Development of Analysis Procedures for Signalized Intersections in the 1985 *Highway Capacity Manual*

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The procedures for signalized intersection analysis covered in Chapter 9 of the 1985 *Highway Capacity Manual* are complex and reflect the actual complexity of operations at these critical conflict points within surface street and highway systems. The concepts and logic behind the development of the procedures are documented with a focus on revisions made to published and unpublished source materials in the final preparation of the manual text.

Preparation of materials for signalized intersections for the 1985 *Highway Capacity Manual* (HCM) was a most difficult and complex task. The change from the 1965 HCM is complete, involving a new analysis approach and radically different level of service measures.

From the earliest involvement of the Highway Capacity and Quality of Service Committee of TRB, two general directions had been set:

1. The new HCM would use critical movement analysis as the basis for signalized intersection analysis, and
2. The measure of effectiveness for level of service would be delay.

Neither of these were startling decisions. Critical movement analysis was not a new methodology. Bruce Greenshields developed it successfully in the early 1940s as a signal timing methodology, and had calibrated saturation flow rates and lost times. The English and the Australians have adopted and calibrated critical movement analysis as a capacity analysis technique and included methodologies published in their own national highway design and analysis manuals.

In the early 1970s, several American researchers, including Donald Berry at Northwestern University and Carroll Messer at the Texas Transportation Institute, were conducting studies to adapt these procedures to U.S. intersections. Critical movement analysis was a desirable alternative for signalized intersection analysis because it closely related the analysis of geometrics, signal design, and traffic, identifying those movements and lanes that controlled the operation of the intersection. As an analytical approach, critical movement analysis provided a more precise model of signalized intersection operation.

The move to delay as a measure of effectiveness was equally logical. The use of load factor in the 1965 HCM resulted in considerable confusion. It was difficult to measure accurately and consistently in the field. Furthermore, the 1965 HCM assumed that load factor and delay were well correlated. By 1970, a number of papers, notably by Adolf May of the University of California at Berkeley, had debunked this theory. Delay was an attractive alternative. It could be measured in the field with little difficulty under stable operating conditions and was most closely related to the driver's perception of service at the intersection.

The first step toward a new methodology appeared in *Circular 212* by the Transportation Research Board (*1*). As part of Project NCHRP 3-28I, JHK & Associates adapted the results of several published studies to form a critical movement analysis procedure. The primary source used was the work of Carroll J. Messer (*2*). The method presented was a basic critical movement analysis approach that resulted in estimates of prevailing vehicle-capacity (V/C) ratios on various groups of lanes and approaches at the intersection. The method did not contain a procedure for estimating delay, and correlated levels of service directly to V/C ratio.

As a result of the findings of NCHRP 3-28I, JHK & Associates were sponsored in Project NCHRP 3-28II to calibrate a critical movement analysis method for the new HCM that would include a delay estimator. The two reports resulting from that study formed the primary source documents for the 1985 HCM (*3*, *4*).

The methodology of the source documents was, however, revised in a number of ways based on recommendations of the JHK Project 3-28II team, the Polytechnic Project 3-28B team, and the Committee on Highway Capacity and Quality of Service. The major modifications made and the logic that led to them are explained in following sections.

BASIC STRUCTURE OF PROCEDURES

The source materials contained two levels of analysis. Operational analysis was provided to cover most situations in which all geometrics, signalization, and demand volumes could be measured or projected. Because of the large number of variables that had to be addressed, the procedure was set up in a modular format to simplify computations. Planning analysis was established for use where little detail was available, such as signalization, and a more general futuristic analysis was sufficient. In basic concept and format, both of these procedures were used in the 1985 HCM.

The operational analysis procedure is presented in the 1985 HCM as a method for determining V/C ratios and delays (level of service) at a signalized intersection, given a detailed description of traffic demand, signal timing, and geometry of the intersection. The manual also provides (in the 1985 HCM,

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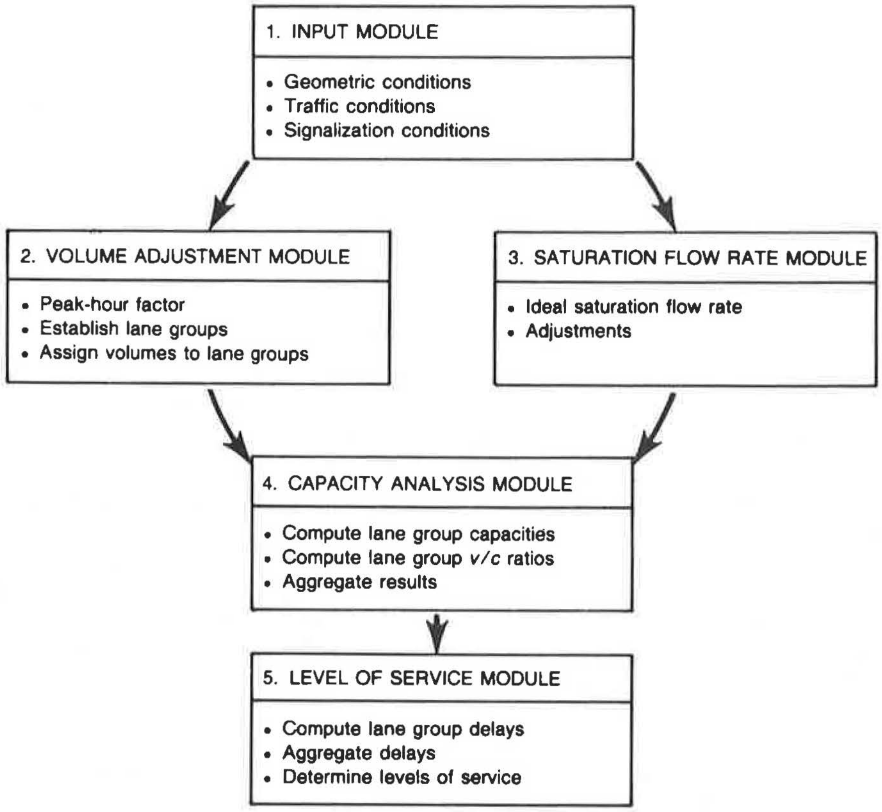

FIGURE 1 Operational analysis modules for signalized intersections.

Figures 9–14) guidance for alternative computational sequences using the same procedures. The analysis modules of the operational analysis procedure in the 1985 HCM are shown in Figure 1 (*5*, see Figure 9-1).

The analysis modules are somewhat simplified from the source documents, and each module is associated with a worksheet used for computations. The analyses conducted in each module are briefly described as follows:

1. Input module—This is merely a worksheet providing for specification of existing or projected traffic, geometric, and signalization conditions at the intersection to be analyzed. It is a summary of required input data.
2. Volume adjustment module—In this module, volumes are adjusted to reflect peak rates of flow, lane groupings for analysis are defined, and an optional lane use adjustment factor may be applied.
3. Saturation flow rate module—The saturation flow rate under prevailing conditions is computed for each lane group. A base saturation flow rate of 1,800 passenger cars per hour per lane (pcphpl) is used, and is adjusted by eight factors, including adjustments for lane width, grade, heavy vehicles, parking conditions, bus blockage, area type, left turns, and right turns. The saturation flow rate is the maximum number of vehicles that could be accommodated by the lane group if the signal were always green for those lanes.
4. Capacity analysis module—In this module, V/C ratios for each lane group and for the intersection as a whole are computed. Critical lane groups are identified, and signal timings may be estimated if they are not known.
5. Level of service module—The average individual stopped delay for each lane group, approach, and for the overall intersection is estimated, and level of service criteria are applied.

This modular organization of the chapter allows for a more orderly and understandable analysis, and highlights key aspects of the results. In following sections the development of level of service (LOS) criteria, and the algorithms involved in each analysis module are discussed.

LEVEL OF SERVICE CRITERIA FOR SIGNALIZED INTERSECTIONS

The level of service criteria selected for signalized intersections are given in Table 1 (*5*, see Table 9-1). From the outset, there

TABLE 1 LEVEL OF SERVICE CRITERIA FOR SIGNALIZED INTERSECTIONS (*5*, Table 9-1)

Level of Service	Stopped Delay per Vehicle (sec)
A	≤ 5.0
B	5.1 to 15.0
C	15.1 to 25.0
D	25.1 to 40.0
E	40.1 to 60.0
F	> 60.0

was very little controversy concerning the delay thresholds themselves. The judgment of the JHK researchers was well founded in data and experience. Only the boundary between LOS A and B came under discussion, and at various times in the process moved from 10 to 5 sec per vehicle, and in one draft, to 7.5 sec per vehicle. The basic discussion involved whether any intersection could ever operate at delays under 5 sec per vehicle, and whether this criterion was reasonable. Data presented by various committee members proved the practicality of the 5 sec per vehicle bound, which was adopted.

Although the LOS criteria were easily established from source materials, the concept of delay as a LOS measure came under continuous review and controversy. The original decision to use delay as a measure of effectiveness was clear. The implications of this, however, did not become obvious until procedures were developed. The implications were severe and caused the Highway Capacity and Quality of Service Committee to review its commitment to delay as a LOS measure on several occasions.

Research made clear that delay was related to several variables: signal progression, cycle length, green times, V/C ratio. For V/C ratios up to approximately 0.90, this listing is in order of decreasing importance. These relationships created a problem in interpretation—delay, and thus LOS, was not well correlated to V/C ratio for most of the range of commonly occurring values. In fact, V/C ratio has only a small impact on delay at values below 0.90. High delays, however, can and do occur (even when V/C ratios are low) primarily when progression is poor, cycle lengths are excessive, and green times are inefficiently allocated. LOS F can and does occur at V/C ratios under 1.00. Thus, LOS F no longer signifies a breakdown, but merely that delay has reached an unacceptable level.

This concept was in itself controversial because what is considered unacceptable in one location may be acceptable in another. A driver at an intersection in midtown Manhattan may expect reasonable high delays, whereas a driver in a small rural or suburban community would not tolerate the same level of delay.

As a result of this, the committee considered relating LOS directly to V/C ratio, as in *Circular 212 (1)*, and also the development of a LOS matrix, with delay on one axis, V/C ratio on the other, and levels of service specified for each cell. After frequent and long discussions, however, the committee held to its original conviction that delay was the most appropriate level of service indicator for signalized intersections.

In presenting the methodology, this decision required that the interpretation and meaning of level of service be clearly and precisely defined. Thus, the modular arrangement of computations highlights the need to consider both V/C ratios (capacity) and delays (level of service) in an analysis. For the first time, and for the only time in the 1985 HCM, the concepts of capacity and level of service are not strongly linked—and both must be carefully considered in an analysis.

The decision to retain delay as an LOS measure had a hidden benefit—it encouraged efficient signal timing. In *Circular 212*, where LOS was keyed to V/C ratio, to improve service, one had to reduce V/C ratios. A low V/C ratio, however, is an indication of much unused green time within the cycle, which is inefficient from the point of view of signal timing. In fact, there is a minimum delay cycle length that generally occurs within a critical V/C range of 0.80 to 0.90 for the intersection as a whole.

Lower V/C ratios increase delay as a result of inefficient cycle lengths and green times. Higher V/C ratios also increase delay as a result of individual cycle failures within the standard 15-min analysis period. Even where the V/C ratio for a 15-min analysis period is below 1.00, individual cycles within that 15-min period may fail. The probability of this occurring increases sharply at V/C ratios higher than the range cited previously. The new manual, with its reliance on delay as a measure of effectiveness, does not encourage signal timing to achieve low V/C ratios, but, in fact, encourages V/C ratios that are as high as possible without producing individual cycle failures. This more closely matches the objective of signal control than previous methods and interpretations.

The retention of delay as the level of service measure caused a change in the planning procedure of the source materials. The planning procedure does not yield delay estimates, but produces a sum of critical volumes that is compared with capacity criteria. Source materials associated this sum with a level of service, which would have been inconsistent with the delay definition of LOS. Thus, the 1985 HCM does not yield a level of service for the planning technique, but merely a judgment as to whether capacity will be exceeded or not. This further reinforces the distinction between capacity analysis and level of service analysis for signalized intersections.

INPUT MODULE

A summary of input data for operational analysis of signalized intersections is provided by the input module. The amount and specificity of data required for the 1985 HCM exceeds that required by previous procedures. In addition to the basic geometrics, signal timings, and traffic flows, the new manual requires data on

1. Pedestrian flows in crosswalks, which influence right-turn adjustment factors;
2. Approach grades, which influence both passenger car and heavy vehicle operation;
3. Parking maneuvers per hour within 250 ft of the intersection. Parking creates friction due to the presence of parked vehicles, and blockage of the right lane as vehicles enter and leave parking spaces; and
4. Arrival type, which is a descriptor of progression quality with a major impact on delay estimates.

To assist analysts dealing with unavailable data, default values are suggested in the manual for several of these, although it is clearly preferable to obtain field values.

VOLUME ADJUSTMENT MODULE

In the volume adjustment module, several analysis steps take place. First, all movement volumes are adjusted to reflect rates of flow during the peak 15 min. This is done by dividing each volume by the appropriate peak hour factor (PHF).

The most critical step of this module is, however, the establishment of lane groups for analysis. The procedure for doing this differs somewhat from source materials. The source documents allow the analyst several options for establishing analysis lane groups. A three-lane approach with no separate turning lanes could, for example, be analyzed as one-, two-, or three-lane groups. The results of analysis could vary somewhat depending on the approach taken. To avoid confusion, Polytechnic University's 3-28B project team established a set procedure for disaggregating the intersection into lane groups. In general, a set of approach lanes is to be analyzed as a single-lane group if there are no restraints on lane use. In such a case, it is assumed that drivers will select their lane to establish an equilibrium operation. Where impediments to lane use exist, such as in the provision of an exclusive left- or right-turn lane, a separate lane group must be established. Thus, a three-lane approach with no exclusive turn lanes must be analyzed as a single-lane group.

There is a case where this standard procedure may not apply. In cases where high left- and right-turn volumes are such that they fully occupy a lane or lanes, a de facto left- or right-turn lane exists, even if it is not so designated by regulation. The 1985 HCM presents a thumbnail procedure, which was developed specifically for the manual, for determining whether such a case exists or not. Where there is a de facto turn lane, it too should be analyzed as a separate lane group.

The procedure assumes that left-turning vehicles through an opposing flow may be approximately expressed as equivalent through vehicles:

$$V_{LE} = v_L \times [1{,}800/(1{,}400 - v_d)]$$

The equivalence is taken as the ratio of 1,800 pcphpl (the ideal saturation flow rate) to 1,400 $-$ v_d, which is the calibrated saturation flow rate for left turns filtering through an opposing flow.

Where a de facto left-turn lane exists, all left turn equivalents would occupy the left lane, with all other vehicles sharing remaining lanes equally. The volume in each of the other lanes is then:

$$(v - v_L)/(n - 1)$$

If the left-lane equivalent volume is less than the average volume in other lanes, it is assumed that some through vehicles will move into and share the left lane to establish equilibrium. In such a case, no de facto left-turn lane exists. Where the equivalent left-lane volume is greater than the average volume in other lanes, a de facto left-turn lane exists and should be separately analyzed.

A similar procedure for de facto right-turn lanes can be applied, except that one right turn is assumed to be roughly equivalent to one through vehicle.

As a last step in this module, a lane use factor can be applied against each lane-group flow rate. Use of this factor was made optional in the 1985 HCM, and has the following significance:

1. Where the factor is applied, results (V/C ratio, delay) are for the worst lane of the lane group.

2. Where the factor is not applied, results (V/C ratio, delay) are for the average lane of the lane group.

The analyst may use either option, but should be consistent when comparing before–after or various intersection analyses.

SATURATION FLOW RATE MODULE

In this module, the ideal saturation flow rate of 1,800 pcphpl is adjusted to reflect prevailing conditions at the intersection:

$$s = s_o N f_w f_{HV} f_g f_p f_{bb} f_a f_{RT} f_{LT}$$

where

s = saturation flow rate for lane group in vehicles per hour;
s_o = ideal saturation flow rate per lane, usually 1,800 pcphpl;
N = number of lanes in lane group;
f_w = adjustment factor for lane width;
f_{HV} = adjustment factor for heavy vehicles;
f_g = adjustment factor for grade;
f_p = adjustment factor for parking;
f_{bb} = adjustment factor for bus blockage;
f_a = adjustment factor for area type;
f_{RT} = adjustment factor for right turns; and
f_{LT} = adjustment factor for left turns.

Most of these factors are simple tabulations taken directly from source materials. Heavy vehicle factors were slightly altered to reflect more recent data collected in Texas (unpublished). The area type factor was somewhat controversial in that many of the factors normally believed to be accommodated by such an adjustment are accounted for elsewhere in the procedure. Pedestrian flows are accounted for in the right-turn adjustment. Peaking is specifically accounted for in the use of flow rates during the peak 15-min period. Nevertheless, the JHK data base indicated that the general environment of central business district (CBD) intersections led to approximately a 10 percent reduction in capacity and service flows. Thus, the factor was retained and is recommended for use.

Right- and left-turn factors, however, underwent extensive development beyond the source materials throughout the preparation of the manual. Fully documented in unpublished technical memoranda to the Committee on Highway Capacity and Quality of Service, the changes reflected some additional data, some revised interpretations of existing data, and the use of harmonic means to calculate complex cases of protected and permitted phasing and turns from shared lanes.

Right- and left-turn factors cover eight different cases under which turns can be made. They may be made from exclusive lanes or from mixed lanes. Signalization can provide for permitted (opposed) turns, protected (unopposed) turns, or a combination of both. Two special cases are provided to handle double exclusive turn lanes and single-lane approaches, which are unique because left and right turns are made from the same lane.

The most dramatic change from source materials involved permitted or opposed left turns. Previous materials developed in Australia, England, and the United States have always assumed that permitted left turns filter through the opposing flow at a calibrated rate for the entire period of the green phase. For the 1985 HCM, this rate was calibrated as 1,400 – v_o. The saturation flow rate could then be multiplied by the ratio of effective green-to-cycle length (g/C) to obtain the capacity for left turns. However, this ignores a critical characteristic of flow at the intersection.

Various portions of the green phase that affect opposed left turns are shown in Figure 2. When the light turns green, the opposing queue of waiting vehicles proceeds through the intersection. During the time it takes this queue to clear, no left turns may proceed because there are no gaps in the opposing platoon. This time is designated as g_q, the green time blocked by an opposing queue.

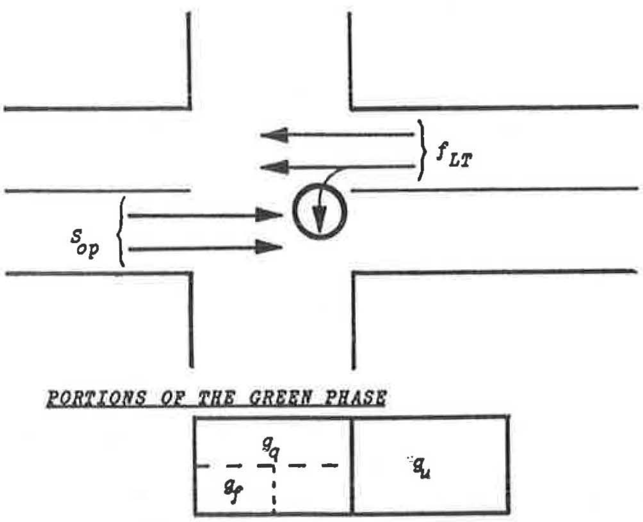

FIGURE 2 Permitted left turns at a signalized intersection.

The remaining green time is g_u, the unsaturated green time during which left turns can filter through the opposing flow at a rate of 1,400 – v_o. Another portion of green time must also be considered. If left turns are made out of a shared lane, through vehicles can proceed during g_q until the first left turning vehicle arrives. The left turner must wait until the opposing queue clears, effectively blocking the left lane for all vehicles. The portion of g_q that can be used by through vehicles in the left lane until the first left-turning vehicle arrives is denoted as g_f.

A complex analytic model was developed specifically for the 1985 HCM to estimate these critical times. Capacity of the left lane was then taken to be the through vehicles that proceed during g_f, together with the left-turning vehicles that can filter through the opposing flow during g_u, and one to two left turners that proceed on the clearance interval at the end of the phase. This capacity was converted to an equivalent left-turn factor for application in the saturation flow module. A worksheet for finding f_{LT} for permitted left turns and the equations that comprise the analytic model is shown in Figure 3 (5, see Figure 9-9).

Green times are as previously defined. Other variables used include the following:

S_{op} = saturation flow rate for opposing lanes in vehicles per hour of green,
N_o = number of opposing lanes,
Y_o = flow ratio for opposing lanes,
C = cycle length in seconds,
P_{LTO} = proportion of left turns in opposing approach flow,
P_{LT} = proportion of left turns in lane group under consideration,
P_L = proportion of left turns in left lane of lane group under consideration,
P_T = proportion of through vehicles in left lane of lane group under consideration,
E_L = through car equivalent for left turns,
f_m = left-turn factor applied only to lane from which left turns are made, and
f_{LT} = left-turn factor for all lanes of the subject lane group.

The model was developed primarily by Carroll J. Messer, and was adopted only after it was clear that no simpler methodology would eliminate the overprediction of left-turn capacity, which results from assuming filtration for the entire green phase.

CAPACITY ANALYSIS MODULE

In the capacity analysis module, simple algebraic manipulations of lane group saturation flow rates and demand flows yield V/C ratios for each lane group and for the intersection as a whole.

Lane group capacities are computed as the product of saturation flow rate and g/C ratio:

$$c_i = s_i \times (g/C)i$$

and V/C ratios for each lane group can be directly computed by dividing lane group demand flow by capacity. For simplicity, intersection V/C ratios are given by the symbol X:

$$X_i = v_i/c_i$$

The module also results in the computation of a critical V/C ratio for the intersection. This is the ratio of the sum of critical flows to the total capacity in critical lane groups accommodating those flows:

$$X_c = \sum_i (v/s)_{ci} \times [C/(C - L)]$$

where

X_c = the critical V/C ratio,
$\sum_i (v/s)_{ci}$ = sum of critical flow ratios (v/s),
C = cycle length in seconds, and
L = lost time per cycle in seconds.

If this critical V/C ratio is under 1.00, then the signal cycle, phase plan, and geometric design are adequate to handle all flows at the intersection. If individual lane groups have V/C ratios greater than 1.00, the green time is inappropriately allocated. If the critical V/C ratio is over 1.00, then a basic change in cycle length, phase plan, or geometry is required to provide adequate capacity.

SUPPLEMENTAL WORKSHEET FOR LEFT-TURN ADJUSTMENT FACTOR, f_{LT}				
INPUT VARIABLES	**EB**	**WB**	**NB**	**SB**
Cycle Length, C (sec)				
Effective Green, g (sec)				
Number of Lanes, N				
Total Approach Flow Rate, v_a (vph)				
Mainline Flow Rate, v_M (vph)				
Left-Turn Flow Rate, v_{LT} (vph)				
Proportion of LT, P_{LT}				
Opposing Lanes, N_o				
Opposing Flow Rate, v_o (vph)				
Prop. of LT in Opp. Vol., P_{LTO}				
COMPUTATIONS	**EB**	**WB**	**NB**	**SB**
$S_{op} = \dfrac{1800\,N_o}{1 + P_{LTO}\left[\dfrac{400 + v_M}{1400 - v_M}\right]}$				
$Y_o = v_o / S_{op}$				
$g_u = (g - CY_o) / (1 - Y_o)$				
$f_s = (875 - 0.625\, v_o) / 1000$				
$P_L = P_{LT}\left[1 + \dfrac{(N-1)\,g}{f_s g_u + 4.5}\right]$				
$g_q = g - g_u$				
$P_T = 1 - P_L$				
$g_f = 2\dfrac{P_T}{P_L}\left[1 - P_T^{\,0.5 g_q}\right]$				
$E_L = 1800 / (1400 - v_o)$				
$f_m = \dfrac{g_f}{g} + \dfrac{g_u}{g}\left[\dfrac{1}{1 + P_L(E_L - 1)}\right] + \dfrac{2}{g}(1 + P_L)$				
$f_{LT} = (f_m + N - 1) / N$				

FIGURE 3 Worksheet and model for permitted left turns *(5)*.

LEVEL OF SERVICE MODULE

In the level of service module, delay for each lane group is estimated and aggregated to obtain approach and intersection delays.

The basic algorithm form was taken from the *Australian Road Capacity Guide*, developed primarily by Akcelik *(6)*. JHK & Associates revised the formula to better fit delay data from U.S. intersections. Carroll Messer introduced a further modification—called the North American Equation—that had a simpler form, yielded similar results to the Australian and JHK equations, and appeared to have the potential to better predict cases in which V/C ratio was marginally higher than 1.00. Messer's equation was adopted for the 1985 HCM as follows:

$$d_1 = 0.38\, C\,[1 - q/C]^2/[1 - (g/C)(X)]$$
$$d_2 = 1.73\, X^2\,\{[X - 1] + [(X - 1)^2 + (16X/c)]^{1/2}\}$$
$$d = (d_1 + d_2)\, PF$$

where

d_1 = first term, or uniform delay in seconds per vehicle;
d_2 = second term, or incremental delay in seconds per vehicle;
d = total delay in seconds per vehicle;
X = V/C ratio for subject lane group;
C = cycle length in seconds;
g/C = green ratio for subject lane group;

TABLE 2 PROGRESSION FACTORS FOR SIGNALIZED INTERSECTION DELAY (5, Table 9-1)

Type of Signal	Lane Group Types	V/C Ratio, X	Arrival Type[a] 1	2	3	4	5
Pretimed	TH, RT	≤ 0.6	1.85	1.35	1.00	0.72	0.53
		0.8	1.50	1.22	1.00	0.82	0.67
		1.0	1.40	1.18	1.00	0.90	0.82
Actuated	TH, RT	≤ 0.6	1.54	1.08	0.85	0.62	0.40
		0.8	1.25	0.98	0.85	0.71	0.50
		1.0	1.16	0.94	0.85	0.78	0.61
Semiactuated	Main street	≤ 0.6	1.85	1.35	1.00	0.72	0.42
	TH, RT[b]	0.8	1.50	1.22	1.00	0.82	0.53
		1.0	1.40	1.18	1.00	0.90	0.65
Semiactuated	Side street	≤ 0.6	1.48	1.18	1.00	0.86	0.70
	TH, RT[b]	0.8	1.20	1.07	1.00	0.98	0.89
		1.0	1.12	1.04	1.00	1.00	1.00
	All LT[c]	All	1.00	1.00	1.00	1.00	1.00

NOTE: TH = through, RT = right turn, and LT = left turn.
[a]See Table 9-2 (5).
[b]Semiactuated signals are typically timed to give all extra green time to the main street. This effect should be taken into account in the allocation of green times.
[c]This category refers to exclusive LT lane groups with protected phasing only. When LTs are included in a lane group encompassing an entire approach, use factor for the overall lane group type. When heavy LTs are intentionally coordinated, apply factors for the appropriate through movement.

c = capacity of subject lane group; and
PF = progression adjustment factor.

Progression adjustment factors are given in Table 2. These were modified from the source document based on suggestions submitted by Donald Berry and others. A basic conceptual disagreement developed in the Highway Capacity and Quality of Flow Committee over whether the impact of progression on delay or capacity diminished as V/C ratios approached 1.00. One group believed that V/C ratio had no impact on the benefits of good progression or detriment of bad progression on delay. Another believed that the impact of progression on delay disappeared as capacity was approached, that is, all values of PF should be 1.00 at capacity.

Extant data did not permit an absolute resolution of the controversy. The factors in Table 2 (5, see Table 9-13) represent a compromise, with the effect of progression on delay diminishing, but not disappearing as capacity is approached. It is useful to note that a project sponsored by NCHRP is currently underway at the Texas Transportation Institute to better calibrate these values.

PLANNING ANALYSIS

The planning analysis procedure was adopted directly from source materials, except for two modifications for handling single-lane approaches. For single-lane approaches, equivalent volumes are used rather than actual volumes. These are based on left-turn equivalents, and are used to approximate the additional friction of a one-lane approach compared to multilane approaches. The second modification considers that left turns in one direction are aided by left turns in the opposing direction which, in effect, create a gap in the opposing vehicle stream.

CLOSING COMMENTARY

The signalized intersection methodology of the 1985 HCM is indeed complex. The computational complexity will be alleviated by the availability of microcomputer software replicating procedures. It is, however, a major advance in the understanding of how intersections operate, and is sensitive to a wide variety of factors that traffic engineers must regularly deal with. Not every question or issue has been fully resolved, and research must clearly continue. Nevertheless, the procedure is a good one that will serve the profession well for years to come.

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