time. For a given green interval and average arrival rate (g, m, and x constant), delay increases with increasing cycle length; failure rate is constant.

These relationships are all to be expected, of course, and, so far, I do not find justification for my uneasiness. However, the last relationship, especially, may illustrate something. In that case, failure rate is constant, even though cycle length and delay are varying. In other words, delay is not a function of failure rate. Failure rate is not a function of cycle length. Both failure rate and delay are functions of green time and arrival rate; delay is, in addition, a function of cycle length. It may be incorrect to compare the 2 quantities, delay and failure rate, in the way that was done.

A point noted from the final chart is that the degree of saturation (in Webster's equation) exceeds unity above a line roughly approximating the 45-second delay line on the 60-second cycle chart. If the delay relationship is correct and the level of service relationship is incorrect, this would indicate that the upper limit of level of service E should be at about that same point. Perhaps the level of service curves, then, should have slopes nearly the same as those for delay. This might be a starting point for revising the level of service curves.

Another point brought out by the final chart is, I think, much more interesting. That is that product \( d \times m \) is nearly constant for a given level of service. In other words, for the conditions of that chart, there is a close correspondence between level of service and total delay (not average individual delay). If this relationship is consistent for other conditions, the result could be extremely meaningful.

I conclude that I have no firm views on the apparent inconsistencies in the authors' final result, except that, possibly, the most disturbing inconsistency is eliminated if one uses total delay rather than average individual delay. I do have the observation that, while delay is an appealing figure of merit, the relationship between service volume and delay is often an erratic one. Normann (9) pointed this out and gave this as one reason for selecting load factor, rather than delay, as a criterion for signalized intersection capacity. Some of our studies have shown similar inconsistencies between service volumes and load factor. May and Pratt's study, referred to by the authors, does not show a consistent relation between load factor and delay except at very low load factor levels. We seem to have need for additional facts.

In reviewing the paper, a question keeps arising: Should we really be surprised if figures of merit developed from bases involving different sets of assumptions fail to agree? Perhaps we should be more surprised if they did agree.

References

JAMES H. LITTLE, Missouri State Highway Commission—It is becoming increasingly more apparent that the search for the most acceptable method of determining intersection approach capacities must continue. The refined computation procedure described in the 1965 Highway Capacity Manual is unquestionably a step forward; however, it is not without its weaknesses. Chang and Berry have discovered apparent discrepancies between some of the Highway Capacity Manual curves, and, as pointed out by the authors of this paper, May and Pratt have found inconsistencies in the ranges of average individual delay associated with the various load factor groups. A study comparing estimated and observed service volumes of 90 signalized intersection approaches in Missouri's 3 largest cities has indicated average errors of 47.7, 32.0, 16.1, and -4.5 percent for service levels B, C, D, and E respectively. Clearly, further refinements of the data and procedures contained in Chapter 6 of the Manual are needed.
The authors of this paper have brought us another step nearer the better solution we seek. By a rather ingenious application of the failure rate design chart developed by Drew and Pinnell, they have demonstrated that the failure rate design method and the current level of service design method are not well correlated. As the authors point out, we would expect the level of service lines of Figures 2 and 4 to be relatively level if good correlation exists. An examination of their charts shows that this would be possible only if the Manual’s vehicles per hour of green for a given level of service were allowed to vary with the allotted green time.

In Figures 3 and 5, the authors show that better correlation exists between the 2 design methods when the flow rates for the various service levels are adjusted to represent peak-period rather than peak-hour rates. In fact, Figure 5 shows that the correlation between the 2 methods is reasonably good if a 2-lane approach is designed for adjusted capacity (level E) flow rates.

In the latter part of their paper, the authors conclude that deviations in the relationship between average individual delay and level of service curves are due in part to the fact that the Manual’s criteria of vehicles per hour of green does not take into account cycle length or g/c ratios. This shortcoming may also explain why the failure rate and level of service curves are not better correlated.

Inasmuch as the authors conclude their paper by recommending that consideration be given to including average individual delay as an index of the level of service and modifying the Manual’s present term of vehicles per hour of green to account for cycle length and g/c ratios, their earlier conclusion that it may not be possible to establish a practical failure rate design level may be premature. For the present, the failure rate method at least provides a good check of the adequacy of a level of service design.

One limitation of the failure rate design method developed by Drew and Pinnell is that the equation for estimating peak flow rate within the peak hour does not apply to the larger metropolitan areas. If applied to a metropolitan area of 1,667,000 or more population, it will indicate a peak-period flow rate less than the average peak-hour flow rate. Wherever possible, actual counts at the site in question, or average peak-hour factors, should be used to determine peak flow rates.

The table developed by the authors (Table 1) for estimating the level of service provided by a given approach, when the lane layout and signal phasing are known, might be useful for a rough check of level of service on approaches similar to those covered by the table; however, because of the many possible combinations of lane width, percent turns, and the like, it is felt that generally it would be better to use the Manual’s charts, or the nomographs developed by Leisch, in making such checks.

The authors’ use of average individual delay curves corresponding to the load factor break points suggested by May and Pratt was of considerable interest to those of us who worked on the previously mentioned Missouri study because we feel these break points are more realistic than those presently in use.

The authors’ finding that average individual delay does not appear to be correlated with level of service, as presently defined by the Manual, is noteworthy. Their suggestion that the correlation might be improved by making the Manual’s criteria of vehicles per hour of green more responsive to the effects of cycle length and g/c ratios deserves serious consideration.

Messrs. Tidwell and Humphreys have made a significant contribution to the store of knowledge concerning signalized intersection capacity and should be congratulated on the result of their efforts.

DAVID SOLOMON, U.S. Department of Transportation, Federal Highway Administration, Bureau of Public Roads—The authors have certainly presented a very useful paper. Their general finding, that there is no correlation between level of service, failure rate, and average individual delay, suggests that it would be useful to investigate at a more fundamental level the basic criteria employed in evaluating intersection performance.
The basic criteria used in these analyses are delay, stops, and travel speed or time. The question is, How do drivers evaluate these criteria? For example, is one minute of delay and one stop less desirable than 20 seconds of delay and 2 stops? Once a better understanding has been obtained of these relationships, it will be possible to design signal timing schemes based on criteria that are important to drivers.

The next questions is, How should research on the desires of drivers be carried out? A number of techniques might be explored. Direct questions might be asked, or a more advanced type interview technique could be employed involving development of an attitude scale.

An experimental approach might be tried in a laboratory. A group of test subjects could be shown films of several traffic situations and asked to evaluate delay and stops in terms of a subjective scale or in terms of the cost they would assign to each level of delay or number of stops.

Field experiments could be tried by giving drivers alternate route assignments, having them evaluate the routes subjectively or in terms of cost, and correlating with the stops and delays. A refinement of this could involve giving test subjects a certain sum of money and requiring them to pay back some of it in return for reduced stops or delays. This could be employed in either a laboratory-type situation or on the street, with car pools, for example.

JOHN E. TIDWELL, JR. and J. B. HUMPHREYS, Closure—The authors would like to thank the Highway Research Board for the opportunity of bringing their findings to the attention of the profession. Appreciation is also expressed to Messrs. Skiles, Little, and Solomon for their discussion comments. Their comments are very appropriate and should be of assistance in the further exploration of this topic.

By way of specific comment, Mr. Skiles' suggestion that total delay and the level of service lines may be correlated is not borne out when other cycle lengths and populations are taken into consideration. Admittedly, the correlation is greatly improved over the average individual delay-level of service relationships. The writers realize that total delay may be a useful index of signal efficiency, but, because total delay intimates that average individual delays may vary depending on average arrivals, we do not recommend it for a level of service index. Miller's work (8) regarding a load factor equation has been followed up. A plot of the 0.1, 0.3, 0.7 load factor lines on the failure rate chart yields peak-hour failure rates of approximately 7, 18, and 32 percent. This compares closely with a simulation study by the writer for Poisson arrivals. A 1.00 load factor was obtained at the 55 percent level. This still does not give a relation of peak-period failure rate to the peak-hour load factor. Additional research in this area should prove useful.

We can find no meaningful explanation for Mr. Skiles' \( m/(x + 1) \) relationship. Following Mr. Skiles' lead, a rationale for the failure rate-delay conclusion is as follows: Two signals may have the same failure rate with one having low arrivals and low green time and the other higher arrivals and longer green time. The signal with the short green obviously must have a longer red phase than the one with the long green time. Therefore, any overflows from a previous cycle or arrivals during a red phase must wait longer in the queue than would be necessary for the signal with a long green phase.

If this paper has generated meaningful discussion, which may lead to "the better solution" referred to by Mr. Little, then the time and effort expended in the preparation of this paper have been very worthwhile.