process on an explanatory model for road-user behavior, rather than on a merely statistical model, ought to favor this responsiveness.

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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 1976. 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 Vattenbyggnadsbyrån (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 intersection layouts and phasing scheme designs.

The computational procedure involves the following basic steps:

1. Preparation

a. Determination of phasing scheme, lane division, and lane types.

2. Signal Timing: First Round

b. Calculation of the saturation flow (s) for each lane. Saturation flow, s, is defined as the highest stable flow in vehicles per hour of green (vphg) during existing conditions. First a base value of s is obtained. It takes into account the proportion of turning vehicles and the degree of conflict with other vehicles and pedestrians in the intersection with green in the same phase. This base value is then adjusted for conditions other than normal regarding the width, slope, length, and markings of the lane, the proportion of heavy vehicles, and so forth.

c. Distribution of the flow on different lanes. If the flow in a lane, q_i, is not known, q_i is calculated so that equal values of q_i/s_i are obtained for the adjacent lane or lanes with the same direction in the approach (Figure 9). This assumes that a driver arriving at the approach selects the lane that will minimize his or her delay before he or she crosses the stop line.

d. Identification of the critical conflict point in the intersection. The q_i/s_i ratios are calculated for each lane, and the critical conflict, defined as the point having the highest total of ratios q_i/s_i of adjoining lanes, $(q_i/s_i)_{max}$, is identified (Figure 10). The sum represents the degree of saturation of the intersection if there are no losses in effective green time.

e. Calculation of cycle, split, and green times. These calculations, carried out according to Webster (19), give the signal timing that results in minimum average delay for fixed time operation of the signals.

3. Signal Timing: Second Round

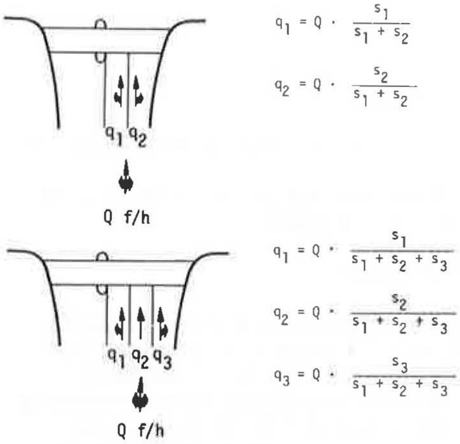
f. Renewal of steps b-e for improved accuracy. The saturation flow for lanes having turning traffic in conflict with opposing vehicle flows or having pedestrian crossings with green in the same phase is a function of the length of the green time and the cycle time. The second round includes procedures for more precise calculations of s for such lanes. The signal timing and other parameters are derived in the first round (b-e) as input. This is only necessary if consequences for individual lanes, such as queue length or delay, are to be calculated.

4. Consequences

g. Calculation of capacity, queue length, proportion of stopped vehicles, and delay. Once s, q, and the signal timing obtained from steps a-f are known, calculating the demanded consequences is a simple arithmetic procedure. Capacity is defined as the highest stable flow in vehicles per hour (vph) during existing conditions.

The manual is divided into a number of separate work moments that are numbered consecutively in the order in which they are to be performed. In Figure 11 a flowchart of the method is illustrated.

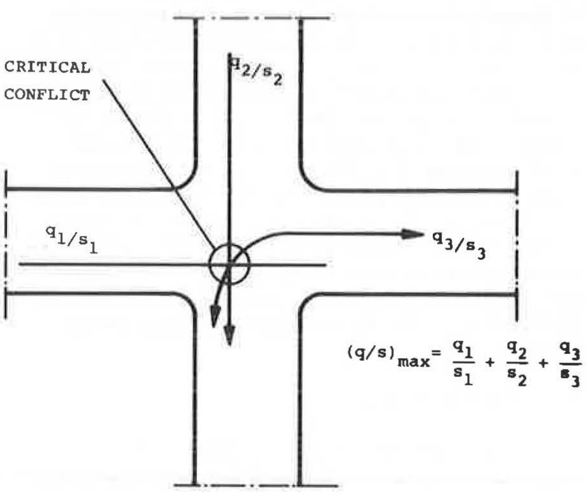
Figure 9. Method used to distribute the flow between lanes in the same direction.



$$q_1 = Q \cdot \frac{s_1}{s_1 + s_2} \qquad q_2 = Q \cdot \frac{s_2}{s_1 + s_2}$$

$$q_1 = Q \cdot \frac{s_1}{s_1 + s_2 + s_3} \qquad q_2 = Q \cdot \frac{s_2}{s_1 + s_2 + s_3} \qquad q_3 = Q \cdot \frac{s_3}{s_1 + s_2 + s_3}$$

Figure 10. Definition of critical conflict.

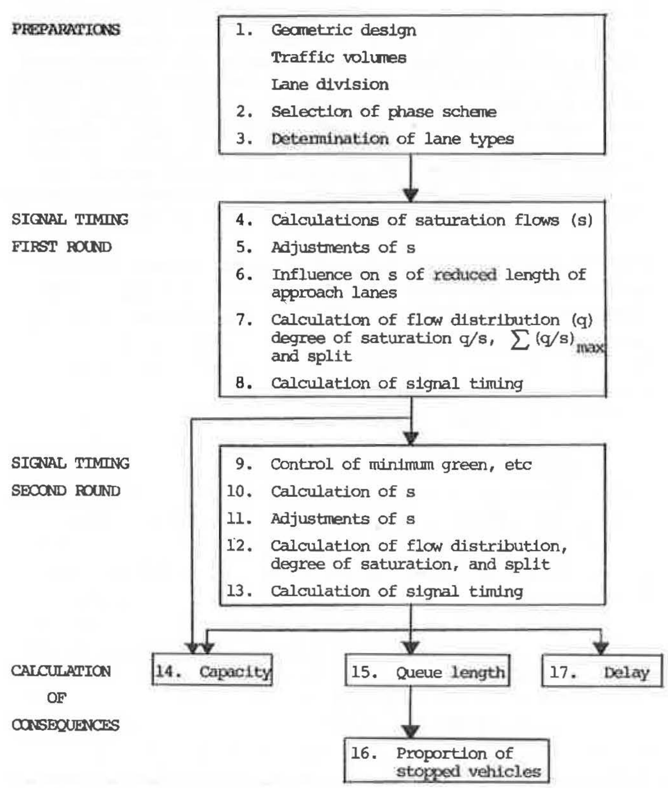


IMPORTANT FEATURES

Lane Configuration and Classification

The models that form the basis of the method assume that each lane carries a single flow of vehicles through

Figure 11. Flow chart for the manual.



the intersection. It is therefore important to make sure that actual traffic behavior corresponds to the lane markings. If this is not the case, the calculations should be performed with the number of lanes actually formed by moving traffic.

If field observation in the actual intersection is not possible, one should assume a minimum lane width of 2.5 m.

Figure 12. Relations among saturation flow, approach width, and number of marked lanes.

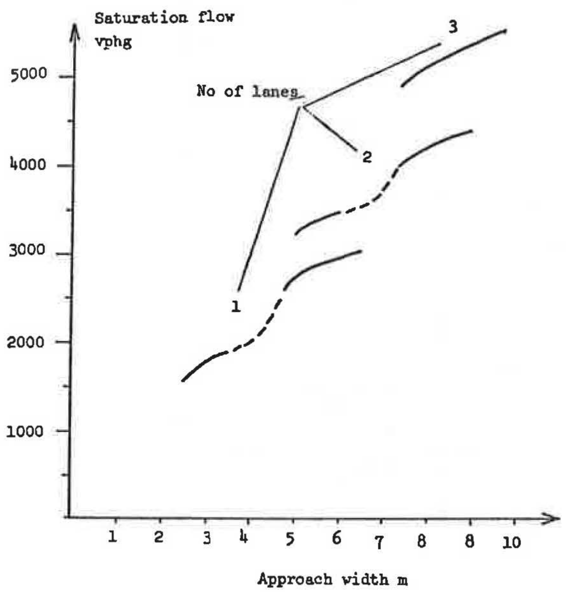
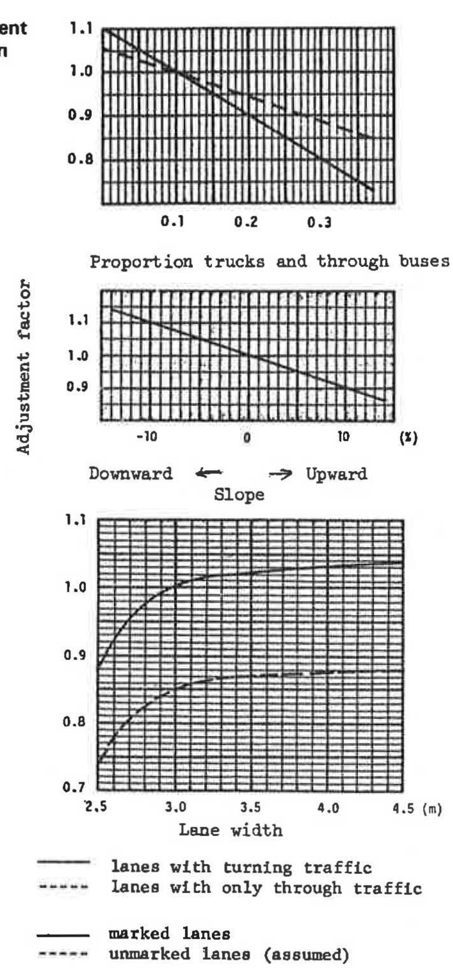


Figure 13. Adjustment factors for saturation flow.



The relations among capacity, approach width, and number of marked lanes in the approach have been evaluated by means of before-and-after field measurements at intersections with different lane arrangements. Figure 12 shows examples of results from these experiments and indicates that capacity is a function of both approach width and number of lanes. The model deals with this by making separate calculations of the capacity for each lane and by adjusting for lane widths other than 3.0 m and for absence of lane markings (Figure 13).

To simplify handling of the method, the lanes are classified as different types, depending on the presence of turning traffic and the degree of conflict experienced by the traffic in the lane. In Figure 14 seven different lane types are defined; lanes with turning traffic that conflicts with both opposing vehicle traffic and pedestrians discharged in the same phase may be assigned to more than one type, for instance to D/F or E/G.

Base Values for Saturation Flow

The base value for the saturation flow (s) for lane type A (only through traffic) is set at 1700 vphg and for type C (only turning traffic without conflict) at 1500 vphg. For type B (some turning traffic without conflict) s varies between 1700 and 1500 vphg, as a function of the percentage turning (0-100 percent).

For lane types with some degree of conflict between turning traffic and opposing flows or pedestrians (types D, E, F, G), determining s is more complicated.

Conflicts Between Left-Turning and Opposing Flows

From practical experience and limited field measurements it was concluded that a primary source of error in older manuals was inadequate handling of conflicts between left-turning vehicles and opposing vehicles on two-way streets. This is particularly important for normal, two-phase controlled intersections where all left turners have to face this conflict. In order to overcome this weakness, a thorough analysis was performed, and a model, suggested by Gordon and Miller in 1966 (20), was applied for stepwise calculations of the different stages of the conflict (Figure 15). These stages are as follows:

1. First part of the green phase, g_k, is when queue discharge from the opposite direction blocks left-turning vehicles in the lane. During g_k, only through or right-turning vehicles, N_k, can be discharged from the lane and then only until the lane becomes blocked by queueing left turners.
2. Remainder of green, $g_g = g - g_k$, is the time during which left-turning vehicles can pass when acceptable gaps occur in the opposing flow. The total discharge during this stage is noted as $N_g = g_g \times s_g$.
3. Inter-green period is that time during which vehicles stalled in the intersection can pass, N_r.

Provided that the signal timing is known from the first round of calculations, g_k can be estimated as the time required to discharge the queue that has been formed in the opposite direction during the previous red phase:

$$g_k = [q_m (c - g)]/s_m \times 1/(1 - q_m/s_m) = [q_m (c - g)]/(s_m - q_m) \quad (8)$$

Figure 14. Classification of lane types.

TYPE	DESCRIPTION	EXAMPLE
A	ONLY THROUGH TRAFFIC	
B	SOME TURNING TRAFFIC WITHOUT CONFLICT	
C	ONLY TURNING TRAFFIC	
D	SOME TURNING TRAFFIC IN CONFLICT WITH PEDESTRIANS	
E	ONLY TURNING TRAFFIC IN CONFLICT WITH PEDESTRIANS	
F	SOME TURNING TRAFFIC IN CONFLICT WITH OPPOSING TRAFFIC	
G	ONLY TURNING TRAFFIC IN CONFLICT WITH OPPOSING TRAFFIC	
D/F	SOME TURNING TRAFFIC IN CONFLICT WITH OPPOSING TRAFFIC AND PEDESTRIANS	
E/G	ONLY TURNING TRAFFIC IN CONFLICT WITH OPPOSING TRAFFIC AND PEDESTRIANS	

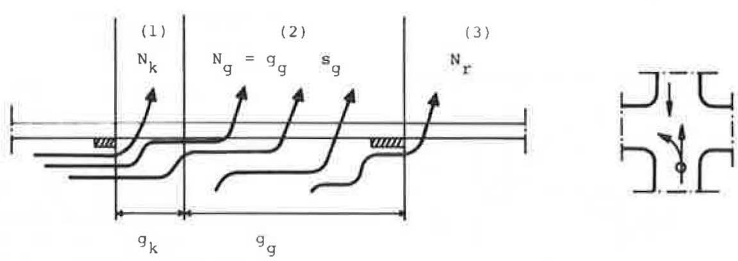


Figure 15. Calculation of saturation flow for lanes with left turning vehicles.

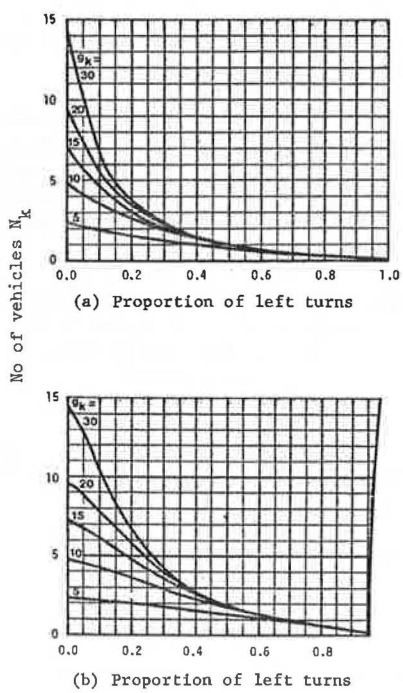


Figure 16. Number of vehicles, N_k, that can be discharged during g_k.

where

g = length of green phase in the opposite direction;
c = cycle time;
q_m = flow vph in opposite direction; and
s_m = saturation flow vphg in opposite direction.

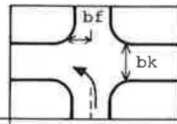
The term $1/(1 - q_m/s_m)$ accounts for the cumulative effect of vehicles arriving at the approach before the queue has been completely discharged. Numerical values for all variables are obtained from the first round of calculations for the opposing lane with the highest q/s ratio.

With g_k known, the number of vehicles, N_k, that can be discharged is a function of the proportion of left-turning vehicles, p_1, in the lane and the number of left-turners that can queue in the intersection without blocking other vehicles in the same lane. N_k can be solved by using general probability theory with the following results in Figure 16, where (a) is the case where no left-turning vehicle can queue without blocking the lane:

$$N_k = \sum_{i=1}^{N-1} [i \times p_1 \times (1 - p_1)^i] + N \times (1 - p_1)^N \qquad (9)$$

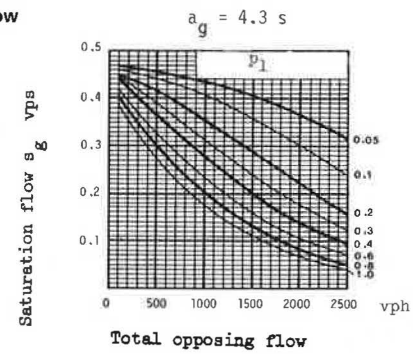
and (b) is the case where one left-turning vehicle can queue without blocking the lane:

Figure 17. Accepted critical gap, a_g, in the conflict between left-turning and opposing vehicles.



		CROSS-STREET WIDTH bk (m)	
		≤10	> 10
OPPOSITE APPROACH WIDTH bf (m)	≤ 8	4.8	5.3
	>8	5.3	5.8

Figure 18. Saturation flow s_g during g_g.



$$N_k = \sum_{i=1}^{N-2} [i(i + 1) \times p_1^2 \times (1 - p_1)^i] + (N - 1) \times N[p_1 (1 - p_1)^{N-1}] + N \times (1 - p_1)^N \qquad (10)$$

where N is the maximum number of vehicles that can be discharged during green ($N = g \times s$). The discharge, N_g, during g_g is a function of g_g, the proportion of left-turning traffic, p_1, and the possibilities for left-turners to cross in gaps in the opposing flow. This conflict is similar to the one that occurs in an unsignalized intersection. The capacity for left turns is in this case a function of the accepted critical gap α_g and the distribution of gaps in the primary stream.

Measurements of α_g were performed at 10 signalized intersections with a method described by Hansson in part 2; α_g was found to vary as a function of the width of the opposite approach, bf, and the cross street, bk, as presented in Figure 17. For busy intersections in large cities, the values obtained for α_g were 0.5 s lower than the values in Figure 17.

The saturation flow s_g during g_g can be expressed as

$$s_g = [q_m \times (e^{-\alpha_g \times q_m})]/[1 - (e^{-\alpha_f \times q_m})] \qquad (11)$$

where α_f is move-up time, which is 1/s.

In the field studies α_f was found to be closely correlated with α_g ($\alpha_f = 0.54\ \alpha_g$). In the manual this is presented in a separate graph for each value of α_g (Figure 18), where critical gap α_g = 4.3 s.

The value for N is then obtained as

$$N_g = g_g \times s_g \qquad (12)$$

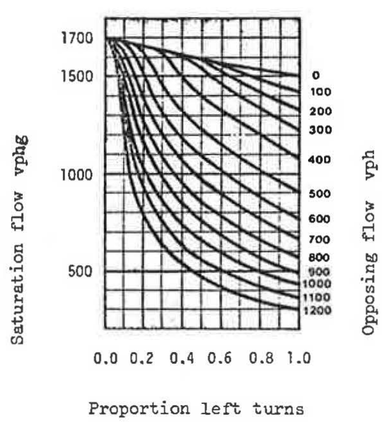
The number of vehicles discharged during inter-green, N_r, is obtained from measurements or observations of the space for queueing vehicles inside the intersection. In Sweden an average space of 8 m per vehicle is assumed. Care should also be taken to ensure that the inter-green is sufficiently long to allow these vehicles to clear the intersection before the next phase begins. Otherwise the intersection can become seriously blocked.

The resulting saturation flow for the lane in vphg is obtained as

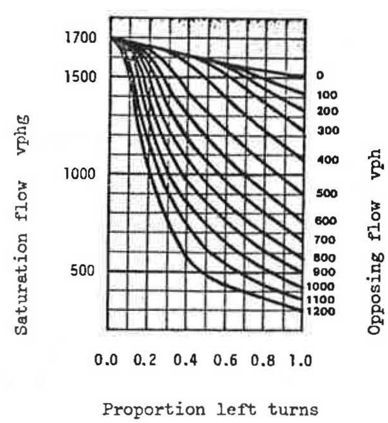
$$s = (3600/g)\ (N_k + N_g + N_r) \qquad (13)$$

The complex and thorough procedure described above can only be carried out when signal timing and saturation flows have been estimated in the first round of calculations. The manual gives two simplified graphs for the first calculation round (Figure 19) for this purpose that are based on series of calculations with the second-round procedure for normal types of intersections. In these graphs s is determined by using the ratio of left-turning vehicles and the total opposing flow as inputs.

Figure 19. Saturation flow for lanes with left-turning vehicles in conflict with opposing flow.

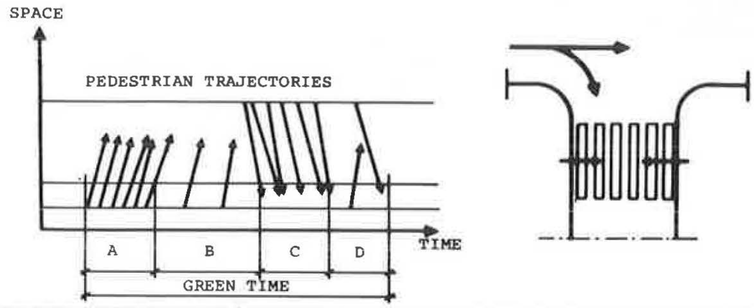


No left-turning vehicle can queue without blocking the lane



One left-turning vehicle can queue without blocking the lane

Figure 20. Calculation of saturation flow for lanes with right-turning movements.



Conflicts Between Turning Vehicles and Pedestrian Movements

In two-phase signal control for intersections of two-way streets, right-turning as well as left-turning vehicles are usually affected by pedestrian movements in the crosswalk of the leg into which the turn is being made. This is particularly true for Sweden, where right turns on red are not permitted. An attempt was therefore made to analyze these conflicts and to develop a theoretical model as a basis for a computational procedure in the manual (3).

Collecting data with time-lapse photographs combined with inductive loops for vehicle detection was performed in four intersections with the described conflicts. A theoretical model was then developed to describe the behavior of a vehicle and a pedestrian as a function of the relative time advantage α_{by} for the party first arriving at the collision point. The reason for this was that it was felt that the right-of-way for the pedestrians during green was not fully respected by the turning vehicles. The model was therefore designed to enable tests of alternative hypotheses of this behavior.

The results from the field studies were evaluated manually, and α_{by} was derived by probit analysis. The vehicles usually required a positive time advantage of 3.2 s if they were not to give way to a pedestrian (standard deviation 4.3 s).

A capacity model based on division of the green phase into four parts was developed. Figure 20 shows A and C, where no vehicles can pass because pedestrian platoons have formed during red, and B and D, where a random arrival of pedestrians into the conflict zone is assumed. Turning vehicles can pass if their time advantage is sufficiently great.

The capacity of the turning flow can be expressed as

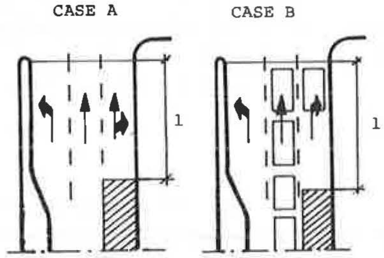
$$K_f = (T_D - T_C + T_B - T_A) \times \{q_p [(e^{-4.2q_p})/(1 - e^{-3q_p})]\} + N \qquad (14)$$

where

K_f = largest number of turning vehicles that can pass per green phase per lane;
$T_A - T_D$ = starting point of intervals A-D;
q_p = pedestrian flow, in number of pedestrians per second, two-way; and
N = number of turning vehicles that can pass during inter-green.

$T_A - T_D$ are functions of the length of the crosswalk and of the size and diffusion of the pedestrian platoons formed during the red phase. The equation, however, includes too many variables to allow graphic presentation. It has therefore been evaluated for a number of typical cases and the results have been put together in Table 3.

Figure 21. Restricted length of approach lanes.



Effect of Restricted Length of Approach Lanes

In built-up areas, the length of the curb lane is often limited by parking, bus stops, or narrowing street width. If the curb lane in an approach is too short, capacity can fall. Two different cases can be distinguished (Figure 21).

Case A. Curb Lane Serving Right-Turning and Through Vehicles

A reduction in saturation flow, s, of the curb lane occurs if the available lane length, l, is smaller than the space occupied by the maximum number of vehicles (8 m per vehicle) that can be discharged during the green time, g.

$$1/8 < (s \times g)/3600 \qquad (15)$$

The reduced saturation flow s' becomes

$$s' = (3600 \times l)/(8 \times g) = (450 \times l)/g \qquad (16)$$

Case B. Curb Lane Serving Only Right-Turning Vehicles

In this case the queue formed during red in the nearby lane might block the curb lane from being used to its full length. The likelihood of such blocking occurring is a function of l, g, and the ratio of right-turning vehicles in combined curb lane and adjacent lane. The number of right turns, N, that can be made during a green phase has been solved with probability theory (Figure 22).

The adjusted saturation flow s' is obtained as

$$s' = N \times (3600/g) \qquad (17)$$

If s' < s for the curb lane, s' should be used in further calculations. In this case the curb lane and the nearby lane are considered one lane with a saturation flow equal to the sum of the individual saturation flows for each lane.

The method can also be applied with minor modifications to lanes for left turners.

Signal Timing

The signal timing method the manual uses was developed by Webster (19) and is based on minimum average delay. An important factor in this method is the estimation of the loss of effective green time per cycle, F. For this purpose the manual includes methods for determining safe clearance times between phases set according to rules established by the Swedish National Traffic Safety Administration.

Traffic signals in Sweden display green plus amber at the end of green. Field measurements have shown that this interval is fully utilized by drivers and that it has therefore been included in the effective green in the manual. An inter-green interval thus consists only

Figure 22. Number of vehicles per cycle that can be discharged from a curb lane for right turns with reduced length.

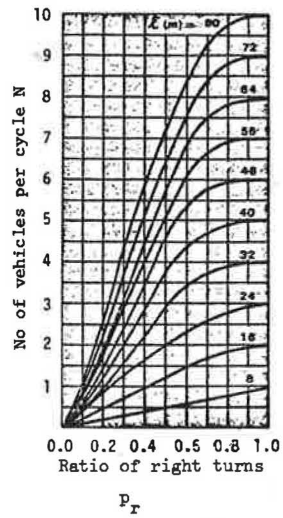


Table 3. Saturation flow of lane with turning vehicles in conflict with pedestrian movements.

Turns	Two-Way Pedestrian Flow per Hour	Green Time (s) 10, Length of Crosswalk (m) 7	10, 14	10, 21	15, 7	15, 14	15, 21	20, 7	20, ≥14	≥30 All Widths
0	250 - ≥1500	1700	1700	1700	1700	1700	1700	1700	1700	1700
0.05	250	1580	1650	1640	1650	1670	1650	1660	1670	1680
	500	1500	1620	1600	1610	1660	1640	1640	1670	1670
	750	1390	1570	1580	1560	1570	1580	1600	1600	1630
	1000	1330	1490	1560	1510	1490	1510	1550	1540	1600
	≥1500	1290	1490	1560	1380	1120	1490	1450	1500	1540
0.10	250	1530	1620	1600	1610	1650	1640	1630	1660	1670
	500	1400	1580	1530	1540	1620	1600	1590	1640	1650
	750	1260	1480	1500	1460	1500	1490	1520	1550	1580
	1000	1150	1380	1460	1380	1370	1390	1450	1450	1520
	≥1500	1090	1380	1460	1170	1280	1350	1280	1350	1420
0.15	250	1430	1580	1560	1560	1630	1600	1590	1630	1650
	500	1270	1510	1460	1470	1580	1540	1540	1610	1620
	750	1070	1390	1400	1360	1410	1400	1440	1470	1530
	1000	960	1230	1360	1250	1230	1270	1350	1340	1450
	≥1500	890	1230	1360	1010	1420	1220	1130	1200	1300
0.20	250	1390	1560	1520	1520	1620	1580	1560	1610	1640
	500	1200	1480	1400	1410	1550	1510	1490	1590	1600
	750	1010	1320	1340	1290	1350	1340	1380	1430	1490
	1000	870	1160	1280	1160	1140	1180	1270	1260	1380
	≥1500	800	1160	1280	890	1030	1120	1020	1110	1210
0.30	250	1300	1500	1450	1440	1580	1530	1510	1580	1610
	500	1040	1390	1280	1300	1490	1420	1410	1540	1560
	750	850	1200	1210	1150	1240	1220	1270	1330	1410
	1000	700	1000	1140	1000	980	1020	1130	1120	1260
	≥1500	630	1000	1140	720	860	960	850	950	1060
0.40	250	1230	1460	1390	1380	1550	1480	1450	1550	1580
	500	930	1320	1190	1200	1430	1350	1330	1490	1510
	750	720	1100	1110	1030	1150	1130	1170	1250	1330
	1000	590	880	1030	880	860	900	1010	1000	1160
	≥1500	520	880	1030	600	740	840	730	830	940
0.50	250	1180	1410	1340	1330	1510	1450	1420	1520	1550
	500	830	1240	1100	1120	1380	1290	1260	1440	1470
	750	640	1000	1020	940	1070	1050	1090	1180	1270
	1000	500	790	930	780	770	810	920	910	1070
	≥1500	450	790	930	520	650	740	640	730	850
0.60	250	1130	1370	1300	1280	1480	1410	1380	1490	1520
	500	750	1180	1030	1050	1320	1230	1200	1400	1440
	750	570	940	950	870	990	980	1020	1120	1210
	1000	440	710	860	710	690	730	840	830	1000
	≥1500	390	710	860	450	570	670	570	660	770
0.80	250	1040	1300	1110	1200	1410	1350	1300	1450	1480
	500	640	1070	910	930	1230	1120	1090	1320	1360
	750	470	830	820	750	910	870	900	1020	1120
	1000	360	600	730	590	580	620	720	710	880
	≥1500	310	600	730	360	470	550	460	550	650
1.0	250	950	1240	1170	1120	1360	1290	1240	1410	1420
	500	550	980	820	840	1150	1030	1000	1250	1300
	750	390	740	730	660	830	780	800	930	1040
	1000	300	510	640	510	490	530	630	620	780
	≥1500	260	510	640	300	400	470	390	470	570

of all red and all red plus amber between green in two conflicting phases.

The total loss of effective green time per cycle, F, is obtained as the sum of the inter-green intervals between the conflicting phases.

The cycle time, c, is obtained as

$$c = (1.5F + 5)/[1 + \Sigma(q_i/s_i)_{max}] \qquad (18)$$

where

F = sum of inter-green intervals per cycle, and

$\Sigma(q_i/s_i)_{max}$ = sum of q/s for the critical lanes, i.e., the lanes that make up the critical conflict point in the intersection.

The distribution of green times that best minimizes delay is derived from

$$g_i = (c - F) \times (q_i/s_i)_{max}/\Sigma(q_i/s_i)_{max} \qquad (19)$$

The green times are checked against minimum green time requirements, and the timing is adjusted if necessary. This is done by adding the extra green time required to F and calculating a new cycle time that is distributed among the phases without disrupting the optimum distribution according to Equation 18.

The manual also includes recommendations for timing of traffic-actuated controls (minimum and maximum green, extension intervals) based on previous research (21).

Capacity, Queue Length, Proportion of Stopped Vehicles, and Delay

All the measures dealt with under this heading are first calculated on a per-lane basis. Totals and averages for an approach or for the whole intersection can also be derived according to instructions in the manual, but they are not always meaningful.

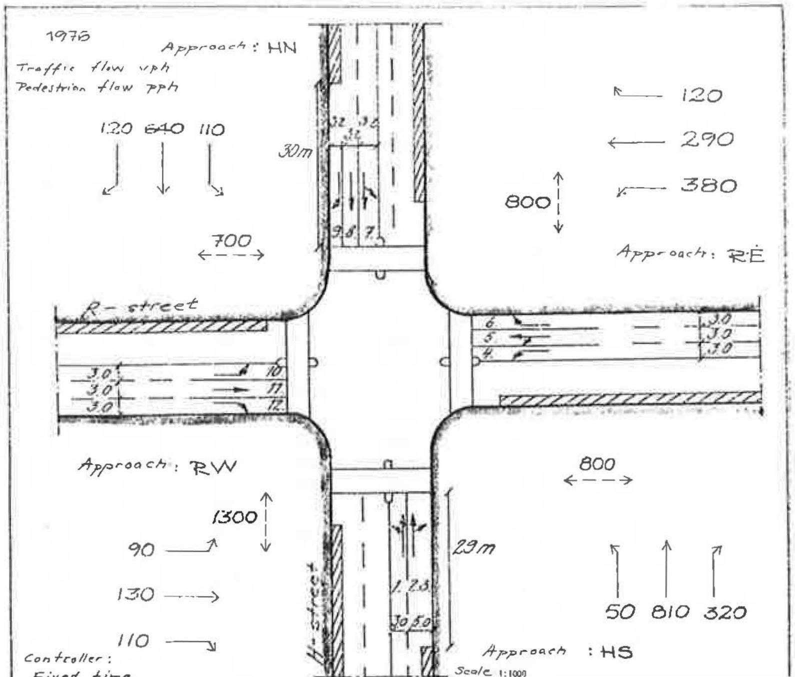
When the signal timing is established, lane capacities, K_i, in vehicles per hour are derived as

$$K_i = g_i/c \times s_i \qquad (20)$$

The degree of saturation, q_i/K_i, becomes highest and equal for all the critical lanes. If it exceeds 1.0, queues build up infinitely, and even at values above 0.8 substantial queueing occurs. The average number of queueing vehicles in lane i at the beginning of green, N_i, is calculated according to Miller (22):

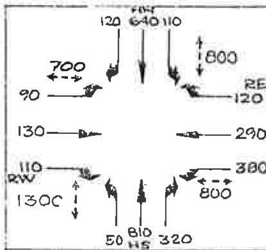
$$N_i = N_{1i} + N_{2i} \qquad (21)$$

Figure 23. Form A plan sketch, traffic volumes, and assumptions.

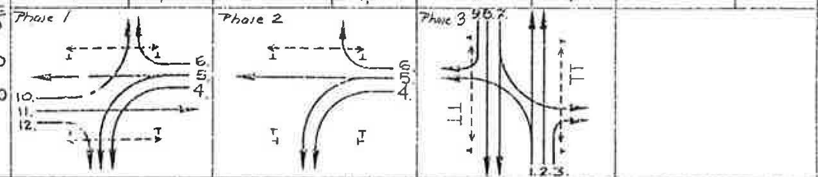


Approach	Speed limit (km/h)	Trucks and through buses %	Bicycles	Slope (%) + up - down	Min. green for pedest. (s)	Min. inter-green for pedestrians (s)	
col. 1	2	3	4	5	6	7	
HS	50	10		—	9	7.0	
RE	50	10		—	9	7.0	
HN	50	10		-3	9	7.0	
RW	50	5		—	9	7.0	

Figure 24. Form C, calculation of signal timing, first round.

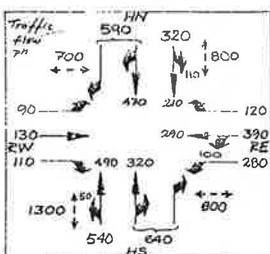


g_{min}	intergreen	g_{min}	intergreen	g_{min}	intergreen	g_{min}	intergreen	
14	4,5	—	4,5	14	4,5			F = 9, C = 53

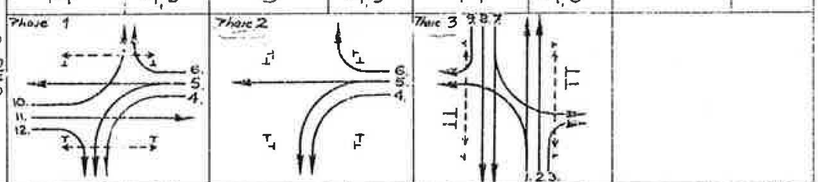


Approach	Lane no i	Green in phase no	Lane type	Turns p	Saturation flow s_i Base value vphg	Adjustments: Trucks and through buses	Slope	Lane width	Other	Adjusted value vphg	Flow q_i vph	q_i/s_i	$f_i = \frac{(q_i/s_i)_{max}}{\Sigma (q_i/s_i)_{max}}$	Green time g_i (s) Base value s	Adjusted value s	g_i max s	Capacity $\frac{s_i \cdot g_i}{c}$ vph	Degree of saturation $x_i = q_i/k_i$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
HS	1	3	D/F	0,12	1350/163?	—	—	—	—	1350	540				27			
	2	3	A	0,0	1700	—	—	0,74	—	1260	640	0,40	0,618	27	27			
	3	3	E	1,0	900	—	—	0,74	—	320*					27			
RE	4	1+2	E/G,C	1,0	900/1550, 1390,	—	—	—	—	1130	280	0,247	0,382	17	17			
	5	1+2	D/F,B	0,13	1490/1670,168?	—	—	—	—	1590	390				17			
	6	1+2	E,C	1,0	1000, 1500	—	—	—	—	1250	120	0,096			17			
HN	7	3	D/F	0,29	1300/870	—	—	1,02	—	890	280				27			
	8	3	A	0,0	1700	—	—	1,02	—	1730	590	0,31		27	27			
	9	3	E	1,0	530	—	—	1,02	—	160*					27			
RW	10	1	E/G	1,0	1000/12??	1,05	1,03	—	—	1080	90	0,083			14			
	11	1	A	0,0	1700	1,03	1,03	—	—	1800	130	0,072			14			
	12	1	E	1,0	900	1,05	1,03	—	—	970	110	0,113	0,175	8	14			
Sum												0,648						

Figure 25. Form C, calculation of signal timing and capacity, second round.



g_{min}	intergreen	g_{min}	intergreen	g_{min}	intergreen	g_{min}	intergreen	
14	4,5	5	4,5	14	4,5			F = 9, C = 61



Approach	Lane no i	Green in phase no	Lane type	Turns %	Saturation flow s_i vpgh Base value vphg	Adjustments: Trucks and through buses	Slope	Lane width	Other	Adjusted value vphg	Flow q_i vph	q_i/s_i	$f_i = \frac{(q_i/s_i)_{max}}{\Sigma (q_i/s_i)_{max}}$	Green time g_i (s) Base value s	Adjusted value s	g_i max s	Capacity $k_i = \frac{s_i \cdot g_i}{c}$ vph	Degree of saturation $x_i = q_i/k_i$
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
HS	1	3	D/F	0,09	1420/1680	—	—	—	—	1420	550				29		670	0,82
"	2	3	A	0,0	1700	—	—	0,74	—	1260	630	0,387	0,558	29	29		770	0,82
"	3	3	E	1,0	1040	—	—	0,74	—	350*					29			
RE	4	1+2	E/G,C	1,0	780/1160,1350	—	—	—	—	880	270	0,307	0,442	23	23		330	0,82
"	5	1+2	D/F,B	0,26	1220/1650,1650	—	—	—	—	1300	400				23		490	0,82
"	6	1+2	E,C	1,0	780, 1500	—	—	—	—	910	120	0,132			23		340	0,35
HN	7	3	D/F	0,39	1330/890	—	—	1,02	—	910	290				29		430	0,67
"	8	3	A	0,0	1700	—	—	1,02	—	1730	590	0,300			29		880	0,66
"	9	3	E	1,0	570	—	—	1,02	—	120*					29			
RW	10	1	E/G	1,0	780/710	1,05	1,03	—	—	770	90	0,117			14		180	0,50
"	11	1	A	0,0	1700	1,03	1,03	—	—	1800	130	0,072			14		410	0,32
"	12	1	E	1,0	780	1,05	1,03	—	—	840	110	0,131	0,189	10	14		190	0,58
Sum												0,694						

Figure 26. Form D, calculation of queue length, proportion of stopped vehicles, and delay.

Approach	Lane no. i	Flow q_i vps	q_i/s_i	$\lambda_i = g_i/c$	Degree of saturation x_i	No of queuing vehicles N_{1i}	N_{2i}	N_i	N_{max} $P_0=20$	Queue length (m) $P_0=20$	Proportion of stopped vehicles p_{si}	$p_{si} \times q_i$ (1/s)	λ_i	$\lambda_i \times c$	a_i	x_i/λ_i	Average delay, d_i Fixed time (s)	Vehicle actuated (s)	Total delay $d_i \times q_i$ vs
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
HS	1	0.153	0,387	0,475	0,82	1,8	4,9	6,7	9,3	75	0,63	0,096	0,22	13,4	1,87	12,2	23,0		3,5
"	2+3	0,175		0,475	0,82	1,8	5,6	7,4	10,0	80	0,64	0,112	0,22	13,4	1,87	10,7	21,7		3,8
RE	4	0,075	0,307	0,377	0,82	1,8	2,9	4,7	7,0	55	0,69	0,052	0,27	16,5	1,87	24,9	37,3		2,8
"	5	0,111		0,377	0,82	1,8	4,2	6,0	8,4	67	0,71	0,079	0,27	16,5	1,87	16,8	30,0		3,3
"	6	0,033	0,132	0,377	0,35	–	1,3	1,3	2,5	21	0,64	0,021	0,22	13,4	0,094	2,8	14,6		0,5
HN	7	0,081	0,319	0,475	0,67	0,5	2,6	3,3	5,4	41	0,57	0,046	0,22	13,4	0,480	8,4	19,6		1,6
"	8+9	0,161	0,314	0,475	0,66	0,5	5,2	5,7	7,8	61	0,60	0,097	0,22	13,4	0,641	4,0	15,7		2,5
RW	10	0,025	0,117	0,230	0,50	0,1	1,2	1,3	2,6	20	0,78	0,020	0,34	20,7	0,250	10	27,8		0,7
"	11	0,036	0,072	0,230	0,32	–	1,7	1,7	3,1	22	0,79	0,028	0,33	20,1	0,075	2,1	20,0		0,7
"	12	0,031	0,131	0,230	0,59	0,2	1,5	1,7	3,1	22	0,79	0,024	0,34	20,7	0,400	12,9	30,2		0,9
Sum		0,881										0,575							19,8
Average												0,66							22

where

$$N_{1i} = (2x_i - 1)/[2(1 - x_i)] \; ; \; x_i = (q_i \times c)/(s_i \times g_i) > ½$$

$$N_{1i} = 0 \; ; \; x_i \leq ½$$

$$N_{2i} = q_i \times (c - g_i) \quad (22)$$

N_1 represents the number of vehicles remaing from the previous green phase and N_2 the number of vehicles arriving during red. N is then adjusted upward by means of Poisson's curves for required probability of overload.

Queue length, L_i, is derived by multiplying the number of queueing vehicles by the average space per vehicle. The manual also includes adjustments of L_i as a function of the proportion of trucks and buses in the lane.

The proportion of stopped vehicles, p_{si}, in lane i is derived from

$$P_{si} = (r_i + N_i \times t_i)/c \quad (23)$$

where $N_i \times t_i$ represents the time from beginning of green until the N_ith vehicle in the queue has started to move. Measurements indicate that t_i normally equals 1.0.

Average delay, d_i, is calculated according to Webster (6):

$$d_i = 0.9 \, [c(1 - \lambda_i)/2(1 - \lambda_i \times x_i)] + \{x_i^2/[2q_i \times (1 - x_i)]\} \quad (24)$$

where λ_i equals g_i/c. For x_i, see Equation 22 above. Adjustment factors for queue length and delay in traffic-actuated controls based on the corresponding values at fixed time controls are also given in the manual.

APPLICATION

In the manual (4) the chapter on signalized intersections occupies about a fourth of the whole book. The important relations are all presented in graphs or tables in order to simplify use of the manual. Computer programs are also being developed for the same purpose.

The layout of the manual is in the form of stepwise calculations (Figure 10), each step leading to a set of figures to be filled into the corresponding column on the form for the calculation. Four different forms are provided for signalized intersections:

1. Plan sketch: traffic volumes and assumptions,
2. Determination of inter-green,
3. Calculation of signal timing and capacity, and
4. Calculation of queue length, proportion of stopped vehicles, and delay.

Figures 23-26 illustrate the use of the forms for an intersection in the downtown area of Stockholm. Figures 23 and 24 represent the first and the second rounds of the signal timing that are filled into form C.

The time required to carry out a complete application, such as the one in the example, may vary between 3 and 6 h, depending on the skill and practice of the engineer. The most time-consuming part is the second round of signal timing, which, however, is only necessary if queue lengths or other measures for individual lanes are requested. If only signal timing and degree of saturation are asked for, the calculations should not take more than an hour.

The manual has been introduced in a number of workshops for traffic engineers in Sweden and has been greatly appreciated for its exactness and flexibility.
A number of important questions still have to be solved by future research, however, for example, the effect of bicycles in the vehicle flow, the influence of different

types of traffic-actuated controls, and the influence of rain, snow, and darkness.

A thorough validation of the methods is also desirable and may take place after the manual has been used for some time.

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19. F. Webster. Traffic Signal Settings. Her Majesty's Stationery Office, London, Road Research Technical Paper 39, 1958.
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21. K.-L. Bang. Optimal Control of Isolated Traffic Signals. TRB, Transportation Research Record 597, 1976, pp. 33-35.
22. A. J. Miller. Settings for Fixed-Cycle Traffic Signals. Operational Research Quarterly, Vol. 14, No. 4, 1963.

Discussion

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PART 1 BY PETERSON

Mr. Peterson initially reviews the historical background leading to development of the 1977 publication on calculation of capacity, queue length, and delay on road traffic facilities. He notes that "it has been clearly shown from measurements in several countries that actual capacity is higher than that arrived at by application of the methods" in referring to previous methods.

No design standards, such as levels of service, are provided; rather, the final decision as to the design is suggested to depend on factors of road safety, cost, and environment. Thus, Mr. Peterson states that "the choice (or design) should be based on that degree of accuracy which is right both socially and economically."

Major additions to the literature are calculations for unsignalized intersections, pedestrians (midblock), bicycle traffic (bike paths), and short weaving sections of 40-60 m.

The manual itself is a fine publication, well laid out with color for emphasis and the use of examples and forms to assist the analyst. I am generally favorably impressed by the Swedish manual and Mr. Peterson's paper. It is, indeed, a forward step in the capacity field.

I do take some minor exception to Mr. Peterson's statement that "it has been clearly shown" that actual capacity is higher than that arrived at by different methods. In my discussion of the other two papers, I shall indicate why I differ. Also, I believe that providing design guides or levels such as levels of service is an advantage because it sets standards. "The choice of that degree of accuracy which is right both socially and economically" can be a controversial choice if it is not based on engineering fact.

PART 2 BY HANSSON

Mr. Hansson's paper presents a method of calculating capacity, queue length, and delay at intersections controlled by yield or stop signs. The method is based on a queuing model (M/G/1) whose most important input parameter is the critical headway determined from field observations at 18 intersections in Sweden (Table 2 in the paper).

Data gathered for the 10 yield and 8 stop approaches provided critical headway (right, straight, and left) and move-up times. The paper notes that "values... are based on these measurements for Swedish conditions, as well as on previous studies elsewhere." Table 1 outlines the values.

I have reviewed the information in Tables 1 and 2 and cannot find any direct relations. Table 1 values are the basis for calculating intersection operations, so it would be interesting to know how they are derived.

A breakdown of Table 2 reveals 12 locations (8 yield and 4 stop) where speeds were 50 km/h, 3 (1 yield and 2 stop) locations at 70 km/h, and 3 locations (1 yield and 2 stop) at 90 km/h. I wonder what level of statistical confidence can be placed on values derived from one or two locations.

I suspect the additional data came from previous studies. In 1974, the Organization for Economic Co-operation and Development (OECD) published Capacity of At-Grade Junctions (7). Table 4 of that publication (here Table 4) summarizes critical time gap values.

Table 4. Summary of values for the critical time gap found by various researchers or used in the national design manuals.

	Through		Right Turning		Left Turning	
Source	Passenger Cars	Trucks	Passengers Cars	Trucks	Passengers Cars	Trucks
Grabe, Wörner	4.1-5.2	6.0-6.5				
Krell	6.4 (second vehicle: 1.82 t; third 2.53 t; etc.)					
Ashworth (median value)	6.5					
Hofwegen (value at a major road flow of 1000 v/h)	5.4		5.0		6.0	
Kell (median value)	5.8		5.4		6.3	
Owens (median value)	6.3					
Thomasson (median value)	8.6					
Knoflacher	6					
French design standards[b]	6	8				
German guidelines for traffic signals						
V = 90 km/h	7-8		6-7		8-9	
V = 50 km/h	6-7		5-6		6.5-7	
English design standards	4-8		8-12			
Dutch design standards	6 (+2 s reaction time)				(7)	6 (9)
Swedish manual						
V = 90 km/h	7.0		7.2		7.5	
V = 70 km/h	6-6.5		6-6.5		6.2-6.8	
V = 50 km/h	5.2-5.8		4.8-5.5		5.3-6.0	

[a] In some cases the time needed to complete the maneuver for one carriageway with two lanes is given.
[b] Sight distances are calculated taking into account the time required for merging.

I have added the Swedish manual values to the bottom of this table, and find they generally compare well. (Note, however, that the values in Table 4 are simplistic. The original texts generally classify time gaps by specific configurations.)

There are no sample calculations in the paper. However, there are two examples in the manual (attached). One is a T-junction, where both roads have speeds of 50 km/h (Figure 27). Traffic flow from the two-lane, one-way secondary road is 200 vph turning left, and 200 vph turning right. The two-lane, two-way primary road carries 500 vph each way. Capacity is calculated as 570 vph for the right turn and 300 vph for the left turn.

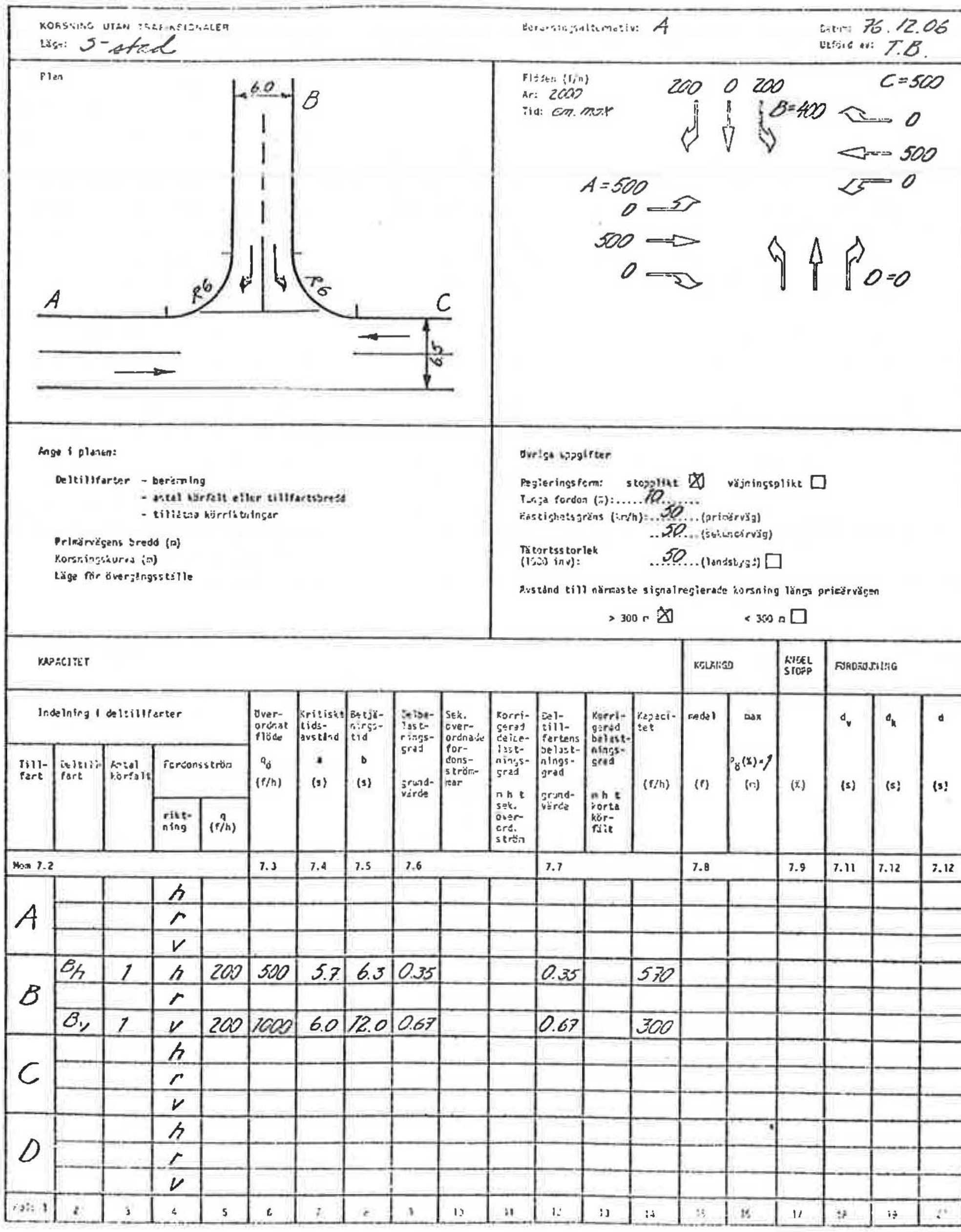
For comparison, I have another OECD exhibit (Figure 12 from page 35, attached here as Figure 28). This figure shows three locations in Britain where comparisons were made between observed flows and those predicted using Tanner's formula (10, 11). I note in site 3, where the critical gap time is identical to that chosen by Hansson, that the results are a perfect match. Sites 1 and 2 have lower critical gap times, resulting in higher capacity for the right-turn movement.

The next figure (Figure 11 from the OECD, here given as Figure 29) compares capacities predicted by five methods, with critical gaps of 5 and 7 s for a straight movement in conflict with two-way primary movements. As may be seen, there is a great deal of variance in predicted capacities. The OECD review concludes with the remark, "The foregoing review of methods of estimating the capacity of major/minor priority junctions has shown that the topic is a very complex one, and that despite the considerable amount of research which has already been completed, a great deal still remains unknown."

Mr. Hansson's paper and procedures add a method for calculating unsignalized intersections that appears to be simpler than others used in some areas.

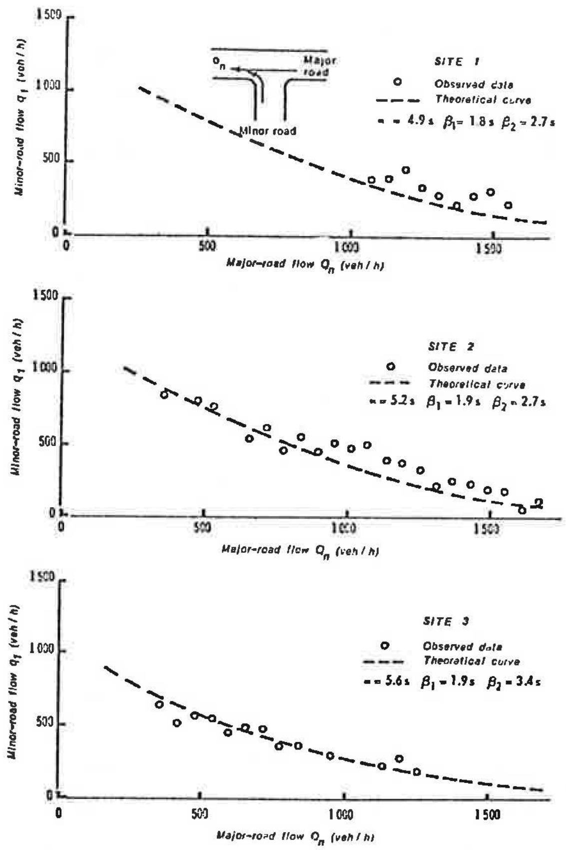
I would like to see an example or two in the paper so

Figure 27. Example 1 from the Swedish capacity manual.



the method could be used by traffic engineers. Thus, comparisons could be made. At present, the paper provides a great deal of background, which has value but does not help the reader make practical use of what is obviously a well-developed method. I hope the Swedish manual can be translated to fill the missing elements.

Figure 28. Comparison of predicted and observed traffic flows (7).



PART 3 BY BÅNG

Mr. Bång has presented a well-written paper outlining the Swedish capacity manual method for calculating, not only capacity, but also queue length, delay, approach grades effect, proportion of stopped vehicles, and signal timing. The method makes use of previous findings by Webster of Great Britain on signal timing and Miller and Gordon of Australia on conflicts between left turns and opposing flow and signal timing, as well as previous Swedish work, especially by Arne Hansson on conflicts between vehicles and pedestrians.

Original procedures are shown for calculating

1. Effect of conflicts between left turns and opposing traffic on two-way streets where (a) a waiting left turn will block the lane and (b) a waiting left turn will not block the lane;
2. Reductions in the capacity of right turns by pedestrians in the crosswalk; and
3. Effects of restricted length of curb lanes where (a) through and right-turning traffic are in the lane and (b) the curb lane serves only right turns.

The paper contains an example of the procedures for a four-way intersection, in downtown Stockholm, with three-phase signal control. There are three examples in the manual: two-phase, three-phase—the one in this paper (Figures 30 and 31)—and four-phase.

For comparison, I calculated capacities for two of the three intersections using the HCM and then compared the results (Tables 5 and 6). Tables 5 and 6 show the comparisons. My calculations were rough, but are adequate for comparative purposes. Overall intersection capacity appears to correlate fairly well. Location A, M-Stad, shows an overall variance of 6 percent, or 220 vehicles. In this example, the HCM would predict more overall capacity than the Swedish method. Location B, Stockholm, shows an overall variance of 4 percent. In this instance, the Swedish manual predicts more capacity than the HCM.

The individual approaches do not compare as well, with variances of 2-50 percent between the methods. One of the major reasons for variations on the high end is signal timing. Table 7 reviews the level of ser-

Figure 29. Comparison of capacities (7).

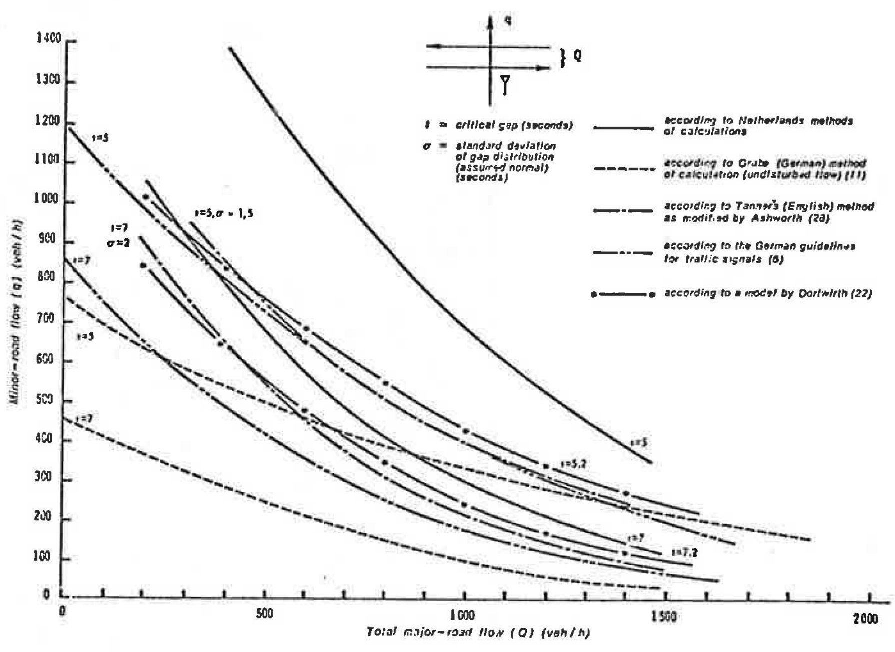


Table 5. Capacity comparison for location A: four-way two-phase intersection with all two-way streets.

Approach	Width (m)	Volume L	Volume S	Volume R	HMC Level E	Capacity*	Difference
BN	5.5	100	200	200	750	590	-160 (-0.21)
BS	5.5	50	150	150	940	470	-470 (-0.50)
AV	6.5	110	760	150	1000	1200	+200 (0.20)
AO	6.5	80	850	170	1060	1270	+210 (0.20)
Total					3750	3530	-220 (-6)

*Pedestrian volume: N-S = 500-700, E-W = 200-150.

Table 6. Capacity comparison for location B: four-way three-phase intersection with all two-way streets.

Approach	Width (m)	Volume L	Volume S	Volume R	HMC Level E	Capacity*	Difference
HN	9.6	110	640	120	1340	1310	-30 (-0.02)
HS	8.0	50	810	320	1260	1340	+80 (0.06)
RV	9.0	90	130	110	700	780	+80 (0.06)
RO	9.0	380	290	120	1090	1150	+66 (0.11)
Total					4390	4580	+190 (+4)

*Pedestrian volume: N-S = 1300-800, E-W = 700-800.

Figure 30. Location A from the Swedish capacity manual.

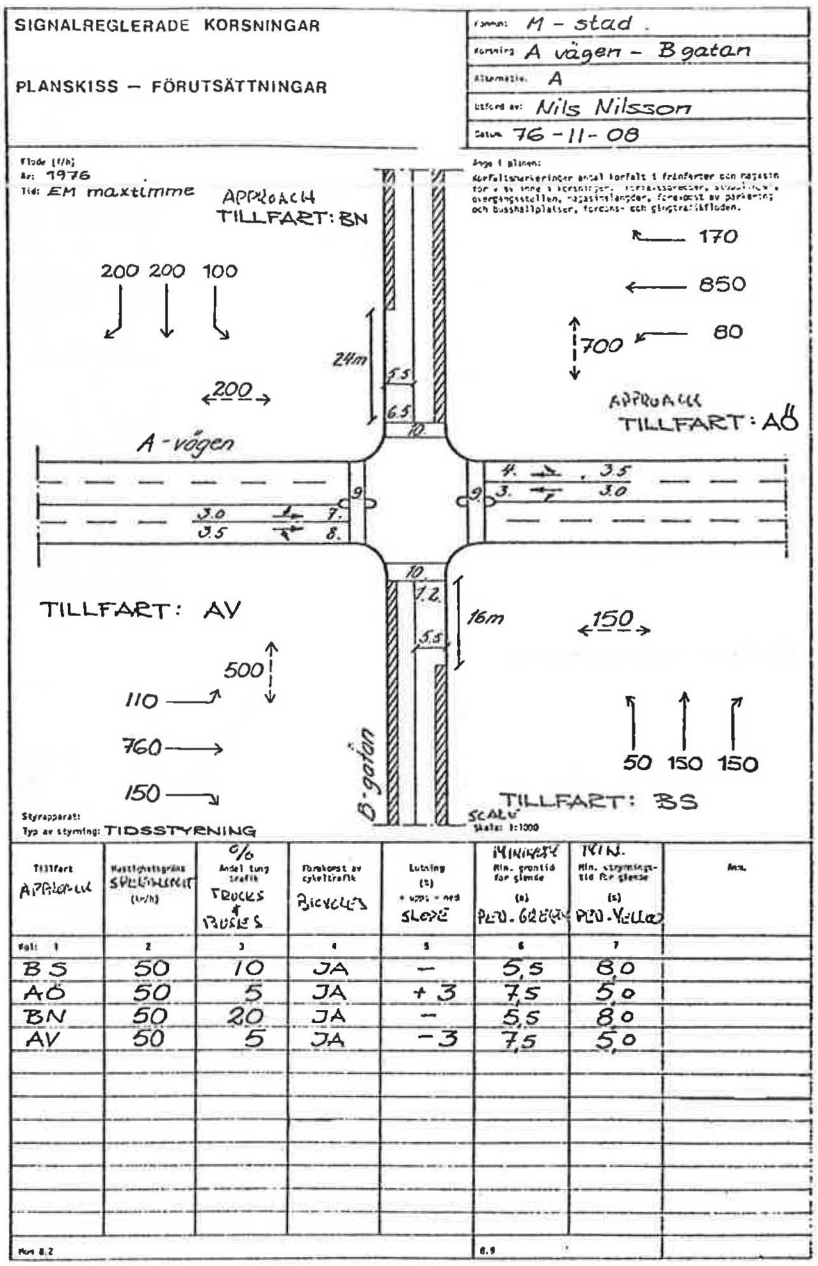


Table 7. Level of service comparison for locations A, B, and C.

Location	Approach	W.	Volume	G/C	HCM LOS "E"	HCM V/C	HCM LOS	Degree of Saturation
A	BN	18	500	0.49	750	0.53	C/D	0.86
	BS	18	350		940	0.37	C	0.74
	AV	21.3	1020	0.40	1000	1.02	E	0.86
	AO	21.3	1100		1060	1.04	E	0.87
Total			2870		3750	0.76	D	
B	HN	31.5	870	0.36	1340	0.65	D	0.67
	HS	26.2	1180		1260	0.93	E	0.82
	RV	29.5	330	0.18	700	0.47	C	0.58
	RO	29.5	790	0.29	1090	0.72	D	0.82
Total			3170		4390	0.72	D	
C	MN	21.3	550		530	1.04	E	
	MS	14.8	280		380	0.74	D	
	IY	19.7	1080		1100	0.98	E	
	IO	19.7	1270		1050	1.15	E	
Total			3180		3060	1.04	E	

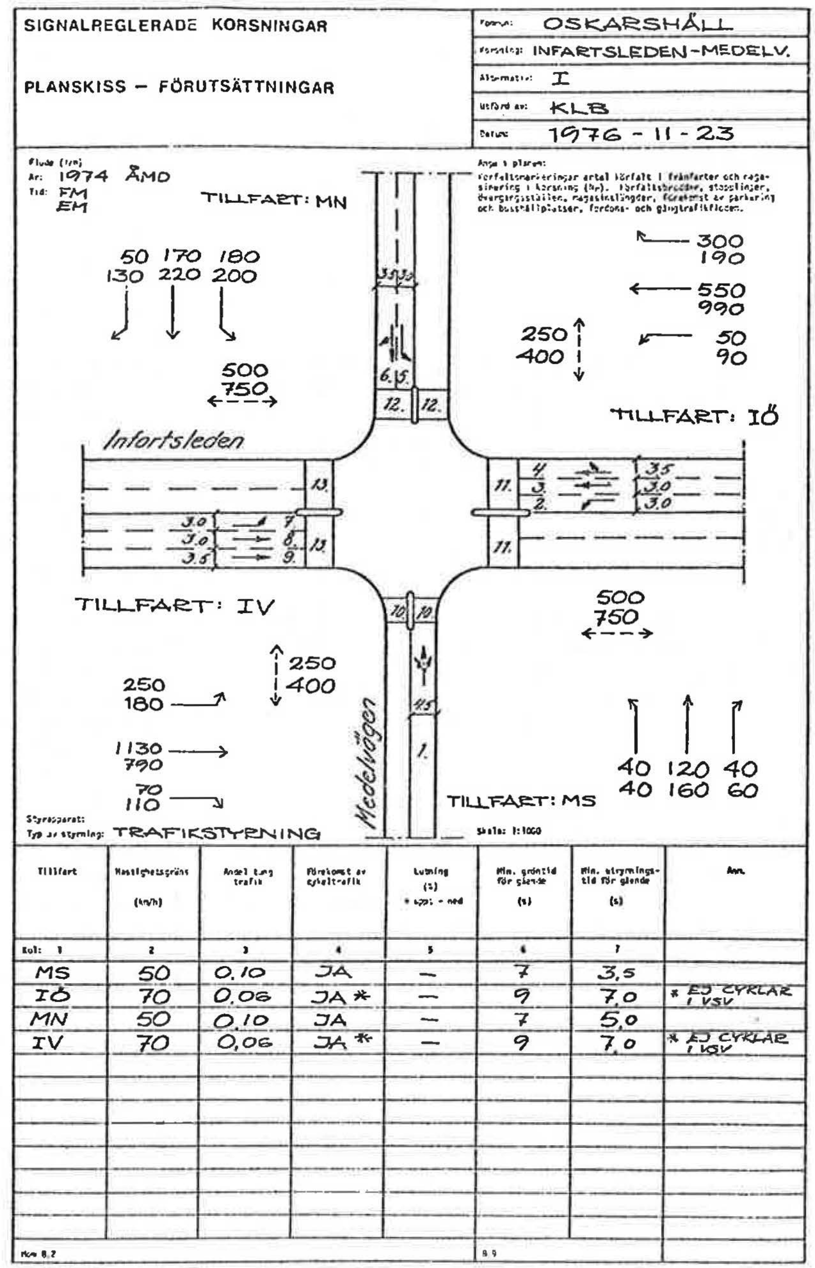


Figure 31. Location C from the Swedish capacity manual.

vice expected with the signal timing shown in the examples. Location A would have been timed differently, based on HCM analysis, and would have evened out the capacity differences. Delay was not calculated for location A. Therefore, there was no opportunity for comparison of that item.

I should note that neither capacity nor delay was calculated for location C (Oskarshall), the four-phase, actuated intersection.

I wish to congratulate Mr. Bång on a well-written paper and on his contribution of a methodology for calculating effects of conflicts between left-turning and opposing flows, conflicts between turning vehicles and pedestrians, delay, and queue lengths.

Analysis by lane and movement should give the traffic engineer an opportunity for specific insights into an intersection's operations. It will obviously not be a favored method for planners or those who seek a quick, simplistic method.

Although the comparison of approach capacities appears to indicate substantial variance between the Swedish manual and the HCM, two factors in the examples could have significance in reducing the variance: (a) signal timing, as previously noted, and (b) use of approach width versus lane. Please note the single lane approaches at location A of 5.5 m. The HCM (and others) treats this width as two lanes (although not specifically), thus providing greater capacity.

Jack A. Hutter, Traffic Institute, Northwestern University, Evanston, Illinois

Because of time and space limitations, my discussion will be directed only to the paper presented by Mr. Bång. This informative paper provides a detailed overview of an analysis procedure that offers some original and unique techniques for evaluating the performance capabilities of signalized intersections.

The primary object of this discussion paper is to highlight those attributes of the Swedish manual that may be applied to the development of an improved U.S. HCM.

In discussing Mr. Bång's paper, I shall carefully avoid the term "intersection capacity analysis," because the Swedish manual provides potential capabilities for evaluations beyond the determination of signalized intersection capacity. Additional analysis techniques include

1. Calculation of optimum signal timing,
2. Determination of queue length,
3. Calculation of the proportion of stopped vehicles,
4. Measures of delay, and
5. Determination of the degree of saturation, similar to a volume-to-capacity ratio that provides a measure of the level of operation of the critical lanes or approaches of an intersection.

Another significant feature of the Swedish manual is the important mathematical relations that are presented in graphic or tabular form for ease of use by the practicing engineer. In addition, the layout of the descriptive procedure is in the form of stepwise calculations, supplemented by clearly labeled work forms, which assist the user in following a fairly detailed and complex analysis procedure.

There are several calculation routines in the Swedish intersection analysis procedure that appear to have superior sensitivity and consistency of application than do the procedures in the HCM. For instance, Swedish calculations are based on the analysis of the individual vehicle streams entering an intersection, rather than aggregated intersection approaches. This enables the user to identify the critical lanes as well as the critical approaches of the intersection. It also provides the analyst with an overview of the total performance potential of the intersection, through the identification of the degree of saturation.

In currently practiced "critical lane analysis" procedures, the user must have access to lane volume data or make assumptions regarding the distribution of the approach lane volumes, which can produce serious inconsistencies in the results of the analysis. The Swedish manual provides a unique lane-volume distribution technique whereby the lane flows are distributed in proportion to the saturation flow values of the adjacent lanes. This technique allows all users to perform this calculation in a consistent manner.

The Swedish manual also provides a technique for quantifying the effect of the conflicts between left-turning vehicles and opposing flows and the conflicts between turning vehicles and pedestrian movements on the performance capabilities of the intersection. This simplified lane classification system enables the user to identify the type and degree of vehicle and pedestrian conflicts and to select corresponding values of saturation flow for each lane type in a consistent manner.

Most intersection analysis techniques are unable to calibrate the utilization or capacity of a curb lane if parking is not prohibited for a specified distance from the intersection approach or exit. The Swedish manual provides procedures for calculating the reduction in saturation flow values for curb lanes of restricted lengths for the two cases where the curb lane serves right-turning and through vehicles or the curb lane serves only right-turning vehicles.

The second object of this discussion is to raise questions regarding various aspects of the Swedish intersection analysis procedures, questions brought about by my lack of practical experience in applying these new techniques to real-world situations, and give to the author an opportunity to share his insights and experiences regarding the use and application of the manual by practicing traffic engineers in Sweden.

The lane-flow distribution technique is designed to produce consistent results by all users. The question is whether this technique has demonstrated that it will simulate actual lane-volume distributions under a wide range of geometric and traffic flow conditions. Also, it is stated that the method is based on theoretical models supported by a limited amount of field survey. One must ask if the experiences of the practicing engineers in Sweden in the application of the techniques supported the validity of the models.

There is an increasing need for American engineers to have models that provide quantitative measures of queue length, stopped vehicles, and vehicle delay for use in environmental and fuel conservation studies. Here the question is whether the techniques in the Swedish manual provide for reasonably accurate and consistent simulation of these variables, insofar as previous research would tend to indicate that flow distributions and traffic performance by lane are highly variable and erratic quantities when measured on a cycle-by-cycle basis.

It was reported that calculations require from 1 to 6 h per intersection, depending on the number of measures desired and the skill and experience of the user. Therefore, one must know the level of acceptance and use of the procedure by practicing engineers in Sweden.

Arthur A. Carter, Office of Traffic Operations, Federal Highway Administration

The authors should be commended for undertaking research into highway capacity in their own country, rather than depending on earlier, increasingly obsolete American criteria, or criteria from other countries. Given the performance differences that we see just from locality to locality even within the United States, I suspect that only concepts, not absolute values themselves, are freely transferable from country to country.

Having been technical editor of the American 1965 Highway Capacity Manual, I shall discuss in general terms how these Swedish efforts relate to the American state of the art.

PART 1 BY PETERSON

First, I was not surprised by Mr. Peterson's observations that capacity flows in several countries appear greater than indicated by U.S. criteria. Typically, I find that foreign flows exceed ours, possibly because of smaller vehicles and different driver behavior. Therefore, I am not particularly concerned about findings that disagree with either our 1950 or 1965 HCM, in terms of absolute numbers.

I am more interested in concepts, trends, and procedures. In this regard, the literature review and analysis of some 900 references, accomplished in 1973, seems particularly impressive. It would appear to be superior, in both numbers and scope, to anything yet done in the United States. Translation into English might be recommended.

Although Mr. Peterson indicates that a complete manual has been developed, his emphasis appears to be on urban intersection operations, the subject of the two other reports presented here, with briefer mention of urban networks, bicycle flows, and pedestrian traffic. He does not specifically cover the basic area of rural uninterrupted flow, either two-lane or multilane, or basic freeway flows. I assume that these fundamentals are included in the manual, presumably in the road sections and street sections categories of the first table. I would be interested in learning more about their handling.

I am confused by Mr. Peterson's comments regarding pedestrian crossings, which seem to stress the between-intersection case at the expense of the at-intersection case. This appears to be in conflict with Mr. Bång's paper, which emphasizes the at-intersection situation. [The authors of this paper revised the draft on which these discussions were based. Therefore, some of the points in this discussion are no longer relevant.]

Mr. Peterson describes the use of models based on queue theory for much of the work as an important innovation, with only limited use of empirical evidence. Although I am probably biased, I continue to feel that in this capacity field, practice frequently varies too widely from theory for us to make empirical evidence secondary. I strongly support validation with empirical data.

The "description of consequences" concept appears to be parallel to our level-of-service concept, but extended to better cover congested conditions, where delays, stoppages, and queueing occur. Its orientation differs somewhat from our procedures in that, in its straightforward form, design and demand volume are both known and performance is unknown. Apparently, application to design problems or to volume determinations is a trial-and-error procedure. This is in contrast with our current procedures, which are most directly oriented to flows obtainable with a given design. There probably is no one answer to which is best; it depends on the particular user's needs.

In this connection, I sympathize with Mr. Peterson concerning his quandary regarding the right degree of accuracy. Here in the United States, we have the same problem. I suspect that the final answer one day may be three entirely separate manuals, at different levels of precision and complexity, for planners, designers, and operators.

PART 2 BY HANSSON

Mr. Hansson's work on the performance of unsignalized intersections is much needed. Relatively little has been done in this field, and that which has been done is mainly on a case-history basis. I assume that four-way stops are not included.

This chapter apparently uses the term "capacity" in its general sense to cover all levels of service, rather than its specific saturation flow sense, because cautions are given against its application where the minor flow is at a load factor (which, as they define it, is similar to our volume-to-capacity ratio, rather than in the U.S. sense) exceeding 0.8 or 0.9. The observation in the report that such conditions normally would warrant traffic signals tends to substantiate the 1965 HCM statement that the true capacities of unsignalized intersections are usually of academic interest only.

The reference to M/G/1 queueing system and M/M/1 queueing model may well be meaningless to many readers of this paper. It would have been helpful to describe these models in general terms.

I am somewhat concerned by the rather large number of assumptions and deliberate omissions, in contrast to the overall considerable detail involved in the method. We have no answers regarding the influence of such factors as primary flow headway distributions, the combining of flows, move-up time, conflicts between flows, relative sizes of flows, and load factor as they define it, but I feel that the related assumptions and omissions may have a significant effect in this unsignalized case. This points up the need for validation, as suggested by the author.

Of greatest concern to me is the author's decision to omit consideration of platooning produced by nearby traffic signals, on the basis that the effect, like that of several other factors, cannot be quantified. In this country, one of the principal factors governing the performance of a particular unsignalized intersection is the proximity of nearby signalized intersections and the relative phasing of the nearest ones on either side of that under consideration. While I agree that the effect is nearly impossible to quantify, I submit that it is a principal reason why some of the work in this country has ended up quoting specific cases rather than general criteria.

With all other conditions being identical, I see one specific location operating well, with periodic breaks in the major flow occurring simultaneously in both directions to permit free crossings, while another never is free of at least one direction of the major flow. Possibly, adjacent signalized intersections are less common in Sweden than in the United States, but I question that this factor could be overlooked in practice in the United States. I agree, therefore, with the author's conclusion that primary flow headways may be the weakest link in the method.

Also of great importance in this country is the nature of the primary road—two-lane two-way, multilane undivided, or multilane divided. This element is considered in the method, and we are interested in their inability in the validation studies to confirm higher critical headways for crossing four-lane roads than for two. Possibly, the influence of adjacent signals, more likely to be found on four-lane roads than on two, is involved.

PART 3 BY BÅNG

Mr. Bång's work on signalized intersections is a valuable contribution to the state of the art, particularly with respect to its consideration of turning movements related to pedestrians. This is one area where the public expects and seems to assume that we have much more information than we actually do.

I was surprised to note that the principal Swedish intersection capacity criteria, prior to the subject work, were based on the original American 1950 HCM rather than the 1965 edition, which is the principal reference in use today. Apparently this is because Sweden was in the forefront in the 1950's and developed its own procedures soon after ours were published.

It would be useful to learn which evaluations were made in Sweden after the 1965 manual was issued that caused them to decide to start over rather than try to adapt its procedures. Such information would be particularly timely as we begin the initial steps of preparation of a new edition of the American manual.

The Swedish procedure appears to emphasize a different range of operational levels than does our manual. While they indicate that their work omits design levels and centers on saturation flow or capacity, which is our level of service E, they suggest that 0.8 times saturation flow be used in practice as a feasible maximum. In effect, then, they closely relate to our level of service D, which of necessity has become our design level for many urban applications even though level C is considered more desirable.

The Swedish procedures then appear to extend from our level D into level F, with a description of consequences throughout including level F, as compared to our levels of service A through E, with only general reference to broad breakdown of level F. This is rather characteristic of foreign signalized intersection capacity investigations and procedures as compared to current U.S. procedures; they typically concentrate on saturation flow to a greater extent than do U.S. procedures.

I agree that any new American method must include consideration of stops, queueing, and delay, which are the elements of intersection performance most visible to users. Our load factor (that is, percentage of fully utilized green intervals) is inadequate where most needed.

It is not clear why the Swedish measures developed on a by-lane basis are "not always meaningful" when expanded to a complete approach or whole intersection; this concerns me.

The conclusion that both width and number of lanes are significant is interesting. This point is controversial, not only within the United States but internationally, where viewpoints range from the 1965 HCM's overall approach width criterion to Australia's basic number-of-lanes procedure.

Regarding the pedestrian effect, I find the tabulation form of presentation quite good and a step toward what I would like to have, to answer citizens' questions. I wonder, though, whether such national criteria could ever be established for the United States as a whole, given the widely differing degrees of respect shown each other by drivers and pedestrians from one city to another. It would seem that city factors would have to be developed, much as both we and our Swedish counterparts dislike them.

It is interesting to note that several of the same factors that have escaped easy solution in this country also defy solution in Sweden, including bicycles, actuation of signals, and weather.

The problem solutions described conform with Mr. Peterson's introductory comments; they relate to situations where the traffic volumes and geometrics are known, and the nature of the resulting operation is desired. It is not indicated whether or not the method can be used effectively for other situations, where either volumes or geometrics are the unknown. (The sample problems that exist in the manual undoubtedly assist greatly in providing an understanding of the procedures.)

At first glance, the typical problem solution time, 3-6 h, looks long. However, given that this involves a detailed solution including queue lengths and so on, something much beyond our current procedures, and that a basic signal timing solution is possible in an hour, it is probably reasonable.

Finally, it is unfortunate that this work, like nearly all other work in this field in recent years, must be tagged as "needing validation." I hope users will soon do sufficient testing and evaluation so that validity can be more firmly established.

Publication of this paper sponsored by Committee on Highway Capacity and Quality of Service.

Weighing Vehicles in Motion

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A scale for weighing vehicles in motion was developed at the University of Saskatchewan. This scale has been successfully operated unmanned at two locations in Saskatchewan for the past 2 years. An expanded evaluation program is currently under way in which the scale will be installed and evaluated in Ontario, Quebec, and New Brunswick by a project committee of the Roads and Transportation Association of Canada.

The relationship between vehicle and axle loads and the structural requirements of bridges and roadways has been and continues to be an area of particular interest to those concerned with the provision and maintenance of bridge and roadway facilities. The need for comprehen-