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A GUIDE FOR DESIGNING AND OPERATING SIGNALIZED
INTERSECTIONS IN TEXAS

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Research Report Number 203-1

Effects of Design on Operational Performance
of Signal Systems

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Texas Transportation Institute
Texas A&M University
College Station, Texas

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ABSTRACT

This report presents a guide for designing and operating signalized intersections to serve rush hour traffic demands. Physical design and signalization alternatives are identified and methods of evaluation are provided. The level of service criteria used in the design guide were selected to expedite the design process. These design criteria are related in subsequent sections of the guide to traffic operational measures which are more descriptive of the quality of traffic flow from the motorists' point of view. The last section of the guide describes procedures for intersection signal timing and evaluation.

Key Words: Intersection Design, Intersection Operations, Critical Lane Analysis, Left Turn Capacity

ACKNOWLEDGMENT

SUMMARY

In order to provide an acceptable level of service to traffic operating along an urban arterial, the signalized intersections must be able to "keep the traffic moving". The ability of a signalized intersection to move traffic is determined by the physical features of the intersection and by the signalization used. Moreover, the geometric design of the intersection will have a direct effect on the ability of the signalization to move traffic. Thus, total system design of a signalized intersection involves concurrent evaluation of the proposed geometric design and traffic control devices as they will function together in the field as an integrated unit.

This report presents a guide for designing and operating signalized intersections to serve rush hour traffic demands. Physical design and signalization alternatives are identified and methods for evaluation are provided. The guide begins with a description of the procedures used to convert given traffic volume data for the design year into equivalent turning movement volumes. All volumes are converted into equivalent straight thru passenger car volumes. This is done to permit turning movement volumes which have different capacities to be converted into equivalent movements having slightly larger equivalent volumes but the same capacity (saturation) flow per lane. The geometric design procedures are applicable to channelization for improved operation and safety of signalized intersections (i.e., addition of left turn or right turn lanes) as well as for use in initial design.

The critical lane analysis technique is then 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. Guidelines and example

problems are presented to assist the engineer in determining satisfactory design alternatives. Signalization alternatives are also described.

Capacities of left turning phases at signalized intersections are estimated based on considerable field data collected during this research. Capacities for left turns with and without left turn bays are provided. In addition, guidelines are provided for designing the length of storage bay required for a given left turning volume. Decreases in capacity are given as the length of the left turn storage is reduced below minimum desirable values.

Operational performance characteristics of the intersection are related to signalization and design alternatives in subsequent sections of the report. The selected design Level of Service criteria are discussed. A signalization timing plan is developed and evaluated for one of the design example problems.

In the last section of the report, a new traffic flow, field evaluation technique is presented. This procedure evaluates the operational measures of effectiveness of saturation (volume-to-capacity) ratio, probability of clearing queues and average vehicle delay from traffic characteristics which can be easily measured at the intersection by only one observer.

Implementation

This report can be used by design and traffic engineers to design, operate, and evaluate signalized intersections. Information from this report is currently being used in the development of the newest edition of the highway and public transportation design manual of the Texas State Department of Highways and Public Transportation.

This report should also provide meaningful information for use in traffic engineering seminars, short courses, and other educational programs.

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INTRODUCTION

In order to provide an acceptable level of service to traffic operating along an urban arterial, the signalized intersections must be able to "keep the traffic moving." The ability of a signalized intersection to move traffic is determined by the physical features of the intersection and by the signalization used. Moreover, the geometric design will have a direct effect on the ability of the signalization to move traffic (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-off decisions between design variables, with their associated costs, and the resulting level of service. Level of Service at an intersection measures or describes the quality of traffic flow afforded motorist on a particular approach to the signalized intersection. Qualitatively speaking, the various levels of service may be characterized as:

Level of Service A - Light traffic on approach, short stable queues exist during red.

Level of Service B - Moderate traffic on approach, stable queues, little additional delay.

Level of Service C - Moderately heavy traffic on approach, moderately long but stable queues during red, moderate but acceptable delay.

Level of Service D - Heavy traffic on approach, long unstable queues, delays sometimes become excessive.

Level of Service E - Heavy flow (capacity) on approach, long queues suffering excessive delays.

Level of Service 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. The procedure seeks to provide Level of Service C traffic conditions, as a minimum, during the peak 15 minute period of the design hour. All operating conditions can be evaluated, however.

Basic design variables for consideration are the number of approach lanes provided, the possible use and length of left and right turn lanes, the combination of traffic movements using the lanes provided, and the type of signal phasing that will be used. The use of at least minimum design standards for the basic design variables of lane width (10 - 12 foot lane widths) and curb return radii (15 - 30 foot radii) will normally provide satisfactory operations during rush hours.

Volume Data Preparation

To illustrate the application of the procedure over a range of initially given volume data conditions, an example problem will be presented. In practice, the designer would begin the analysis at the appropriate step in the volume preparation procedure depending on the given traffic data.

Step 1, Average Daily Traffic.

In this example it is assumed that the given volume data are the forecasted, design year, average daily traffic (ADT) volumes. These two-way, ADT volumes are shown in Figure 1, Step 1.

An overall summary of the given traffic and operational conditions is as follows:

STEP 1 : AVERAGE DAILY TRAFFIC

10,900

1500   1300

13,700   9900   12,250

8100

2300   1050

ADT,
VEHICLES
PER DAY   11,450

STEP 2 : DESIGN HOUR MOVEMENT
VOLUMES
385

50   65 *

270

*65

875   660   330   430

115 *   35

540

*115   70

DHV,
VEHICLES   725   * $\bar{D} = 0.5$
PER HOUR
DHV = ADT $\cdot$ K $\cdot$ $\bar{D}$

STEP 3 : DESIGN PERIOD VOLUMES

482

63   81

338

125   81

1094   825   413   538

144   44

675

144   88

907

DPV,
VEHICLES
PER HOUR
DPV = DHV $\cdot$ PF

STEP 4 : ADJUST FOR TRUCKS AND
BUSES
506

66   85

355

131   85

1148   866   433   564

151   46

709

151   92

952

ECV,
EQUIVALENT
CARS PER HOUR
ECV= DPV $\cdot$ [I + T(E - I)]

STEPS IN VOLUME DATA PREPARATION

FIGURE I

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Volumes	:	1985 ADT
Design Hour Factor	:	K = 10%
Directional Distribution:		D = 67%
Trucks and thru buses	:	T = 5%
Population, 1985	:	400,000

Step 2, Design Hour Movement Volumes.

The two-way, ADT volumes are first converted into approach movement volumes for the design hour being analyzed. The A.M. peak hour is assumed in this example. The P.M. design hour also would be checked. If the given volumes are in ADT, the A.M. design hour volumes for left turns on one approach become the departure leg's right turns during the P.M. peak hour, etc.

The A.M. design hour volumes are shown in Figure 1, Step 2. The directional peak flows are assumed to be from left-to-right and bottom-to-top. Other factors being equal, the location and orientation of the intersection in the metropolitan area dictates the peak directions of flow. The larger of the two design hour, directional, movement volumes flowing between legs "a" and "b" is calculated from

$$DHV_{ab} = ADT_{ab} \cdot K \cdot \overline{D} \qquad \text{[heavier direction]} \qquad (1)$$

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" (Figure 1, Step 1), K is the design hour factor (10% = 0.10) and $\overline{D}$ is the average directional distribution split (decimal equivalent) between the approaches. In this example problem, $\overline{D}$ is either 0.67 or 0.50. The off-peak direction movement volume between legs "a" and "b" is calculated from

$$DHV_{ab} = ADT_{ab} \cdot K \cdot (1.00 - \overline{D}) \qquad \text{[lighter direction]} \qquad (2)$$

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Step 3, Design Period Volumes.

The time period used to evaluate the level of service at the intersection is the peak 15 minute period of the design hour. The traffic volume flow rates during this 15 minute period consistently exceed the average for the design hour by approximately 20 - 30 percent. These peaking factors have been found in Texas to vary with the population of the city and are given in Table 1.

TABLE 1

VARIATION OF PEAKING FACTOR, PF, WITH POPULATION OF CITY IN DESIGN YEAR

Population	PF
<100,000	1.35
100,000 - 300,000	1.30
300,000 - 500,000	1.25
>500,000	1.18

Source: References (1) and (2).

Design period flow rates for each movement are calculated from

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

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

Step 4, Equivalent Passenger Car Volume, ECV.

This design procedure converts all design period volumes of mixed traffic (5% trucks and thru buses in this example) into an equivalent number of

passenger cars. It is assumed that one truck or thru bus is equivalent to two passenger cars ($\underline{3}$). Thus, the equivalent passenger car volumes (ECV) in Figure 1, Step 4, are calculated from the design period mixed traffic flow rates of Figure 1, Step 3, by

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

where T is the decimal equivalent of the percent trucks and thru buses in the traffic stream (0.05) and E_T is the passenger car equivalent for trucks and thru buses. Since E_T is assumed to be 2.0,

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

Geometric Movement Volumes

The next step in the design guide requires that the individual ECV turning movement volumes (Figure 1, Step 4, and Equation 5) be defined by the way they will be combined by the geometric design of the intersection. Eight basic geometric movement volumes (GMV_m) would exist at a high-type, four-leg intersection having left turn bays on all approaches, as depicted in the left intersection of Figure 2. When a left turn bay, or 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, e.g., $GMV_{1A} = ECV_{1t}$. The adjacent thru-right geometric movement volume would be calculated as $GMV_{4A} = ECV_{th} + ECV_{rt}$. However, when a free right turn lane is provided, the ECV right turning volume (ECV_{rt}) can be neglected.

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

Geometric Design Volumes
_____

All of the geometric movement volumes (GMV_m) are further adjusted to
account for the (proposed) design and operational features of the intersection.
These equivalent-effects volumes, named the geometric design volumes GDV_m, are
calculated from:

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

GDV_m = Geometric design volume for movement "m" of Figure 2, "cars"/hr.

   U = Lane utilization factor (Table 2).

   W = Lane width factor (Table 2).

  TF = Turning movements factor (Equation 7).

GMV_m = Geometric movement volume of movement "m". Sum of one or more
         turning movement volumes (ECV's from Equation 5) as illustrated in
   Figure 2.
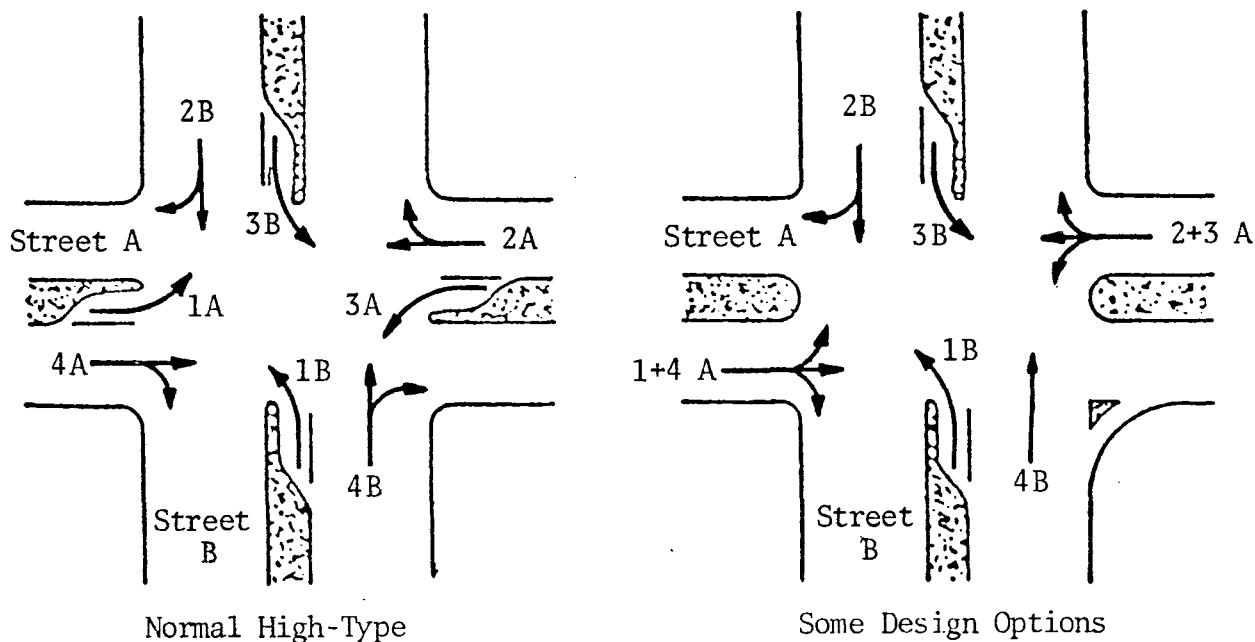
FIGURE 2   DEFINITION OF MOVEMENTS

Lane Utilization - As the number of lanes serving a movement(s) increases, there is an increasing tendency for one lane to become more highly utilized than the others. This effect is accounted for by the lane utilization factor, U, presented in Table 2.

Lane Width - Lanes 10 feet or more in width have little influence on rush hour traffic flow rates as reflected by the lane width adjustment factor, W, given in Table 2. However, thru lanes less than 11 feet wide may experience related safety problems. The width of a lane may not include any pavement used for or appreciably affected by parking.

TABLE 2

VOLUME ADJUSTMENT FACTORS FOR LANE UTILIZATION AND WIDTH

Factor	Conditions	Value
Lane Utilization, U	Number of Lanes	
used in Equation 6	1 2 ≥ 3	1.0 1.1 1.2
Lane Width, W	Average Lane Width, Feet	
used in Equation 6	9.0 - 9.9 ≥ 10.0	1.1 1.0

Source: Reference (4).

Turns - The effects of left and right turning vehicles on flow are given by the turning factor, TF, as:

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

where L adjusts for the effects of left turns and R for right turns; L may be L_1, L_2, or L_3 as described subsequently.

For designs providing no left turn bay, the left turn adjustment factor, L, for the combined left-thru-right movement volume is calculated from:

$$L_1 = P_L (E - 1.0) \qquad \text{[for left-thru-right only]} \quad (8)$$

where P_L is the decimal fraction of the total approach volume turning left and E is the appropriate equivalence factor from Table 3.

TABLE 3

LEFT TURNING EQUIVALENT, E, FOR APPROACH CONDITIONS

Intersection • Signal Phasing	Traffic Movement	Number of Opposing Thru Lanes	Opposing Volume, CPH[+], ECV				
			200	400	600	800	1000
No Protected Turn Phase							
• Two-Phase							
No Bay on Approach	Left & Thru	1	2.0	3.3	6.5	16.0*	16.0*
	Left & Thru	2	1.9	2.6	3.6	6.0	16.0*
	Left & Thru	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*	10.4*
	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 & Thru	1	2.2	4.5	11.0*	11.0*	11.0*
	Left & Thru	2	2.0	3.1	4.7	11.0*	11.0*
	Left & Thru	3	2.0	2.9	4.2	6.0	11.0*
With Bay on Approach	Left	1	1.8	3.3	8.2*	8.2*	8.2*
	Left	2	1.7	2.4	3.6	5.9	8.2*
	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

[+]Includes total thru volume (from Figure 1, Step 4) on the approach opposing the left turn being analyzed. The opposing volume also includes any turning volume(s) (left and/or right) for which no separate turning lane (bay) is provided.

*Turning capacity only at end of phase. Not recommended for design. Add additional thru lane, turning lane, or protected left turn phasing.

For designs providing a left turn bay, the left turn adjustment factor for the left turn movement, only, is

$$L_2 = \frac{1700 \cdot E}{S} - 1.0 \qquad \text{[for left turn only]} \qquad (9)$$

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

The desired minimum left turn bay storage length for a given equivalent turning volume is presented at the top of Figure 3. Shorter bay storage lengths result in saturation flow rates, S, less than 1700. The storage length does not include either the taper section of the left turn bay or any length of the bay that may be provided beyond the usual stop line. For normal urban street conditions, a taper length of 70 to 100 feet may be considered appropriate; for high-type urban facilities and rural highways, it should be 150 to 300 feet.

When a left turn bay is provided, it is necessary to account for any blockage effects that left turns may cause the thru-right movement. The left turn adjustment factor to be applied only to the thru-right movement is calculated from

$$L_3 = \frac{1700 - S}{1700 \ (N-1) + S} \qquad \text{[for thru-right only]} \qquad (10)$$

where S is the saturation flow of the left turn from Figure 3, as in Equation 9, and "N" is the number of lanes serving the adjacent thru-right movement.

When a separate right turn lane is provided, the right turning volume and right turn lane are not 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 to zero, i.e., R = 0, in Equation 7, if right-turns-on red will be permitted. If a detailed
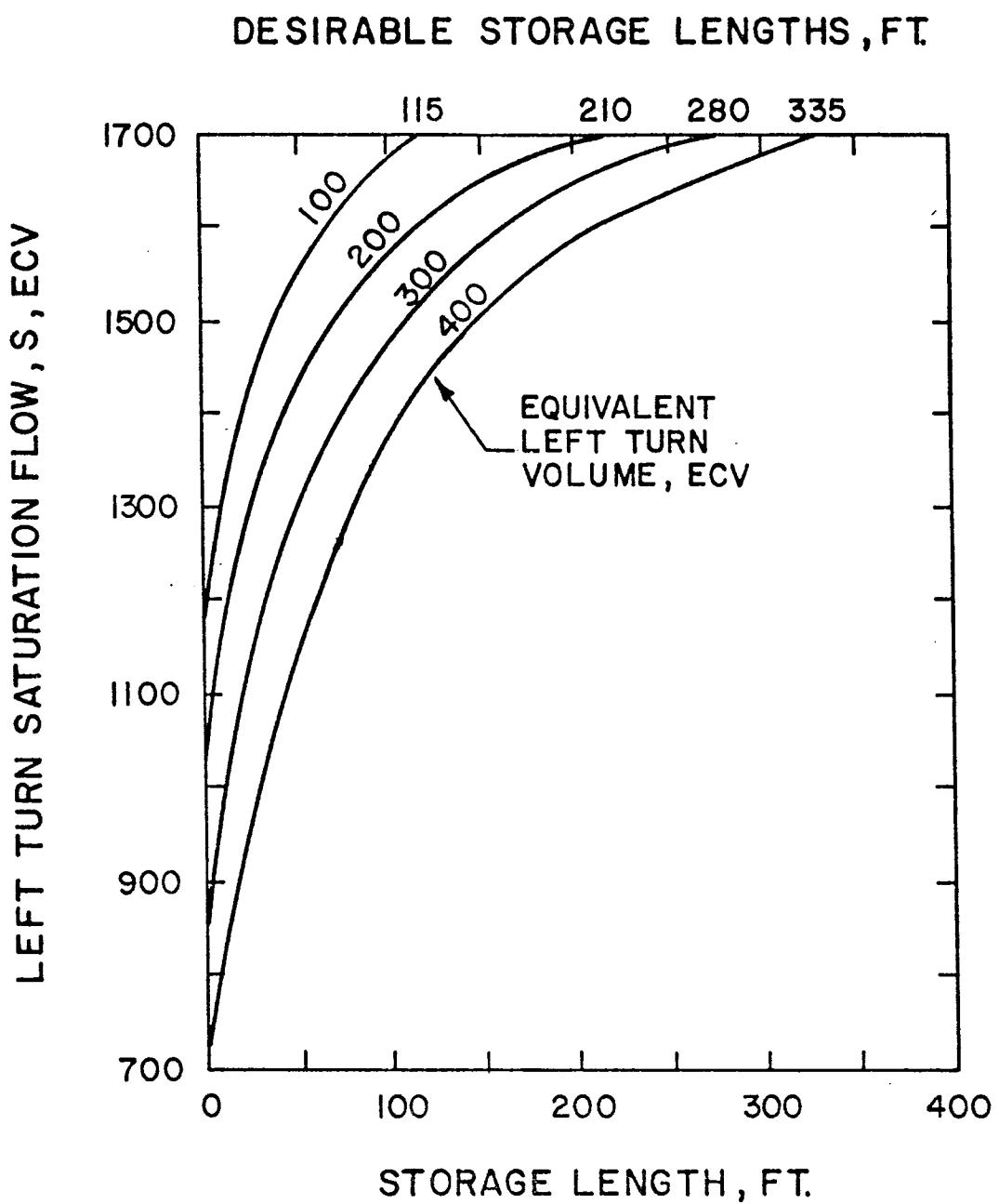
SATURATION FLOW OF LEFT TURN PHASE
AS A FUNCTION OF BAY STORAGE LENGTH
AND TURNING VOLUME

FIGURE 3

analysis is desired, the following approach may be used to estimate the effects of right turns as related to design and operational variables. The effects of right turning vehicles (5) are given by:

$$R = \frac{5 \cdot P_R}{c} \qquad (11)$$

R = Right turning adjustment factor in Equation 7.

P_R = Decimal fraction of movement combination turning right.

c = Related curb return radius, feet.

In addition, the estimated number of vehicles turning right-on-red are subtracted from the thru-right geometric movement volume combination (GMV_m). This estimate, which should not exceed 0.5 of the right turning volume, is calculated from:

$$ROR = 50 \cdot \frac{P_e}{1 - P_e} \qquad (12)$$

ROR = Estimated right-on-red volume, veh./hr.

P_e = Estimated decimal fraction of traffic in curb lane turning right.

Capacity - It is assumed throughout this procedure that the capacity of a normal protected thru lane is 1750 passenger cars per hour green (6). This is equivalent to a minimum average headway of 2.06 seconds per car. The type of signalization also affects the capacity of the intersection as will be reflected in the critical lane analysis technique to follow. Several types of signalization might be considered in the design.

Critical Lane Volumes for Each Street

The critical lane analysis technique is used to evaluate the acceptability of a design. To begin the analysis, the geometric design volume for each movement(s) GDV_m, calculated in Equation 6, is divided by the number of useable lanes provided to serve the movement(s) to obtain a design volume per lane, V_m, of

$$V_m = \frac{GDV_m}{N}$$

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

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 ways by the type of signalization used, as shown in Table 4 and illustrated in Appendix A. For a given type of signalization on a street, one movement or a sum of two movements will be larger (critical). Different sums of critical lane volumes will usually result with different types of signal phasing.

Level of Service Evaluation for Intersection

To evaluate the acceptability of the design, it is necessary to calculate the sum of the critical lane volumes (ΣV) for both (all) intersecting streets at the intersection from:

$$\Sigma V = \Sigma V_{max} \text{ (Street A)} + \Sigma V_{max} \text{ (Street B)}$$

The sum of the critical lane volumes at the intersection is then compared against maximum values established for a given level of service as presented in Table 5. Level of Service "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;

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TABLE 4

METHOD FOR CALCULATING SUM OF CRITICAL
LANE VOLUMES FOR EACH STREET AT INTERSECTION

Type of Signal Phasing On Street A or B*	For Streets A and B of Figure 2, Calculate the Maximum Lane Volume Sum of 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"	
Overlap Phasings of	
• 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

*See Appendix A for illustrations of signal phasings.

TABLE 5

LEVEL OF SERVICE MAXIMUM SUM OF CRITICAL LANE VOLUMES AT
SIGNALIZED INTERSECTIONS FOR USE IN DESIGN

Level of Service	Traffic Flow Condition	Volume to Capacity Ratio X	Maximum Sum of Critical Lane Volumes, ΣV, at Intersection		
			Two-Phase* 2ϕ	Three-Phase* 3ϕ	Multi-Phase* $\geq 4\phi$
A	Stable	≤ 0.6	900	855	825
B	Stable	≤ 0.7	1050	1000	965
C	Stable	≤ 0.8	1200	1140	1100
D	Unstable	$\leq 0.85^y$	1275	1200	1175
E	Capacity	≤ 1.0	1500	1425	1375

*Number of critical phases. See Appendix A for illustrations of signal phasings.
^{y}Modified in this project.

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while multiphase signalization has protected left turning on both streets. Capacity values are based on practical peak hour cycle lengths.

Design Example Problems

Two design alternatives that might be considered to serve the projected traffic are evaluated in Appendix B. Some assumptions made in the examples were selected to illustrate computational procedures of the critical lane analysis design guide rather than optimum design practice.

Left Turn Capacities (1)

Providing the necessary left turn capacity at an intersection is frequently necessary to relieve congestion when left turn demand volumes exceed practical service volume limits. Initial problems often arise on approaches having no protected turning phase or left turn bay. To determine if a left turn bay is warranted on an approach when all traffic on the street is permitted to move simultaneously on a common green indication, the following procedure should be used:

1) Determine peak period turning movement volumes in equivalent passenger cars per hour. See Figure 1, Step 4.

2) Obtain street's green phase to cycle length ratio, G/C. Do not include yellow as part of green phase, G.

3) Compare equivalent left turning volume with 0.8 of capacity given in Table 6. Level of Service C is assumed, as in Table 5.

If the equivalent left turning volume exceeds 80 percent of the capacity given in Table 6, or known capacity of the existing approach as determined from a field study, then a channelized or otherwise designated left turn lane is warranted.

A left turn phase may also be needed even though a left turn bay is pro-

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TABLE 6

ESTIMATED CAPACITY OF LEFT TURNING MOVEMENT

WITHOUT PROTECTED SIGNAL PHASE OR LEFT TURN BAY

(IN CARS PER HOUR DURING PEAK PERIOD)

	Total Opposing Volumes, CPH*, ECV				
	200	400	600	800	1000
G/C = .3					
N = 1	132	50	50	50	50
N = 2	148	95	53	50	50
N = 3	149	101	66	50	50
G/C = .4					
N = 1	209	122	50	50	50
N = 2	223	166	119	50	50
N = 3	223	171	131	97	50
G/C = .5					
N = 1	286	200	114	50	50
N = 2	298	237	185	122	50
N = 3	298	241	195	156	122
G/C = .6					
N = 1	363	278	194	108	50
N = 2	373	307	251	187	122
N = 3	373	311	259	214	175
G/C = .7					
N = 1	441	356	274	192	105
N = 2	447	378	317	252	188
N = 3	448	381	323	273	228

G/C = Actual green/cycle length.

N = Number of opposing thru lanes.

* = Includes left turn, thru, and right turn volumes. However, does not include left turn volume if left turn bay is provided.

vided, when left turning and opposing approach volumes are heavy and would otherwise move on the same phase. To determine if a separate left turn phase is warranted, the following procedure should be used:

1) Determine peak period turning movement volumes in equivalent passenger cars per hour. See Figure 1, Step 4.

2) Obtain street's green phase to cycle length ratio, G/C. Do not include yellow as part of green phase, G.

3) Compare equivalent left turning volume with 0.8 of capacity given in Table 7. Level of Service C is assumed, as in Table 5.

If it is found that the equivalent left turn volume exceeds 80 percent of the capacity of the left turn lane given in Table 7, the following alternatives should be considered: (1) accept a lower level of service on the left turn lane; (2) increase the G/C (for the street from which the left turn is being made, if possible) in which case the thru-plus-right volume from the same direction as the left turn would operate at a higher level of service; or (3) add a separate traffic signal phase for the left turning movement in order to accommodate it. Check the resulting level of service on all movements as described in a later section of this guide.

TABLE 7

LEFT TURNING CAPACITY OF SINGLE PHASE
WITH UNPROTECTED TURNING AND ADEQUATE BAY LENGTH
(IN CARS PER HOUR DURING PEAK PERIOD)

	Total Opposing Thru and Right Turning Volumes, CPH, ECV*				
	200	400	600	800	1000
G/C = .3					
N = 1	232	82	82	82	82
N = 2	260	159	82	82	82
N = 3	261	168	105	82	82
G/C = .4					
N = 1	368	204	82	82	82
N = 2	392	276	187	114	82
N = 3	393	285	207	147	100
G/C = .5					
N = 1	503	333	181	82	82
N = 2	524	394	292	208	138
N = 3	525	401	309	236	177
G/C = .6					
N = 1	639	463	307	164	82
N = 2	655	512	397	302	223
N = 3	656	518	410	324	254
G/C = .7					
N = 1	775	593	434	291	153
N = 2	787	630	502	396	307
N = 3	788	634	512	413	331

G/C = Actual green/cycle length.

N = Number of opposing thru lanes.

* = Also includes left turn volume if no left turn bay is provided.

TRAFFIC FLOW MEASURES OF EFFECTIVENESS

The evaluation of the level of service at an intersection may be desired in the form of an estimation of operational measures, or the evaluation may be based on traffic flow data collected in the field from the existing intersection. Typical traffic flow operational measures of effectiveness include: (1) saturation ratio, (2) delay, (3) minimum delay cycle length, (4) Poisson's probability of failure, (5) probability of queue clearance or overflow, and (6) load factor. Most of these operational measures describe traffic flow conditions on a single approach and, frequently only for a single traffic movement. Only minimum delay cycle length is an overall intersection performance measure. However, average approach values for the intersection can be calculated for the other measures.

Intersection Signal Phasing

A normal, four-legged intersection having multiphased signalization will have four critical volume phases, as was shown in Figure 2 and Table 4. The sum of these four critical phases, including green plus yellow times, is

$$\phi_{A1} + \phi_{A2} + \phi_{B1} + \phi_{B2} = C \tag{14}$$

or

$$(G+Y)_{A1} + (G+Y)_{A2} + (G+Y)_{B1} + (G+Y)_{B2} = C \tag{15}$$

where: ϕ_{A1} = First critical phase on Street A, sec.

C = Cycle length, sec.

G = Green time of phase, sec.

Y = Yellow (clearance) time of phase, sec.

Effective Green

The effective green time (g) is defined as that portion of the green plus yellow time when saturation capacity flow occurs. It is known that saturation flow conditions do not begin until approximately 2.0 seconds after the start of the green. Saturation flow conditions end about 2.0 seconds before the yellow clearance time expires. Thus,

$$(G+Y) \doteq 2.0 + g + 2.0 \doteq g + 4.0 \tag{16}$$

Rearranging terms, the effective green, g, equals approximately

$$g \doteq G + Y - 4.0 \tag{17}$$

or

$$g = G + Y - L \tag{18}$$

where: g = Effective green time, sec.

G = Actual green, sec.

Y = Yellow clearance, sec.

L = Lost time, sec. ($\doteq$ 4.0)

In all calculations to follow, it is assumed that the lost time, L, is 4.0 seconds. Practically speaking, however, the actual green time and effective green time are about the same if the yellow clearance time is established according to basic intersection approach speed and width criteria. That is,

$$g \doteq G \tag{19}$$

where: g = Effective green, sec.

G = Actual green, sec.

Movement Capacity on a Phase

The number of vehicles, NV, that can move into an intersection from one

approach movement during one phase is

$$NV = g \cdot \frac{S}{3600} \qquad (20)$$

where S is the saturation capacity flow of the approach in vehicles per hour of green time. For example, if the saturation flow is 3500 vehicles per hour green for a two lane approach, then for a 30 second effective green, g;

$$NV = 30 \cdot \frac{3500}{3600} = 29.17 \text{ vehicles per phase (or per cycle)}$$

The number of vehicles that can enter the intersection per hour from the approach, the capacity CAP, is

$$CAP = NV \cdot \frac{3600}{C} \; ; \qquad \frac{veh.}{cycle} \cdot \frac{cycles}{hour} \qquad (21)$$

or

$$CAP = \frac{g \cdot S \cdot 3600}{3600 \cdot C} \; \frac{veh.}{hr.}$$

Thus, $\qquad CAP = \frac{g \cdot S}{C} \; \frac{veh.}{hr.} \qquad (22)$

For example, if g = 30, S = 3500, and C = 70 seconds, the capacity of the movement is

$$CAP = \frac{30 \cdot 3500}{70} = 1500 \quad \frac{vehicles}{hour}$$

Saturation Ratio, X

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

$$X = \frac{Volume}{CAP} = \frac{Q}{\frac{g}{C} \cdot S} \qquad (23)$$

For the total approach movement, then

$$X = \frac{Q \cdot C}{g \cdot S} \qquad (24)$$

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where: X = Saturation (volume-to-capacity) ratio of the signal phase

Q = Approach movement volume, vph

C = Cycle length, sec.

g = Effective green time of phase serving movement, sec.

S = Saturation flow of approach, vphg

Continuing with the example, if Q = 1200 vph on the approach, then with CAP = 1500 vph

$$X = \frac{Q}{CAP} = \frac{1200 \text{ vph}}{1500 \text{ vph}} = 0.8$$

Substituting all variables at once into Equation 24, yields

$$X = \frac{Q \cdot C}{g \cdot S} = \frac{1200 \cdot 70}{30 \cdot 3500} = 0.8$$

The saturation ratio is a good quantitative descriptor of what traffic operating conditions will be like. When $X > 0.85$, vehicle delay on the approach becomes very large and queues frequently fail to clear the approach at the end of the green phase.

Critical Lane Development

The critical lane design process is developed in the following section. Beginning with Equation 15

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

one substitutes the definition of effective green, g, and lost time, L, given in Equation 18

$$(g+L)_{A1} + (g+L)_{A2} + (g+L)_{B1} + (g+L)_{B2} = C$$

Separating the variables,

$$g_{A1} + g_{A2} + g_{B1} + g_{B2} = C - L_{A1} - L_{A2} - L_{B1} - L_{B2}$$

Letting "n" be the number of critical phases (4 here) and L the average lost time per phase (assumed to be 4.0 seconds), then

$$g_{A1} + g_{A2} + g_{B1} + g_{B2} = C - nL \tag{25}$$

Solving for g in the saturation ratio formula (Equation 24) and substituting in Equation 25 results in

$$\frac{Q \cdot C}{X \cdot S}_{A1} + \frac{Q \cdot C}{X \cdot S}_{A2} + \frac{Q \cdot C}{X \cdot S}_{B1} + \frac{Q \cdot C}{X \cdot S}_{B2} = C - nL$$

Or dividing by the cycle length, C,

$$\frac{Q}{X \cdot S}_{A1} + \frac{Q}{X \cdot S}_{A2} + \frac{Q}{X \cdot S}_{B1} + \frac{Q}{X \cdot S}_{B2} = \frac{C - nL}{C}$$

If the saturation ratio is selected to be the same for all critical phases, then

$$\frac{1}{X} \left[\frac{Q}{S}_{A1} + \frac{Q}{S}_{A2} + \frac{Q}{S}_{B1} + \frac{Q}{S}_{B2} \right] = \frac{C - nL}{C}$$

or

$$\frac{Q}{S}_{A1} + \frac{Q}{S}_{A2} + \frac{Q}{S}_{B1} + \frac{Q}{S}_{B2} = X \cdot \frac{C - nL}{C} \tag{26}$$

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

$$\frac{V_{A1}}{1750} + \frac{V_{A2}}{1750} + \frac{V_{B1}}{1750} + \frac{V_{B2}}{1750} = X \cdot \frac{C - nL}{C}$$

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

$$V_{A1} + V_{A2} + V_{B1} + V_{B2} = 1750 \cdot X \cdot \frac{C - nL}{C}$$

or the sum of the critical lane volume for a given level of service saturation ratio and design cycle length is

$$\Sigma V = 1750 \cdot X_{L.O.S.} \cdot \frac{C - nL}{C} \tag{27}$$

The results of Equation 27 are plotted in Figure 4 for two X ratios, X = 1.0 or capacity and X = 0.8. Also shown in Figure 4 are desirable design hour cycle lengths. These cycle lengths, 55 seconds for two-phase, 65 second for three-phase, and 75 seconds for four-phase were selected as basic design criteria. From these desired cycle lengths, the capacity values and other sum of critical lane service volumes in Table 5 were determined. Slightly higher capacities are possible at longer cycle lengths, but these should only be used as an interim treatment for an existing operational problem. When a wide range of long cycle lengths is required to serve peak hour volumes, coordinated signal operations along an arterial are not practical.

Minimum Delay Cycle Length

The average vehicular delay experienced by an approach movement having Poisson arrivals can be calculated using Webster's rather lengthy formula:

$$ d = \frac{C \left(1 - \frac{g}{C}\right)^2}{2 \left(1 - \frac{Q}{S}\right)} + \frac{1800 \, X^2}{Q \, (1 - X)} - 0.65 \left[\frac{C}{(Q/3600)^2}\right]^{1/3} X^{2 + 5 \, g/C} \tag{28} $$

where: d = Average delay per vehicle, sec./veh.

g = Effective green for movement, sec.

C = Cycle length, sec.

Q = Approach movement volume, vph.

S = Approach saturation flow, vphg.

X = Saturation ratio, QC ÷ gS.

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 5 shows how the average delay varies with cycle length for each of three

24
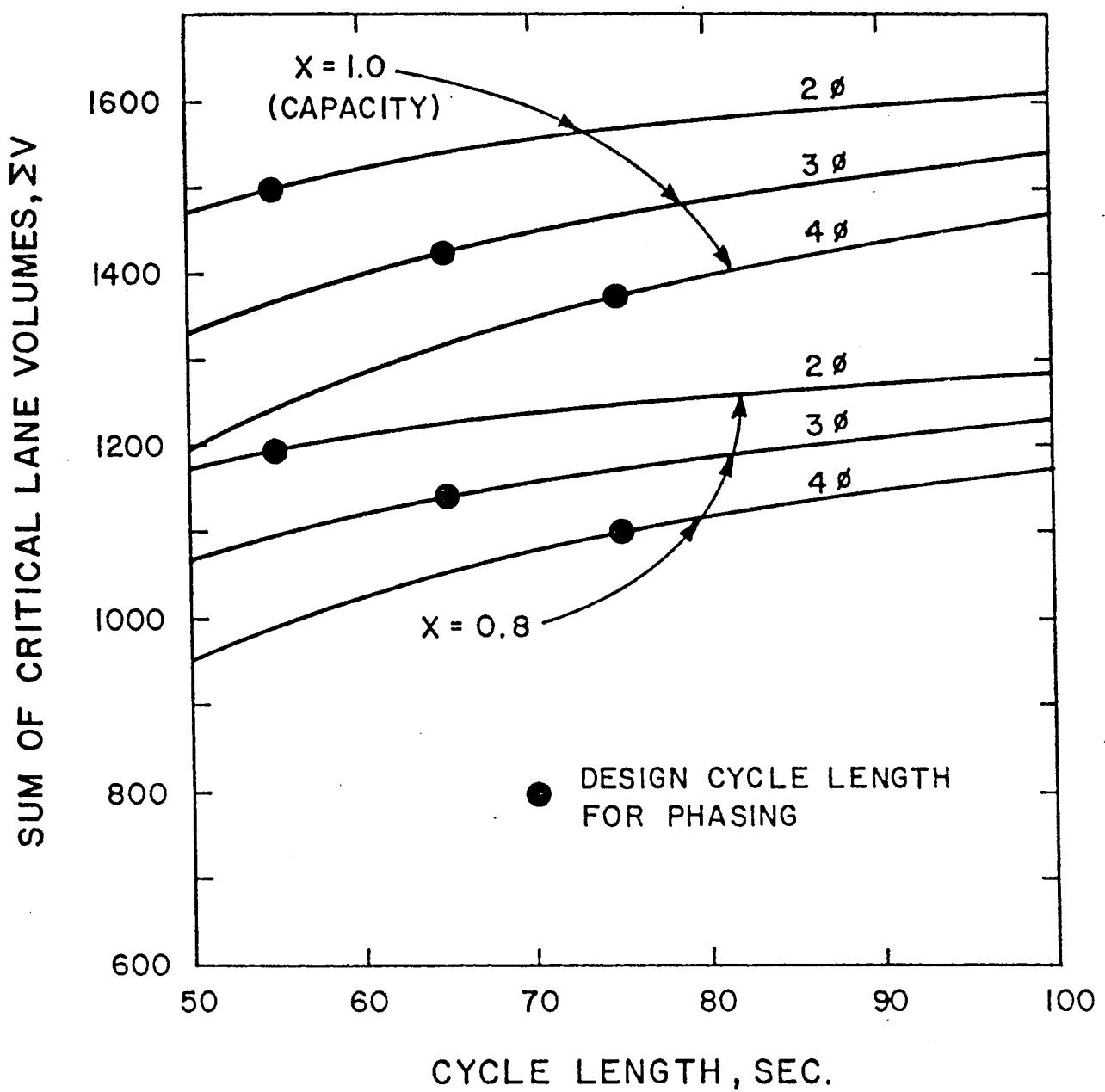
VARIATION IN SUM OF CRITICAL LANE
VOLUMES WITH CYCLE LENGTH AS A
FUNCTION OF SATURATION RATIO

FIGURE 4
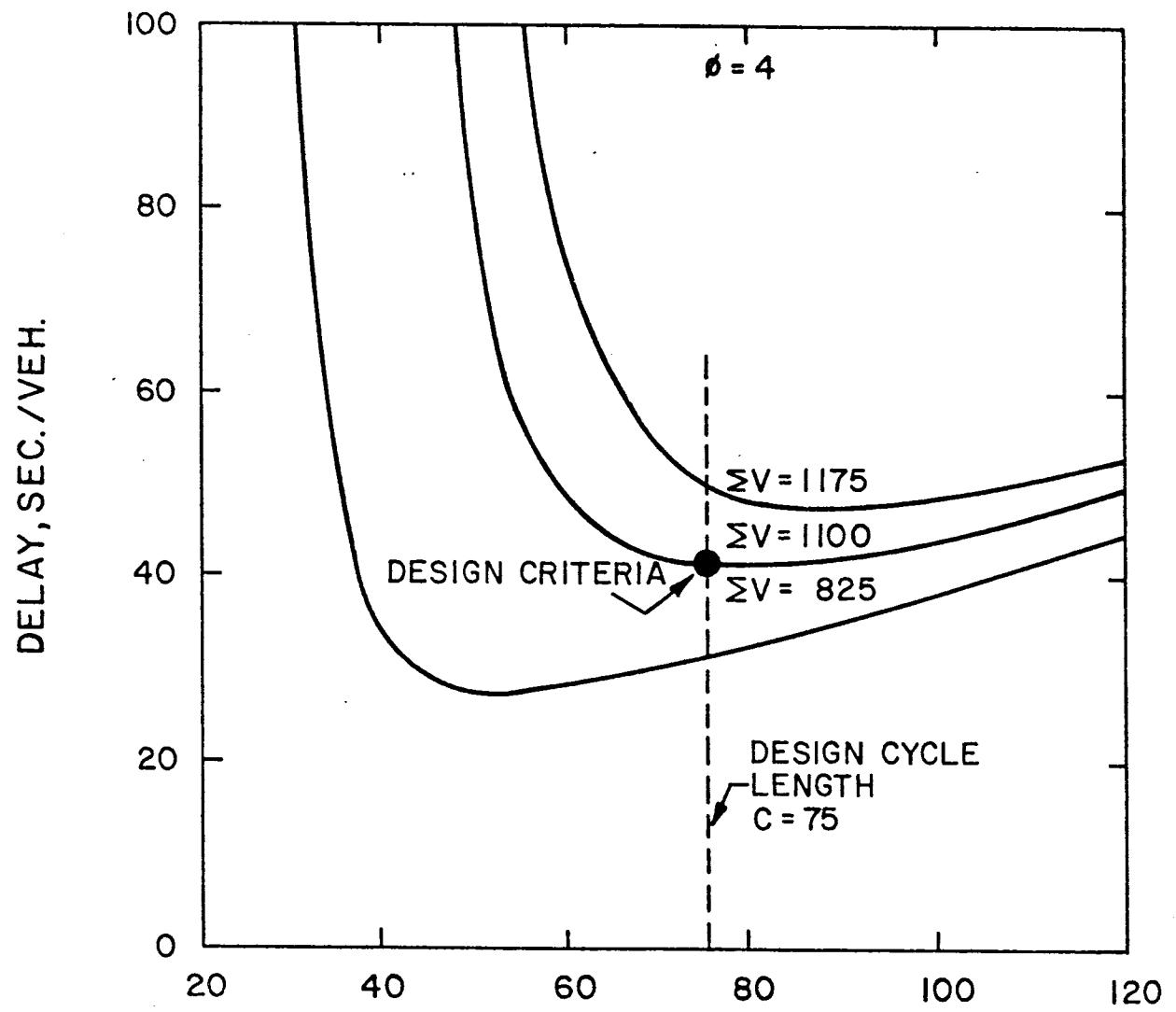
DELAY VERSUS CYCLE LENGTH AS A
FUNCTION OF SUM OF CRITICAL
LANE VOLUMES

FIGURE 5

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 5 for multi-(4) phase signals.

A minimum delay cycle length exists for each sum of critical lane volumes, or related level of service. The minimum delay cycle lengths are 55, 78, and 88 seconds for the sums of critical lane volumes of 825 ("A"), 1100 ("C"), and 1175 ("D"), respectively. Multi-(4) phase intersections designed to Level of Service C could operate at almost any normal range (60 - 90 seconds) of peak hour cycle lengths. However, intersections having a sum of critical lane volumes of 1175 ("D") will be heavily congested at cycle lengths less than 70 seconds.

Figure 6 summarizes the relationships between the design variables and operational measures of 1) sum of critical lane volumes, 2) saturation ratio, 3) minimum delay cycle length, 4) type of signal phasing and 5) Level of Service "C" design criteria. Level of Service C design criteria results in minimum delay cycle lengths which are also desirable cycle lengths 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 has been used in the Highway Capacity Manual of 1965 (2) to define level of service. Traffic engineers have also used Poisson's probability of cycle failure (7). Miller has 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 (8). A summary of these new models developed by Miller is presented subsequently.
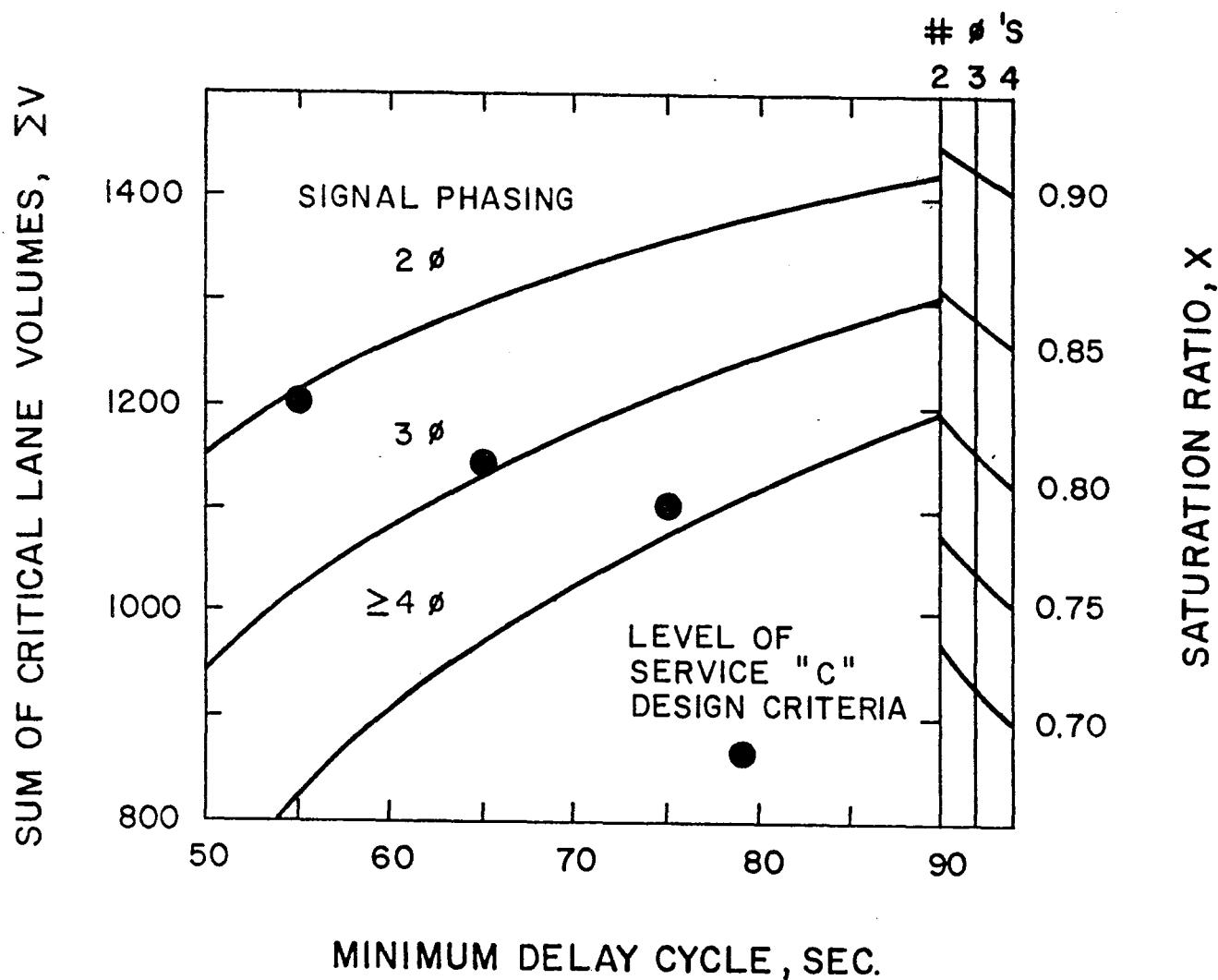
RELATIONSHIP BETWEEN DESIGN
VARIABLES AND OPERATIONAL
MEASURES

FIGURE 6

● Probability of Queue Failure, P_f:

$$P_f = e^{-1.58\phi} \qquad (29)$$

● Load Factor, LF:

$$LF = e^{-1.3\phi} \qquad (30)$$

● Probability of Queue Clearing, P_c:

$$P_c = 1 - e^{-1.58\phi} \qquad (31)$$

where

$$\phi = \frac{1 - X}{X} \sqrt{\frac{S \cdot g}{3600}}$$

$$X = \frac{Q \cdot C}{g \cdot S} = \text{The saturation ratio for movement.}$$

Figure 7 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 a cycle length of 75 seconds is the minimum delay cycle length, the saturation ratio, X, is 0.78, the load factor, LF, is 0.36, Miller's probability of cycle failure, P_f, is 0.30, and Poisson's probability of failure is 0.18 (18%).

Figure 8 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 seconds, respectively. Few cycles fail to clear stopped queues at X (saturation) ratios less than 0.7. Acceptable queue failure rates still exist at 0.8 but larger values result in unstable operation and increasingly excessive delays.

29
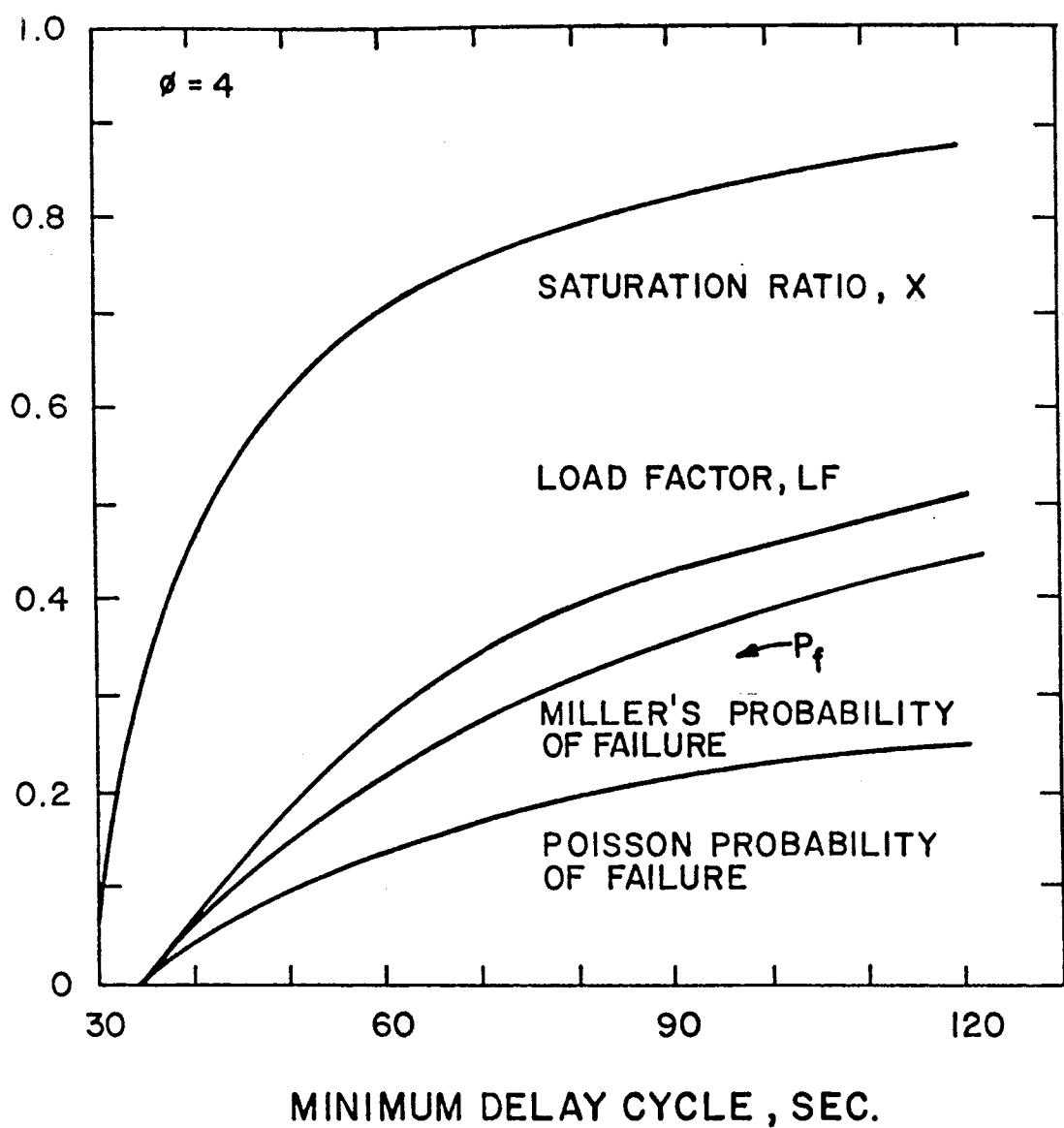
OPERATIONAL MEASURES OF EFFECTIVENESS
AS RELATED TO MINIMUM DELAY
CYCLE LENGTH

FIGURE 7

OPERATIONAL MEASURES OF EFFECTIVENE
AS RELATED TO VOLUME

FIGURE 8

PRETIMED SIGNAL PHASING AND TIMING

It is important to have a satisfactory predetermined timing plan for a pretimed traffic signal when the signal is placed in operation. Timing adjustments are often needed based on field observations but there is a need to start with a good timing plan. Signal timing must satisfy vehicular and pedestrian requirements.

There are several methods available for timing traffic signals. The method to be presented is a modification of Webster's method which seeks to determine the length of phases at the intersection such that the total delay at the intersection is minimized for a given cycle length.

To begin, it is assumed that the following data are given or have been determined:

- Geometry of intersection

- Number of dials used

- Signal phasing

- Equivalent straight through passenger car volumes per lane

As an example, assume it is desired to develop the signal timing plan for the design example problem illustrated to completion in Appendix B, Example Problem 2. The assumed geometry and calculated equivalent straight through passenger car volumes are shown in Figure 9-a. The assumed signal phasing is given in Figure 9-b. Level of Service C or better is assumed to be required.

The following procedure should then be followed:

1. Select Vehicle Clearance Intervals. Vehicle clearance intervals are based on approach speeds: 3 second yellow for speeds up to 35 MPH; 4 second yellow for 35 to 50 MPH; and 5 second yellow for speeds 50 MPH or more. The maximum length for the yellow period is 5 seconds. A one to two second all red

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STREET B
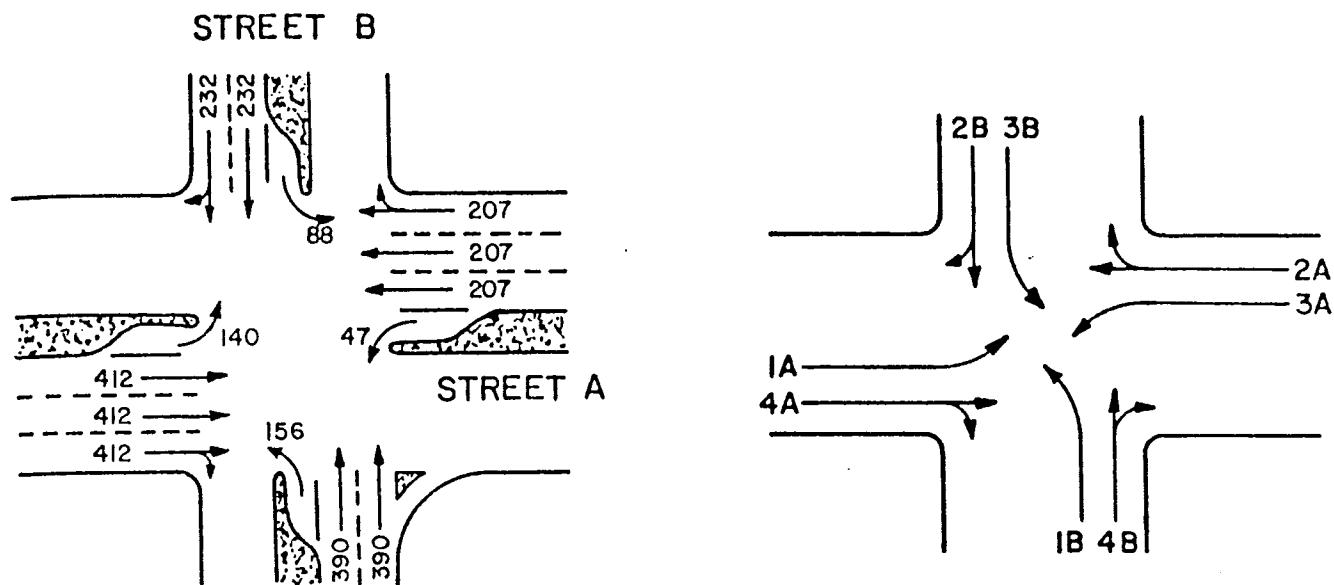
9-A EQIVALENT STRAIGHT THROUGH PASSENGER CAR
VOLUMES PER LANE (EQ. 13) ON MOVEMENTS

ϕ_B (2+3)	10
ϕ_B (2+4)	13
ϕ_B (1+4)	15
ϕ_A (2+3)	10
ϕ_A (2+4)	11
ϕ_A (1+4)	16
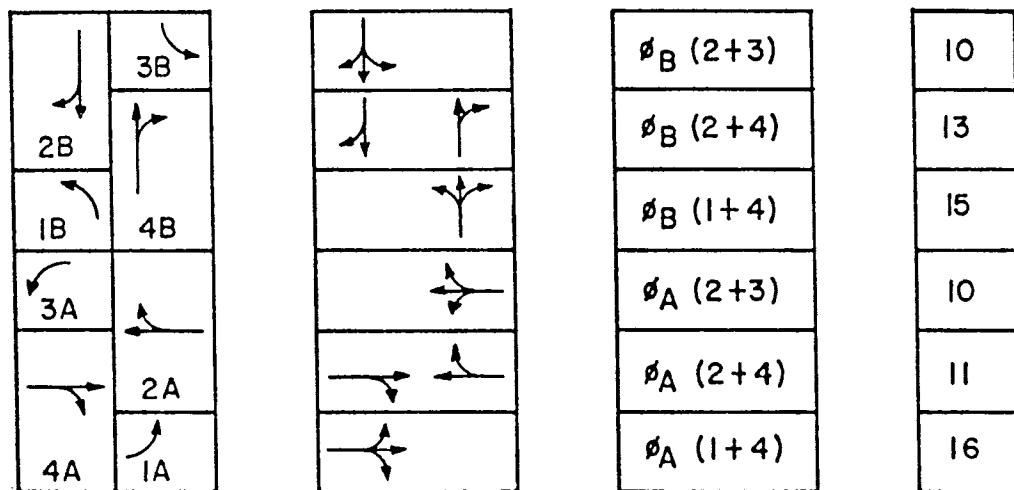
9-B SIGNAL MOVEMENTS, PHASING, NUMBERING, AND TIMES

SIGNALIZATION OF EXAMPLE DESIGN
PROBLEM FROM APPENDIX B

FIGURE 9

interval may also be used at isolated intersections or where the street being crossed is wide. Also, the all red interval should be considered at intersections where the 85-percentile speed is 45 MPH or more. An all red interval can help at some intersections where there is accident experience.

For the example problem, it is assumed that the approach speeds are 40 MPH; therefore, a 4.0 second yellow clearance period has been chosen for each phase.

2. <u>Determine Pedestrian "Walk" Time</u>. The length of time provided for the pedestrian to start across the street is based on the number of pedestrians counted during the traffic volume count.

a. If there is a sufficiently high volume of pedestrians to justify the use of pedestrian "Walk" and "Don't Walk" signals, the "Walk" period should be 7 seconds or more in length.

b. If the pedestrian volume is relatively high but not high enough to justify the installation of "Walk" and "Don't Walk" signals, a minimum of 5 seconds is used for this initial interval.

c. If there is only an occasional pedestrian, a minimum of 3 seconds can be used for the initial interval.

3. <u>Determine the Pedestrian Clearance Time</u>. The pedestrian clearance time can be based on a pedestrian walking speed of 4 feet per second measured from the near side curb to half the width of the farthest lane or from the near side curb to the curb (or pavement marking) for a refuge island in the center of the street being crossed (see Manual on Uniform Traffic Control Devices). Also consider school children and senior citizens.

4. <u>Compute the Minimum Crossing Time</u>. The minimum crossing time is equal to the pedestrian "Walk" time plus the pedestrian clearance time. No major street phase should be less than 15 seconds in length. No left turn phase should be less than 10 seconds.

In the example, Street B is 64 feet wide and Street A is 84 feet wide. A pedestrian count shows that pedestrians only occasionally cross both streets.

Street A Movements 2 or 4:

$$3 \text{ sec. ("Walk" period)} + \frac{64' - 6'}{4 \text{ ft/sec}} = 3 \text{ sec.} + 15 \text{ sec.} = 18 \text{ sec.}$$

Street B Movements 2 or 4:

$$3 \text{ sec. ("Walk" period)} + \frac{84' - 6'}{4 \text{ ft/sec}} = 3 \text{ sec.} + 20 \text{ sec.} = 23 \text{ sec.}$$

Minimum phase times are as follows:

Street	Phase	Time	Subtotal	Total
A	1 + 4	10		
	2 + 4	8		
	2 + 3	10	28	
B	1 + 4	10		
	2 + 4	13		
	2 + 3	10	33	61

5. <u>Calculate Sum of Critical Lane Volumes as Defined in Table 4.</u>

Street A

$$V_{1A} = 140 \qquad V_{3A} = 47$$
$$V_{2A} = \frac{207}{347} \qquad V_{4A} = \frac{412}{459} = V_A$$

Street B

$$V_{1B} = 156 \qquad V_{3B} = 88$$
$$V_{2B} = \frac{232}{388} \qquad V_{4B} = \frac{390}{478} = V_A$$

Sum of critical lane volumes, ΣV, $= V_A + V_B = 459 + 478 = 937$

6. <u>Determine Minimum Delay Cycle Length From Figure 6.</u>

Given : $\Sigma V = 937$, $\phi = 4$ critical phases

Solution: Minimum delay cycle length; $C_o = 62$ seconds.

7. <u>Select Trial Cycle Length.</u> This cycle length should lie within the bounds: $0.85 C_o \le C \le 1.25 C_o$ if at all possible. It may be as long as $1.5 C_o$ if C_o is relatively short, as it might be for off-peak operations. Cycle lengths during off-peak periods should normally be from 40 to 60 seconds. Longer cycle lengths may be required during peak periods but the cycle length

should not exceed 90 seconds. A 75 second cycle will be required.

8. <u>Calculate Trial Phase Splits.</u> The phase splits are calculated from a modification of Webster's method for minimizing delay for a given cycle length.

a. Calculate phase split between Streets A and B. Phase lost times, L, are 4.0 seconds; n_A, n_B, and n_{A+B} are the number of critical lane volumes (phases) on streets A, B, and intersection total. (See p. 47).

$$\text{St. A} = \frac{V_A}{V_A + V_B}[C - n_{A+B} \cdot L] + n_A \cdot L = \frac{459}{459 + 478}[75 - (2 + 2) \cdot 4] + 2 \cdot 4 =$$

$$(0.49) \cdot (59) + 8 = 37 \text{ sec.}$$

St. B = C - St. A = 75 - 37 = 38 sec.

b. Check minimum phase split between streets A and B.

Street	Minimum	Trial	
A	28	37	(OK)
B	33	38	(OK)

c. Calculate trial green times for movements for Street A.

$$(G + Y)_{1A} = \frac{V_{1A}}{V_{1A} + V_{2A}}[\text{St. A} - n_A \cdot L] + L = \frac{140}{140 + 207}[37 - 2 \cdot 4] + 4 = 16 \text{ sec.}$$

$$(G + Y)_{2A} = \text{St. A} - (G + Y)_{1A} = 37 - 16 = 21 \text{ sec. } (\geq 18)$$

$$(G + Y)_{3A} = \frac{V_{3A}}{V_{3A} + V_{4A}}[\text{St. A} - n_A \cdot L] + L = \frac{47}{47 + 412}[37 - 2 \cdot 4] + 4 = 7 \text{ sec.}$$

$$(G + Y)_{3A} < 10.0 \text{ second left turn minimum; therefore } (G + Y)_{3A} = 10 \text{ sec.}$$

$$(G + Y)_{4A} = \text{St. A} - (G + Y)_{3A} = 37 - 10 = 27 \text{ sec. } (\geq 18)$$

d. Calculate trial green times for movements for Street B.

$$(G + Y)_{1B} = \frac{V_{1B}}{V_{1B} + V_{2B}}[\text{St. B} - n_B \cdot L] + L = \frac{156}{156 + 232}[38 - 2 \cdot 4] + 4 = 16 \text{ sec.}$$

$$(G + Y)_{2B} = \text{St. B} - (G + Y)_{1B} = 38 - 16 = 22 \text{ sec.}$$

This value is less than minimum crossing time of 23 sec. Therefore,

$(G + Y)_{2B} = 23$ sec. and $(G + Y)_{1B} = 38 - 23 = 15$ sec. (≥ 10)

$(G + Y)_{3B} = \dfrac{88}{88 + 390} [38 - 2 \cdot 4] + 4 = 10$ sec. (≥ 10)

$(G + Y)_{4B} = 38 - 10 = 28$ sec. (≥ 23)

Borrowing considerable green time from one movement to meet another movement minimum green may result in excessively large delay on those movements whose times were reduced as indicated in Figure 10. Sometimes the needed time can be obtained from the cross street. However, it may be necessary to: 1) increase the cycle length, 2) change the signal phasing, 3) add a pedestrian refuge island or 4) add additional lanes in order to provide an acceptable solution. (In this example, a 75 second cycle was required to provide L.O.S. "C" on movement 4A.)

    e. Calculate saturation ratio and level of service for each movement.

Movement	G+Y	g=G+Y-L	V	$X = \dfrac{V \cdot C}{g \cdot 1750}$	L.O.S.
1A	16	12	140	0.50	A
2A	21	17	207	.52	A
3A	10	6	47	.34	B
4A	27	23	412	.77	C
1B	15	11	156	.61	A
2B	23	19	232	.52	A
3B	10	6	88	.63	B
4B	28	24	390	.70	C

Level of Service "C" or better is provided on all movements (from Table 5). Thus, the signal timing plan will be acceptable.

    f. Signal timing plan. (Required clearance phases not shown).

Street	Ending Movement	Phase	Time, Sec.
A	1A	1 + 4	16
	4A	2 + 4	11
	2 & 3A	2 + 3	10
B	1B	1 + 4	15
	4B	2 + 4	13
	2 & 3B	2 + 3	10
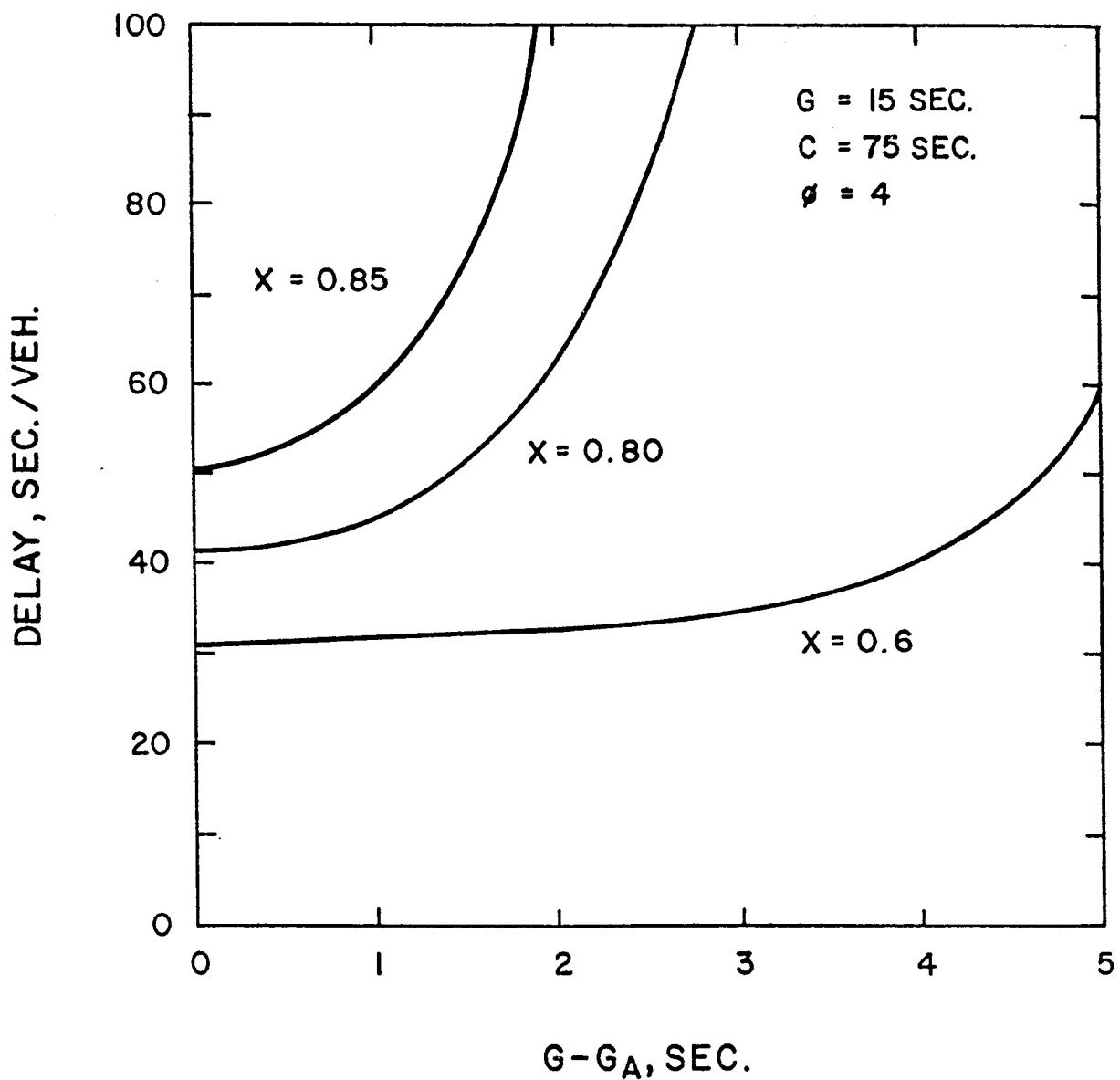
INCREASE IN DELAY AT INTERSECTION IF
ACTUAL GREEN TIME (G_A) IS DIFFERENT
FROM METHOD'S CALCULATED VALUE (G)
FOR EQUALLY LOADED MOVEMENTS

FIGURE 10

FIELD EVALUATION OF SIGNAL OPERATIONS

Sometimes it is desired to evaluate the operations of an existing signal system. Extensive field procedures frequently have been used to measure delays on the approaches to the intersection and other operational measures of effectiveness. The following procedure is presented to assist in evaluating operating conditions with a minimum of field personnel and to provide a basis for evaluating the level of service at pretimed signalized intersections.

Level of Service Measures

Table 8 presents the measures of effectiveness which are evaluated together with previously published level of service criteria for each. Different measures may yield slightly different levels of service when evaluated for the same approach, particularly when comparing delay with the other measures.

TABLE 8

LEVEL OF SERVICE CRITERIA FOR OPERATIONAL MEASURES
OF EFFECTIVENESS ON SIGNALIZED MOVEMENTS

OPERATIONAL MEASURE	LEVEL OF SERVICE					
	A	B	C	D	E	F
Saturation Ratio, X^{+}	$\leq.6$	$\leq.7$	$\leq.8$	$\leq.85^{y}$	≤1.0	>1.0
Probability of Clearing Queues, P_c^{*}	$\geq.95$	$\geq.90$	$\geq.75$	$\geq.50$	$<.50$	
Average Approach[x] Delay, d, sec./veh.	≤ 15	≤ 30	≤ 45	≤ 60	> 60	

Source: [+] Reference (9), [*] Reference (6), [x] Reference (10),

[y] Modified in this project.

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Field Data Collection

One person is required to collect the necessary field data for each approach studied at the same time. A skilled observer might be able to study more than one, however. The following data are required for the time period being evaluated:

1. Cycle length, C, in seconds.

2. Green time of movement, G, in seconds. (No yellow)

3. Calculate the effective red time, R, from C - G.

4. Measure the time it takes each queue to clear its approach after the start of green (visually average the lanes).

5. Record each time the phase clears the queue.

6. Calculate the average queue clearance time, T, in seconds.

Table 9 illustrates field data recorded for an approach during moderate rush hour traffic flow. Two cycles failed to clear the queues during this study.

Estimated Probability of Clearing Queues and Delay

Estimates of the probability of clearing queues using Miller's model and average vehicle delay in seconds per vehicle using Webster's approximation model (11) are obtained by applying the following procedure to Figure 11. These procedures will be illustrated for an example recorded data set presented in Table 9.

It is first necessary to calculate the average saturation ratio, X, existing on the approach during the study period from:

$$X = \frac{T - 2.0}{G} \cdot \frac{C}{R + T - 2.0} \tag{32}$$

where the variables are defined in Table 9. Thus, the saturation ratio is

$$X = \frac{15.0 - 2.0}{18} \cdot \frac{75}{57 + 15.0 - 2.0} = 0.77$$

TABLE 9

FIELD DATA COLLECTED FOR EVALUATING
OPERATING CONDITIONS AT A PRETIMED SIGNALIZED
INTERSECTION APPROACH

LOCATION: Southbound Texas Avenue at University
ESTIMATED SATURATION FLOW, S = 3400 VPHG
C = 75 sec., G = 18 sec., R = C - G = 57 sec.

Time of Day	Time to Clear Queue (sec.)	Queue Cleared?	Average Queue Clearance, T
5:00 p.m.	14	Yes	
	13	Yes	
	7	Yes	
	15	Yes	
	17	Yes	
	9	Yes	
	15	Yes	
	15	Yes	
	21	No	
	19	Yes	
	21	No	
	14	Yes	15.0 sec.
5:15 p.m.			

METHOD FOR ESTIMATING DELAY AND
PROBABILITY OF CLEARING QUEUES

FIGURE 11

Next, calculate the quantity: sG

$$sG = \frac{S \cdot G}{3600} = \frac{3400 \cdot 18}{3600} = 17.0$$

Draw a horizontal line across the upper central vertical scale at the calculated X (saturation) ratio value in Figure 11 (0.77). Extend this horizontal line to the right until the existing G/C ratio is reached. (The existing G/C ratio is 18/75 or 0.24.) Extend a line vertically downward and note the intercept value of f_1 (0.32). This value will be used later in the delay calculation.

Draw a vertical line across the left central horizontal scale, the sG scale, at the calculated value for sG (17.0). Extend this line first vertically until it intersects the previously drawn horizontal line for X. The intersection point is Miller's estimate for probability of clearing queues. Thus,

● Estimated P_c = 0.85 (85%)

This compares with an observed value of 10/12 = 0.83. Field comparisons of the probability of clearing queues made from one day's study may not always compare closely to theoretical values due to the binary nature of queue clearances and the assumption made of Poisson arrival flow on the approach.

Next, extend the vertical line (sG = 17.0) downward in Figure 11 until the appropriate X (saturation) ratio curve is reached (0.77). Then extend a horizontal line from this point to intercept the vertical f_2 scale. Note the value of f_2 (0.10).

Calculate the value of delay as illustrated in Figure 11. The calculated value for the delay on the approach studied from 5:00 to 5:15 p.m. is:

$$d = (f_1 + f_2) \cdot C$$

$$d = (0.32 + 0.10) \cdot 75$$

$$d = 31.5 \text{ sec./veh.}$$

Level of Service Summary

Comparing the calculated values for the X (saturation) ratio (0.78), probability of clearing queues (0.85) and delay (31.5 sec./veh.) with those level of service limits given in Table 8 indicates that traffic flow conditions were at Level of Service "C". Field observations would confirm these indications.

It is envisioned that this procedure will provide an efficient but consistent field evaluation procedure for pretimed signalized intersections.

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APPENDIX A

EXAMPLES OF SIGNAL PHASINGS

Movement Sequence on Street A (or B)	Phasing Sequence	Phase Number Sequence	Number of Phases on Street	Number of Critical Phases (Movements) on Street "n"
		$\phi(1+2+3+4)$	1	1
		$\phi(2+4)$ $\phi(1+4)$	2	1 or 2
		$\phi(1+4)$ $\phi(2+4)$	2	1 or 2
		$\phi(2+4)$ $\phi(2+3)$	2	1 or 2
		$\phi(2+3)$ $\phi(2+4)$	2	1 or 2
		$\phi(2+4)$ $\phi(1+3)$	2	2
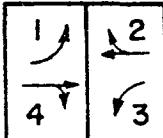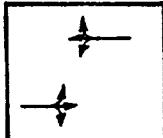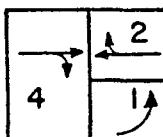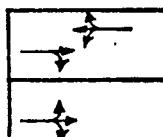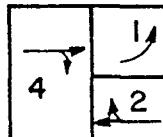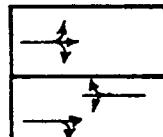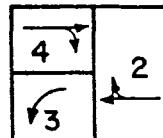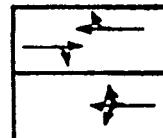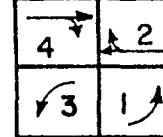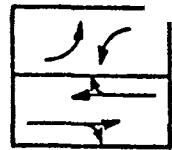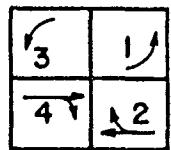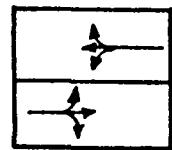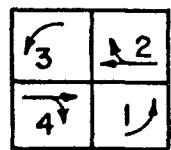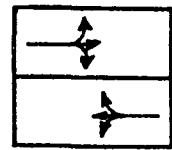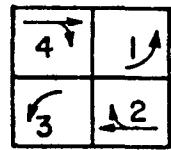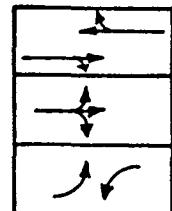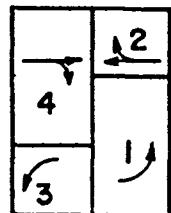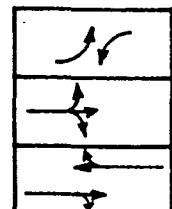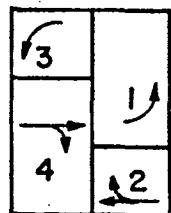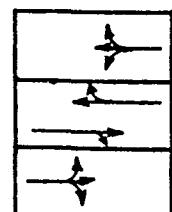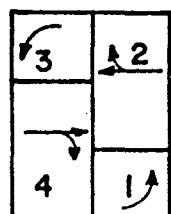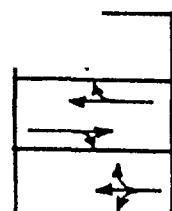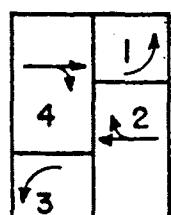
47

φ(1+3)

φ(2+4)   2   2

φ(2+3)

φ(1+4)   2   2

φ(1+4)

φ(2+3)   2   2

φ(2+4)

φ(1+4)   3   2

φ(1+3)

φ(1+3)

φ(1+4)   3   2

φ(2+4)

φ(2+3)

φ(2+4)   3   2

φ(1+4)

φ(1+4)

φ(2+4)   3   2

φ(2+3)

APPENDIX B

VOLUME DATA
(Equivalent Cars Per Hour)

PROPOSED DESIGN
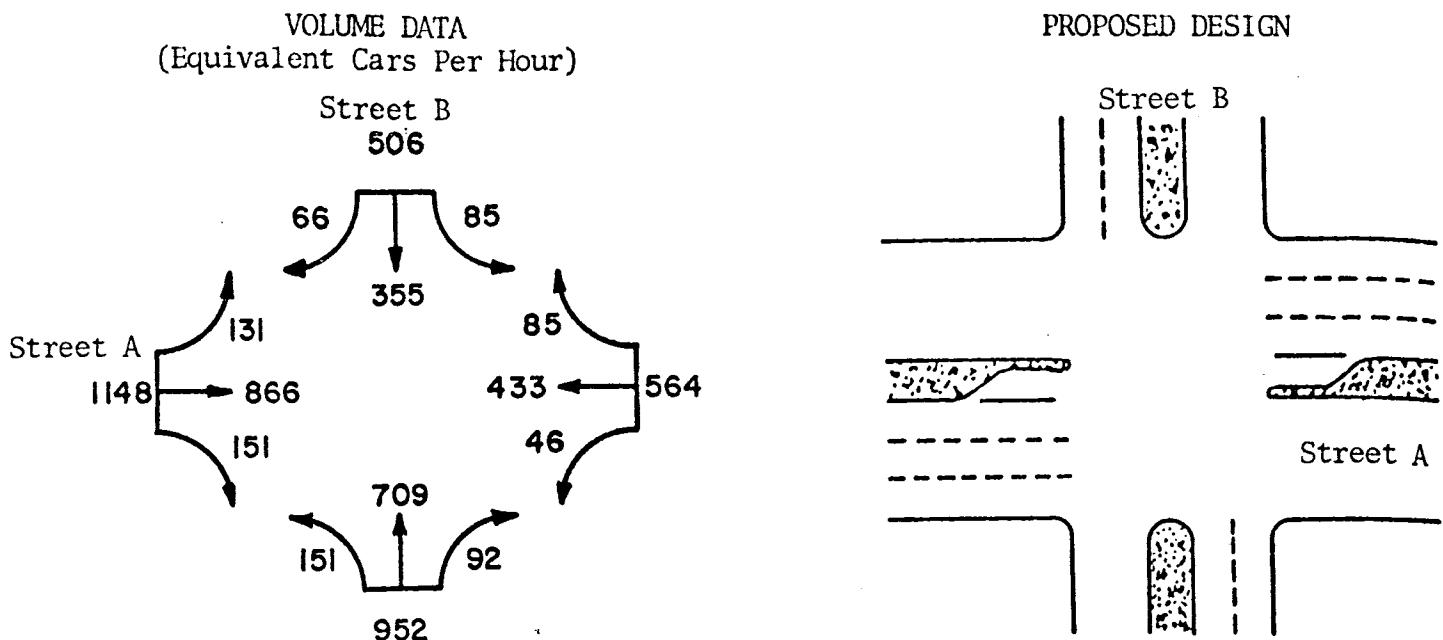
Given: Traffic volume data in equivalent cars per hour for the intersection as shown in Figure 1, Step 4.

All lane widths are 12 feet.

All left turn bay storage lengths are 100 feet.

All right turn effects are neglected. (R = 0.0)

Street A Phasing: Three phases; dual lefts leading.

Street B Phasing: One phase.

Determine:

Acceptability of proposed design.

Solution:

Left turn adjustment factors.

$$L_{1A} = \frac{1700 \times 1.03}{1640} - 1.0 = 0.07 \qquad L_{1+4B} = (151/952) \times (3.9 - 1.0) = 0.46$$

$$L_{3A} = \frac{1700 \times 1.03}{1700} - 1.0 = 0.03 \qquad L_{2+3B} = (85/506) \times (11.0^* - 1.0) = 1.68$$

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

* Not recommended for design.

Adjusted volumes. $[GDV_m = U \cdot W \cdot TF \cdot GMV_m]$

Q_{1A} = 1.0 x 1.0 x 1.07 x 131 = 140

Q_{2A} = 1.2 x 1.0 x 1.0 x 518 = 622

Q_{3A} = 1.0 x 1.0 x 1.03 x 46 = 47

Q_{4A} = 1.2 x 1.0 x 1.012 x 1017 = 1235

Q_{1+4B} = 1.1 x 1.0 x 1.46 x 952 = 1529

Q_{2+3B} = 1.1 x 1.0 x 2.68 x 506 = 1492

Design volume per lane for each movement. $[V_m = GDV_m \div N]$

V_{1A} = 140 V_{1+4B} = 765

V_{2A} = 207 V_{2+3B} = 746

V_{3A} = 47

V_{4A} = 412

STREET B

Sum of critical lane volumes.

V_{3A} =   47

V_{4A} = 412

V_{1+4B} = 765

            1224 > 1140

Therefore, unacceptable design.

746 746

207
207
207

140   47

412
412
412

765 765

STREET A
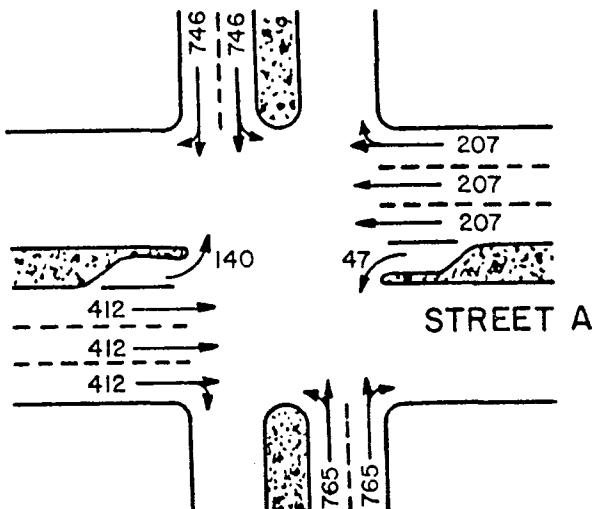
EXAMPLE PROBLEM NO. 2

VOLUME DATA
(Equivalent Cars Per Hour)
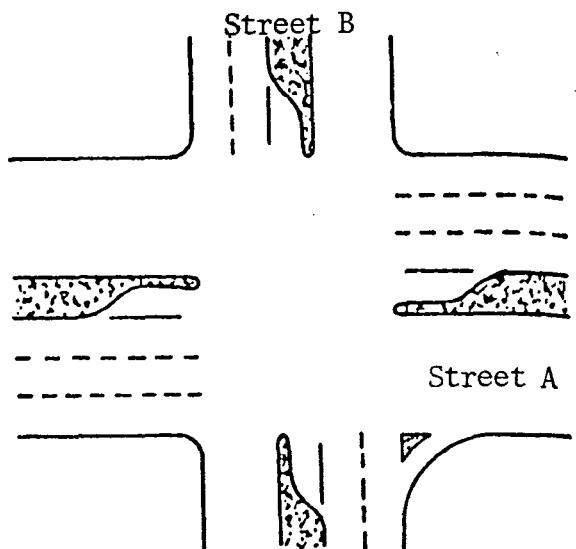
PROPOSED DESIGN

Street B
506

Street B

66   85

355

Street A   131   85

1148   866   433   564

151   46

709

151   92

952

Street A

Given: Traffic volume data in equivalent cars per hour for the intersection as shown in Figure 1, Step 4.

All lane widths are 12 feet.

All left turn storage lengths are 100 feet, except for 1B (150').

All right turn effects are neglected.   (R = 0.0)

Street A Phasing: Three phases; dual lefts leading.

Street B Phasing: Three phases, dual lefts leading.

Determine:

Acceptability of proposed design.

Solution:

Left turn adjustment factors.

$$L_{1A} = \frac{1700 \times 1.03}{1640} - 1.0 = 0.07 \qquad L_{1B} = \frac{1700 \times 1.03}{1700} - 1.0 = 0.03$$

$$L_{3A} = \frac{1700 \times 1.03}{1700} - 1.0 = 0.03 \qquad L_{3B} = \frac{1700 \times 1.03}{1700} - 1.0 = 0.03$$

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

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Adjusted volumes. $[GDV_m = U \cdot W \cdot TF \cdot GMV_m]$

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

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

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

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

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

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

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

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

Design volume per lane for each movement. $[V_m = GDV_m \div N]$

$V_{1A} = 140$	$V_{1B} = 156$
$V_{2A} = 207$	$V_{2B} = 232$
$V_{3A} = 47$	$V_{3B} = 88$
$V_{4A} = 412$	$V_{4B} = 390$

STREET B

Sum of critical lane volumes.

$V_{3A} = 47$

$V_{4A} = 412$

$V_{3B} = 88$

$V_{4B} = \underline{390}$

$\qquad 937 \; < \; 1100$

Therefore, acceptable design.

STREET A