# HEADWAY APPROACH TO INTERSECTION CAPACITY

Donald S. Berry, Northwestern University; and
P. K. Gandhi*, Howard, Needles, Tammen and Bergendoff, Kansas City

The capacity of an approach to a signalized intersection is a function of approach headways of vehicles as the queue of waiting vehicles is discharged into the intersection and of lost time due to starting delay and to utilization of only part of the yellow light interval. This paper suggests a method for computing capacity of signalized intersections from measurements of headways, starting delays, and utilization of the yellow light and presents some preliminary data on its application.

•BARTLE, SKORO, AND GERLOUGH (1) measured starting delays and approach headways for loaded portions of cycles for many intersections in Los Angeles and proposed a formula for computing approach capacity of a fixed time signal. The formula for capacity of each cycle is $n = (g + a - d)/h$, where g is length of green in seconds, a is length of yellow, d is average starting time delay for first vehicle in peak hours, and h is average approach headway for loaded portions of cycles.

Webster (2) utilized the saturation flow of the approach (in vehicles per hour of effective green) as the basic measure of capacity. Saturation flow was measured by recording the number of vehicles that cross the stop line in saturated phases during successive 2-sec intervals after the start of the green and with information on commercial vehicles and turns (15). In computing saturation flow, data in the first few periods after the start of the green were excluded. Effective green time is chosen so that the product of saturation flow and effective green time equals the average number of vehicles passing during the combined green and yellow intervals. Effective green is green plus yellow minus lost time. Lost time averages about 2 sec per phase excluding all-red intervals.

Miller (3) also utilizes the concepts of saturation flow, lost time, and effective green but makes capacity computations by lane of approach. In the Australian Guide (14), saturation flow is defined as the reciprocal of the average time headway, in which the average headway for the loaded portion of a cycle is measured from the start of the green and 1 sec is subtracted because of starting time delay. Lost time is taken as the longer of intergreen time minus ½ sec or the travel time through the intersection plus 2½ sec. Effective green is defined such that the number of vehicles that cross the stop line in a fully saturated phase is equal to the product of the saturation flow and the proportion of effective green time.

In both the British and Australian methods, saturation flow values are given for ideal conditions. In defining capacity for prevailing conditions, through-car equivalents are utilized to correct for commercial vehicles and turns, and reduction factors are used to correct for effects of parking and grades.

## METHOD

The method outlined in this paper differs from the Australian and British methods primarily in the procedures used for determining saturation flow and lost time. Effective green is determined from measurements of starting delay, utilization of the

*Mr. Gandhi was with Northwestern University at the time of this research.

yellow, and length of the green as in the numerator of Eq. 1. This equation also takes into account the fact that the number of vehicles entering a loaded cycle is one more than the number of headways. Average values for headways, starting delays, and utilization of the yellow are used in Eq. 2 to compute the lane capacity or approach capacity in vehicles per hour.

$$n = \frac{g + \lambda y - D}{\bar{h}} + 1 \tag{1}$$

$$Cap = \frac{3,600 \,(g + \bar{\lambda} y - \bar{D} + \bar{h})}{C\bar{h}} \tag{2}$$

where

- $n$ = number of vehicles discharged from one approach during one loaded cycle;
- Cap = capacity of the signalized approach in vehicles per hour;
- $D$ = starting time delay in seconds elapsing from beginning of green to instant the rear wheels of the first vehicle cross the reference line (usually the stop line);
- $\bar{h}$ = average headway time, in seconds, for all vehicles in a compact platoon that cross the reference line (in Eq. 2, $\bar{h}$ is the average for a large sample of cycles);
- $\lambda$ = proportion of length of yellow light, for a loaded cycle, which is utilized up to the time the last vehicle in a compact platoon crosses the reference line;
- $C$ = length of signal cycle in seconds;
- $g$ = length of green in seconds; and
- $y$ = length of yellow in seconds.

## EXPERIMENTAL STUDY

Data were taken at one 18-ft approach to a three-phased signalized intersection to evaluate the proposed method and to study its application in evaluating effects of weather and visibility on capacity (4). The south approach of the intersection of Ridge and Church Streets, Evanston (Fig. 1), was selected because there were no left-turning traffic, no opposing flow, no vehicles parked, standing, or stopping, practically no commercial vehicles, and practically no pedestrians to interfere with right-turning vehicles. Those few cycles with buses and pedestrian interference were excluded from the study. All data were taken on weekdays during the evening peak period when about 95 percent of the cycles were loaded cycles. Cycle length was 60 sec with 17 sec green and 3 sec yellow.

Starting delays were measured with a stopwatch that makes one revolution in 10 sec. The stop line, which is 24 ft from the intersection as determined by a prolongation of the curblines, was used as the reference line. A second stopwatch was started as the rear wheels of the first vehicle crossed the stop line and was stopped when the last vehicle in the compact platoon crossed the stop line with its rear wheels. The elapsed time, T, shown on this second stopwatch, was then divided by the number of vehicles, less one, to determine the average headway, h, for the compact platoon. For a loaded cycle, the utilization of the yellow was $\lambda y = D + T - g$.

Results for 14 peak periods of data collection are given in Table 1, with data segregated according to weather and visibility conditions. Average queue discharge factors are shown for 60 loaded cycles on each of the days along with capacity values for each day as computed from Eq. 2 and as observed for 60 loaded cycles.

Statistical tests were performed to examine consistency among results for days having the same conditions. Tests were conducted at the 1 percent significance level under the null hypothesis that the mean headways come from the same population. For the 4 "dry-night" days, the mean values for each of the 4 days were not significantly different from the 4-day average. For the 5 "dry-daylight" days, 1 day had a mean headway significantly different from the 5-day average. Also, mean headway values for the two "wet-night" studies were significantly different from each other, perhaps because of the differences in intensity of rainfall. Such differences are being given further study by collecting additional data for other intersections.

Statistical tests were also performed using the null hypothesis that the mean headways for each set of adverse weather and visibility conditions were the same as for dry-daylight conditions. Comparisons were made between dry-night and dry-daylight conditions and between wet-night and dry-night conditions. The null hypothesis had to be rejected for all significance levels above 1 percent in both cases, indicating that adverse weather significantly increased headways.

Table 1 also shows that capacity values computed from Eq. 2 check closely with observed counts of 60 loaded cycles, as would be expected. Computed values for adverse weather and visibility are substantially lower than those for dry-daylight conditions. For dry pavement, capacity at night was 9.4 percent lower than that for daytime; wet-night capacities were 16.2 percent lower than those for dry-daytime conditions. Observed values for dry-daylight conditions are as much as 32 percent higher than those computed by Highway Capacity Manual (12) methods.

Othman and Rapino (5) studied utilization of the yellow for loaded cycles at the same intersection, for both the south and north approaches, and by lane for the south approach. Results given in Table 2 show values for $\lambda y$ for the south approach that are not significantly different from those for the north approach where the reference line was 3 ft closer to the intersection. Average values by lane for the south approach were similar (1.53 and 1.56 sec) and were somewhat higher than the 1.43 sec average per lane reported by Leong for Australian conditions (6).

The effects of position of the reference line on starting delay and utilization of the yellow were also examined by Othman and Rapino (5), for the south approach at Ridge and Church Streets, Evanston. A reference line 12 ft from the intersection would correspond to a position where the front of the average passenger car would be entering the intersection as its rear wheels cross the reference line. If the stop line remained 24 ft from the intersection (Fig. 1), use of a reference line 12 ft from the intersection instead of 24 ft would increase measured starting delay and increase utilization of the yellow. These effects tend to cancel each other when Eq. 2 is used to compute capacity. Further study is needed at other intersections to determine the location of the reference line best suited to varying positions of stop lines in relation to the intersecting curblines.

## APPLICATION

Equation 2 can be applied to compute capacity for prevailing conditions at existing signalized intersections, as was done in the experimental study, without need for corrections for commercial vehicles, turns, parking, and so forth. Observers would measure headways, starting delays, and utilization of the yellow for loaded cycles and then utilize Eq. 2 to compute the capacity. In the event that there are no loaded cycles, typical values of $\bar{\lambda}$ can be used based on driver practice in the area. Corrections for turns, commercial vehicles, and so forth are not needed if the traffic conditions during field studies are typical of the average peak-hour conditions.

Utilization of this method for intersections not yet signalized, or where changes are expected in physical layout, controls, or traffic characteristics, will require development and use of standard values and adjustments as was done for the Australian method. This will involve compiling average values for starting delays and utilization of the yellow and lane headways (in through-car units). Also needed will be through-car equivalents for right turns and commercial vehicles, left-turn reduction factors as affected by opposing flow, and correction factors for parking, buses, grades, and lane width.

Some data on saturation flows and on reduction factors have already been collected in the United States. Passenger-car headways by lane and passenger-car equivalents for trucks and turning vehicles have been evaluated by Carstens for streets in Ames, Iowa (13). Effects of opposing flow on passenger-car equivalents for left turns have been studied in Australia (3) and at Northwestern University (7, 8).

**Table 1. Capacity computed from queue discharge factors.**

| Weather Visibility and Date | Number of Loaded Cycles | Loaded Cycle Performance: Starting Delay, $\bar{D}$ | Headways $\bar{h}$ | Headways $\sigma$ | Utilization of Yellow, $\bar{\lambda}y$ | Computed Capacity (vph) | Observed Count for 60 Cycles |
|---|---|---|---|---|---|---|---|
| Dry-day | | | | | | | |
| 3/22/71 | 60 | 2.379 | 1.107 | 0.047 | 0.967 | 905 | 904 |
| 3/23/71 | 60 | 2.607 | 1.086 | 0.049 | 1.574 | 941 | 942 |
| 3/25/71 | 60 | 2.490 | 1.074 | 0.039 | 1.746 | 968 | 968 |
| 3/29/71 | 60 | 2.485 | 1.071 | 0.047 | 1.293 | 944 | 946 |
| 4/15/71 | 60 | 2.457 | 1.089 | 0.054 | 1.460 | 941 | 944 |
| Average | | 2.483 | 1.085 | 0.049 | 1.408 | 940 | 941 |
| Dry-night | | | | | | | |
| 11/17/70 | 60 | 2.434 | 1.167 | 0.098 | 0.300 | 823 | 835 |
| 11/18/70 | 60 | 2.483 | 1.178 | 0.083 | 1.301 | 865 | 869 |
| 11/21/70 | 60 | 2.555 | 1.176 | 0.040 | 1.816 | 890 | 896 |
| 11/22/70 | 60 | 2.458 | 1.178 | 0.084 | 1.089 | 855 | 858 |
| Average | | 2.482 | 1.175 | 0.099 | 1.126 | 858 | 864 |
| Wet-night | | | | | | | |
| 11/16/70 | 60 | 2.670 | 1.256 | 0.070 | 1.408 | 810 | 813 |
| 2/4/71 | 60 | 2.762 | 1.318 | 0.135 | 1.193 | 762 | 772 |
| Average | | 2.716 | 1.287 | 0.112 | 1.300 | 786 | 792 |
| Snow-day | | | | | | | |
| 3/18/71 | 60 | 2.714 | 1.282 | 0.059 | 1.696 | 808 | 799 |
| 3/19/71 | 60 | 2.683 | 1.255 | 0.063 | 2.088 | 844 | 846 |
| Average | | 2.698 | 1.269 | 0.062 | 1.892 | 826 | 822 |
| Snow-night | | | | | | | |
| 2/12/71 | 60 | 2.638 | 1.283 | 0.064 | 2.073 | 829 | 821 |

**Figure 1. Site of experimental study.**

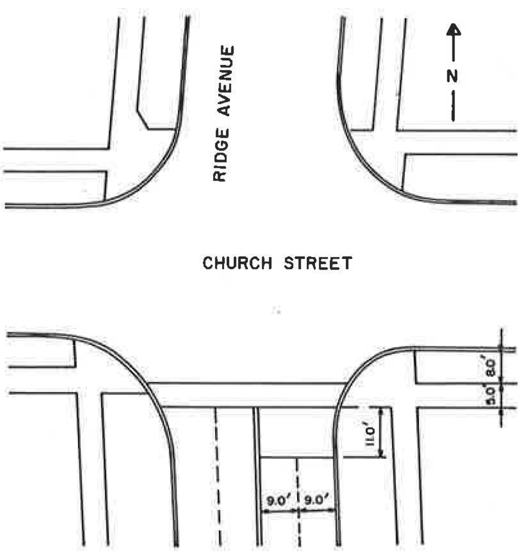


**Table 2. Utilization of yellow light for loaded cycles.**

| Approach | Evening Peak $\lambda y$ | Evening Peak $\sigma_{\lambda y}$ | Evening Peak $\lambda$ | Morning Peak $\lambda y$ | Morning Peak $\sigma_{\lambda y}$ | Morning Peak $\lambda$ |
|---|---|---|---|---|---|---|
| South | | | | | | |
| Both lanes | 1.89 | 0.78 | 0.63 | 1.92 | 0.71 | 0.64 |
| Curb lane | 1.56 | 0.82 | 0.52 | | | |
| Median lane | 1.53 | 0.65 | 0.51 | | | |
| North | | | | | | |
| Both lanes | 2.15 | 0.65 | 0.72 | 2.18 | 0.77 | 0.73 |

Note: Sample size varied from 38 to 99 loaded cycles (5).

Other studies by graduate students at Northwestern University have examined headways for double left turns on separate phasings (9) and on discharge rates by lane (10). Additional graduate students are extending these studies to additional intersections. We also are examining possible effects of reaction time in measuring starting time delay, as has been considered in Australia (3), and effects of grade of approach.

## RECOMMENDATIONS

Additional data on discharge performance of typical signalized intersection approaches of different widths should be collected by lane for different types of streets and areas to determine average starting delays, through-car headways, and utilization of the yellow for loaded cycles. This type of data collection might be incorporated in the information-gathering procedure for updating the Highway Capacity Manual (12), which is in the planning stage. In addition, extensive sampling should be carried out at a few approaches to evaluate through-car equivalents for left turns as affected by opposing flow (8), for commercial vehicles, and for effects of pedestrians, parking, bus stops, and grades.

These procedures for capacity computations should be extended to determining service volumes for levels of service other than capacity, utilizing level-of-service criteria such as delay, queue length, percentage of loaded cycles, and percentage of vehicles stopping.

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