Part 2. Capacity of Unsignalized Intersections

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This paper presents a method for calculating capacity, queue length, and delay at unsignalized intersections that was developed for and recommended by the new Swedish capacity manual. The method is based on a queuing model that considers each lane in the approaches controlled by yield or stop signs as a service position. Service time is calculated for each stream of vehicles in the lane, as a function of primary road flow rates and gap acceptance parameters. The lane capacity is assumed to be the reciprocal of the mean of these service times. Starting from the actual flow-to-capacity ratio, the queuing model estimates queue length, distribution and mean delay. The method is believed to apply to most intersections controlled by yield or stop signs. The most important parameter in the model is critical headway, which was measured at 18 Swedish intersections. The results range between 3.3 and 7.5 s, depending primarily on direction, speed limit, and type of control; they generally agree well with those of previous studies. A model similar to the one for unsignalized intersections was developed for traffic circles with short weaving sections.

Intersections without traffic signals were rather summarily treated in the previous Swedish capacity manual (5), and in the 1965 HCM (6). Nevertheless, a considerable number of total traffic delays in urban areas occur in such intersections; thus their performance and design have attracted increasing interest from planners and engineers. Simultaneously, a number of field studies and theoretical works have facilitated the development of useful calculation procedures (1); some countries have already developed such procedures (7, 8).

In the work on the new Swedish capacity manual (4), whose background and objectives are described in part 1 of this article, the subject of unsignalized intersections was considered a priority area, and investments were made in theoretical developments as well as in field studies. This paper presents the results of some of those efforts.

As explained in part 1, the calculation method is descriptive in nature. The model chosen is theoretical and based on road-user behavior. It is believed that, for the problem being considered, such a model ought to achieve greater consistency, with regard to the numerous influencing factors, than a purely statistical empirical approach.

Intersections here mean at-grade intersections, excluding weaving sections and those ramp connections common to grade-separated interchanges. Traffic circles, which are attracting attention in Sweden now, are treated in a separate chapter in the manual. The method recommended by the manual for signalized intersections is presented in part 3.

OVERVIEW

The method recommended for calculating capacity, queue length, and delay is based on a simple queuing theory model. Each lane in the minor road is represented by an M/G/1 queuing system with Markov-type arrivals and a general distribution of service times.

Models of this type easily lend themselves to the generalizations required to analyze any real intersection with interactions among a number of traffic flows. Similar approaches, using either the M/M/1 or the M/G/1 model familiar to queuing theorists, have been used by Surti (9) and others. More sophisticated models, such as Tanner's (10), that do not require assuming independent service times cannot be generalized to the same extent. Although some of the simpler assumptions inherent in the suggested model can be corrected for by choosing suitable parameter values, some accuracy has thus been sacrificed in order to gain comprehensiveness and consistency with regard to the many influencing factors.

PROCEDURE

The basic method procedure is described in Figure 2, where only the basic interaction between one minor and one major road flow is considered; the latter proceeds uninterrupted. The scheme can be explained in the following terms.

1. Frequency function \( f_1(h_1) \) describes the frequencies of headways of different size \( h_1 \) in the major road flow.

2. Distribution function \( G(\alpha) \) expresses the probability that a headway (gap or lag) of size \( h_1 = \alpha \) will be accepted by a waiting vehicle on a minor road.

3. Statistical distribution of the service time, \( d_s \), depends on frequency of headways and probability of acceptance of occurring headways. The capacity, \( C \), in vehicles per second, of the minor road approach must approximately equal the reciprocal of the mean service time \( d_s \), in seconds.

4. Frequency function \( f_2(h_2) \) describes the frequencies of headways of different size \( h_2 \) in the arriving minor road flow.

5. Statistical distribution of waiting time, \( d_w \), i.e., delay minus a retardation and acceleration component, depends on service times for vehicles first in queue and on frequency of intervals between arrivals. The same applies to queue lengths: the mean queue length, \( N \), in vehicles is related to mean waiting time, \( d_w \), in seconds and arrival rate, \( q_2 \), in vehicles per second through

\[
N = d_w \times q_2 \quad (1)
\]

Input parameters for the model in Figure 2 are headway distributions in the two flows, with means \( 1/q_1 \) and \( 1/q_2 \), and distribution of accepted headways.

In the calculation procedure, the function \( f_2(h_2) \) is assumed to be exponential, while \( f_1(h_1) \) depends on number of lanes and mean flow rate.

The gap-acceptance distribution, \( G(\alpha) \), is simplified to a step function and defines a critical gap as \( a \) seconds for one vehicle and assumes a constant increment for each additional vehicle. The values of the critical gap and the increment depend on variables describing intersection design and control and do, in

![Figure 2. Analysis procedure for interaction between a major and a minor road flow.](image-url)
fact, to a large extent, summarize the influence of such variables.

In an actual intersection, a number of such interactions are superimposed on each other; each interaction describes the crossing or converging movement between one major (interrupted) and one minor (interrupted) flow from a pair of lanes. The recommended procedure for calculating capacity, queue length, and delay for minor road approaches to an intersection considers these interactions in the specific order (Figure 3) that follows.

A. Minor road approaches are divided into lanes, combining two or more lanes from the same approach that have one or more directions in common. Left-turning vehicles from the major road have to be considered as well.

B. For each traffic stream from the lanes considered, a corresponding total primary flow \(q_i\) is calculated. This flow depends on the number of interactions in which vehicles from the secondary stream must yield. The influence of primary flows in all these interactions is reduced to a single value.

C. For each secondary stream, as above, a critical gap is calculated and will depend on direction of movement for the secondary stream.

D. For each secondary stream, theoretical mean service time is calculated by using the results of B and C and an assumption regarding the headway distribution of the total primary flow.

E. Some service times thus calculated will have to be corrected, because some streams, such as a left-turning stream from the major road, are primary in some interactions but secondary in others. This corresponds to preemptive priority in queuing theory.

F. Mean service time and hence capacity can now be obtained for each lane. In case two or more streams share one lane, their mean service times are weighed according to their respective arrival rates. The capacities of adjacent unreserved lanes are simply added.

G. Mean queue length and mean waiting time are obtained for each lane by using the flow-to-capacity ratio and the queuing theory model. This model also gives the probability of stopping (at a yield-signed intersection) and then the total delay, as mean waiting time plus acceleration losses for stopped vehicles.

Such a calculation procedure of necessity contains a number of assumptions, some verified through extensive measurements or calculations, many not. A few of the more important of these assumptions will be discussed.

Conflict Patterns

One basic aspect of the manual is the unambiguous hierarchy of different vehicle streams in an intersection. It is assumed that signed or other legally stated conventions are observed, so that (for example) a vehicle traveling straight ahead on a major road approach is not affected by minor road vehicles.

Measurements taken in Sweden indicate that this is true when the load factors (flow to capacity ratio) of the minor road approaches do not exceed 0.8-0.9. As load factors increase, some minor road vehicles tend to accept very short gaps, which forces the major road flow to slow down or even halt.

The manual does not apply to such conditions, which would normally, if they were frequent, warrant the installation of traffic signals.

For the T-intersection in Figure 4, it is thus assumed that stream Cb yields to Ab, stream Ca to Ab, Ba, and Be, and stream Be to Ab and Ac. The manual requires calculating an equivalent total primary flow for each secondary stream. In the case of one secondary stream's crossing different primary streams from one or more lanes, the flows of these streams are simply added together for total flow. Theoretically, this result would give a correct total headway distribution only when all the individual distributions are exponential. The mean will always be correct, though, and, as shown by Tanner (11) for the M/D/1 headway-generating model, the effect of "reasonable aberrations" is probably negligible.

In the case of converging conflicts, such as the one between Cb and Ab in Figure 4, it is generally unclear to what extent the stream with legal priority (in this case Ab) should be included as primary flow with re-

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**Figure 3.** Steps in calculating capacity and delay for a minor road approach.

A. Divide approach into lanes. Consider each lane. The lane considered is assumed to contain three streams.

B. Determine total primary flow \(q_i\) (each stream).

C. Determine critical gap \(G_{ij}\) (each stream).

D. Calculate service time \(t_{ij}\) (each stream).

E. Correct service time \(t_{ij}\) (each stream).

F. Calculate mean service time. Calculate capacity per lane.

G. Calculate mean queue length. Calculate delay per lane.

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**Figure 4.** Interactions at a yield- or stop-signed T-intersection.
spect to the secondary stream (Ch). Where the exit has only one lane and no acceleration lane or broad shoulders used as such, all vehicles in the priority stream must be included, just as for a crossing conflict. Note, however, that the critical gap might be different.

When there is more than one lane in the exit where the convergence takes place, the priority flow effective in the conflict will be determined by the lane distribution of the priority flow and by a number of design features such as number of lanes in the approach, lane markings, and islands.

Rather than trying to consider all these factors, which, at least in the case of lane distributions, is virtually impossible in urban areas, the manual recommends that priority flow in a converging conflict be divided by the number of exit lanes before adding it to the total primary flow. This general rule may be modified or disregarded for special designs or known lane distributions.

Other assumptions have to be made regarding the effect of streams turning off the primary road from some secondary streams where no direct conflict occurs but where some influence is possible. In Figure 4, for example, it is likely that some right-turning vehicles from approach A prevent some gaps from being used by vehicles from approach C (by not signaling their intention to turn).

On the basis of the field studies described below, different hypotheses on the effects of such streams were tried. Although their presence did have some effect at smaller intersections, it was decided to exclude streams not in conflict with the secondary flow. Accordingly, critical gaps and lags were defined and measured at the point of conflict. It might be noted here that the German manual (8) differs in this aspect, including such streams as Ac in Figure 4 with a factor of 0.5 (also independent of geometry).

As an application of the principles outlined above, the total priority flows presented below are calculated for the example in Figure 4.

Secondary Flow

| q2 | qBc | q1 = qA0 + qAc/nA |
| q2 | qBc | q1 = qA0 + qAc/nA |
| q2 | qBc | q1 = qA0 + qAc/nA |

where

- q2 = secondary flow in vehicles per second;
- q1 = corresponding primary flow in vehicles per second;
- qAc = vehicle stream between approach x and exit y; and
- nA = number of lanes in exit y (or multiples of 3 if lanes are not marked).

### Critical Gaps

By the critical gap for vehicles in a certain secondary stream we mean a time headway in the total primary flow that is just acceptable to the average driver for passing through the intersection. This parameter affects capacity in a nearly exponential manner and further summarizes the influence of several interesting design variables (from a capacity and delay point of view). As will be described below, extensive measurements were taken to determine this parameter and its dependence on different variables.

The values recommended in the manual are based on these measurements for Swedish conditions, as well as on previous studies elsewhere (1, the bibliography). The most important determinants are stream or direction of travel, speed limit, and type of priority (stop or yield sign), the effects of which are expressed by the base values in Table 1.

Corrections for the following factors are added to or subtracted from the base values (Table 1): width of main road, existence of a median on main road, radius for right-turning vehicles, angle between major and minor roads, major road one-way or not, percentage of heavy vehicles, and size of urban area. Some of these factors affect the critical gap for some streams only.

The German capacity manual (8) considers the same factors as Table 1 plus main-road width for estimating critical gap. Although the influence of the different factors is similar, the recommended values are generally 0.5-1.5 s above those given in Table 1. This difference has not been verified as real or originating from differences in definitions and measurement techniques.

The use of a single parameter to describe gap-acceptance behavior merits some comment.

First, no distinction is made between lag (the first headway when arriving in an empty approach lane) and gap. The term "critical gap" as used in the manual implies some weighted average between critical gap and critical lag, as measured without distinguishing between the two.

Second, the concept of move-up time (the increment in headway a second queuing vehicle requires to follow the first vehicle) has not been mentioned. This parameter is as important as critical gap in estimating capacity or mean service time. From the measurements, we may conclude that move-up time comprises about 60 percent of the critical gap and depends on different factors in much the same way. This relation is implied in the manual without explicitly calculating move-up time.

### Service Time and Capacity

Assuming that gap-acceptance behavior and any given total primary flow are as outlined, the capacity of a secondary stream can be calculated analytically thus:

\[
C = q1 \sum q2 \int_s^{(a-1)q1} f_t(h_1)dh_1
\]

where

- C = capacity in vehicles per second;
- q1 = total primary flow in vehicles per second;
- a = critical gap in seconds;
- \(a_0 = 0.5\) a or move-up time in seconds; and
- \(f_t(h_1)\) = headway distribution of the primary flow, with mean 1/q1s.

Inverting this capacity value yields what may be called the mean service time during queuing conditions.

### Table 1. Recommended base values for critical gaps.

<table>
<thead>
<tr>
<th>Speed (km/h)</th>
<th>Sign</th>
<th>Primary Approach Stream</th>
<th>Secondary Approach Stream</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Left Turn</td>
<td>Right Turn</td>
</tr>
<tr>
<td>50</td>
<td>Yield</td>
<td>5.0</td>
<td>4.8</td>
</tr>
<tr>
<td>70</td>
<td>Stop</td>
<td>5.0</td>
<td>5.5</td>
</tr>
<tr>
<td>90</td>
<td>Stop</td>
<td>5.8</td>
<td>6.0</td>
</tr>
</tbody>
</table>
If several streams share one lane, their mean service times have to be weighted according to arrival rates. Several expressions for the headway distribution $f_i(h_i)$ in Equation 2 have previously been suggested and tried (1, 12, 13). Comparisons between these results and some from the measurements described below indicate that the shape of the distribution depends on several factors, some of which cannot be quantified in an urban area (platoon dispersions from nearby traffic signals or circles, curb frictions, and so forth).

Of the quantifiable factors, the number of streams or lanes was found to be the most important. Fewer lanes for the primary flow, as well as the presence of nearby traffic signals, increase platooning and thereby the coefficient of variation of the headway distribution.

The mean service time flowcharts in the manual are based on three different headway distributions with different coefficients of variation representing three classes of such conditions. The choice of these particular functions can be disputed, and further studies on this point are being undertaken. Regarding the effect of platooning, it should be noted that traffic signal densities are generally lower in Sweden than in the United States, for example.

The use of inverse capacity as mean service time will give an overestimate for nonqueuing conditions, because the constant move-up time is always included. The service time during nonqueuing conditions, which then equals total waiting time, can similarly be calculated for various headway distributions. The difference obtained equals move-up time, at zero primary flow, and then decreases as the primary flow (and hence the mean service time) increases.

In order to represent capacity correctly, this difference is disregarded in the calculation of mean service times but is compensated for on an approximate, average basis in the ensuing calculation of delay. Figure 5 shows an example of the resulting service time flowcharts for the case of primary flow in two lanes and no nearby traffic signals.

Where the secondary stream must cross or turn left onto the main road and a median allows this in two steps, the mean service time may be considerably reduced (and capacity increased). This effect has been calculated theoretically by Tanner (10), whose results are used in the manual.

Now we shall consider the case of one lane used by two or more secondary streams. If several streams jointly use several unreserved lanes, the calculations are performed with averages per lane. Instead of computing a weighted average of the mean service times, the manual directly computes a load factor defined as the ratio of actual flow to theoretical capacity. Remembering that mean service time, as defined here, equals the reciprocal of capacity, we obtain load factor as

$$B = \sum_k q_k \left(\frac{\bar{d}_k}{d_k}\right)_k$$

where

- $B =$ load factor for a lane, or capacity ratio;
- $k =$ index for streams utilizing the lane;
- $q_k =$ flow rate in vehicles per second and per lane of stream $k$; and
- $\bar{d}_k =$ mean service time in seconds for stream $k$.

The products within the sum of Equation 3 represent the degrees of capacity use of individual streams in the lane and are called 'part-capacity ratios' in the manual. Before adding these ratios in Equation 3, however, two types of corrections generally have to be made.

First, corrections for queuing in primary flows must be made. Some streams that are primary to the secondary stream are in turn secondary in some other conflicts; this prevents some otherwise acceptable gaps from being used by the secondary stream. The effect for the latter is the same as if the vehicles from this primary approach were, on arrival, immediately placed first in the queue of secondary stream vehicles. The effect can be estimated with the help of the theory for queues with preemptive priority. The size of the correction depends on the part-capacity ratio of those primary streams where queuing may occur. A similar correction is used in the German capacity manual, although with some differences in the approximations (8, 13).

Second, corrections for short lanes are made. Lanes of a limited length are initially ignored or, rather, considered part of the adjacent lanes. If the adjacent lane is thus used by two or more streams, the capacity ratio is reduced by an amount that depends on the degree of possible use of the short lane. This degree of use is estimated by elementary probability theory.

The capacity of the lane can now be obtained from actual lane flow and the load factor of Equation 3. It should be noted that this capacity is a theoretical concept and not necessarily the maximum lane flow. Increasing lane flow to such a level would entail assuming that other (primary) flows remain constant and may exceed the validity of some assumptions such as constant critical gap. Nevertheless, the load factor of

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**Figure 5.** Calculation of mean service time from primary flow and critical gap in two lanes.
Equation 3 is a valuable concept for analyzing intersection performance and can be further used for calculating queue lengths and delay.

**Queue Lengths and Delay**

Assuming independent service times from a distribution with a certain mean and variance and a Markov arrival process, the mean queue length in a lane can be obtained by Pollaczek's formula,

\[
N = B + \frac{B}{1 - B} \times \frac{1 + C^2/2}{C^2} 
\]

(4)

where

- \(N\) = mean queue length in vehicles, including the vehicle being served;
- \(B\) = load factor (see Equation 3); and
- \(C^2\) = \(\text{Var}(d_i)/\langle d_i \rangle^2\), or the coefficient of variation for the distribution of service times.

For \(C^2 = 1\), the \(M/M/1\) queuing model is obtained.

In a preliminary version of the manual, the variance of the service time distribution was estimated theoretically as a function of critical gaps and total primary flows in much the same way as the mean service time.

In the present version, however, the manual has been simplified in this respect, and the coefficient of variation, \(C^2\) in Equation 4, is assumed to be a function only of the load factor. This gives a unique relation between mean queue length and load factor. This relation was calibrated partly by a simulation model, partly with the help of data from the measurements described later on. The final function is shown in Figure 6.

Compared to the \(M/M/1\) model, Figure 6 sometimes gives slightly lower queue lengths, although the theoretical coefficient of variation is always larger than one. This is because service times are not quite independent; a shorter lag is required by a queuing vehicle (the move-up time is shorter than the critical gap). The differences are quite small, however.

A larger error results from assuming the coefficient of variation within a lane to be dependent only on average service time rather than on the mean service times of all the individual streams in the lane. This will cause some underestimates for lanes used by equal proportions of streams with widely different mean service times and some overestimates for reserved lanes.

The effect of this approximation has been accepted in view of the rather extensive simplification in procedure it makes possible. A slight improvement, substituting a family of curves representing different types of lanes for Figure 6, is being considered for future versions of the manual.

When designing an intersection, the higher fractiles of the queue length distribution, exceeded only by a few percent of arriving vehicles, are more interesting than the mean. These fractiles have been calculated as for the \(M/M/1\) model, and mean queue length is obtained from Figure 6. Finally, queue lengths are converted from number of vehicles to meters.

The delay incurred by a minor road vehicle when passing through the intersection can be divided into different components, as shown by the time-space diagram of Figure 7.

Waiting time includes service time as well as time spent standing in queue and can be thought of as the flow-dependent part of delay. Its mean for a certain lane with one or more secondary streams is easily obtained by using

\[
\bar{d}_w = \frac{N}{q_2} 
\]

(5)

where

- \(\bar{d}_w\) = mean waiting time in seconds;
- \(N\) = mean queue length in vehicles from Figure 6; and
- \(q_2\) = flow in vehicles per second in the lane being considered.

Equation 5 is quite general, although of course the results suffer the same constraints as do mean queue length calculations.

The remainder of total delay, called "running delay," is caused by acceleration and retardation. In the manual, running delay is calculated for each stream in the lane as a function of speed or speed limit, radius for turning vehicles, and probability of stopping.

General parameter values are used for the first two variables, while the probability of stopping for a yield sign intersection is

\[
P_s = 1 - P_r(1 - P_a) 
\]

(6)

where

- \(P_s\) = probability of having to stop;
- \(P_r\) = probability of accepting the first lag (depending on total primary flow and critical gap); and
- \(P_a\) = probability of queue (equal to load factor, according to queuing theory).
The running delay is averaged for all streams in the lane and added to the mean waiting time to give mean delay per lane. From this sum, however, must be subtracted a certain correction to compensate for double counting. Retardation loss and running time along the queue are also included in waiting time during queuing conditions.

The present version of the manual does not include calculation of queue length and delay for lanes on the primary road, with the exception of reserved lanes for left-turning vehicles. But extending the manual in this respect is being considered and has in fact already been done in a computerized version.

**TRAFFIC CIRCLES**

Studies of traffic circles in Sweden and England (14) indicate principal operating differences between weaving sections of different lengths. Where a length exceeds 30 to 45 m and capacity is sufficient, there is a "true" weaving in the sense used by the 1965 HCM. For such long weaving sections the Swedish manual recommends a procedure similar to that of the 1965 HCM.

For shorter weaving sections, which are frequent in Swedish traffic circles, a procedure much like the one described above for intersections is recommended. The studies mentioned show that such traffic circles actually operate as a series of T-intersections that gives priority to the traffic inside the traffic circle.

Capacity, queue length, and delay are therefore calculated for each approach in turn, as described in Figure 3, although with different charts and parameters.

There are two major differences, however. First, for a certain secondary stream the total primary flow in the different points of conflict is totaled, as for the intersection. In a converging conflict, however, the primary flow is reduced by a factor that depends on the geometry of the weaving section and of the exit used. Second, the critical gap depends primarily on the direction of the secondary stream (right turning, straight ahead, or left turning) and on the weaving section's width-to-length ratio.

In addition, only one chart (as in Figure 5) is given for mean service time. Very little data are available on headway distributions in traffic circles, and generally the calculation model for traffic circles is felt to be less reliable than the one for intersections. No doubt this is mainly due to more degrees of freedom in road-user behavior and hence to the decreasing reliability of behavioral models. Still, the method is believed to be an improvement over existing empirical formulas, perhaps most of all with regard to consistency.

**MEASUREMENT OF CRITICAL HEADWAYS**

**Background and Purpose**

Gap-acceptance behavior has been shown to be of vital importance in modeling the performance of an unsignalized intersection, and a number of previous studies have undertaken to define and measure parameters relevant to such behavior (1, 7, 15, 16). The literature survey preceding the manual gives a fairly complete bibliography on the subject and a comparison between and an analysis of results.

The main purposes of the measurements made on unsignalized intersections were to validate these results for Swedish conditions and to further study the influence of some design variables on gap acceptance. In most intersections studied, service time, delay, and similar variables were measured alongside gap acceptance parameters in order to obtain some preliminary tests of the model.

**Definitions**

Several different distribution functions for accepted headways can be defined and have been used in the literature (1, 17). These definitions vary with respect to the types of choice situation studied (gaps and lags or both, one or more vehicles accepting one gap, and so forth), statistical population defined (drivers or gaps), and points of reference for studying headways (intersection limits, conflict points, and so forth).

For these measurements, gaps and lags were referred to points of conflict between the primary and secondary vehicles involved. In the final analysis, gaps and lags were pooled to produce a flow-dependent average. Critical gaps were determined by using probit analysis with correction for bias according to Ashworth (17, 18). By using these definitions, the results are believed to be compatible with the theoretical models for calculating capacity and delay.

**Sample**

The sample should be representative of the most frequent intersection designs. It should also, if possible, admit pair-wise comparisons of the effects of the more important variables, such as type of control (yield or stop), speed limit, cross section of a major road, lane types in secondary approaches, location (urban, rural), and size of flows. In order to facilitate such comparisons, intersections with more than normal disturbances of any kind were excluded.

Eighteen intersections were selected on the basis of a nationwide inventory. Total incoming vehicle flows to these intersections ranged from 6000 to 34,000 average daily traffic (ADT). Total number of lanes in all approaches ranged from three to twelve. Nine of the intersections are situated in urban areas (speed limits 50 or 70 km/h) and three in rural areas (speed limits 60 or 90 km/h). The study considers ai-grade intersections only. No ramp connections with the acceleration lanes common to grade-separated interchanges were included, nor were any weaving sections or traffic circles.

**Method**

An automatic data-collection technique, based on detectors and a data log, was developed and used. In the minor-road approaches, presence-type magnetic loop detectors were installed. These were combined with pneumatic tube detectors in a logic circuit to minimize errors. Two detectors were installed in each lane, so that both the first and second vehicles in a queue could be detected. Passage times in the major road lanes were usually registered by pneumatic tube detectors. Manual switches were used to indicate choice of exit for passing vehicles, lane changes, and different types of disturbances. Figure 8 shows a typical installation in a T-intersection.

The effective analyzed time of observation in each intersection ranges from 6 to 10 h divided into continuous periods of 20 to 30 min and distributed over different peak and off-peak flow conditions.

**Results**

The critical headways obtained for minor-road approaches are listed in Table 2, together with values of the move-up time. Gaps and lags have been pooled in this table.
The confidence limits for the critical headways in the table vary between ±0.1 and ±1.0 s, and for the move-up time between ±0.1 and ±0.3 s.

The critical headways for vehicles turning left from a major road vary between 4.6 and 6.1 s for the four intersections where this parameter was studied (gaps and lags pooled). The highest value pertains to a road section where indexes m and t denote measured and theoretical values, respectively. No systematic errors could be found. The agreement seems to be good, although of course the error in estimating critical gap remains to be added.

The results regarding service time and delay were averaged over continuous 20-min periods and compared with different theoretical models, using a sample of 55 such periods with different flow conditions and from different intersections. For mean service time, and comparing with the theoretical model finally selected and described above (with actual critical gaps), the following regression line (with a correlation coefficient of 0.85) was obtained:

\[ \Delta t_m = -1.11 + 0.96 \Delta t_d \]  

where indexes m and t denote measured and theoretical values, respectively. No systematic errors could be found. The agreement seems to be good, although of course the error in estimating critical gap remains to be added.

**COMMENTS**

The calculation method presented above is believed to be a valuable tool for analyzing intersection performance as well as for designing intersections themselves. Like most such tools, however, the method presumes some skill and good judgment on behalf of the user. The Swedish capacity manual, perhaps more than most such manuals, leaves it to the user to decide the context in which the method is to be used and how to apply the results.

Flexibility has also been an important objective in constructing the method. New research results and practical experiences are expected to continue flowing in, and it should be possible to accommodate these in the manual without changing its entire structure. The stepwise calculation process selected is believed to be useful in this respect.

The accuracy of the method cannot yet be judged, although some preliminary results appear promising. Much work remains to be done before the manual can be considered to be validated.

The responsiveness of the method to changes in various design parameters is perhaps as important as overall accuracy. The attempt to base the calculation...
process on an explanatory model for road-user behavior, rather than on a merely statistical model, ought to favor this responsiveness.

REFERENCES


Part 3. Capacity of Signalized Intersections

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Calculations of signal timing and capacity have been performed in Sweden by using methods based on the 1950 and 1965 Highway Capacity Manuals. These methods, however, give results that differ by as much as 50 percent from observed flows. A comprehensive development was therefore undertaken by the Swedish National Road Administration from 1971 to 1973. After extensive literature reviews, theoretical analyses, and field studies, a method was developed for calculating signal timing, capacity, queue length, proportion of stopped vehicles, and delay. The method is based on calculating saturation flows separately for each lane, which makes the method applicable to all geometric designs and phasing schemes. Adjustment factors for vehicle-actuated control are also included. Study emphasis was on the relations among approach width, lane markings, and capacity; conflicts between left-turning and opposing flows; and conflicts between turning vehicles and pedestrians. The signal-timing method is based on the minimum average delay criterion proposed by Webster.

For 20 years the calculations of signal timing and capacity done in Sweden have been based on a manual developed by Nordqvist in 1958 (5). This manual was, in turn, largely based on the 1950 Highway Capacity Manual (HCM). Some attempts have also been made to promote the use of the 1965 HCM (6). These manuals, however, have been found to have some serious drawbacks. They do not give optimum cycle time, and capacity values often differ from observed flows by as much as 30-50 percent. Furthermore, no guidance is provided for calculating more complex geometric designs and signal phasing schemes.

To overcome these deficiencies, a comprehensive study including the development of calculation methods for roads as well as for different types of unsignalized and signalized intersections was initiated in 1971 by the Swedish National Road Administration (NRA). Chief investigator for the work was Professor Stig Nordqvist at Vattenbyggnadsbyran (VBB); I was responsible for the part covering signalized intersections; and Arne Hansson was responsible for unsignalized intersections. The manual was completed and published in Swedish in 1977 (4).

An overview of the different phases of the work is presented as part 1 of this paper.

SCOPE AND OBJECTIVES

The manual was designed for calculations of capacity (defined as maximum flow at given conditions), queue length, proportion of stopped vehicles, and delay. These calculations serve to describe the consequences of a given set of geometric and traffic parameters. No recommendations for design standards are given, because the manual does not include all aspects, such as safety, capital costs, and external effects on the environment, that would have to be considered.

The manual does include forms and examples for practical application. The purpose of this paper is to present an overview of the proposed method and some of the material dealing with signalized intersections. The method is based on theoretical models supported by a limited number of field surveys. Each model explains the behavior of traffic in a critical conflict or geometric design that affects the discharge rate at the stop line. By identifying the true reasons for each effect, adjustment factors such as city size and location within the city can be neglected or given only minor importance. This should greatly improve the accuracy of the results as well as the possibility of their being reproduced.

METHOD PROCEDURE

Each lane is treated individually in the method. This means that the manual can be used for practically all...