Critical Lane Analysis for Intersection Design

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This paper presents a new critical lane analysis as a guide for designing signalized intersections to serve rush-hour traffic demands. Physical design and signalization alternatives are identified, and methods for evaluation are provided. The procedures used to convert traffic volume data for the design year into equivalent turning movement volumes are described, and all volumes are then converted into equivalent through-automobile volumes. The critical lane analysis technique is applied to the proposed design and signalization plan. The resulting sum of critical lane volumes is then checked against established maximum values for each level of service (A, B, C, D, E) to determine the acceptability of the design. We provide guidelines, a sample problem, and operational performance characteristics to assist the engineer in determining satisfactory design alternatives for an intersection.

To provide an acceptable level of service to drivers operating along an urban arterial, the signalized intersections must keep the traffic moving. The ability of a signalized intersection to move traffic is determined by the physical features of the intersection, by the type of signalization used, and by the geometric design (1). Thus, total system design of a signalized intersection involves concurrent evaluation of the proposed geometric design and signalization as an operational system.

Designing a signalized intersection frequently involves making trade-offs between design variables (and their associated costs) and the resulting level of service. Level of service at an intersection describes the quality of traffic flow afforded motorists on a particular approach to the signalized intersection. The various levels of service may be characterized qualitatively as follows:

Level of Service	Description
A	Light traffic on approach, short stable queues during red
B	Moderate traffic on approach, stable queues, little additional delay
C	Moderately heavy traffic on approach, moderately long but stable queues during red, moderate but acceptable delay
D	Heavy traffic on approach, long unstable queues, sometimes excessive delays
E	Heavy flow (capacity) on approach, long queues, excessive delays
F	Heavily congested traffic conditions, more traffic demand than signal capacity

DESIGN PROCEDURE FOR SIGNALIZED INTERSECTIONS

The critical lane analysis technique is used in this procedure to determine if a proposed design will provide an acceptable level of service. Level C is the minimum during the peak 15-min period of the design hour; however, all operating conditions can be evaluated.

Basic design variables include the number of approach lanes provided, the possible length and use of left and right turn lanes, the combination of traffic movements using the lanes provided, and the type of signal phasing. Minimum design standards for the basic design variables of lane width [3.0 to 3.7 m (10 to 12 ft)] and curb return radii [4.6 to 9.1 m (15 to 30 ft)] will normally provide satisfactory operation during rush hours.

Volume Data Preparation

We will use a sample problem to illustrate the application of the procedure over a range of initially given volume data conditions. In practice, the given traffic data would dictate the appropriate step in the volume preparation procedure at which the designer should begin the analysis.

Step 1. Average Daily Traffic

In our sample problem we assumed that the given volume data are the forecast, design year, average daily traffic (ADT) volumes. These two-way ADT volumes are shown in step 1 of Figure 1. Our given traffic and operational conditions are as follows: Volumes are 1985 ADT; design hour factor (K) is 10 percent; directional distribution (D) is 67 percent; trucks and through buses (T) is 5 percent; and population for 1985 is 400 000.

Step 2. Design Hour Movement Volumes

We first converted the two-way ADT volumes into approach movement volumes for the design hour being analyzed. The morning peak hour is assumed in this example. The evening design hour could also be checked, because, if the given volumes are in ADT, the morning design hour volumes for left turns on one approach become the right turns on the departure leg during the evening peak hour.

The morning design hour volumes are shown in step 2 of Figure 1. The directional peak flows move from left to right and from bottom to top. Other factors being equal, the location and the orientation of the intersection in the metropolitan area dictate the peak directions of flow. The larger of the two design hour, directional, movement volumes flowing between legs a and b is calculated from

$$DHV_{ab} = ADT_{ab} \cdot K \cdot \bar{D} \quad (1)$$

where DHV_{ab} is the design hour, peak direction movement volume between legs a and b, ADT_{ab} is the average daily traffic interchanging between legs a and b (step 1, Figure 1), and $\bar{D}$ is the average directional distribution split (decimal equivalent) between the approaches. $\bar{D}$ is either 0.67 or 0.50. The off-peak direction movement volume between legs a and b is calculated from

$$DHV_{ab} = ADT_{ab} \cdot K \cdot (1.00 - \bar{D}) \quad (2)$$

Step 3. Design Period Volumes

We used the peak 15-min period of the design hour to evaluate the level of service. The traffic volume flow rates during this period consistently exceed the average for the design hour by approximately 20 to 30 percent. These peaking factors have been found to vary with the population of the city (1, 2) as follows. According to the Highway Capacity Manual (1) and others (2), these peaking factors have been found to vary with the population of the specific city in the following manner: For populations under 100 000 the peaking fac-

Figure 1. Steps in volume data preparation.

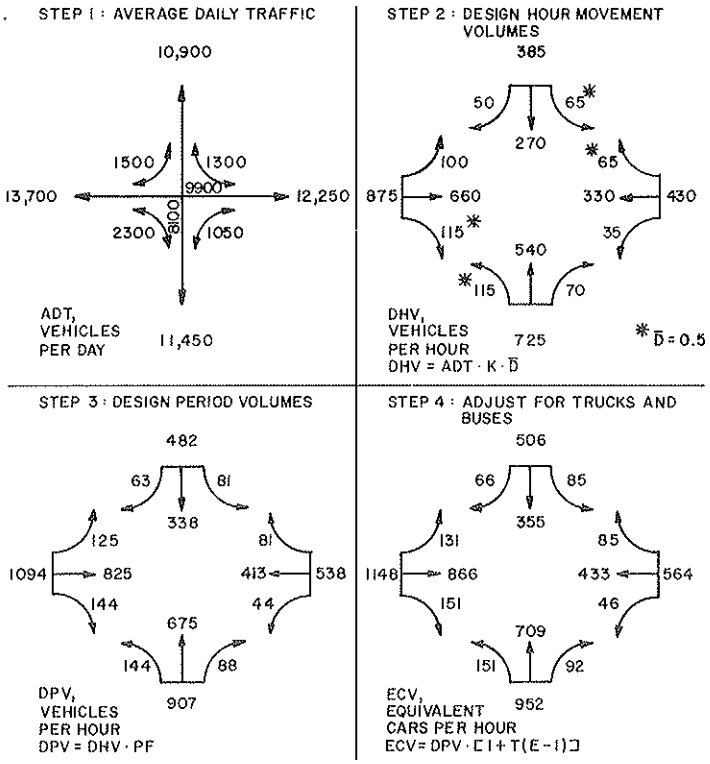
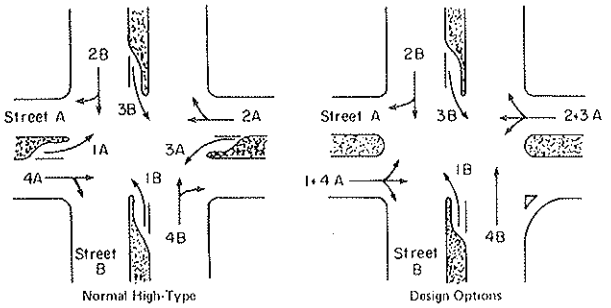


Figure 2. Definition of movements.



tor is 1.35; for those 100 000 to 300 000 it is 1.30; for those 300 000 to 500 000 it is 1.25; and for over 500 000 it is 1.18.

Design period flow rates for each movement are calculated from

$$DPV_{ab} = DHV_{ab} \cdot PF \quad (3)$$

where DPV_{ab} is the design period volume from leg a to b as given in step 3 of Figure 1; DHV_{ab} is the design hour volume from step 2 of Figure 1; and PF is the peaking factor given above, in this case 1.25 for a population of 400 000.

Step 4. Equivalent Passenger Automobile Volume

This design procedure converts all design period volumes of mixed traffic (5 percent trucks and through buses in this example) into an equivalent number of automobiles. One truck or through bus is equal to two automobiles (2). Thus, the equivalent automobile volumes (ECV) in step 4 of Figure 1 are calculated from the design period mixed traffic flow rates of step 3, by

$$ECV_{ab} = DPV_{ab}[1.0 + T(E_T - 1)] \quad (4)$$

where E_T is the automobile equivalent for trucks and through buses. Since E_T is assumed to be 2.0,

$$ECV_{ab} = DPV_{ab}(1.0 + T) \quad (5)$$

Geometric Movement Volumes

The next step in the design guide requires that the individual ECV turning movement volumes (step 4 of Figure 1 and Equation 5) be defined by the way they will be combined in the geometric design of the intersection. Eight basic geometric movement volumes would exist at an urban arterial, four-legged intersection having left turn bays on all approaches, as depicted on the left intersection of Figure 2. When a left turn bay or a separate left turn lane is provided on an approach, the left turn geometric movement volume is the same as its corresponding turning movement volume in ECV from Equation 5 ($GMV_{1A} = ECV_{lt}$). The adjacent through-right geometric movement volume would be calculated as $GMV_{4A} = ECV_{th} + ECV_{rt}$. If a channelized or free right turn lane is provided, we can dispense with the ECV right turning volume (ECV_{rt}).

When an approach does not have a left turn lane, the left turning movement volume (ECV_{lt}) is added to the appropriate through-right movement volume forming a combined left-through-right geometric movement volume [$GMV_{(1+4)A} = ECV_{lt} + ECV_{th} + ECV_{rt}$] for the eastbound approach of the right intersection shown in Figure 2.

Geometric Design Volumes

All of the geometric movement volumes are further adjusted to account for the (proposed) design and operational features of the intersection. These equivalent-effect volumes are calculated from

$$GDV_m = U \cdot W \cdot TF \cdot GMV_m \quad (6)$$

where GDV_n is the geometric design volume for movement (m) of Figure 2, vehicles per hour; U is the lane utilization factor (3); W is the lane width factor; TF is the turning movement factor (Equation 7); and GMV_n is the geometric movement volume of movement (m). The sum of one or more turning movement volumes (ECVs from Equation 5) is illustrated in Figure 2.

Lane Utilization

As the number of lanes serving a movement increases, there is an increasing tendency for one lane to be used more than the others, which is accounted for by the lane utilization adjustment factor (U). These factors are given below.

Number of Lanes	Factor
1	1.0
2	1.1
≥3	1.2

Lane Width

Lanes 3 m (10 ft) or more in width have little influence on rush-hour traffic flow rates as reflected by the lane width adjustment factor below (1 m = 3.3 ft). The width of a lane does not include any pavement used for or appreciably affected by parking. Through lanes less than 3.4 m (11 ft) wide may experience related safety problems.

Width (m)	Factor
2.7 to 3.0	1.1
≥3.0	1.0

Turns

The effects of turning vehicles on flow are given by the turning factor as

$$TF = 1.0 + L + R \quad (7)$$

where L, which adjusts for the effects of left turns and R for right turns, is L_1, L_2, or L_3 and is described as follows.

For an approach having no left turn bay, the left turn adjustment factor to be applied to a combined left-through-right movement volume is calculated from

$$L_1 = P_L(E - 1.0) \quad (8)$$

where P_L is the decimal fraction of the total approach volume turning left and E is the appropriate left turn equivalent factor from Table 1 (1).

For an approach having a left turn bay, the adjustment factor is

$$L_2 = (1700 \cdot E)/S - 1.0 \quad (9)$$

where S is the saturation flow of the left turn bay obtained from Figure 3 for a given storage length and equivalent left turning ECV from step 4 of Figure 1. The left turn equivalent factor (E) is obtained from Table 1.

The desired minimum left turn bay storage length, which does not include either the taper section or any length of the bay beyond the usual stop line, for a given equivalent turning volume is presented at the top of Figure 3 (see the other paper by Messer and Fambro in this Record). Shorter bay storage length results in saturation flow rates less than 1700. For normal urban street conditions, a taper length of 21.3 to 30.5 m (70 to 100 ft) may be considered appropriate; for higher types of urban facilities and rural highways, it should be 45.7 to 91.4 m (150 to 300 ft).

When a left turn bay is provided, any blockage effects that left turns may cause the through-right movement are calculated from

$$L_3 = (1700 - S)/[1700(N - 1) + S] \quad (10)$$

where N is the number of lanes serving the adjacent through-right movement.

When a separate right turn lane is provided, neither right turning volume nor right turn lane is analyzed. In other cases, the analysis of right turns depends on accuracy requirements. From the viewpoint of practicality and simplicity, the adjustment factor (R) for most designs can be set at zero (R equals zero in Equation 7 if right turns on red will be permitted). If a detailed analysis is desired, the following approach may be used to estimate the effects of right turning vehicles (4):

$$R = (5 \cdot P_R)/c \quad (11)$$

where P_R is the decimal fraction of movement combination turning right and c is the related curb return radius (m). In addition, the estimated number of vehicles turning right on red is subtracted from the through-right geometric movement volume combination (GMV_n). This estimate, which should not exceed 0.5 of the right turning volume, is calculated from

$$ROR = 50[P_e/(1 - P_e)] < 100 \quad (12)$$

where ROR is the estimated right-on-red volume (vehicles/h), and P_e is the estimated decimal fraction of traffic in the curb lane turning right.

Capacity

It is assumed throughout this procedure that the capacity of a normal protected through lane is 1750 automobiles/green·h (5). This is equivalent to a minimum average headway of 2.06 s/automobile. In addition, the normal protected left turn capacity is 1700 automobiles/green·h (5). The type of signalization also affects the capacity of the intersection and will be reflected in the critical lane analysis technique to follow.

Critical Lane Volumes for Each Street

To begin the analysis of a design's acceptability, the geometric design volume for each movement, calculated in Equation 6, is divided by the number of usable lanes provided to serve the movement to obtain a design volume per lane (V_n) of

$$V_m = GDV_m/N \quad (13)$$

If there is a left turn bay on an approach, then N equals 1 for the separated left turning movement, and N equals 1, 2, or 3 for the through-right movement, depending on the number of through lanes provided in the design. If no left turn lane is provided, then N would be the total number of approach lanes, since GDV_n is the total approach movement volume.

Figure 2 defined the movements at a typical intersection to be considered in the critical lane analysis technique. For each street, these movements may be combined in different patterns according to the type of signalization used, as shown here.

Signal Phasing	Maximum Lane Movements
One phase with no left turn bay	1 + 4 or 2 + 3
One phase with left turn bays	1, 2, 3, or 4
Two phases (no overlap) with bays	
Dual lefts leading	1 or 3 + 2 or 4
Dual lefts lagging	2 or 4 + 1 or 3
Leading left	1 or 4 + 2 or 3
Lagging left	2 or 3 + 1 or 4
Three phases (overlap) with bays	
Dual lefts leading	1 + 2 or 3 + 4
Dual lefts lagging	1 + 2 or 3 + 4
Leading left	1 + 2 or 3 + 4
Lagging left	1 + 2 or 3 + 4

For any given type of signalization on a street, one movement or a sum of two movements will be larger (critical). Different types of signal phasings will usually result in different critical lane volumes.

Level of Service Evaluation for Intersection

To evaluate the acceptability of the design, it is necessary to total the critical lane volumes (ΣV) for the intersecting streets.

$$\Sigma V = \Sigma V_{max}(\text{street A}) + \Sigma V_{max}(\text{street B}) \quad (14)$$

This sum is then compared with maximum values established for a given level of service as presented in Table 2. Here, level C is recommended for the design of urban signalized intersections. The maximum service volumes vary slightly depending on the signalization used: Two-phase signalization has no protected left turning on either street; three-phase has protected turning on only one of the two streets; multiphase signalization has protected left turning on both streets. A detailed description of the selection of total critical lane volumes relative to level of service criteria will be presented later.

Design Sample Problem

We shall now present a design alternative for the projected traffic. Some assumptions made in the problem were selected to illustrate computational procedures of the critical lane analysis design technique rather than optimum design practice.

Our purpose was to determine the acceptability of the proposed design, given the following: Traffic volume data are presented in equivalent automobiles/h for the intersection (step 4 of Figure 1); all lanes are 3.7 m (12 ft) long; all left turn storage lengths are 30.5 m (100 ft) except for 1B [66 m (150 ft)]; all right turn effects are ignored (R = 0.0); and both A and B streets have three phases with dual lefts leading (Figure 4).

These are the left turn adjustment factors:

$L_{1A} = [(1700 \times 1.03)/1640] - 1.0 = 0.07$

$L_{1B} = [(1700 \times 1.03)/1700] - 1.0 = 0.03$

$L_{3A} = [(1700 \times 1.03)/1700] - 1.0 = 0.03$

$L_{3B} = [(1700 \times 1.03)/1700] - 1.0 = 0.03$

$L_{4A} = \{(1700 - 1640)/[(1700 \times 2) + 1640]\} = 0.012$

Now, given the volumes adjusted according to Equation 6,

$GDV_{1A} = 1.0 \times 1.0 \times 1.07 \times 131 = 140$

$GDV_{2A} = 1.2 \times 1.0 \times 1.0 \times 518 = 622$

$GDV_{3A} = 1.0 \times 1.0 \times 1.03 \times 46 = 47$

$GDV_{4A} = 1.2 \times 1.0 \times 1.012 \times 1017 = 1235$

$GDV_{1B} = 1.0 \times 1.0 \times 1.03 \times 151 = 156$

$GDV_{2B} = 1.1 \times 1.0 \times 1.0 \times 421 = 463$

$GDV_{3B} = 1.0 \times 1.0 \times 1.03 \times 85 = 88$

$GDV_{4B} = 1.1 \times 1.0 \times 1.0 \times 709 = 780$

and the design volume per lane for each movement found by Equation 13,

$V_{1A} = 140$ $V_{1B} = 156$

$V_{2A} = 207$ $V_{2B} = 232$

$V_{3A} = 47$ $V_{3B} = 88$

$V_{4A} = 412$ $V_{4B} = 390$

The sum of the critical lane volumes (Figure 5) for street A will be

$$\left.\begin{matrix} V_{3A} = 47 \\ V_{4A} = 412 \end{matrix}\right\} (47 + 412 > 140 + 207)$$

and those for street B will be

$$\left.\begin{matrix} V_{3B} = 88 \\ V_{4B} = 390 \end{matrix}\right\} (88 + 390 > 156 + 232)$$

for the critical lane volume

$\Sigma V = 937 < 1100$

Therefore, this is an acceptable design for a service level B.

CRITICAL LANE VOLUME DESIGN CRITERIA

This section describes the theory of the critical lane analysis technique and operational measures used as a basis for selecting the design criteria of Table 2.

Effective Green

The effective green time is defined as that portion of the signal phase when saturation (capacity) flow occurs. Its duration is

$$g = G + Y - L \quad (15)$$

where g is effective green time (s); G is actual green (s); Y is yellow clearance (s); and L is lost time (4 s). Saturation flow conditions are assumed to begin in about 2 s after the start of green and to end about 2 s before the yellow clearance time expires. The total lost time per phase is 4 s in this paper. The effective green and actual green times are about the same if the yellow clearance interval is established according to basic intersection approach speed and width criteria.

Saturation Ratio

The saturation ratio of the signal phase (X) serving a movement could more descriptively be called the volume-to-capacity ratio, since

$$X = \text{volume/capacity} = Q/[(g/C) \cdot S] = (Q \cdot C)/(g \cdot S) \quad (16)$$

Table 1. Left turning equivalent for opposing ECV.

Intersection and Signal Phasing	Traffic Movement	Number of Opposing Through Lanes	Factor by Opposing ECV[a]				
			200	400	600	800	1000
No protected turn phase							
Two-phase							
No bay on approach	Left and through	1	2.0	3.3	6.5	16.0[b]	16.0[b]
	Left and through	2	1.9	2.6	3.6	6.0	16.0[b]
	Left and through	3	1.8	2.5	3.4	4.5	6.0
With bay on approach	Left	1	1.7	2.6	4.7	10.4[b]	10.4[b]
	Left	2	1.6	2.2	2.9	4.1	6.2
	Left	3	1.6	2.1	2.8	3.6	4.8
Three-phase							
No bay on approach	Left and through	1	2.2	4.5	11.0[b]	11.0[b]	11.0[b]
	Left and through	2	2.0	3.1	4.7	11.0[b]	11.0[b]
	Left and through	3	2.0	2.9	4.2	6.0	11.0[b]
With bay on approach	Left	1	1.8	3.3	8.2[b]	8.2[b]	8.2[b]
	Left	2	1.7	2.4	3.6	5.9	8.2[b]
	Left	3	1.7	2.4	3.3	4.6	6.8
Protected turning							
No bay	Left	Any	1.2		1.2		1.2
With bay	Left	Any	1.03		1.03		1.03

[a] In automobiles per hour. Includes total through volume (from step 4, Figure 1) on the approach opposing the left turn being analyzed. The opposing volume also includes any turning volume (left or right or both) for which no separate turning lane is provided.
[b] Turning capacity only at end of phase. Not recommended for design. Add additional through lane, turning lane, or protected left turn phasing.

Figure 3. Saturation flow of left turn phase as a function of bay storage length and turning volume.

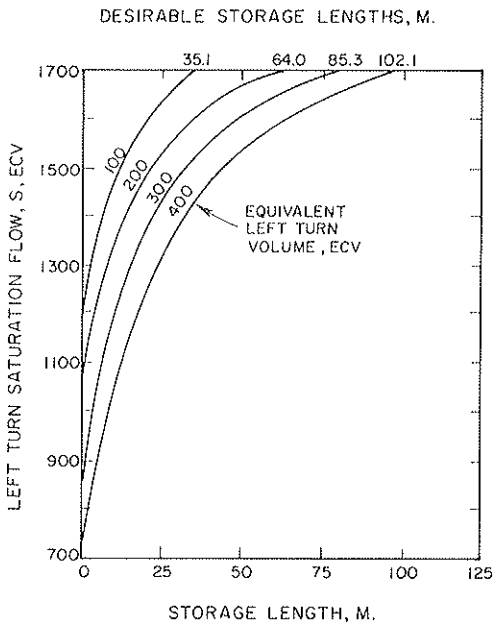


Figure 4. Volume data and proposed design of example.

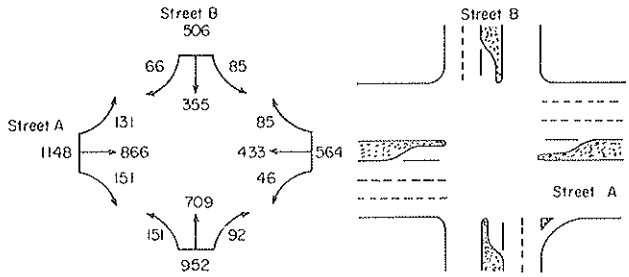


Figure 5. Design lane volumes for streets A and B.

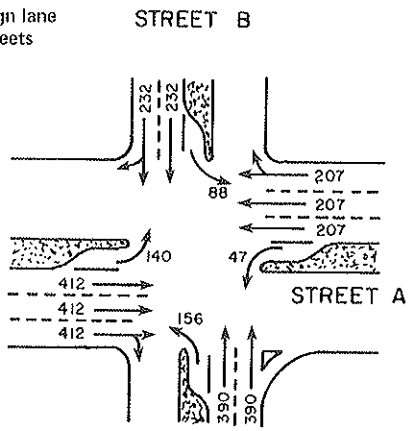


Table 2. Level of service and maximum sum of critical lane volumes at signalized intersections.

Level of Service	Traffic Flow Condition	Volume to Capacity Ratio	Critical Lane Volumes		
			Two-Phase	Three-Phase	Multiphase
A	Stable	≤0.6	900	855	825
B	Stable	≤0.7	1050	1000	965
C	Stable	≤0.8	1200	1140	1100
D	Unstable	≤0.85[a]	1275	1200	1175
E	Capacity	≤1.0	1500	1425	1375

[a] Modified (1).

where Q is approach movement volume (vehicles/h); C is cycle length (s); and S is saturation flow of approach (vehicles/green·h).

The saturation ratio is a good tool for describing quantitatively what traffic operating conditions will be like. When X > 0.85, vehicle delays on the approach become very long, and queues frequently fail to clear the approach at the end of the green phase.

Critical Lane Development

A normal, four-legged intersection having multiphased signalization will have four critical volume phases, as was shown in Figure 2. The sum of these four critical phases, including green plus yellow times, is one cycle. The equation for this requirement is

$$C = \phi_{A1} + \phi_{A2} + \phi_{B1} + \phi_{B2} \quad (17)$$

or

$$C = (G + Y)_{A1} + (G + Y)_{A2} + (G + Y)_{B1} + (G + Y)_{B2} \quad (18)$$

where ϕ_{A1} is the first critical phase on street A (s).

Substituting the equivalent effective green (g) and lost time (L) from Equation 15 into Equation 18 for each critical phase and rearranging terms yield

$$g_{A1} + g_{A2} + g_{B1} + g_{B2} = C - nL \quad (19)$$

where n is the number of critical phases per cycle. Solving for g in the saturation ratio formula (Equation 16), substituting in Equation 19, and dividing by C results in

$$(C - nL)/C = [Q/(X \cdot S_{A1})] + [Q/(X \cdot S_{A2})] + [Q/(X \cdot S_{B1})] + [Q/(X \cdot S_{B2})] \quad (20)$$

If the saturation ratios are selected to be the same for all critical phases, then

$$(C - nL)/C = (1/X)[(Q/S_{A1}) + (Q/S_{A2}) + (Q/S_{B1}) + (Q/S_{B2})] \quad (21)$$

or

$$[X \cdot (C - nL)/C] = Q/S_{A1} + Q/S_{A2} + Q/S_{B1} + Q/S_{B2} \quad (22)$$

Since the ratios Q to S are the same on a per lane basis as for the total approach movement and if Q/N = V and S/N = 1750 according to the critical lane analysis procedure, then

$$X \cdot [(C - nL)/C] = (V_{A1}/1750) + (V_{A2}/1750) + (V_{B1}/1750) + (V_{B2}/1750) \quad (23)$$

where V_{A1} is the critical lane volume on phase A_1. Thus,

$$V_{A1} + V_{A2} + V_{B1} + V_{B2} = 1750 \cdot X \cdot [(C - nL)/C] \quad (24)$$

It follows that the sum of the critical lane volumes (ΣV) for a given level of service saturation ratio ($X_{L.O.S.}$) and design cycle length (C) is

$$\Sigma V = 1750 \cdot X_{L.O.S.} \cdot [(C - nL)/C] \quad (25)$$

The results of Equation 25 are plotted in Figure 6 for two X ratios: 1.0 (capacity) and 0.8. Also shown in Figure 6 are desirable design hour cycle lengths: 55 s for two-phase, 65 s for three-phase, and 75 s for four-phase. These lengths were used to determine the capacity values and other sums of critical lane service volumes in Table 2.

Minimum Delay Cycle Length

The average vehicle delay (d) in seconds per vehicle experienced by an approach movement having Poisson arrivals can be calculated by using Webster's rather lengthy formula:

$$d = \{[C(1 - g/C)^2]/[2(1 - Q/S)]\} + \{(1800 \cdot X^2)/[Q(1 - X)]\} - 0.65\,[C/(Q/3600)^2]^{1/3}\, X^{2+5(g/C)} \quad (26)$$

To illustrate average delay conditions at an intersection, assume that a four-phase signalized intersection has equal approach volumes and green times. The four critical lane volumes are also equal, but their magnitude can change. Figure 7 shows how the average delay varies with cycle length for each of the volume conditions, as given by the sum of the critical lane volumes. The sums of the critical lane volumes of 825, 1100, and 1175 correspond to levels of service A, C, and D respectively in Table 2 for multiphase (four) signals. The minimum delay cycle lengths are 55, 78, and 88 s for the sums of critical lane volumes of 825 (A), 1100 (C), and 1175 (D) respectively. Multiphase (four) intersections designed to level C could operate at almost any normal range of peak-hour cycle lengths (60 to 90 s). However, intersections having a sum of critical lane volumes of 1175 (D) will be heavily congested at cycle lengths less than 70 s.

Figure 8 summarizes the relationships between the design variables and operational measures of (a) sum of critical lane valumes, (b) saturation ratio, (c) minimum delay cycle length, (d) type of signal phasing, and (e) level C design criteria. The latter result in the minimum delay cycle lengths desirable for operating coordinated arterial signal systems during the peak hours.

Other Operational Variables

There are several other traffic operational measures besides delay that are descriptive of the traffic conditions existing on a movement at a signalized intersection. Load factor was used in the Highway Capacity Manual (2) to define level of service; traffic engineers have also used Poisson's probability of cycle failure (6); and Miller recently extended the probability of queue failure concept to include the effects of queue spillover from one cycle to the next as queued vehicles fail to clear during the green (7). A summary of these new models developed by Miller follows.

$$\text{Probability of queue failure} = e^{1.58\phi} \quad (27)$$

$$\text{Load factor} = e^{1.3\phi} \quad (28)$$

$$\text{Probability of queue clearing} = 1 - e^{1.58\phi} \quad (29)$$

where $\phi = [(1 - X)/X]\,[(S \cdot g)/3600]^{1/2}$.

Figure 9 presents average approach values at an intersection having four critical phases for these operational measures together with Poisson's probability of failure and saturation ratio as a function of minimum delay cycle length. When 75 s is the minimum delay cycle length, the saturation ratio (X) is 0.78; the load factor (LF) is 0.36; Miller's probability of queue failure (P_f) is 0.30; and Poisson's probability of failure is 0.18 (18 percent).

Figure 10 illustrates operating characteristics on a single approach movement as volume conditions increase. The effective green time and cycle length were held constant at 35 and 70 s respectively. Few cycles fail to clear stopped queues at saturation ratios less than 0.7. Acceptable queue failure rates still exist at 0.8, but larger values result in unstable operation and increasingly long delays.

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Figure 6. Variation in sum of critical lane volumes with cycle length as a function of saturation ratio.

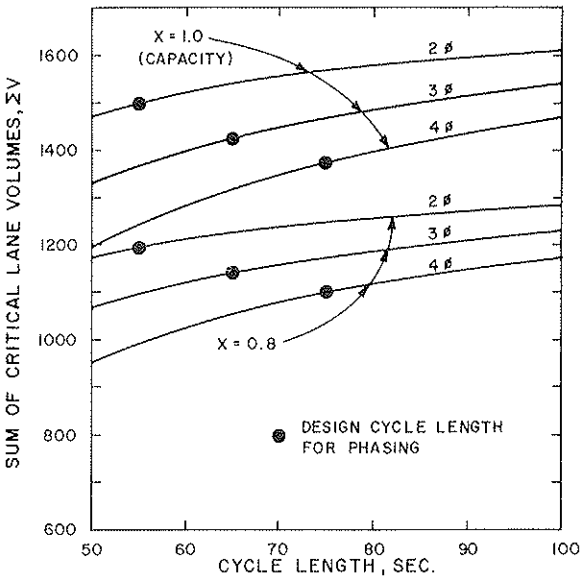


Figure 7. Delay versus cycle length as a function of sum of critical lane volumes.

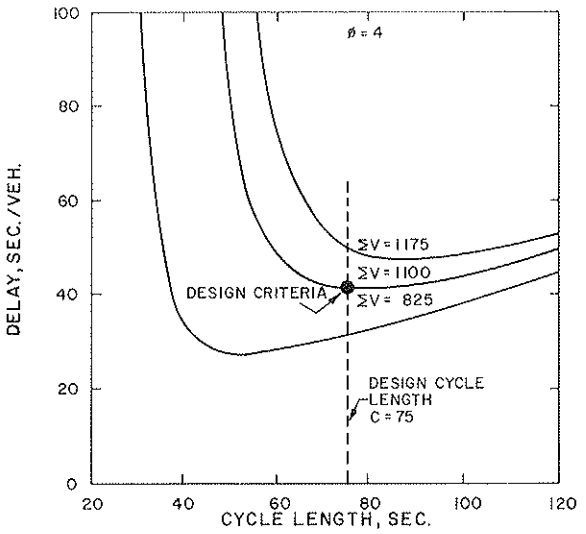


Figure 8. Relationship between design variables and operational measures.

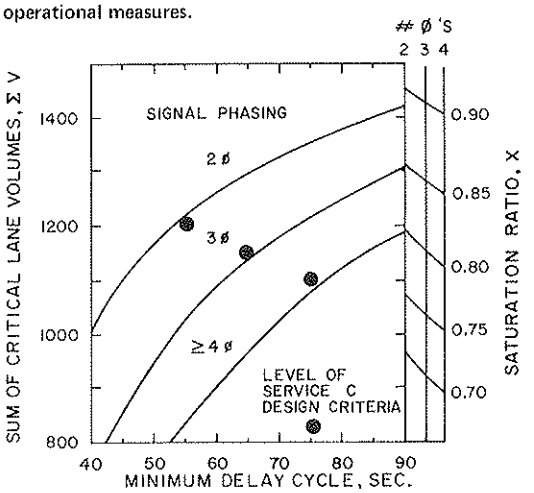


Figure 9. Relationship between operational measures of effectiveness and minimum delay cycle length.

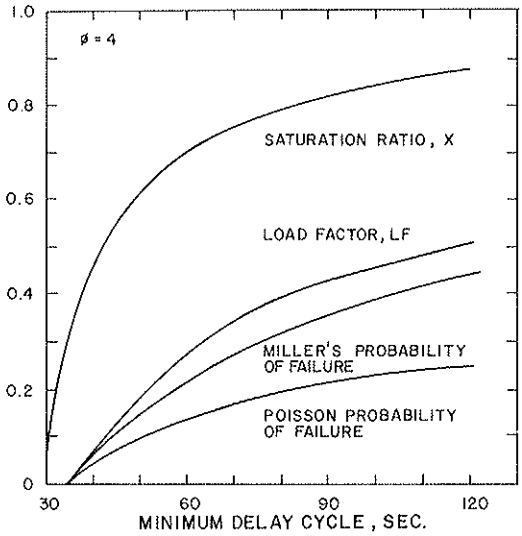
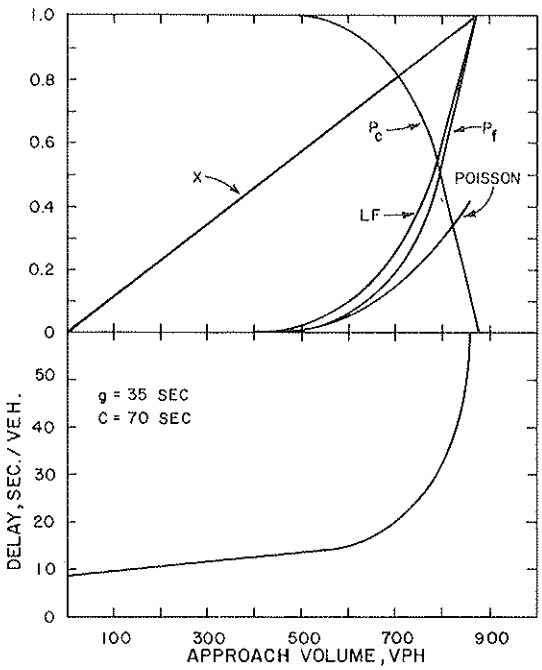


Figure 10. Relationship between operational measures of effectiveness and volume.



or policies of the Federal Highway Administration. We alone are responsible for the facts and accuracy of the data presented. This paper does not consitute a standard, specification, or regulation.

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Discussion

Stephen G. Petersen, Gaithersburg, Maryland

In the federal government, the phrase "adequate public facilities" is becoming commonplace. This phrase, when directed at builders and developers, tells them that their projects can only go forward if all the public facilities required to serve a development are in place and capable of meeting the needs of a specific project. Many facilities are included: fire protection, police protection, schools (if a residential project), water, sewer, and finally transportation but more specifically roads. The test of adequacy is generally easily met except for two facilities, sewers and roads. Usually both are impacted; but if sewers are not, then roads are almost certain to be a problem for any development lying in the developed urban area.

As a test of road adequacy, the two Maryland suburban counties of Washington, D.C., have generally adopted a technique known as critical movement analysis or critical lane analysis that was based on a paper of mine (8). The technique was selected as a planning tool because it is relatively simple to apply, avoids the correction factors common to the Highway Capacity Manual (HCM), and derives an answer relevant to the whole intersection in one step rather than as part of a multistep process.

A comparison of the techniques described in the authors' paper and in mine reveal the same basic elements. The authors, however, describe a more sophisticated set of correction factors for a more accurate result. It is encouraging to see this kind of effort, because, if we are going to place an expensive development at the mercy of one or two intersections near it, then we had better be sure that our techniques are reasonable representations of real-life conditions.

I had several questions and comments, which are listed below.

1. The source cited for volume adjustment for the lane utilization factor is work done in New Jersey in the early 1960s. However, it is understood that these values were confirmed by field data collected in a study of unprotected left turns at opposing flows of up to 1000 vehicles/h. What happens beyond this volume? Does lane utilization become essentially even at lower levels of service (higher approach volumes)? If so, does this factor need to become a variable? This is a question for the local area because a difference shows up between the data published in the paper and the technique as applied and can make a difference as great as one level of service.
2. Table 1 deals with opposing volumes of only 1000 vehicles/h. Presumably left turns in opposition to volumes in excess of this can only be accommodated by protected turning. However, this is not stated, and for opposing traffic in multiple lanes the factors have not reached turning capacity. Is the number of approach lanes independent of the opposing volume? Also, it is not clear which phasing is in effect when the term "three-phase" signal is used under the No Protected Turn Phase.
3. I would like to see a level of service comparison between this technique and that used in the HCM.
4. I attempted to work through this method with a more heavily loaded, less sophisticated intersection and found that one must be extremely careful to follow the phasing rules for critical lane volumes explicitly for each street or the result of the calculations will be totally erroneous. This need to carefully select signal phasing and to apply factors detracts from the use of the technique as a planning tool. I recognize that the intent of the research was for design purposes, but I hope it can be adapted with some simplification as a planning tool. In planning, the critical lane technique is a valuable communication tool among various professionals, but its effectiveness is lost when correction factors and selection of different phasing arrangements transform too many steps into variables.
5. The authors state, but I emphasize again, that the critical lane values in Table 2 are the result of deliberately selecting cycle lengths that produce less than maximum capacity even though Webster says they minimize delay. The effects of this show up when one tries to compare the level of service determined by field examination with that predicted by the method. In an example in which such a comparison was possible, the technique showed level of service E, whereas the description of the actual condition was closer to level D. This suggests that there is a need either to more carefully relate the description of level of service to the results of the analysis or, more likely, to quantify level of service to some measure such as delay in order to relate critical lane totals to specific measures of total intersection delay.
6. Also, in reviewing the design volume discussion, I notice the use of the peak 15 min for design purposes. It is possible that this factor has distorted comparisons between field observations that deal with a total hour of flow and critical lane totals that are keyed to a peak 15-min period.

In summary, I believe the authors have assimilated a considerable amount of useful information toward the development of improved intersection design techniques that deal with the whole intersection and the interaction within it. The technique now needs to be expanded to account for maximum practical cycle lengths, for other intersection configurations, and for situations controlled by STOP signs and not likely to be signalized. Particular emphasis needs to be place on these aspects in planning situations such as that described at the beginning of these comments.

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Frederick A. Wagner, Alan M. Voorhees and Associates, Inc.

The authors are to be commended for their excellect paper. It synthesizes a substantial range of knowledge on intersection capacity and level of service in a systematic manner. It will guide the traffic engineer through a step-by-step procedure for determining critical lane volumes and corresponding levels of service at an intersection, given the intersection design, traffic control, and traffic flow characteristics. Critical lane volume is inherently more appealing and more fundamentally valid than the established intersection capacity analysis techniques set forth in the Highway Capacity Manual.

This discussion addresses several points: (a) potential refinements in certain detailed aspects of the technique, (b) the need to develop a way to best use the technique for existing operational intersections (in addition to design intersections), (c) the possible invalidity of equations based on lost time and cycle length relationships, and (d) the need for systematic empirical validation of this technique and alternative methods of intersection capacity and level of service analysis.

POTENTIAL REFINEMENTS

One of the appealing attributes of the critical lane analysis technique is its relative simplicity. Certain aspects of the technique, however, border on oversimplification and result in the risk of losing precision. I shall give a few examples (not exhaustive) in which refinements might make the analysis technique more precise without significantly increasing its complexity.

Lane Width Adjustment

Messer and Fambro consider only two classes of lane width: < 3 and ≥ 3 m (10 ft). A volume adjustment factor of 1.10 is applied if lane width is less than 3 m (10 ft). This is perhaps their most serious oversimplification. All other major (American, Australian, and British) capacity analysis techniques utilize a more refined adjustment for lane width (or total approach width). The Australian factors for adjusting saturation flow are given below (1 m = 3.3 ft).

Lane Width (m)	Adjustment (%)	Lane Width (m)	Adjustment (%)
2.4	−12	4.0	+3
2.7	−7	4.3	+4.5
3.0 to 3.7	0	4.6	+6

The reciprocals of these factors would be used to adjust volumes. However, one unresolved debate concerns the use of number of lanes or the use of approach width. Based on a questionnaire survey in 1974 by the TRB Committee on Highway Capacity, approach width versus number of lanes was the number one priority research problem.

Right Turn Adjustment (Pedestrian and Parking)

The Messer-Fambro technique computes a right turn adjustment as a function of only two variables: fraction of geometric movement volume turning right and curb return radius. Two other factors seem to be extremely important. First, pedestrians on the intersection crosswalks create a major interference to right turning vehicles and in downtown areas especially can make right turns as difficult as left turns. Second, parking conditions on the approach will also significantly influence the effect of right turns. The authors do not consider parking except to say that "the width of a lane may not include any pavement used for or appreciably affected by parking." Parking prohibited for a short distance back from the stop lane provides extra approach space for storage and movement of right turn vehicles.

The British developed a method for determining effective loss of approach width as a function of clear distance from the stop line to the nearest parked automobile and green time. An analogous approach might be incorporated into the critical lane analysis technique.

Grade

Although the great majority of intersection approaches are relatively flat, those on even moderate grades have significantly altered capacity characteristics. A simple volume adjustment factor could be incorporated for positive and negative grades. Webster found that for each 1 percent of upgrade or downgrade saturation flow rate decreases or increases by 3 percent.

Cycle Length and Left Turns

On intersection approaches with heavy opposing flow, unprotected left turns are made difficult by few gaps. Most left turns are made at the end of the green phase (during yellow). Messer and Fambro give left turn equivalency factors that cover such situations, but they do not consider how drastically cycle length affects left turn capacity. For example, if virtually all left turns are made during yellow intervals, left turn capacity will be 50 percent higher with a 60-s cycle (60 yellows/h) than with a 90-s cycle (40 yellows/h). This fact should be reflected in the technique, especially considering the propensity among traffic engineers to use only long signal cycles. (More on this later.)

APPLICATION AT EXISTING INTERSECTIONS

The authors indicate that their paper is intended as a guide for designing signalized intersections. The techniques could also be adapted to evaluating capacity and level of service at existing intersections. After all, most traffic engineers devote far more time to the evaluation of operation changes at existing intersections than to the physical redesign of intersections or the design of entirely new ones.

We should emphasize the procedures that directly measure capacity flow rates associated with each geometric movement volume rather than rely on estimates based on generalized adjustment factors. Most intersections have at least a few measurable loaded cycles during peak periods. Direct measurement of a relatively small number of loaded cycles may provide a more precise estimate of intersection capacity than the various adjustment factors do. There is an opportunity here to develop a unified approach that incorporates both direct measurement, where possible, and utilization of generalized volume adjustment factors only where necessary. For example, a redesign of a problem intersection may only involve changing one or two intersection approaches. An analysis of the redesign could be based on direct empirical data for the unchanged approaches and could utilize the generalized adjustment factor technique only for those approaches having substantially changed designs.

POSSIBLE INVALIDITY OF BASIC EQUATIONS

The various equations set forth in the authors' paper for determining level of service, average delay, minimum delay cycle length, probability of queue failure, load factor, and probability of queue clearing, all evolve from Webster's original work. He divided the green-plus-yellow phase for a given intersection approach into effective green time (when traffic discharges at a constant saturation flow rate) and lost time (when zero flow occurs).

The authors' Equation 25 gives the sum of critical lane volumes (ΣV) as a function of basic saturation flow rate (1750 vehicles/h), volume to capacity ratio ($X_{L.O.S.}$), cycle length (C), lost time (L), and number of phases (n). The factor nL/C is a crucial one. It represents the fraction that total lost time on all phases is of cycle length. Implicitly, Equation 25 and others using nL/C show that conditions will improve with longer cycle length, because lost time will then represent a smaller fraction of total time.

The equations are theoretically correct, but are they valid empirically? In 1974 Moskowitz argued eloquently:

> Green plus yellow is a better predictor of volume during saturated phases than green alone (or green plus yellow minus lost).
>
> Observations at intersection approaches where cycle length was modified do not show that higher volumes can be served if longer cycle lengths are used.
>
> If the cycle length is long enough to provide optimum splits, given minimum green constraints, short cycles almost always result in better service (loss delay) than long cycles.
>
> The habit of using long cycles causes more unnecessary delay to Americans than any other single fallacy in the traffic engineering profession.
>
> Green plus yellow (with no correction for lost time) should be used in measuring saturation flow and in computing capacity values. This would cause traffic engineers to re-think the fallacious habit of using long cycles in the attempt to increase throughput.

Moskowitz explained that on multilane approaches it is almost impossible for saturated flow to last throughout a long green interval on all lanes of the approach. Messer and Fambro account for underutilized lanes by applying a correction factor for multilane approaches. However, their adjustment factors ignore cycle length. If total approach really is less productively used as green time increases, it might be possible to develop adjustment factors that consider both number of lanes and green phase duration.

A serious question needs to be answered: Are the many theoretical equations that depend on full saturation flow during the effective green period to explain traffic flow at signalized intersections valid when compared with actual traffic operation?

SYSTEMATIC VALIDATION NEEDED

Carefully designed experiments are needed to validate the basic theoretical equations of intersection operation and to compare current alternative methods for evaluating capacity and level of service (HCM method, British saturation flow method, Australian method, critical lane volume analysis, multiple regression equations developed in Canada).

These experiments should stress sensitivity tests in which conditions would be changed at field intersections and the resulting changes in capacity, if any, and other operating characteristics would be compared with changes predicted by the theoretical methods. Only through this type of scientific experimentation will it be possible to determine the validity of the various approaches and to isolate the weaknesses of current methods.

Authors' Closure

We would like to express our appreciation to Petersen and Wagner for their discussions. Their comments from the perspective of planning and operation engineers point out the different levels of accuracy required in transportation planning, design, and operations.

Petersen questions the values of our utilization factors and their relationship to approach volume. These values are representative of the moderate to high volume conditions common to intersection design. The user may wish to follow these lane utilization formulas for increased accuracy:

$$U_2 = 1.1 + 0.9\, e^{-0.18m} \quad (30)$$

for two lanes, and

$$U_3 = 1.2 + 1.8\, e^{-0.13m} \quad (31)$$

for three lanes, where m is average approach arrivals per cycle (3).

Table 1, very important but slightly complicated, was developed according to a cycle length of 70 s in all cases. Two- and three-phase equivalents were based on G/C ratios of 0.51 and 0.36 respectively. With these splits, left turning capacity is at or near zero at opposing volumes of 1000 vehicles/h except for three-lane approaches (4). The user may opt for an arbitrary maximum limit on the left turn equivalent factor (E) in Table 1 and use this value for all heavier volume cases (for instance, E = 11.0) (3, p. 234).

Petersen notes the need for care during the selection of signal phases. If an approach does not have a separate left turn bay (lane), then 1 + 4 or 2 + 3 must be used.

Wagner suggests general potential refinements for the analysis technique. In particular, the user may wish to use the lane width adjustments presented by Wagner with the following conversions: 1.14, 1.08, 1.00, 0.97, 0.96, 0.94. The effects of pedestrians, parking, and grades were not included in the design procedure. Users may also wish to include these.

Wagner expresses his personal concern about the overall concept of phase lost time and cites Moskowitz's paper, which we had also reviewed. We believe the equations provide practical operational solutions using a lost time of 4 s/phase.

Several research projects on capacity will be conducted in the National Cooperative Highway Research Program and by the Federal Highway Administration during the next few years. We hope this paper will aid in the development of practical intersection capacity analysis techniques for planning, design, and operation needs.

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