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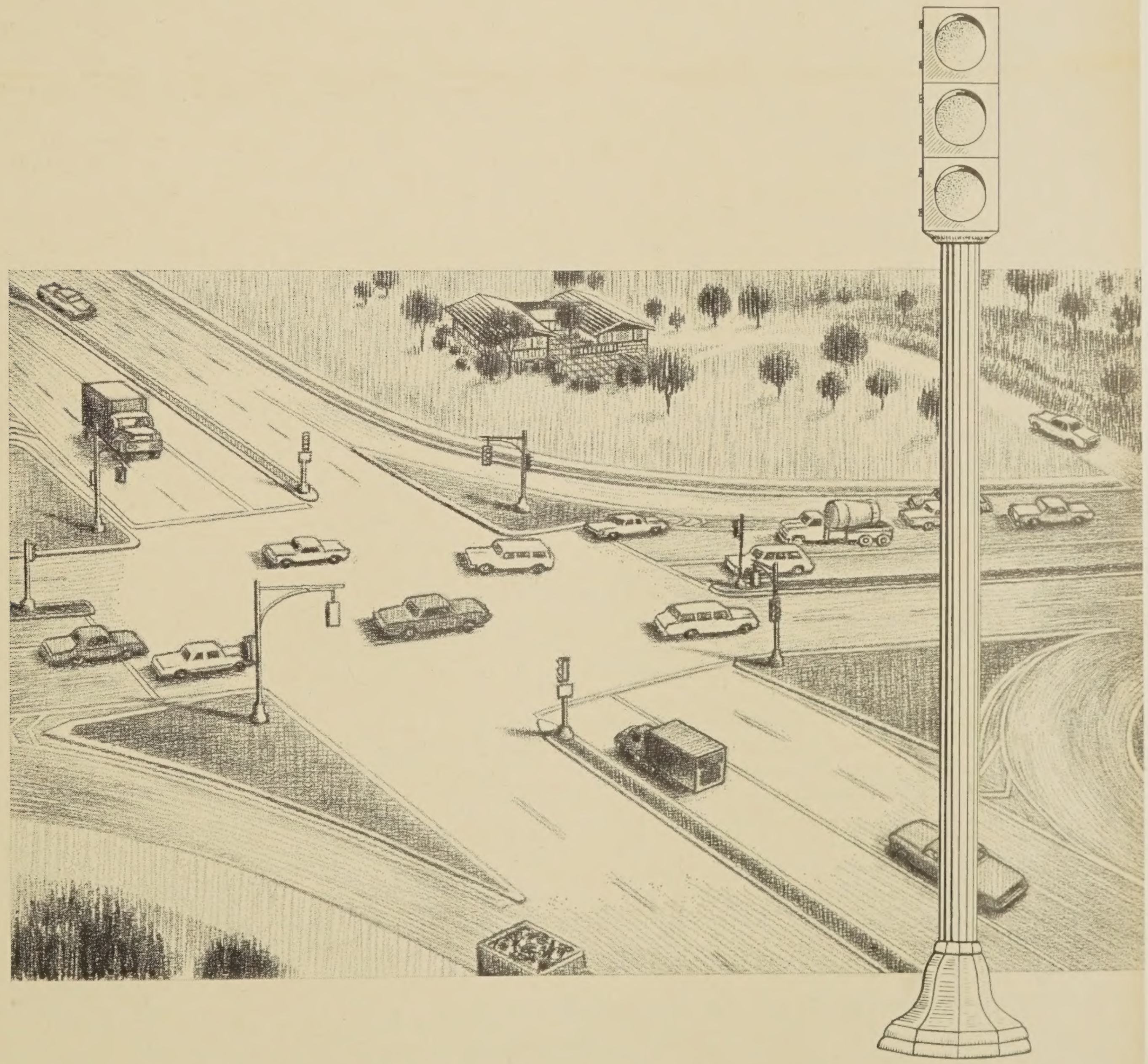

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SPECIAL ISSUE

Design capacity charts for signalized intersections

IN THIS ISSUE

Capacity Analysis Techniques for Design of Signalized Intersections

U.S. DEPARTMENT OF TRANSPORTATION
ALAN S. BOYD, Secretary

FEDERAL HIGHWAY ADMINISTRATION
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Capacity Analysis Techniques for Design of Signalized Intersections

Installment No. 1

ponsored by the
FFICE OF ENGINEERING AND OPERATIONS
UREAU OF PUBLIC ROADS

by [1] JACK E. LEISCH, Vice President and Chief Highway Engineer, DeLeuw, Cather & Co. of Canada Ltd.

INTRODUCTION

After publication of the 1950 Highway Capacity Manual, the author presented a procedure for the graphic solution of the capacity of signalized intersections to simplify the work required by the computational procedures in the Manual. Publication of the 1965 Highway Capacity Manual has provided a revised and comprehensive basis for computations of signalized intersections. Many users of the graphic procedure had expressed a desire for revised charts that reflected the revisions in the new Manual.

In this article, which will be presented in two installments, the author has again filled the need for a graphic procedure incorporating current knowledge. In addition to updating the original charts, new charts have been prepared to cover capacity procedures for which calculations previously required extensive application of judgment. Full discussion of the principles and procedures in the application of the charts in addition to sample problems have been included. The information presented provides a graphic procedure for the capacity analysis of most signalized street and highway intersections.

The current set of charts comprises 20 nomographs. Eighteen of the nomographs are presented in this first installment together with the appropriate application procedures and sample problems. The other two nomographs and the remainder of the article will be included in the second installment to be published in the October issue, Vol. 34, No. 10, of PUBLIC ROADS.

HIGHWAY CAPACITY has become recognized as an essential discipline of highway planning and design. Its importance became apparent with the publication of the 1950 *Highway Capacity Manual* (*1*),[2] which has been superseded by a 1965 *Highway Capacity Manual* (*2*), hereafter referred to as the *Manual*. This new publication was based on considerable research and operational experience; it is a sophisticated and highly useful tool for the traffic and highway engineer in the planning, design, and operation of highways and highway systems.

Capacities of streets and the many interchanges associated with freeways in urban areas are determined largely by the at-grade intersections and ramp terminals. To make less cumbersome the capacity analyses required for design of intersections, a set of nomographs based on the 1950 *Highway Capacity Manual* was prepared and published in a 1951 issue of *Public Roads, A Journal of Highway Research* (*3*). Because of the popularity of the nomographic procedure and its adoption by numerous agencies, a revised set of charts, based on the data of the 1965 *Manual* has been developed. These charts and explanations for their use are reproduced in this article.

Development of Capacity Charts

The nomographs printed here generally conform to the format of those originally charted (*3*) so that a person familiar with the previous charts can use the new charts without having to learn a different procedure. The nomographs have been devised to simplify and reduce the work that otherwise would be required by the long-hand, computational procedures set out in chapter 6 of the *Manual*.

The basic data for intersection capacities are shown in figures 1 and 2 for 1-way and 2-way streets, without parking and with parking, expressed in terms of volumes on one approach—vehicles per hour of green—observed for different conditions of loading, metropolitan area size, peak hour factor, and location within the metropolitan area. The results are representative of average conditions derived from actual observations of some 1,100 intersection approaches. The average conditions are indicative of 10-percent right turns, 10-percent left turns, 5-percent trucks and through buses, and no local bus stops for pickup and discharge of passengers in the vicinity of the intersection approach.

Attainable volume of discharge (service volume) per approach, shown in figures 1 and 2, for a given or desired degree of loading must be adjusted for specific conditions. The additional adjustment factors are applied as multipliers in accordance with the values and instructions described in the *Manual*, pages 138–143. These include adjustments for the G/C ratio (signal timing), turning movements, proportion of trucks and through buses, and

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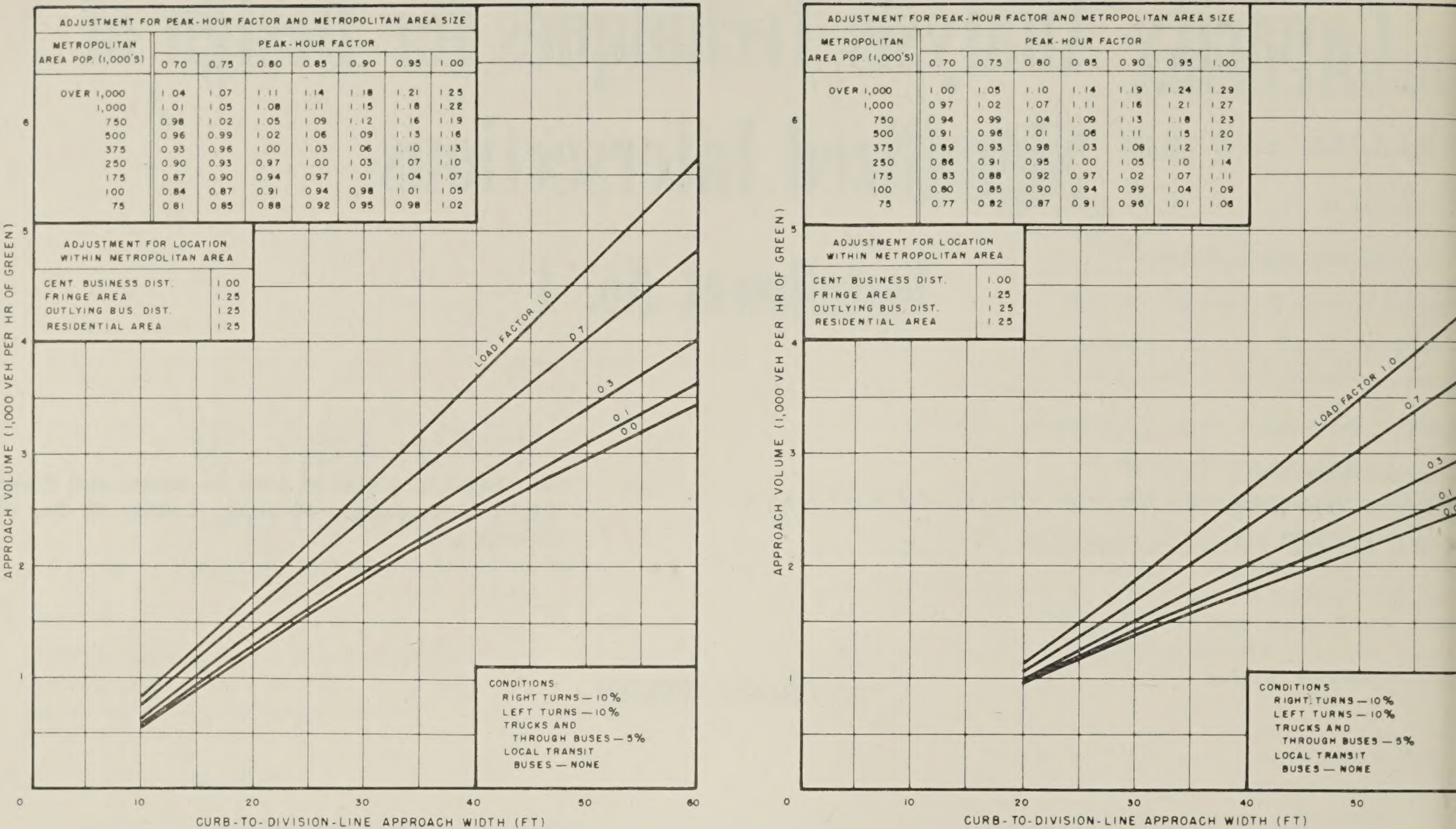
[2] References identified by italic numbers in parentheses are listed on p. 208.



A. Two-way street without parking (Manual Fig. 6.8)

B. Two-way street with parking (Manual Fig. 6.9)

Figure 1.—Urban intersection approach service volumes, basic data for 2-way streets.

bus stops. The charts presented here incorporate all of these adjustments, so that for any known condition, the intersection capacity can be obtained directly without reference to the *Manual* adjustment values. In constructing the charts, all of the adjustments have been precisely accounted for and no short cuts or approximations have been made, except for slight consolidation of values in the right- and left-turn adjustments in charts 3–15. Therefore, the results obtained by the use of these charts, for all practical purposes, are the same as those that would be obtained by use of the method in the *Manual*. Several refinements and additional (rational) procedures for conditions not covered in the *Manual* are also presented. These are included under *Special Conditions, Part 3,* and in charts 17 and 18 for separate turning lanes.

Capacity and Levels of Service

The *1950 Highway Capacity Manual* (*1*) related the ability of a highway to carry traffic to two levels—*practical capacity* and *possible capacity*. *Practical capacity* is the maximum number of vehicles that can be accommodated under prevailing roadway and traffic conditions without unreasonable delay or restriction to the driver's freedom to maneuver. *Possible capacity* represents the maximum number of vehicles that can be accommodated under prevailing roadway and traffic conditions, regardless of the effect of delaying drivers and restricting their freedom to maneuver.

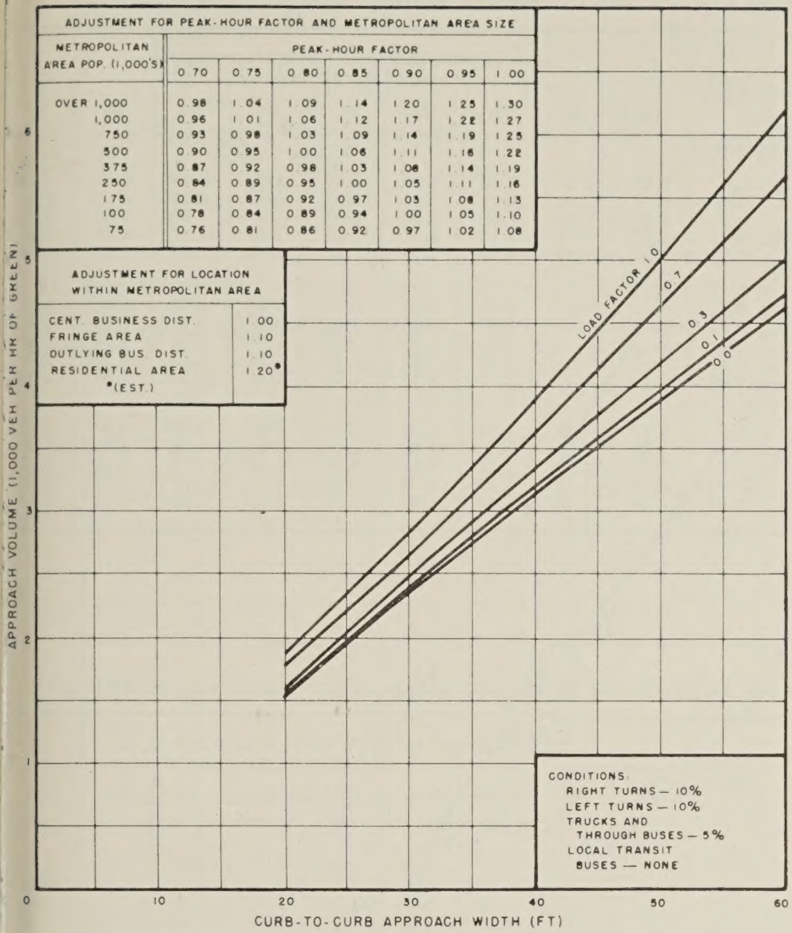
Design policies of the American Association of State Highway Officials (AASHO), published during the 1950's, accepted this concept and used the same terms for *capacity*. The AASHO practice, however, produced an additional term *design capacity* defined as the *practical capacity* or lesser value determined for use in designing the highway to accommodate the design volume. The original capacity charts (*3*) were predicated on *design capacity*, an adjusted value generally less, numerically, than *practical capacity*, whereas *possible capacity* was evaluated by application of appropriate conversion factors to *design capacity*.

The *Manual* (*2*) introduced the *level of service* concept, eliminated the terms *practical capacity* and *design capacity*, and substituted the single word *capacity* for what had been referred to as *possible capacity*. The six *levels of service* A through F designated in the *Manual* are a qualitative measure of operating conditions from excellent to intolerable, including *capacity*. *Level of service* constitutes the composite effect of speed and traveltime, traffic interruptions, freedom to maneuver, safety, driving comfort and convenience, and operating costs. An attainable hourly volume of traffic, or a maximum *service volume*, is designated in the *Manual* for each *level of service*. A *service volume* is the maximum number of vehicles that can be accommodated during a specified time period while operating conditions are maintained that correspond to the selected or specified *level of service*.

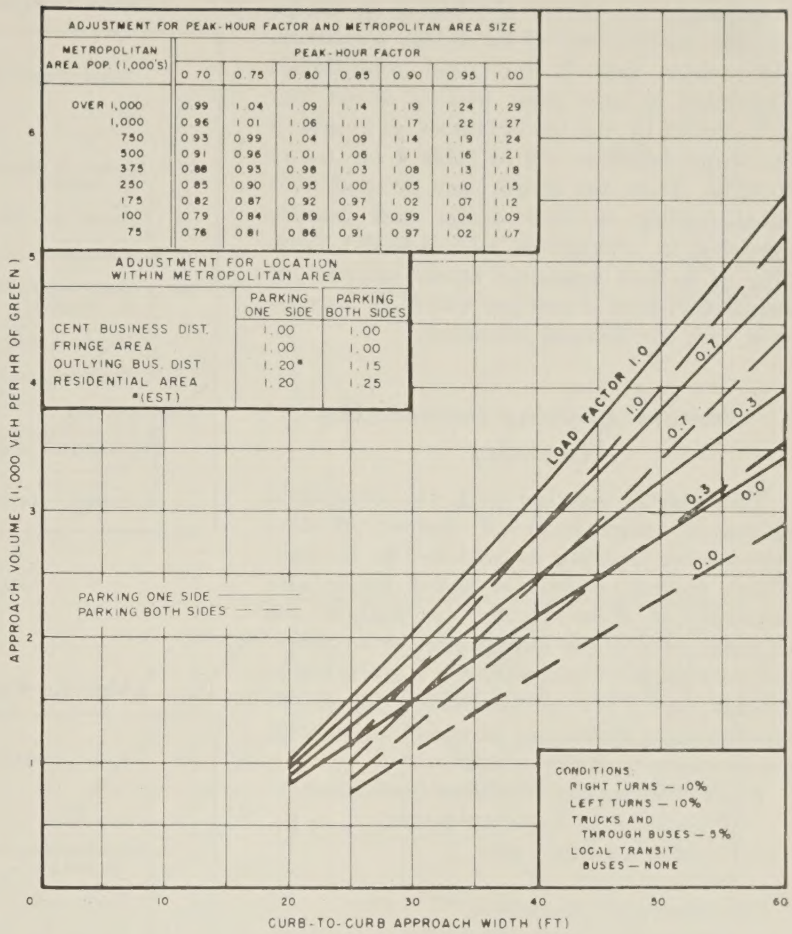
Table 1.—Levels of service as related to load factor for individual isolated intersection approaches [1]

Level of service	Traffic flow description	Load factor
A	Free flow	0.0
B	Stable flow	≤0.1
C	Stable flow	≤0.3
D	Approaching unstable flow	≤0.7
E	Unstable flow—capacity	≤1.0
F	Forced flow	-------

[1] *Highway Capacity Manual*, 1965, p. 131.



A. One-way street without parking (Manual Fig. 6.5)



B. One-way street with parking (Manual Figs. 6.6 and 6.7)

Figure 2.—Urban intersection approach service volumes, basic data for 1-way streets.

The 1965 AASHO Policy on Geometric design of Rural Highways (*4*) refers to the *Manual* for basic values, but as in the previous policies, continues to use the terms *design capacity* and *possible capacity*. Although the terminology is different, the overall concepts in each publication are compatible. For example, the AASHO terminology *design capacity* is the same in essence as the *Manual* terminology *maximum service volume* for a selected *level of service*. Also, numerically, the AASHO terminology *possible capacity* is identical to the *Manual* terminology *capacity*. The relations presented in the charts, therefore, are equally applicable to the *Manual* and to the AASHO procedures.

The *load factors* ranging from 0.0 to 1.0, as shown on the curves of the basic data in figures 1 and 2, are indicative of *levels of service*. The *load factor* is a ratio of the number of green signal intervals that are fully utilized by traffic during the peak hour to the total number of green intervals for that approach during the same period. For intersection conditions, the *Manual* considers the load factor as an appropriate measure of the *levels of service*, since the loading is something the driver sees and interprets in terms of degree of congestion. The relation is shown in table 1.

Level of service B—*load factor* of not more than 0.1—is considered in the *Manual* to be suitable for design of intersections under typical rural conditions. *Level of service* C—*load factor* of not more than 0.3—normally is recommended for design of intersections in urban areas. *Level of service* E with operation at a load factor of 0.85 is taken to be representative of *possible capacity*. Although a factor of 1.0 sometimes may be approached, a lesser factor such as 0.85 generally indicates the maximum loadings that can be achieved repetitively and sustained over a period of 1 hour. Using these load factors, the relation between *design capacity* and *possible capacity*, for rural intersections and for different street types in urban areas, are summarized in table 2. The values shown are the ratios of attainable volumes per hour of green (average conditions) at 0.3 load factor (0.1 for rural conditions) to the attainable volumes at 0.85 load factor. Therefore, *design capacity* can be converted directly to *possible capacity* by multiplying *design capacity* by the appropriate factor (*f*) in table 2.

Table 2.—Factor *f* for conversion of design capacity to possible capacity

Street type and parking conditions	Factor *f* [1] when W_A, in feet, is—								
	10	15	20	25	30	35	40	50	60
2-way:									
No parking	1.20	1.20	1.20	1.20	1.21	1.23	1.25	1.27	1.30
With parking	------	------	1.10	1.14	1.18	1.21	1.25	1.31	1.34
Rural [2]	1.28	1.28	1.28	1.30	1.32	1.35	1.38	1.41	1.44
1-way:									
No parking	------	------	1.15	1.13	1.12	1.12	1.13	1.15	1.17
Parking one side	------	------	1.10	1.13	1.16	1.18	1.20	1.25	1.30
Parking both sides	------	------	------	1.25	1.25	1.25	1.27	1.32	1.37

[1] Ratio of attainable volume at 0.85 load factor to that at 0.30 load factor, except for rural conditions
[2] Ratio of attainable volume at 0.85 load factor to that at 0.10 load factor

Although the charts are based on *design capacity* (*service level* C for urban conditions and *service level* B for rural conditions), conversion to any other *level of service* can be achieved by the use of factors (f) in table 3 for 2-way facilities and in table 4 for 1-way facilities. Thus, the charts may be used with equal facility to find *design* and *possible capacities* in accordance with AASHO practice, or to find *maximum service volumes* for any desired level of service, A to E, in accordance with the *Manual* procedure.

Factors Affecting Intersection Capacity

The charts, together with the designated procedure, incorporate all factors affecting intersection capacity covered in the *Manual*. These factors, which are listed below, also indicate the data needed to analyze the capacity of a given intersection or to determine its required geometry for a given traffic volume, and they are accounted for in the charts either directly or in combination with supplementary charts or tables.

Basic Physical and Operating Conditions:
- One-way or two-way operation,
- Parking condition, and
- Width of approach.

Environmental Conditions:
- Metropolitan area population,
- Location within metropolitan area,
- Peak-hour factor, and
- Load factor.

Traffic Characteristics:
- Traffic volume to be served on each approach,
- Turning movements,
- Trucks and through buses, and
- Local buses (bus stops).

Control Measures:
- Traffic signals—functional type and phasing—and
- Degree of channelization and approach lane markings.

Table 3.—Factor *f* for adjustment to various levels of service, 2-way facilities

Level of service	Load factor	Factor f when approach width W_A, in feet, is—								
		10	15	20	25	30	35	40	50	60
2-way street—no parking:										
A No backlog	0.0	0.85	0.86	0.87	0.88	0.89	0.90	0.89	0.87	0.85
B	0.1	0.90	0.91	0.91	0.92	0.92	0.92	0.93	0.92	0.91
C Design capacity	0.3	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
D	0.7	1.14	1.14	1.14	1.14	1.15	1.16	1.17	1.18	1.20
E Possible capacity	0.85	1.20	1.20	1.20	1.20	1.21	1.23	1.25	1.27	1.30
2-way street—with parking:										
A No backlog	0.0	----	----	0.95	0.93	0.91	0.89	0.88	0.86	0.84
B	0.1	----	----	0.97	0.96	0.95	0.94	0.93	0.91	0.89
C Design capacity	0.3	----	----	1.00	1.00	1.00	1.00	1.00	1.00	1.00
D	0.7	----	----	1.06	1.09	1.11	1.14	1.17	1.22	1.24
E Possible capacity	0.85	----	----	1.10	1.14	1.18	1.21	1.25	1.31	1.34
Rural highway:										
A No backlog	0.0	0.92	0.95	0.96	0.96	0.97	0.97	0.97	0.97	0.96
B Design capacity	0.1	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
C	0.3	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11
D	0.7	1.21	1.21	1.22	1.23	1.25	1.27	1.29	1.31	1.33
E Possible capacity	0.85	1.28	1.28	1.28	1.30	1.32	1.35	1.38	1.41	1.44

Table 4.—Factor *f* for adjustment to various levels of service, 1-way facilities

Level of sevice	Load factor	Factor f when approach width W_A, in feet, is—								
		10	15	20	25	30	35	40	50	60
1-way street—no parking:										
A No backlog	0.0	----	----	0.95	0.95	0.95	0.94	0.94	0.94	0.93
B	0.1	----	----	0.97	0.97	0.97	0.96	0.96	0.96	0.95
C Design capacity	0.3	----	----	1.00	1.00	1.00	1.00	1.00	1.00	1.00
D	0.7	----	----	1.12	1.09	1.07	1.07	1.08	1.11	1.13
E Possible capacity	0.85	----	----	1.15	1.13	1.12	1.12	1.13	1.15	1.17
1-way street—parking one side:										
A No backlog	0.0	----	----	0.90	0.89	0.89	0.89	0.88	0.87	0.86
B	0.1	----	----	0.93	0.93	0.93	0.93	0.92	0.91	0.90
C Design capacity	0.3	----	----	1.00	1.00	1.00	1.00	1.00	1.00	1.00
D	0.7	----	----	1.07	1.08	1.10	1.12	1.14	1.17	1.22
E Possible capacity	0.85	----	----	1.10	1.13	1.16	1.18	1.20	1.25	1.30
1-way street—parking both sides:										
A No backlog	0.0	----	----	----	0.88	0.85	0.84	0.83	0.83	0.82
B	0.1	----	----	----	0.91	0.90	0.90	0.89	0.88	0.88
C Design capacity	0.3	----	----	----	1.00	1.00	1.00	1.00	1.00	1.00
D	0.7	----	----	----	1.17	1.17	1.17	1.18	1.22	1.25
E Possible capacity	0.85	----	----	----	1.25	1.25	1.25	1.27	1.32	1.37

Design of Nomographs

Chart makeup

The charts presented here for the graphic solution of intersection capacities are nomographs of the stepped variety. They perform a series of multiplications and algebraic additions necessary for various adjustments to find design capacities for given roadway and traffic conditions. The first two charts are simple arrangements for determining the design capacity of one approach to a signalized intersection for average conditions. Chart 1 is for 2-way, and chart 2 for 1-way, facilities. The upper part of each nomograph is a plot of the curves from figures 1 and 2 with a load factor of 0.3 for the different types of facilities (except for rural highways), parking conditions, and location within the metropolitan area. The curves relate the approach width to design capacity for average conditions in terms of vehicles per hour of green. The lower part of the chart is a proportional graph that converts—for a given signal timing—the design capacity to a volume in vehicles per hour. The third graph unit on the right adjusts this volume to a given metropolitan size.

The two charts are applicable to situations where only approximate solutions are required or where specific traffic characteristics are not known. They also form the basis for developing additional nomographs for specific conditions. The upper curves, five in chart 1 and eight in chart 2, are the basis for the 13 detailed nomographs in charts 3–15. Each curve in the succeeding charts, expanded to a family of curves representing various percentages of trucks in the traffic stream, forms the upper section of a separate nomograph. The signal-timing-adjustment (G/C ratio) part of the graph in charts 1 and 2 is used to form the last section of the succeeding charts. The intermediate parts of the nomographs, charts 3–15, account successively for the effects of right turns, left turns, and metropolitan area size.

These nomographs are supplemented by: chart 16, which provides adjustments for conditions where there are bus stops at the intersection; chart 17, which determines capacities of separate right- and left-turn lanes **without** separate signal indication; and chart 18, which determines capacities of separate right- and left-turn lanes **with** separate signal indication.

The last two nomographs, charts 19 and 20 (to be published in Oct. 1967 issue), are designed for use in planning street systems and in preliminary design, or for review of plans where approximate but quick solutions are desired in terms of total or overall intersection capacity. The charts are augmented by several tables and special conditions that can be used for complete analyses of practically any form of signalized intersection problem.

Definitions and chart terminology

The factors affecting capacity, referred to in the charts, are defined in the following statements. To reduce these factors to simple terms on the charts and in the examples, a system of symbols is employed. The factors used at the outset to organize the charts are (1) 1-way or

EXAMPLE

Given

Two-way, 66-ft Street
No Parking

Metro Pop. — 400,000
Fringe Area

$W_A = 33'$

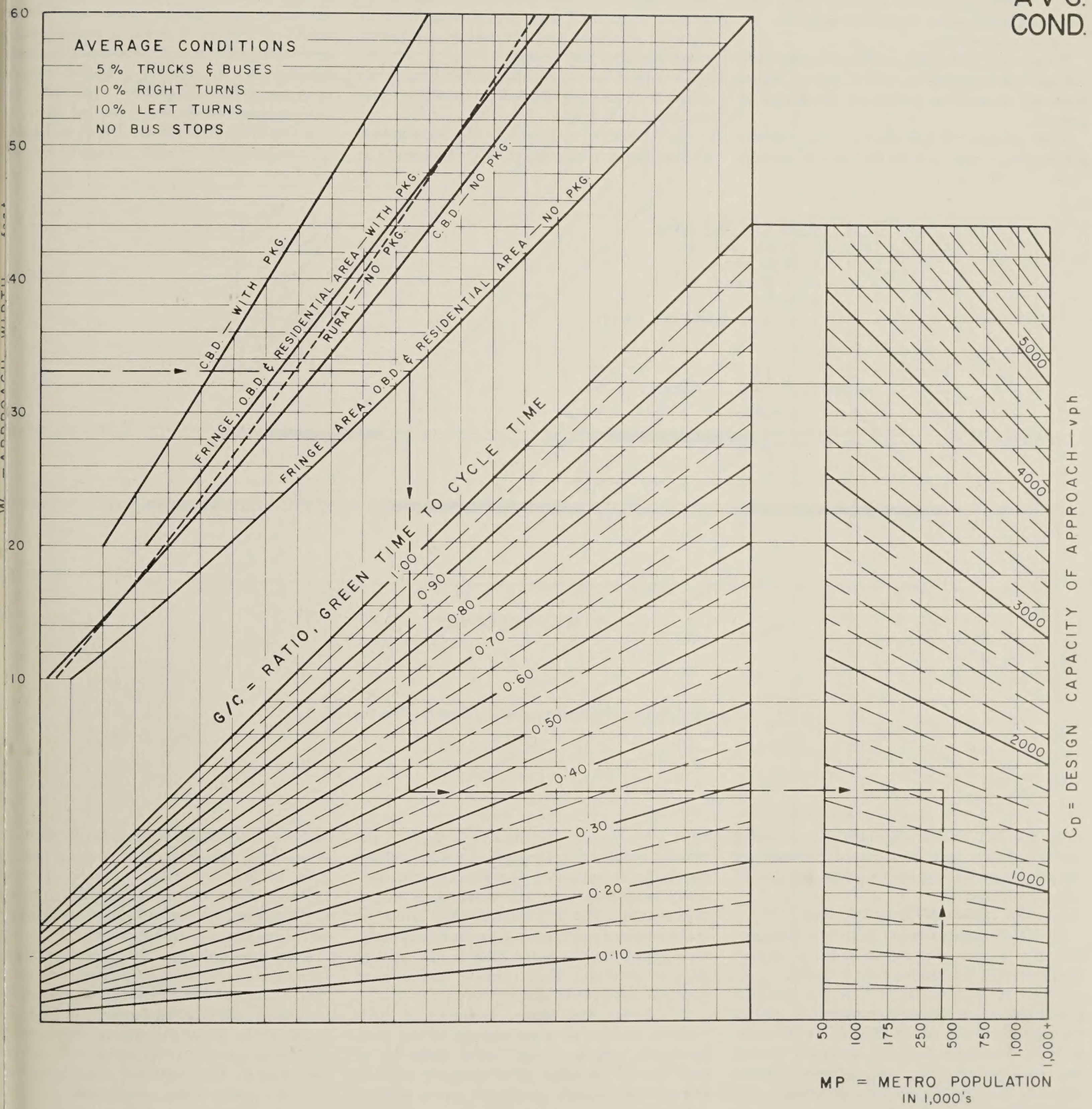
$G/C = 0{\cdot}50$

Solution

$C_D = 1500$ vph

1

AVG. COND.



DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

TWO-WAY FACILITIES — AVERAGE CONDITIONS

CHART I

2-way operation, (2) parking conditions, and (3) location within the metropolitan area. The various combinations of the three conditions produce the 13 separate charts numbered 3–15. These conditions are:

1-way or 2-way Operation

There are major differences in the operation of 1-way and 2-way approaches which are reflected in the capacities attained. Because of this, analysis procedures for the 1-way and 2-way approaches are handled separately.

Parking Conditions

No parking (*NO PKG*) is defined as no standing and no stopping on the approach, other than an occasional passenger discharge or pickup.

With parking (*PKG*) means that vehicles are present, standing attended or unattended, which the principal land use is for business activity.

Residential Area (*RES*) is that portion of a municipality, or an area within the influence of a municipality, in which the dominant land use is residential development, but where small business areas may be included.

Area outside city environs (*RURAL*) is characterized by open country. This area is not related to metropolitan area but is grouped here as a type of area to be considered in capacity analyses and for which a separate chart has been developed.

The other factors accounted for directly on charts 3–15 and the symbols adopted for them are noted as follows:

W_A Width of approach to the intersection, in feet. For a 1-way street, it is normally the

MP Metropolitan area population and the peak-hour factor both have a significant effect on intersection capacity. Several pertinent points are noted:

• Nine population groups are considered in the *Manual*, including a variety of metropolitan areas from small single cities to widespread urbanized areas composed of several cities. These are listed in table B on charts 3–14. Higher capacities are associated with the larger metropolitan areas. Application of resultant criteria to satellite communities requires judgment and/or local study to establish whether the community is better considered as a separate independent small city or an outlying part of the large central city.

• The peak-hour factor (*PHF*) is a measure of consistency of traffic demand. For inter-

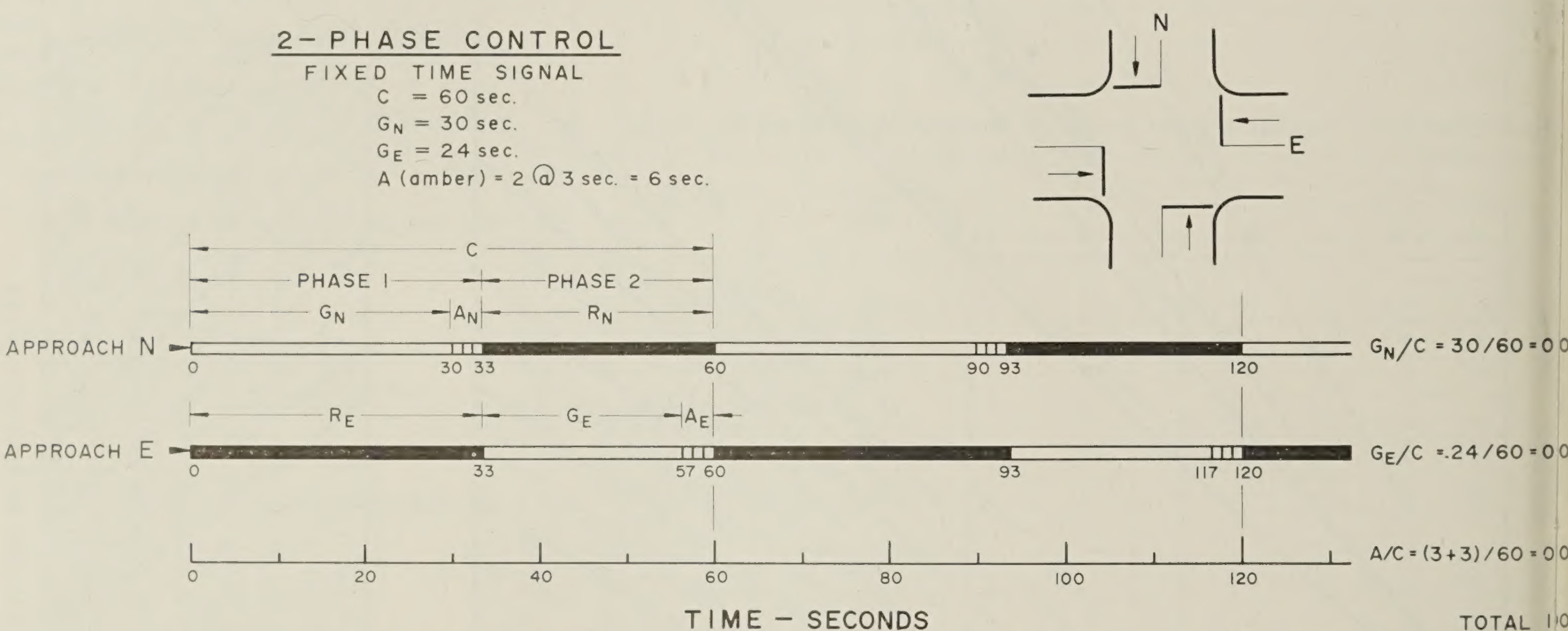


Figure 3.—Signal phasing and G/C ratio illustrated.

along the curb on the approach. For capacity considerations, only the actual presence or absence of parked vehicles applies, not posted parking regulations.

Location Within Metropolitan Area

Central Business District (*CBD*) is that portion of a municipality in which the dominant land use is for intense business activity.

Fringe Area (*FRNG*) is that portion of a municipality immediately outside the central business district in which there is a wide range in type of business activity, generally including small commercial, light industrial, warehousing, automobile service activities, and intermediate strip development, as well as some concentrated residential areas.

Outlying Business District (*OBD*) is that portion of a municipality or an area within the influence of a municipality, normally separated geographically by some distance from the central business district and its fringe area, in

total curb-to-curb width. For a 2-way, undivided street, it is the actual width utilized by traffic approaching the intersection, normally measured from the outer curb to the division line. For a 2-way, divided street, it is the width of the approach roadway, normally measured from the outer curb to the curb or edge of median. W_A is always exclusive of specially designated turning lanes.

T Trucks and buses, exclusive of light delivery trucks, as a percentage of the total approach volume. Local buses stopping to pick up and discharge passengers are counted along with through buses and are presented in the charts.

R Right-turning vehicles expressed as a percentage of the total volume on the approach.

L Left-turning vehicles expressed as a percentage of the total volume on the approach.

sections, it is defined as the ratio of the number of vehicles accommodated during the peak hour to four times the number of vehicles accommodated during the *highest* 15 consecutive minutes. *Higher* capacities are associated with the larger peak-hour factors.

• For convenience in the solution of problems, the effects of metropolitan area population and peak-hour factor are consolidated in a single adjustment factor (*MP*) on charts 3–14. Table B on the charts is entered first with metropolitan area population and peak-hour factor. The resultant adjustment factor is then applied in the nomograph.

• Sometimes the peak-hour factor may not be available, particularly on new facilities planned for the future. Then the application of the *MP* adjustment in the charts may be accomplished by the use of a metropolitan (metro) area population only, labelled on the curves. That is, population is used directly

EXAMPLE

Given

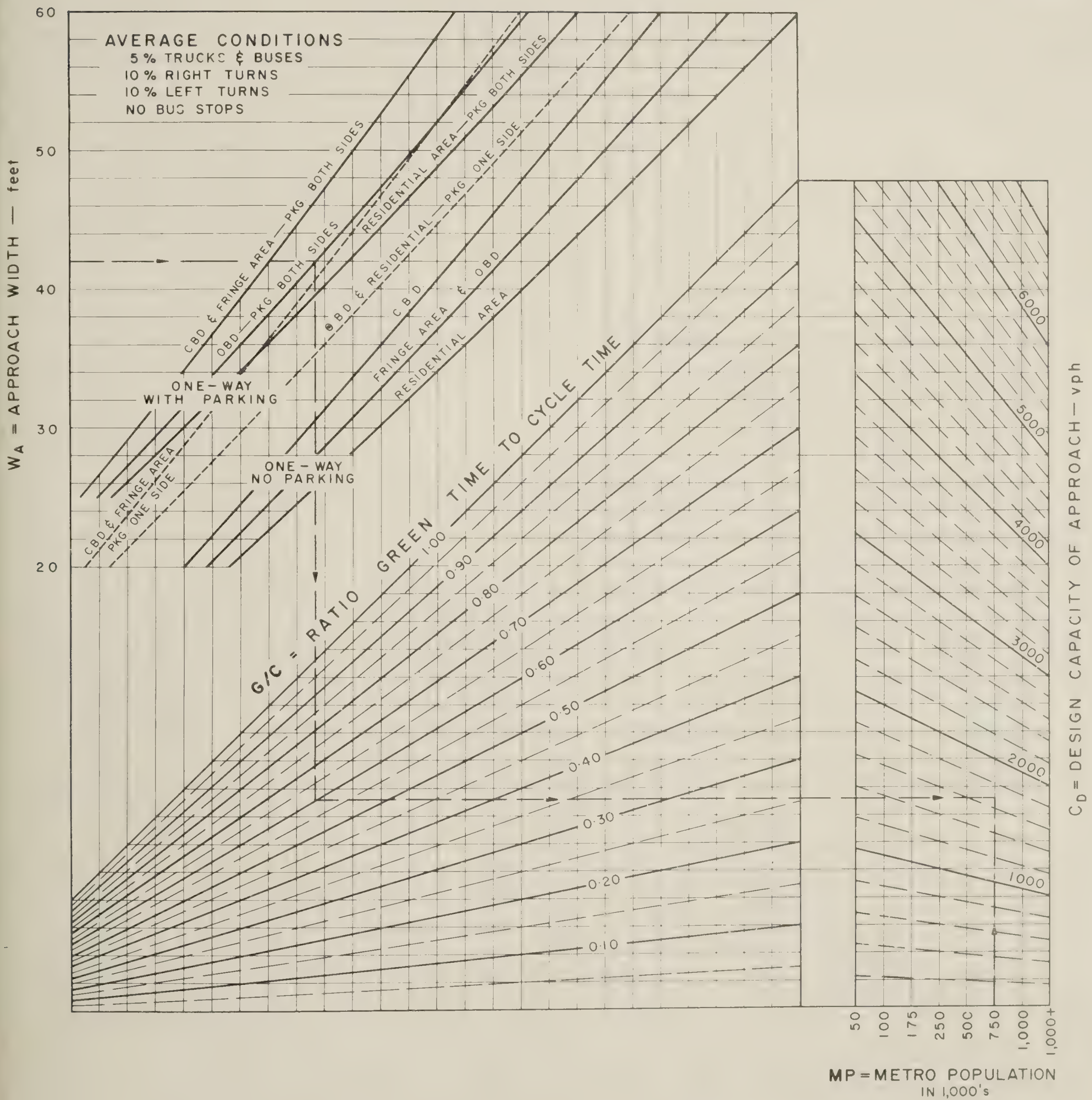
One-way, 42-ft Street
Parking Both Sides
Metro Pop. — 750,000
Outlying Business District
W_A = 42'
G/C = 0·60

Solution

C_D = 1700 vph
C_P = 1700 x 1·28 = 2170 vph
(Factor 1·28 from Table 2)

2

AVG. COND.



DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

ONE-WAY STREET — AVERAGE CONDITIONS

CHART 2

in the chart without using table B data. The indicated population size, however, incorporates an average or representative peak-hour factor. This built-in peak-hour factor ranges from 0.80 for a metro population of 50,000 to 0.90 for a metro population of more than 1 million, based on the averages of data collected for nearly 800 signalized urban intersections.

G/C Proportion of total time during the peak hour that the signal is green for the movement of traffic from the one approach. For a fixed-time signal, in which the length of intervals of the green and red indications are held constant throughout the hour, the hourly proportion of green for a given approach is measured directly by the ratio of G to C, where G is the green interval in seconds, and C is the total cycle (including green, amber, and red intervals) in seconds. Phase sequences and the formation of the G/C ratio are illustrated in figure 3 for a standard 2-phase signal. For 3-phase control, three bars would be shown, and three green, red, and amber intervals would constitute a cycle.

The signal phasing may be altered during different periods of the day. A 3-dial control, which provides different signal phasings for the a.m. peak period, the p.m. peak period, and the off-peak periods, is a popular form of fixed-time control in urban areas. The 3-dial signal setting allows for greater efficiency in utilization of the signal by fitting more closely the demand of traffic during the several periods throughout the day. For this reason, traffic information should provide one design hour volume (*DHV*) for the a.m. peak period and a separate *DHV* for the p.m. peak period. Additional peaks may be considered, such as those occurring during summer weekends, heavy truck operations, etc.

The G/C principle for analysis is also applicable, with some modification, to actuated control and to progressive signal systems. Procedures for other than fixed-time control are covered in part 3 under the heading *Signal Systems Other Than Fixed Time.*

PART 1—2-WAY FACILITIES

Intersections With Average Conditions

Capacity analyses for planning and preliminary design stages generally are accomplished in simplified form. As specific conditions relating to turning movements and truck percentages are not known under such circumstances, procedures using average conditions are appropriate. Chart 1 allows the determination of design capacities for conditions with and without parking for different locations within metropolitan areas of different sizes and for conditions within rural areas. Average conditions constitute 5-percent trucks and buses, 10-percent right turns, 10-percent left turns, and no bus stops.

Problem 1

What is the design capacity of a 2-way street, 66 feet wide curb-to-curb, with parking prohibited, in a fringe area within a metropolitan (Metro) size of 400,000 population? Major intersections are signalized. Specific data regarding commercial vehicles, turning movements, etc., are not known, but conditions are assumed to be average. Half the time during the hour can be allotted to green on this street.

Solution: Using $W_A = 66/2 = 33$ and $G/C = 0.50$, and following the arrows indicated in chart 1, it is determined that design capacity $C_D = 1{,}500$ v.p.h. in one direction. If parking were permitted, C_D would be 1,070 v.p.h.

Problem 2

A major street consisting of a narrow median and two 21-foot traveled ways with no parking, in a residential area within a metro size of 100,000 population, is to have a signal installed at a cross street. Conditions are assumed to be average. If the cross street requires a G/C of 0.35, what would be the design capacity of the major street? If the approach volume, *DHV*, is 1,350 v.p.h., what width of traveled way is required in each direction, without exceeding design capacity, assuming a lane width of not less than 11 feet?

Solution: Normally the amber periods, for purposes of capacity analyses, are considered to be 10% of the cycle. Therefore, G/C available for the major street approaches is $1.00 - 0.35 - 0.10 = 0.55$. Proceeding through chart 1 with $W_A = 21$, residential area without parking, $G/C = 0.55$, and $MP = 100{,}000$, a value of $C_D = 910$ is found.

To handle an approach volume of 1350 v.p.h., the required width of approach is found by proceeding through the chart in reverse order. Proceeding through the chart with $MP = 100{,}000$, $C_D = 1{,}350$, $G/C = 0.55$, and residential area without parking, $W_A = 31$ feet is found. Using 11-foot lanes, the required width of approach is 33 feet.

Problem 3

In a central business district of a 250,000 population metro area, a 2-way 58-foot street with parking intersects a 2-way 44-foot street with no parking. The former is to accommodate a peak-hour volume of 620 v.p.h. in one direction. If conditions are assumed to be average and a 60-second cycle is used, 6 seconds of which are allotted to amber, what should be the green interval on the 58-foot street for operation at design capacity? What would be the resultant green interval and design capacity of one approach on the 44-foot street? What would be the possible capacity of this approach?

Solution: Enter chart 1 at left with $W_A = 58/2 = 29$, proceed right to *CBD* with parking curve, then down to lower graph until a horizontal projection of 620 v.p.h. is intersected: $G/C = 0.43$. G on 58-foot street $= 60 \times 0.43 = 26$ seconds. G on 44-foot street $= 60 - 26 - 6 = 28$ seconds; and $G/C = 28/60 = 0.47$.

For the design capacity of the 44-foot street using $W_A = 44/2 = 22$, *CBD* without parking, and $G/C = 0.47$, in chart 1, $C_D = 730$ v.p.h. in one direction.

For the possible capacity of the 44-foot street, using $f = 1.2$ from table 2 for a 22-foot approach with no parking, $C_P = 730 \times 1.2 = 880$ v.p.h. in one direction.

Streets Without Parking

Charts 3 and 4 include the adjustments for specific intersection conditions on 2-way streets where parking is prohibited. Chart 3 is applicable to central business districts, and chart 4 to all other sections within the metropolitan area—fringe, outlying business, and residential. Charts 3 and 4 will often be used by proceeding from the left side to bottom scales, as shown by arrows. The charts can also be used in the reverse order to obtain the width of approach required to handle a given volume. Or, with a given approach volume and a given width, the necessary ratio of green time to cycle time can be determined readily. When the approach width and the G/C ratio cannot be altered, but capacity must be increased, the increase through elimination of left turns and/or changing the bus-stop

TABLE A — ADJUSTMENT FACTOR (f) FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet								
		10	15	20	25	30	35	40	50	60
A No Backlog	0·0	0·85	0·86	0·87	0·88	0·89	0·90	0·89	0·87	0·85
B	0·1	0·90	0·91	0·91	0·92	0·92	0·92	0·93	0·92	0·91
C Design Capacity	0·3	1·00	1·00	1·00	1·00	1·00	1·00	1·00	1·00	1·00
D	0·7	1·14	1·14	1·14	1·14	1·15	1·16	1·17	1·18	1·20
E Possible Capacity	0·85	1·20	1·20	1·20	1·20	1·21	1·23	1·25	1·27	1·30

*NOTE:

Condition where volume of left turning vehicles can be handled without a separate signal indication; before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5.

EXAMPLE

Given

W_A = 25' L = 10 %
T = 8% MP = 500,000 C.B.D.
R = 25% G = 36 sec., G/C = 0·60
B = No bus stop

Solution

Design capacity
C_D = 1040 vph

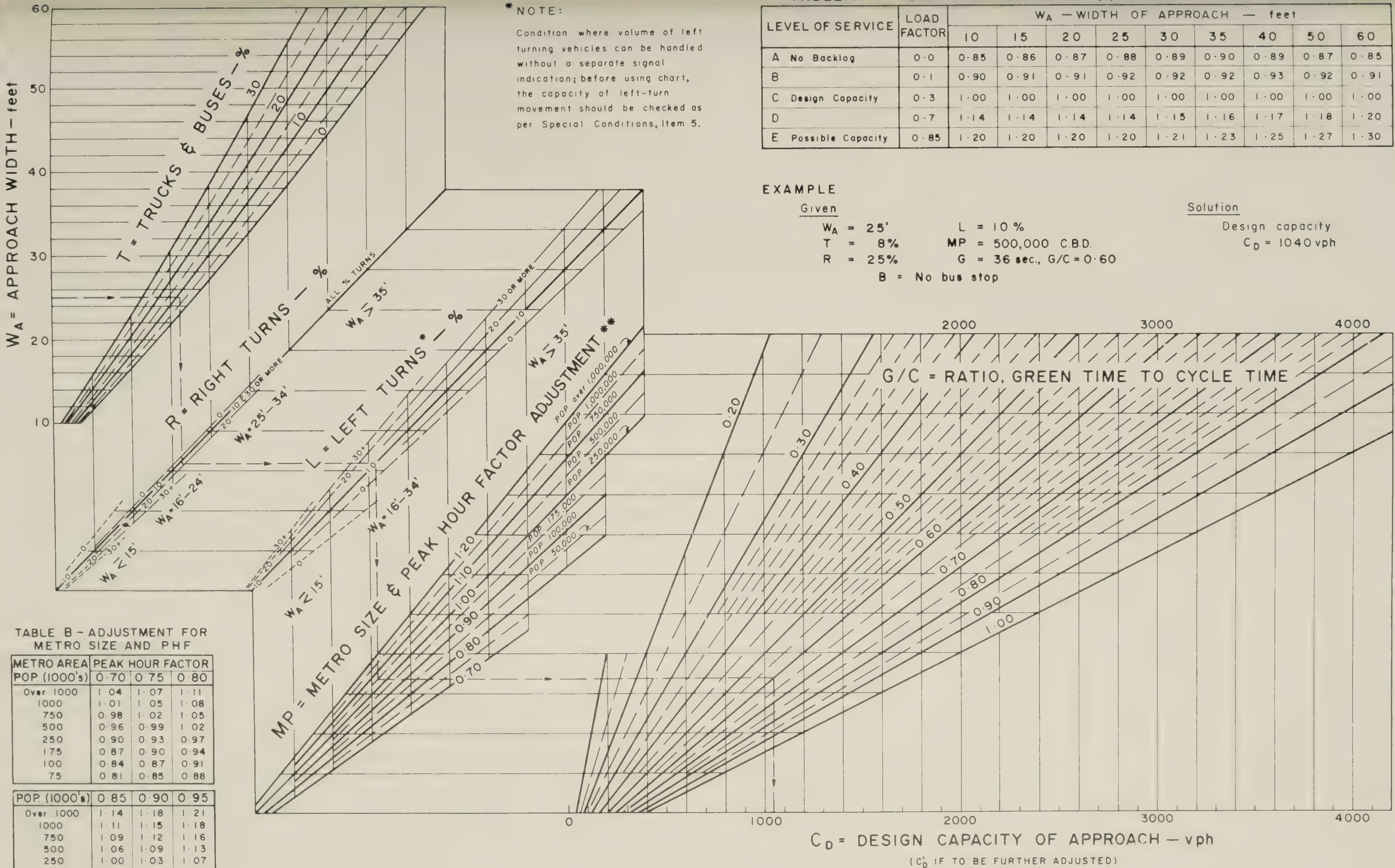


TABLE B – ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP. (1000's)	PEAK HOUR FACTOR 0·70	0·75	0·80
Over 1000	1·04	1·07	1·11
1000	1·01	1·05	1·08
750	0·98	1·02	1·05
500	0·96	0·99	1·02
250	0·90	0·93	0·97
175	0·87	0·90	0·94
100	0·84	0·87	0·91
75	0·81	0·85	0·88

POP. (1000's)	0·85	0·90	0·95
Over 1000	1·14	1·18	1·21
1000	1·11	1·15	1·18
750	1·09	1·12	1·16
500	1·06	1·09	1·13
250	1·00	1·03	1·07
175	0·97	1·01	1·04
100	0·94	0·98	1·01
75	0·92	0·95	0·98

** Use Table B if PHF is known to find adjust factor; otherwise use Population directly.

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

TWO-WAY STREET — NO PARKING — C. B. D.

CHART 3

3

C.B.D.

condition with the aid of chart 16, can be found on the charts.

Charts 3 and 4 apply to intersection approaches that have the conditions described on the charts. The direct use of the charts as indicated by example arrows, however, is applicable specifically to the condition in which the volume of left-turning vehicles on the approach can be handled without requiring a separate signal indication. A check for the capacity of left-turn movement should always be made when using charts 3 and 4 and also charts 5–15, as explained in *Part 3, Special Conditions* under the heading *Check for Capacity of Left Turn*. For simplicity in demonstration and better understanding of chart use, examples 4–8 are purposely selected so that the volume of left-turning vehicles does not exceed the capacity of the left-turn movement. The maximum service volume of left-turning vehicles that can be accommodated without a separate signal indication on major streets is generally in the range of 80 to 120 v.p.h., and a left-turning volume of 100 v.p.h. can be used as an average. Further examples illustrate the use of this important control.

Where bus stops are provided at the intersection for pickup and discharge of passengers, design capacity values obtained from charts 3 and 4 are corrected by multiplying the result by an appropriate adjustment factor from chart 16. When the charts are used in reverse order, to find G/C or W_A, C_D is divided by the bus adjustment factor before the charts are entered.

Where levels of service other than C (design capacity) are desired, including level of service E (possible capacity), the resultant value of C_D in charts 3 and 4 should be multiplied by an appropriate adjustment factor from table A shown at upper right on charts 3 and 4. When the charts are used in reverse order, C_D is divided by the level of service factor before the charts are entered.

Problem 4

Determine the design capacity of one approach on a 50-foot, 2-way street without parking, located in the *CBD* of a metro area with a population of 500,000. Other conditions at the intersection are: $T=8\%$, $R=25\%$, $L=10\%$, $G=36$ seconds, $C=60$ seconds, and no bus stop.

Solution: Enter chart 3 with $W_A=25$ and follow the arrows according to each condition; find $C_D=1{,}040$ v.p.h.

Problem 5

Determine the design capacity and possible capacity of one approach on a 64-foot 2-way street without parking, where the cycle is 60 seconds and green interval is 27 seconds. The intersection is located in the *CBD* of a metro area with a population of 250,000. Other conditions are: $T=12\%$, $R=15\%$, $L=7\%$, and bus stop on near side serving 46 buses per hour.

Solution: Enter chart 3 at left with $W_A=32$ and using $T=12\%$, $R=15\%$, $L=7\%$, $MP=$ pop. 250,000, and $G/C=27/60=0.45$; find $C'_D=930$. Using chart 16—no parking, near side—$B=46$ and $W_A=32$ (see arrows on chart) find adjustment factor, $F_B=0.81$. Design capacity $C_D=930\times0.81=750$ v.p.h., in one direction.

From table A in chart 3, for possible capacity and $W_A=32$, $f=1.22$. Possible capacity $C_P=750\times1.22=920$ v.p.h. in one direction.

Problem 6

A 2-way street on which there will be no parking is planned to cross an existing street. The facility is located in the *CBD* of a metro area having a population of 250,000 and a peak-hour factor (*PHF*) of 0.80. According to the volume on the existing street, 33% of the cycle time must be allotted to green on that street. Determine the needed width of pavement on the new street if the *DHV* in one direction is 1,200 v.p.h. and other conditions are: $T=14\%$, $R=12\%$, $L=5\%$, no bus stop, and C is to be set at 70 seconds with an allowance of 6 seconds for amber per cycle.

Solution: The time available for the amber periods and for the green on the new street is $1.00-0.33=0.67$. Therefore, $(G+\text{amber})\div C=0.67$, or $(G+6)\div70=0.67$; $G=41$ seconds, and $G/C=0.59$.

Enter chart 3 at bottom with a peak-hour volume of 1,200 v.p.h. in one direction, and proceed through the chart, turning at $G/C=0.59$, $MP=0.97$ (determined from table B), $L=5\%$, $R=12\%$, and $T=14\%$; find $W_A=31.5$ feet. In proceeding through the chart, it was necessary to assume for the L and R turning points that either $W_A=(16\text{ to }34)$ or $W_A\geqq35$. If the proper assumption is made initially, the answer is determined directly. If the assumption is incorrect, a second trial is required to determine the correct answer. If 11-foot lanes are to be used, and operation is generally balanced by direction, the new street should be the nearest multiple doubled for both directions; $33\times2=66$ feet wide.

Problem 7

Determine the design capacity of an intersection approach on a street with a narrow median and two 30-foot pavements in the fringe area of a city having a metro population of 700,000 and *PHF* of 0.89. Other conditions are: $T=9\%$, $R=14\%$, left turns prohibited, $G=31$ seconds, $C=65$ seconds, and a bus stop on the far side serving 70 buses per hour.

Solution: As a preliminary step, find $MP=1.11$ by interpolation in table B, chart 4; $G/C=31/65=0.48$; and $F_B=0.95$ in chart 16—no parking, far side—using 70 buses, $W_A=30$ and $(L+R)=14\%$. Proceed through chart 4 from upper left with $W_A=30$ and follow the arrows according to each condition, as illustrated on the chart. Find initial $C'_D=1{,}460$; with adjustment for local buses, design capacity, C_D, is $1{,}460\times0.95=1{,}390$ v.p.h. in one direction.

Problem 8

In a residential area of a city with a metro population of 1 million, a 2-way, 40-foot parkway is to be crossed by a new street. On the critical approach of the existing parkway the projected peak-hour traffic volume in one direction is 900 automobiles of which 135 turn right and 80 turn left. On the critical approach of the new street, the design volume is 1,680 v.p.h. of which 7% are trucks; turning movements are 10% and 4% to the right and left, respectively. There will be no parking and no bus stops at the intersection on either facility. If 12-foot lanes are to be used, how many lanes are required on one approach of the new facility?

Solution: It is first necessary to determine the proportion of green time required for the existing parkway. From chart 4, using $W_A=20$, $T=0\%$, $R=135/900=15\%$, $L=80/900=9\%$, no bus stops, and intersecting from a volume (equal to C_D) of 900 v.p.h., it is found that $G/C=0.43$.

Assume $C=60$ seconds and total amber period is 6 seconds. Then, G for parkway traffic is $0.43\times60=26$ seconds, and G for traffic on the new highway is $60-26-6=28$ seconds.

To determine the required width of the new facility, enter the bottom of chart 4 at a design volume of 1,680 v.p.h., and proceed up and to the left using $G/C=28/60=0.47$, $MP=$ pop 1,000,000, $L=4\%$, $R=10\%$, and $T=7\%$; find $W_A=35.5$ feet. Three 12-foot lanes, therefore, are required on the new highway in each direction of travel.

Streets With Parking

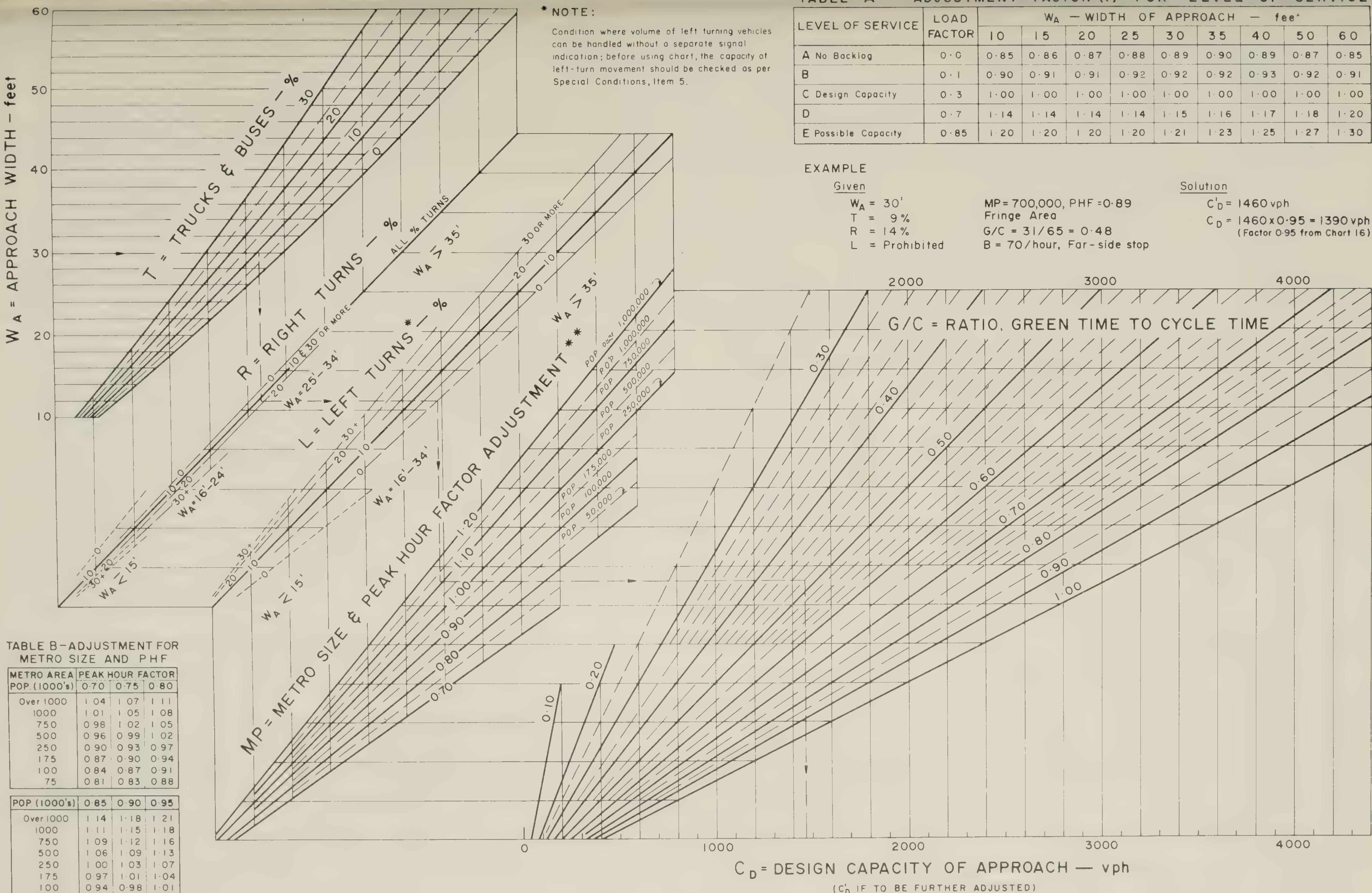
Charts 5 and 6 include the adjustments for specific intersection conditions on 2-way streets with parking permitted. (In this article, as in the *Manual*, parking parallel to the curb is the only type considered; diagonal parking would obviously have a much different effect on traffic flow.) Chart 5 is applicable to central business districts, and chart 6 to all other sections within the metropolitan area—fringe, outlying business and residential.

The procedure through the charts is the same as for charts 3 and 4 as previously described. Also, the direct use of charts 5 and 6 applies to the condition where the volume of left-turning vehicles on the approach can be accommodated without requiring a separate signal indication. For simplicity in presentation, examples 9–13 were selected so that this condition is satisfied, as it would be ordinarily when the volume of left-turning vehicles on one approach does not exceed 100 v.p.h. Adjustments for bus stops, in accordance with chart 16, and for levels of service other than C, in accordance with table A, shown at upper right of charts 5 and 6, are accomplished in the same way as for charts 3 and 4.

Problem 9

Determine the design capacity and possible capacity of one approach on a 2-way 60-foot street, with parking in the central business district of a metro area having a population of 750,000. Other conditions are: $T=8\%$, $R=14\%$, $L=0\%$ (left turns prohibited), $G=34$ seconds, $C=65$ seconds, and a far-side bus stop that serves 70 buses during the peak-hour period.

Solution: Enter chart 5 at left with $W_A=$[illegible] and proceed through the chart using the conditions listed. The initial result, as shown by the arrows in the chart, is a design capacity C'_D of 900 v.p.h.



*NOTE:

Condition where volume of left turning vehicles can be handled without a separate signal indication; before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5.

TABLE A — ADJUSTMENT FACTOR (f) FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet: 10	15	20	25	30	35	40	50	60
A No Backlog	0·0	0·85	0·86	0·87	0·88	0·89	0·90	0·89	0·87	0·85
B	0·1	0·90	0·91	0·91	0·92	0·92	0·92	0·93	0·92	0·91
C Design Capacity	0·3	1·00	1·00	1·00	1·00	1·00	1·00	1·00	1·00	1·00
D	0·7	1·14	1·14	1·14	1·14	1·15	1·16	1·17	1·18	1·20
E Possible Capacity	0·85	1·20	1·20	1·20	1·20	1·21	1·23	1·25	1·27	1·30

EXAMPLE

Given

W_A = 30'
T = 9%
R = 14%
L = Prohibited

MP = 700,000, PHF = 0·89
Fringe Area
G/C = 31/65 = 0·48
B = 70/hour, Far-side stop

Solution

C'_D = 1460 vph
C_D = 1460 x 0·95 = 1390 vph
(Factor 0·95 from Chart 16)

TABLE B—ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP. (1000's)	PEAK HOUR FACTOR 0·70	0·75	0·80
Over 1000	1·04	1·07	1·11
1000	1·01	1·05	1·08
750	0·98	1·02	1·05
500	0·96	0·99	1·02
250	0·90	0·93	0·97
175	0·87	0·90	0·94
100	0·84	0·87	0·91
75	0·81	0·83	0·88

POP (1000's)	0·85	0·90	0·95
Over 1000	1·14	1·18	1·21
1000	1·11	1·15	1·18
750	1·09	1·12	1·16
500	1·06	1·09	1·13
250	1·00	1·03	1·07
175	0·97	1·01	1·04
100	0·94	0·98	1·01
75	0·92	0·95	0·98

** Use Table B if PHF is known to find adjust. factor; otherwise use Population directly.

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

TWO-WAY STREET — NO PARKING — O.B.D., FRINGE & RESIDENTIAL AREAS

CHART 4

4 FRNG. O.B.D. RES.

Because there is a bus stop on the far side, with parking, the lower part of chart 16-B is used: for 70 buses, *CBD*, $W_A=30$ feet, and $(L+R)=14\%$, $F_B=0.98$.

Design capacity, adjusted for the bus stop, is $C_D=900\times0.98=880$ v.p.h. Adjustment factor for possible capacity for a 30-foot approach is $f=1.18$ (table A on chart 5); $C_P=880\times1.18=1,040$ v.p.h.

Problem 10

Determine the design capacity of a 32-foot approach of a 2-way street with parking in an outlying business district of a metro area having a population of 400,000. Other conditions: $T=5\%$, $R=16\%$, $L=7\%$, $G=36$ seconds, $C=62$ seconds, and a near-side bus stop handling 50 buses during the peak-hour period.

Solution: Using $W_A=32$, and following the arrows in chart 6 for the conditions indicated, $C_D'=1,220$ v.p.h.

Adjustment for buses is found in the upper part of chart 16-B. Using $B=50$, outlying-area turning line and $(L+R)=23\%$, in conjunction with $W_A=24$ and $W_A=36$, an adjustment of $F_B=1.06$ and 1.08, respectively, is found. Interpolating for $W_A=32$, $F_B=1.07$.

Design capacity of one approach, $C_D=1,220\times1.07=1,310$ v.p.h.

Problem 11

As shown in figure 4, an east-west 52-foot, 2-way street operates with parking on the south side only. Two lanes are available for moving traffic on both the west and east approaches. The intersection is located in the fringe area of a city having a metro population of 200,000. If the $PHF=0.75$ and the evening peak-hour traffic is as indicated in figure 4, determine the signal timing required on the east-west street to accommodate this traffic, using a 60-second cycle.

Solution: For the west approach, as there is parking, chart 6 is applicable. The fact that there is no parking on the north side of the street or on the east approach does not affect the operation or the analysis of the west approach, which does have parking. Enter chart 6 at upper left with $W_A=31$ feet and proceed through the chart using $T=12\%$, $L=40/660=6\%$, $R=60/660=9\%$, and $MP=0.89$ (adjustment factor interpolated from table B for metro population of 200,000 and $PHF=0.75$). The horizontal projection from the MP turning line is then intersected by a vertical projection from a C_D value on the bottom scale equal to the approach volume of 660 v.p.h.; find $G/C=0.40$.

For the east approach, as there is no parking, chart 4 is applicable. Enter chart with $W_A=21$ feet and, using $T=9\%$, $R=50/810=6\%$, $L=80/810=10\%$ and $MP=0.89$, find the point of intersection on the G/C scale for C_D equal to an approach volume of 810 v.p.h.: $G/C=0.50$.

The required signal timing for the east-west street, therefore, is the larger of the two G/C values or 0.50, and a G of $0.50\times60=30$ seconds.

Rural Conditions

Capacities and service volumes representative of different levels of service for specific intersection conditions in rural areas may be determined by using chart 15. The general format of the nomograph and procedure for application of the nomograph is the same as for charts 3–6. The fourth set of turning lines in chart 15 differs from a similar set in charts 3–6 because only the peak-hour factor is accounted for in chart 15 as population is irrelevant. Normally, all rural conditions are represented by a peak-hour factor of 0.70, for which the adjustment factor in the chart is 1.00.

On recreational routes operating under peak-flow conditions that build up to a *PHF* of 1.00, the capacities and service volumes according to the *Manual* may be adjusted upward by 40%. Thus an adjustment factor of 1.40 is indicated in the chart for *recreational route—high-peak flow.* An intermediate condition, referred to as *recreational route—peak flow* is introduced in the chart assuming a *PHF* of 0.85, with a corresponding adjustment factor of 1.20.

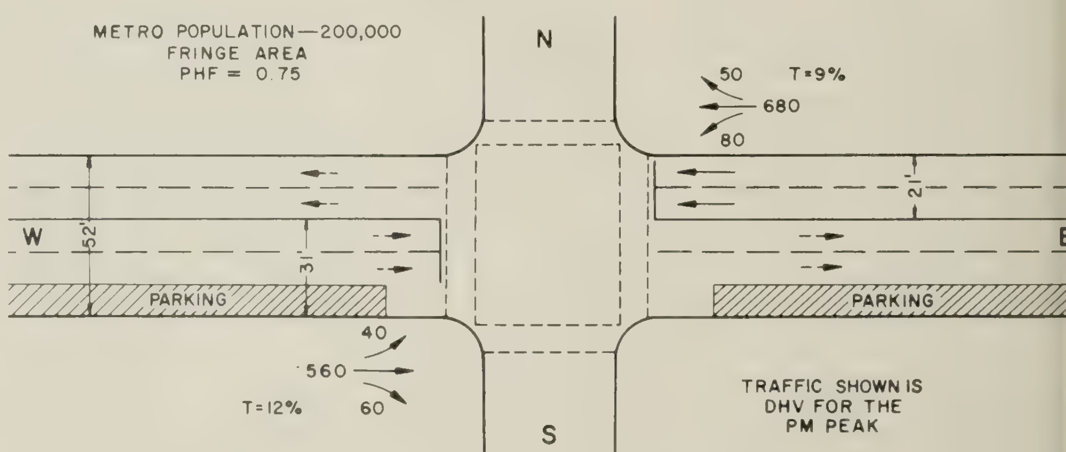


Figure 4.—Problem 11 illustrated.

Problem 12

Determine the design capacity and possible capacity of an intersection approach of a 2-way 4-lane highway in a rural area operating under normal peaking characteristics. Other conditions are: lane width = 11 feet, $T=15\%$, $R=12\%$, $L=7\%$, and available $G/C=0.55$.

Solution: Enter chart 15 with $W_A=22$ feet and proceed through the chart, using the conditions given, as indicated by the arrows; find $C_D=740$ v.p.h. $C_P=740\times1.29=950$ v.p.h.

Problem 13

The capacities indicated in the above problem are representative of normal conditions. If the same facility during the summer weekends carries recreational traffic, what would be the design capacity and possible capacity during peak flows on these weekends? Controlling conditions are the same except that truck traffic is negligible, and right and left turns are reduced to 6% and 4%, respectively.

Solution: Enter chart 15 with $W_A=22$ and proceed through the chart turning at $T=0\%$, $R=6\%$, $L=4\%$, $PHF=0.85$ and $G/C=0.55$ find $C_D=1,100$ v.p.h., and $C_P=1,100\times1.29=1,420$ v.p.h.

Problem 14

If the facility in Problem 13 was expected to handle traffic from a much expanded resort area under conditions of extremely high-peak flows, approaching a peak-hour factor of 1.00, what would be the maximum 1-way volume that could be accommodated at such times? All other conditions are assumed to be the same.

Solution: Proceed through the chart as in Problem 13 but turn on $PHF=1.00$; find $C_P=1,280\times1.29=1,650$ v.p.h. in one direction.

Intersections With Separate Turning Lanes

Frequently, on modern highways and streets, exclusive lanes on intersection approaches are provided for left- or right-turning movements. Such lanes may be in addition to the regular width, or they may be specifically marked for turns within the basic approach width. Exclusive turning lanes may be controlled by separate signal indications, or they may be handled without additional signal phases.

Capacities of various forms of turning lanes are given in charts 17 and 18. Procedures for analyzing intersection approaches with turning lanes, including illustrative problems, are presented below. Traffic carrying capabilities are expressed in terms of design capacity and possible capacity. No distinction normally is made for other (intermediate) levels of serv-

TABLE A — ADJUSTMENT FACTOR (f) FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet								
		10	15	20	25	30	35	40	50	60
A No Backlog	0·0	—	—	0·95	0·93	0·91	0·89	0·88	0·86	0·84
B	0·1	—	—	0·97	0·96	0·95	0·94	0·93	0·91	0·89
C Design Capacity	0·3	—	—	1·00	1·00	1·00	1·00	1·00	1·00	1·00
D	0·7	—	—	1·06	1·09	1·11	1·14	1·17	1·22	1·24
E Possible Capacity	0·85	—	—	1·10	1·14	1·18	1·21	1·25	1·31	1·34

*NOTE:

Condition where volume of left turning vehicles can be handled without a separate signal indication; before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5.

EXAMPLE

Given

W_A = 30'
T = 8%
R = 14%
L = 0%

MP = 750,000
C.B.D.
G/C = 34/65 = 0·52
B = 70/hr., Far-side stop

Solution

C'_D = 900 vph
C_D = 900 x 0·98 = 880 vph
C_P = 880 x 1·18 = 1040 vph
(Factor 0·98 from Table 16-B; Factor 1·18 from Table A, above)

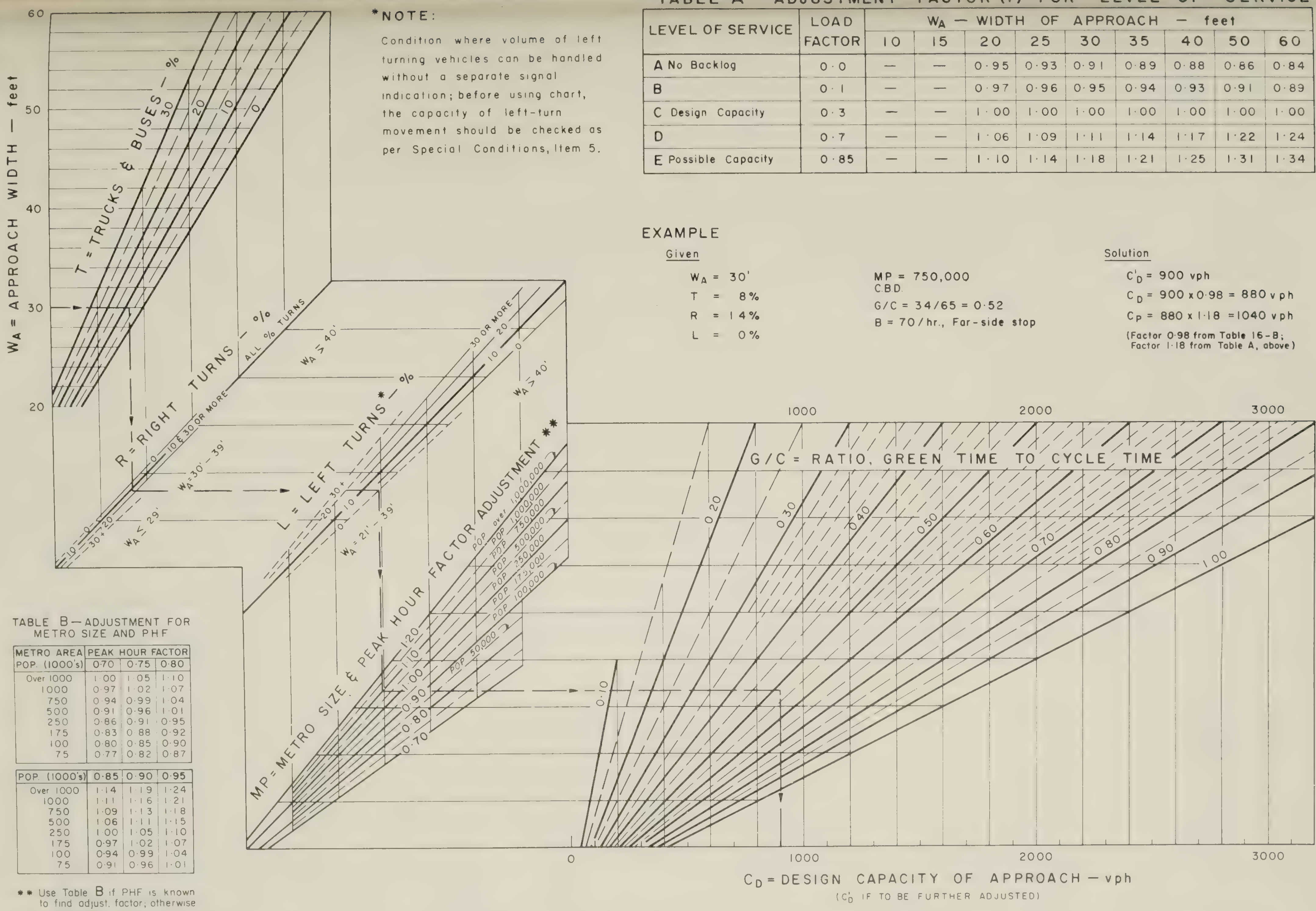


TABLE B — ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP. (1000's)	PEAK HOUR FACTOR 0·70	0·75	0·80
Over 1000	1·00	1·05	1·10
1000	0·97	1·02	1·07
750	0·94	0·99	1·04
500	0·91	0·96	1·01
250	0·86	0·91	0·95
175	0·83	0·88	0·92
100	0·80	0·85	0·90
75	0·77	0·82	0·87

POP. (1000's)	0·85	0·90	0·95
Over 1000	1·14	1·19	1·24
1000	1·11	1·16	1·21
750	1·09	1·13	1·18
500	1·06	1·11	1·15
250	1·00	1·05	1·10
175	0·97	1·02	1·07
100	0·94	0·99	1·04
75	0·91	0·96	1·01

** Use Table B if PHF is known to find adjust. factor; otherwise use Population directly.

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS
TWO-WAY STREET — WITH PARKING — C.B.D.
CHART 5

5
C.B.D.

because data is scarce on the operation of such facilities; however, approximate relations to several other levels of service are indicated in table 5.

The capacities of turning lanes presented in charts 17 and 18 are applicable to all types of intersections, involving 1-way streets, 2-way streets, rural highways, and high-type facilities.

Table 5.—Levels of service conversion factors for separate turning lanes

Level of service	Adjustment factor
A and B	0.90
C	1.00
D	1.20
E	1.30

Turning lanes without separate signal indication

Capacities of exclusive left- and right-turn lanes where no separate signal indication is used for the turning movement are covered in chart 17. The usual form of a separate turning lane is depicted by the sketch in chart 17–A for the left-turn lane, and by the sketches in charts 17–C and 17–D for the right-turn lane. The same capacity relations apply to the form of turning lane that is part of the basic approach width.

LEFT-TURN LANE.—The design capacity of an exclusively left-turn lane, when traffic in all lanes on the approach is permitted to move simultaneously on a common green indication, is given in charts 17–A and 17–B. Additional terms introduced in these charts are as follows:

D_3 Effective length of left-turn lane, in feet, for the storage of turning vehicles, exclusive of crosswalk and taper.

V_3 Volume of traffic turning left on one approach, in vehicles per hour.

T_3 Trucks and buses turning left, expressed as a percentage of the total left-turning volume V_3 on one approach.

V_o Volume of through traffic on the opposite approach, in vehicles per hour, that is in direct conflict during the same period of time with the left-turning movement on the approach in question.

T_o Trucks and buses, expressed as a percentage of the total through volume V_o on the opposite approach.

C_{D3} Design capacity of the separate left-turning lane in vehicles per hour.

The capacity of a left-turn lane is determined primarily by the volume of traffic opposing the left turn during the green signal indication. Normally, on major streets in downtown areas and on wide major streets in outlying areas, it seldom will be possible for more than two vehicles to turn left per cycle (such turns usually have to be made on the amber signal). For design purposes, between one and two left-turning vehicles accommodated per cycle are assumed to be appropriate. Design capacity of a left-turn lane, for the conditions described, is predicated on 1.6 vehicles per cycle. Chart 17-B gives this relation in terms of vehicles per hour as dependent upon the length of cycle.

On some streets, where the opposing through volume is relatively light, the capacity of a left-turn lane may be much greater than indicated above. For such a condition, the design capacity of a left-turn lane per hour of green is estimated as the difference between 1,200 and V_o, both figures expressed in terms of passenger vehicles. Chart 17-A provides a solution for this condition, being the relation between V_o, G/C, and design capacity C_{D3}. To express the capacity in terms of vehicles of all types, factors for the percentage of trucks and buses in the opposing through movement T_o and that in the left-turn movement T_3 are applied. To determine the design capacity of a left-turn lane, C_{D3} should be found on both charts 17-A and 17-B, and the larger of the two results used. Usually on major streets, the values from chart 17-B will govern. The possible capacity of a left-turn lane, whether determined by chart 17-A or 17-B, is 1.3 times the design capacity.

Lane length is another control of capacity of an added turning lane. If the lane is not long enough to store the vehicles that can make the turn on the proper green interval, the capacity otherwise possible cannot be attained. Chart 17-E gives the length of added lane needed to accommodate given volumes for different signal timings. With the chart in this form, the lane length can be determined both for capacity volumes and for known smaller turning volumes for a specific condition. Since control values are in terms of passenger vehicles only, the adjustment for percentage of trucks and buses is included.

The required (minimum) length of the added left-turn lane is determined as the distance needed to store 1.5 times the average number of turning vehicles that will accumulate per cycle, recognizing the maximum number that actually can move during the green interval. Occasionally, the maximum number of vehicles that could be stored during some cycles could be twice the average number owing to variations in arrival rates. A desirable length of turning lane, therefore, is also indicated on the chart and is predicated on two times the average number of *storing* vehicles, or a length 33% greater than the minimum dimension shown. A length of 25 feet is used for each passenger vehicle, and 40 feet for each truck or bus.

The minimum length applies to the full width of the turning lane. This full length is not available for use unless it is preceded by a suitable taper. For normal 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. This taper is in addition to the minimum length of turning lane shown in Chart 17-E.

To determine the capacity of an intersection approach with a separate left-turn lane, when traffic in all lanes on the approach is permitted to move simultaneously on a common green indication, the following procedure should be followed:

(1) Obtain C_{D3} from chart 17-A or 17-B.

(2) Obtain C_D of combined through and right-turning movement from charts 3–6 or 15, using:

W_A = approach width exclusive of left-turn lane, and

L = 0%.

(3) Obtain D_3 from chart 17-E for V_3 or C_{D3}, whichever governs.

(4) Determine possible capacity, when required:

Left-turn lane, $C_{P3} = 1.3 \times C_{D3}$.

Through and right-turning movement, combined, $C_P = f \times C_D$.

Values for f are given in table A on charts 3–6 or 15.

In this procedure, the design capacity of the intersection approach cannot be taken as the sum of separate values for the capacity of the left-turn lane and that of the through-plus-right lanes because traffic on an intersection approach follows a given pattern, or maintains a consistent proportion of the different movements—through, left, and right. Thus, the capacity of the left-turn lane and that of the through-plus-right lanes should be kept separate and, in the analysis, the volumes of the individual movements should be compared with their capacities.

Usually, the through-plus-right movement determines the G/C on the approach, and the left-turn volume is then checked for capacity of the left-turn lane based on this G/C. If the latter movement is less than design capacity the requirements are met. However, if the left-turning volume exceeds the design capacity of the left-turn lane, the following alternatives may be considered: (1) accept a lower level of service on the left-turn lane, not to exceed possible capacity; (2) increase the G/C to provide operation of the left-turn lane at design capacity, in which case the through-plus-right volume would be less than design capacity and would operate at a higher level of service; or (3) change the signal phasing to include a separate signal indication for the left-turning movement in order to accommodate it at design capacity.

Problem 15

Determine the G/C required, during the p.m. peak period, to handle traffic without exceeding design capacity on the east approach of the intersection shown in figure 5.

Solution: Signal timing based on a combined through-plus-right volume is obtained from chart 4 using $W_A = 20$, $T = (30+65) \div (250+750) = 10\%$, $R = 250 \div 1000 = 25\%$, $L = 0\%$ (because left turn is on separate lane), MP = 500,000 population, and approach volume (equated to C_D) = 250 + 750 = 1,000 v.p.h.; find $G/C = 0.57$.

Then check in charts 17-A and 17-B the capacity of the left-turn lane with this signal timing. Entering chart A with $V_o = 350$ v.p.h. and proceeding to right and bottom wit

NOTE:

Condition where volume of left turning vehicles can be handled without a separate signal indication; before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5.

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet								
		10	15	20	25	30	35	40	50	60
A No Backlog	0·0	—	—	0·95	0·93	0·91	0·89	0·88	0·86	0·84
B	0·1	—	—	0·97	0·96	0·95	0·94	0·93	0·91	0·89
C Design Capacity	0·3	—	—	1·00	1·00	1·00	1·00	1·00	1·00	1·00
D	0·7	—	—	1·06	1·09	1·11	1·14	1·17	1·22	1·24
E Possible Capacity	0·85	—	—	1·10	1·14	1·18	1·21	1·25	1·31	1·34

EXAMPLE

Given

W_A = 32'
T = 5%
R = 16%
L = 7%

MP = 400,000
O.B.D.
G/C = 36/62 = 0·58
B = 50/hr., Near-side stop

Solution

C'_D = 1220 vph
C_D = 1220 x 1·07 = 1310
(Factor 1·07 from Chart 16-B)

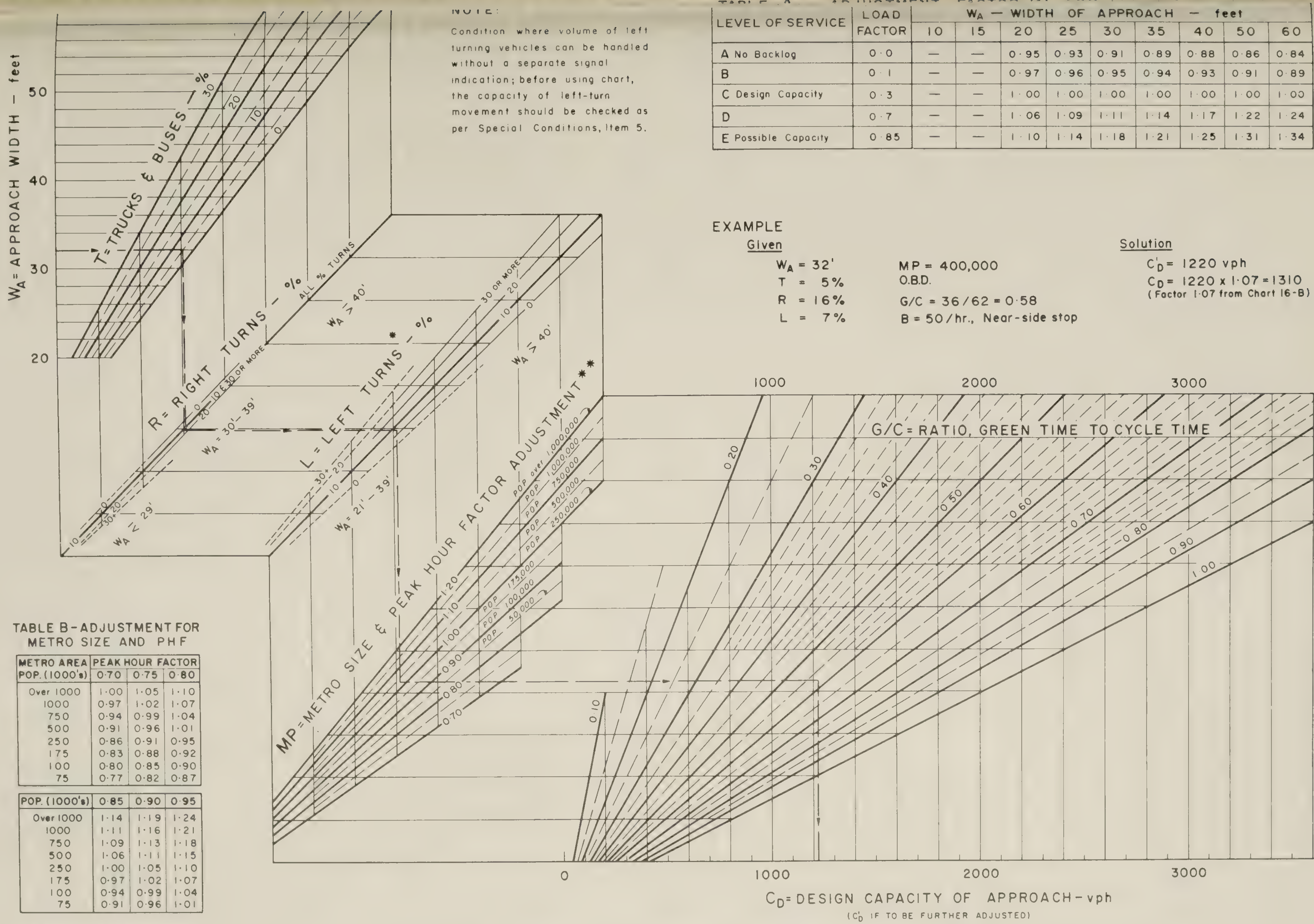


TABLE B - ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP. (1000's)	PEAK HOUR FACTOR 0·70	0·75	0·80
Over 1000	1·00	1·05	1·10
1000	0·97	1·02	1·07
750	0·94	0·99	1·04
500	0·91	0·96	1·01
250	0·86	0·91	0·95
175	0·83	0·88	0·92
100	0·80	0·85	0·90
75	0·77	0·82	0·87

POP. (1000's)	0·85	0·90	0·95
Over 1000	1·14	1·19	1·24
1000	1·11	1·16	1·21
750	1·09	1·13	1·18
500	1·06	1·11	1·15
250	1·00	1·05	1·10
175	0·97	1·02	1·07
100	0·94	0·99	1·04
75	0·91	0·96	1·01

** Use Table B if PHF is known to find adjust factor; otherwise use Population directly.

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS
TWO-WAY STREET — WITH PARKING
O.B.D., FRINGE & RESIDENTIAL AREA
CHART 6

6

FRNG.
O.B.D.
RES.

$T_o = 20/350 = 6\%$, $G/C = 0.57$ and $T_3 = 30/240 = 13\%$ (see arrows), the design capacity of the left-turn lane is $C_{D3} = 270$ v.p.h. Chart 17-A governs here as the value of 80 v.p.h. for $C = 72$ seconds from chart 17-B (see arrows) is much less. Thus, the indicated left-turn volume of 240 v.p.h. can be accommodated.

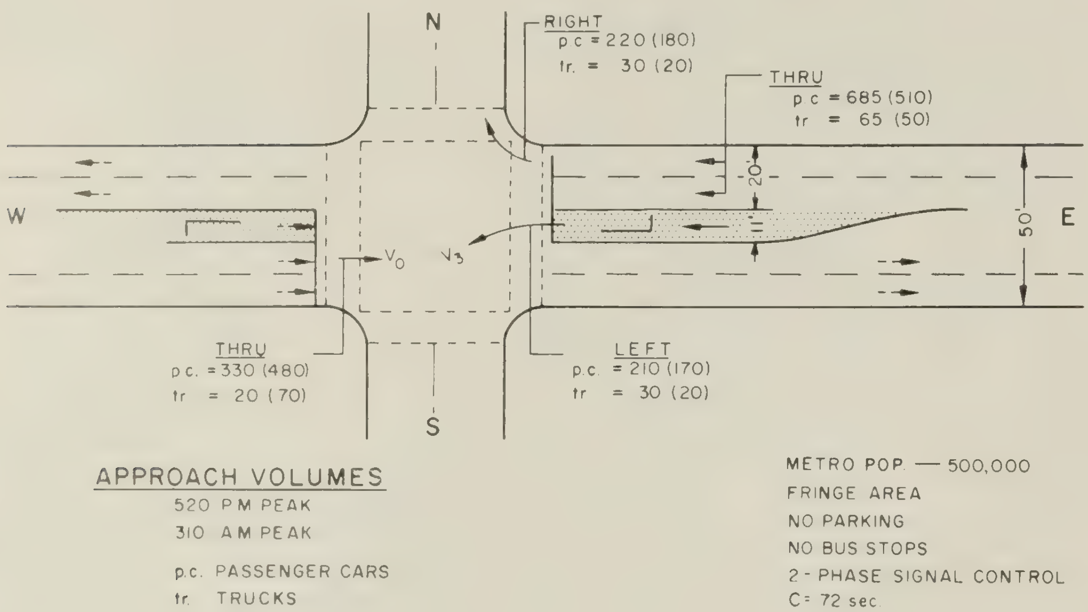


Figure 5.—Problems 15 and 20 illustrated.

The G/C required to handle the p.m. peak-hour traffic on the east approach without exceeding design capacity, therefore, is 0.57 (controlling value for the through-plus-right movement).

The required length of left-turn lane is obtained from chart 17-E. Using $V_3 = 240$ v.p.h., $C = 72$ seconds and $T_3 = 13\%$, find $D_3 = 270$ feet (see arrows).

RIGHT-TURN LANE.—Charts 17-C and 17-D give the design capacity of an exclusively right-turn lane, when traffic in all lanes on the approach is permitted to move simultaneously on a common green indication. The following additional terms are introduced in the charts:

D_2 Effective length of right-turn lane, in feet, for the storage of turning vehicles, exclusive of crosswalk and taper.

a Width of right-turn lane, in feet.

V_2 Volume of traffic turning right on one approach, in vehicles per hour.

T_2 Trucks and buses turning right, expressed as a percentage of the total right-turn volume, V_2 on one approach.

C_{D2} Design capacity of the separate lane for right-turn movement in vehicles per hour.

The capacity of a right-turn lane is largely dependent on the proportion of truck traffic T_2 and the G/C ratio available for movement of traffic in the lane. Charts 17-C and 17-D show design capacity in terms of these factors. Right-turn lane capacity is also dependent on the radius of the turn, the amount of pedestrian interference, and the length of lane provided. From available data, distinction has been made for two general conditions in regard to radius and pedestrians. In each case, the intersection sketches shown with the charts are indicative of these conditions.

Chart 17-C represents average normal curb return, that is, corner radius at edge of pavement, and pedestrian interference, based on an average flow of 600 vehicles per hour of green including 5 percent trucks. The width of lane is not a factor here because either the small corner radius or pedestrian interference, or both, is the principal control. Chart 17-C is applicable generally to ordinary street conditions and geometry of streets in built-up areas.

Better conditions with adequate curb return and moderate to light pedestrian interference are represented in chart 17-D, based on a design capacity of right-turn lane of 800 vehicles per hour of green with 5% trucks per 10 feet of lane width. With no pedestrian interference, the design capacity base is 900 vehicles per hour of green with 5% trucks per 10 feet of lane width. Such high-type conditions are not covered directly in the *Manual;* however, where the movement is free of pedestrians, values corresponding to turning lanes with separate signal indication are suggested in the *Manual*. Accordingly, service volume bases of 800 and 900 v.p.h. are considered to be indicative of these designs.

The length of right-turn lane required to handle the demand volume of right-turning vehicles, or the volume corresponding to th capacity of the right-turn lane, is given i chart 17-E. Right-turn lane applicatio method of measurement, and the design c taper are the same as discussed for the left turn lane above.

To determine the capacity of an inter section approach with separate right-tur lane, when traffic in all lanes on the approac is permitted to move simultaneously on common green indication, the following pro cedure should be followed:

(1) Obtain C_{D2} from chart 17-C or 17-D.

(2) Obtain C_D of combined through an left-turning movement from Charts 3–6 15, using:

W_A = approach width exclusive right-turn lane, and

$R = 0\%$.

(3) Obtain D_2 from chart 17-E for V_2 C_{D2}, whichever governs.

(4) Determine possible capacity, wh required:

Right-turn lane, $C_{P2} = 1.3 \times C_{D2}$

Through and left-turn movement, co bined, $C_P = f \times C_D$.

Values for f are given in table A charts 3–6 or 5.1

In this procedure, the design capacity of t intersection approach cannot be taken as t sum of separate values of the capacity of t right-turn lane and that of the through-pl left lanes, owing to the traffic pattern dema of the intersection. Separate comparisons the volumes of individual movements wi their capacities are necessary. This aspect h been previously covered with regard separate left-turn lanes.

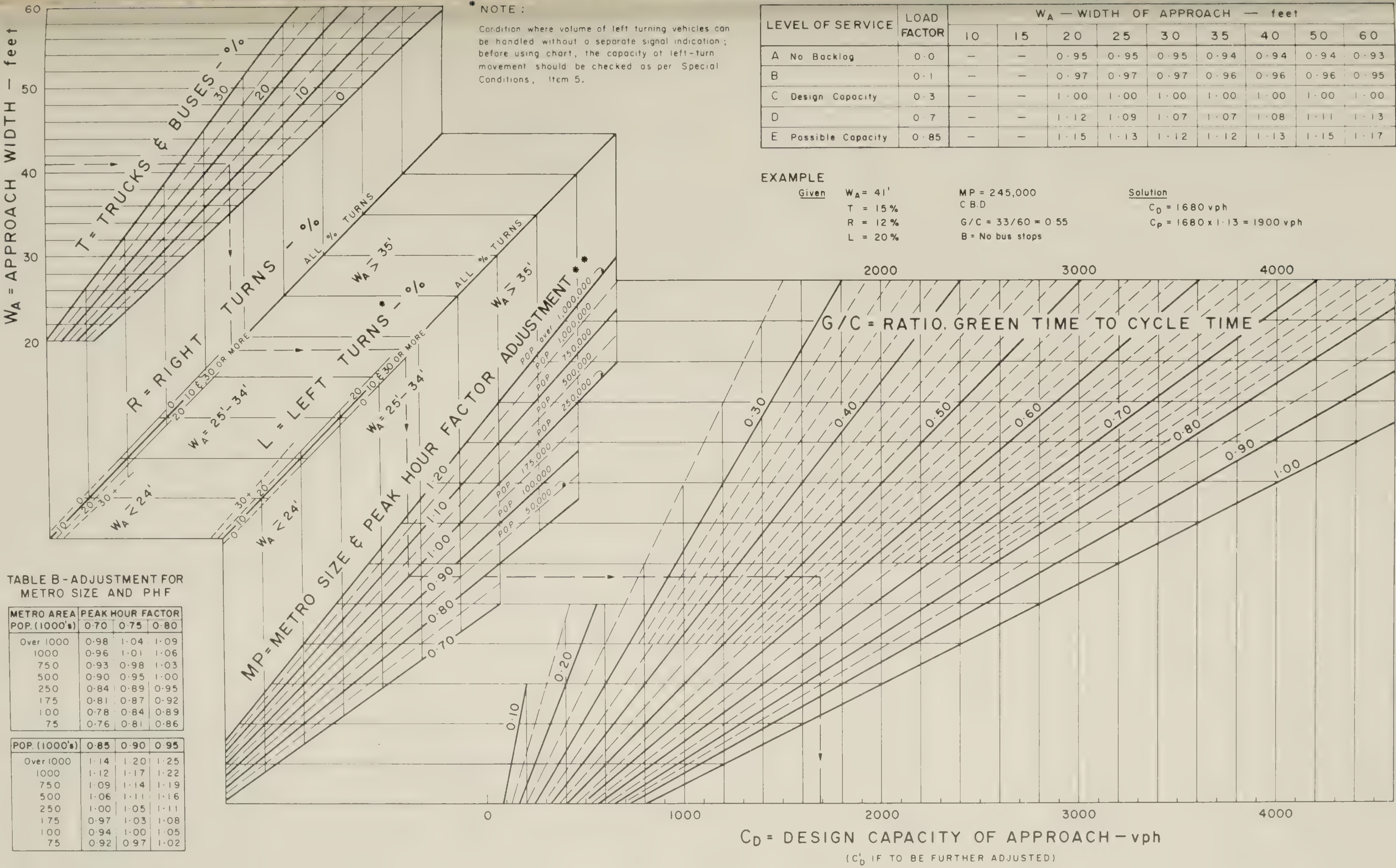
Problem 16

What is the maximum approach vol that can be accommodated without exceedi design capacity for the conditions indicated figure 6. What maximum approach volu can be accommodated without exceeding p sible capacity? What should be the length f the right-turn lane?

Solution: Using the same percentage f trucks in the right-turning movement as the total approach volume, and $G/C = 36/6$ 0.58 in chart 17-C, find $C_{D2} = 330$ v.p.h.

From chart 3, using $W_A = 22$, $T = 12\%$, I 0%, $L = 5\%$, $MP = 250{,}000$ population, d $G/C = 0.58$, the design capacity of combi through and left-turning movement is C 870 v.p.h. On this basis, the right-turn volume $V_2 = (870 \times 25) \div (100 - 25) = 290$ v.p. Because this is less than C_{D2}, the maxim approach volume without exceeding des capacity is $870 + 290 = 1{,}160$ v.p.h.

Possible capacity of the through-plus-f movement is $870 \times 1.20 = 1{,}040$ v.p.h., and c responding right-turning volume is $(1{,}040 \times 5)$ $\div (100 - 25) = 350$ v.p.h. The latter is than the possible capacity of the right-t lane ($C_{P2} = 330 \times 1.30 = 440$ v.p.h., from c 17-C). The maximum approach volume w out exceeding possible capacity is, theref $1{,}040 + 350 = 1390$ v.p.h. The length of ri turn lane required to handle the volume of and 350 v.p.h. is found in chart 17-E; $D_2 =$ and 330 feet, respectively.



* NOTE:

Condition where volume of left turning vehicles can be handled without a separate signal indication; before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5.

TABLE A — ADJUSTMENT FACTOR (T) FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet								
		10	15	20	25	30	35	40	50	60
A No Backlog	0·0	—	—	0·95	0·95	0·95	0·94	0·94	0·94	0·93
B	0·1	—	—	0·97	0·97	0·97	0·96	0·96	0·96	0·95
C Design Capacity	0·3	—	—	1·00	1·00	1·00	1·00	1·00	1·00	1·00
D	0·7	—	—	1·12	1·09	1·07	1·07	1·08	1·11	1·13
E Possible Capacity	0·85	—	—	1·15	1·13	1·12	1·12	1·13	1·15	1·17

EXAMPLE

Given
W_A = 41'
T = 15%
R = 12%
L = 20%

MP = 245,000
C.B.D.
G/C = 33/60 = 0.55
B = No bus stops

Solution
C_D = 1680 vph
C_P = 1680 x 1·13 = 1900 vph

TABLE B - ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP. (1000's)	PEAK HOUR FACTOR 0·70	0·75	0·80
Over 1000	0·98	1·04	1·09
1000	0·96	1·01	1·06
750	0·93	0·98	1·03
500	0·90	0·95	1·00
250	0·84	0·89	0·95
175	0·81	0·87	0·92
100	0·78	0·84	0·89
75	0·76	0·81	0·86

POP. (1000's)	0·85	0·90	0·95
Over 1000	1·14	1·20	1·25
1000	1·12	1·17	1·22
750	1·09	1·14	1·19
500	1·06	1·11	1·16
250	1·00	1·05	1·11
175	0·97	1·03	1·08
100	0·94	1·00	1·05
75	0·92	0·97	1·02

** Use Table B if PHF is known to find adjust factor; otherwise use Population directly

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

ONE-WAY STREET — NO PARKING — C.B.D.

CHART 7

7

C.B.D.

Both Left- and Right-Turn Lanes.—For the condition where lanes are added for both left and right turns, chart 17 is used as before to find capacities and lengths of the separate turning lanes. The following steps are used in the analysis of the intersection approach:

(1) Obtain C_{D3} from charts 17-A or 17-B, and C_{D2} from charts 17-C or 17-D.

(2) Obtain C_D of through movement from charts 3–6, using:

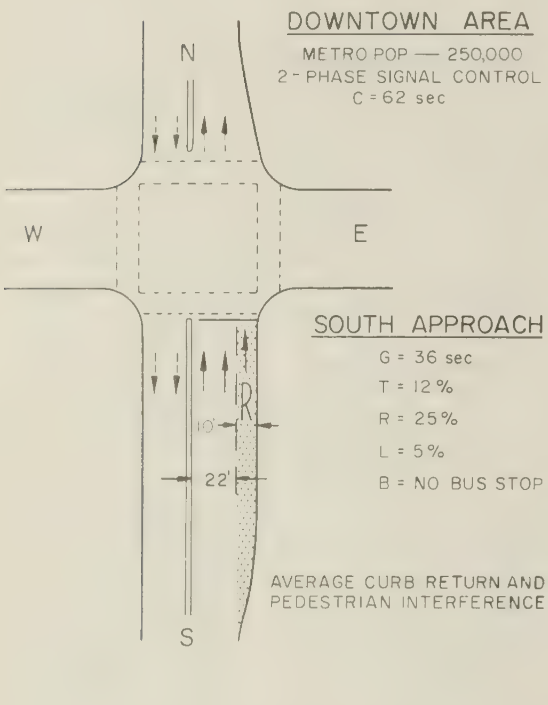


Figure 6.—Problem 16 illustrated.

W_A = approach width exclusive of turning lanes, and
$L = 0\%$ and $R = 0\%$.

(3) Obtain D_2 and D_3 from chart 17-E.

(4) Determine possible capacity, when required:

Left-turn lane, $C_{P3} = 1.3 \times C_{D3}$.
Right-turn lane, $C_{P2} = 1.3 \times C_{D2}$.
Through movement, $C_P = f \times C_D$ (obtain values of f from table A on charts 3–6 or 15).

Problem 17

What is the maximum approach volume that can be handled on the east approach, shown in figure 7, without exceeding design capacity? A large lumber mill to the north on the crossroad accounts for a sizable proportion of vehicles and high percentage of trucks turning right.

Solution: Assuming a large through movement from the east, so that chart 17-B governs, the design capacity of the separate left-turn lane is $C_{D3} = 105$ v.p.h. The design capacity of the separate right-turn lane, using chart 17-*D* ($G/C = 31/56 = 0.55$, $a = 12$, $T_2 = 30\%$ and curve II), is $C_{D2} = 480 \times 0.90 = 430$ v.p.h. Factor of 0.90, according to the footnote in chart 17, is to account for the rural condition of design capacity based on level of service B.

The design capacity of the through movement is found in chart 15, using $W_A = 22$, $T = 6\%$, $R = 0\%$, $L = 0\%$ and normal rural conditions; $C_D = 920$ v.p.h.

On this basis, $V_3 = (920 \times 7) \div (100 - 7 - 28) = 100$ v.p.h., and $V_2 = (920 \times 28) \div (100 - 7 - 28) = 395$ v.p.h.

Since V_2 and V_3 are less than C_{D2} and C_{D3}, the maximum approach volume which can be handled without exceeding design capacity is $920 + 100 + 395 = 1{,}415$ v.p.h.

Turning lanes with separate signal indication

Chart 18, which is similar to chart 17, gives the design capacity and the required length of a right- or left-turn lane, when traffic on this lane moves on a green indication that is separate from the green indications for other traffic on the approach—that is, a right- or left-arrow indication for the turning movement. The only additional term introduced in chart 18 is G', the green interval, in seconds, of separate signal indication for the movement of traffic on a separate turning lane.

With a separate signal indication, the right- or left-turning movement is assumed to be free from interference of other traffic streams and pedestrian movements. For intersections having normal street geometry, the design capacity of a turning lane, corresponding to level of service C, is 800 v.p.h. with 5% trucks in the stream per 10 feet of lane width. For high-type intersections with channelization, the design capacity of a turning lane, corresponding to a level of service C, is increased to 900 v.p.h. including 5% trucks per 10 feet of lane width. The base of 900 v.p.h. for high-type design represents an appropriately adjusted value of the 800 v.p.h. base for normal design indicated in the *Manual*.

Charts 18-A and 18-B give a solution for design capacity in terms of G'/C, width of lane a (10, 11, or 12 feet), and percentage of trucks and buses turning, T_2 or T_3. The design capacity of the turning lane for a given set o[f] conditions is the same whether the moveme[nt] is to the left or to the right and whether th[e] lane is within the normal pavement width o[f] the approach or is an added lane.

The possible capacity of a turning la[ne] with a separate signal indication is 1.3 tim[es] the design capacity given in charts 18-A an[d] 18-B. Values for other levels of service m[ay] be adjusted in accordance with factors give[n] in table 5. For design capacity in rural area[s] predicated on level of service B, the resul[ts] in charts 18-A and 18-B usually would b[e] multiplied by a factor of 0.90.

Chart 18-C provides a solution for the re[quired] length of turning lane that is identic[al] with chart 17-E. Usually the separate sign[al] phase is green while other through mov[e]ments are stopped, and the storage lane mu[st] be long enough to prevent blocking of [the] through lane. To assure that all or nearly [all] vehicles are accommodated during each pha[se], the desirable length given in the chart, bas[ed] on twice the average number of turni[ng] vehicles arriving per cycle, should be us[ed] where possible.

Another consideration in the determinati[on] of turning lane lengths with separate sig[nal] indication is that the lane should also be lo[ng] enough to allow entry of turning vehicles pa[st] a line of stopped, through vehicles. To achie[ve] this, the length of turning lane can also [be] determined in chart 18-C by using the throu[gh] volume divided by the number of throu[gh] lanes to enter the chart. The minimum leng[th] found, based on 1.5 times the average numb[er] of vehicles arriving per cycle, is the desi[gn] indicated to meet the requirement of throu[gh] traffic storage. Frequently, this aspect ca[lls] for a longer lane than may be required just [to] store the turning vehicles. In any event, t[he] larger of the two determinations should [be] used as the length of turning lane.

The procedure for determining the ma[xi]mum approach volume without exceedi[ng] capacity is the same as that previously [ex]plained in conjunction with chart 17. With [a] left-turn lane having a separate signal in-

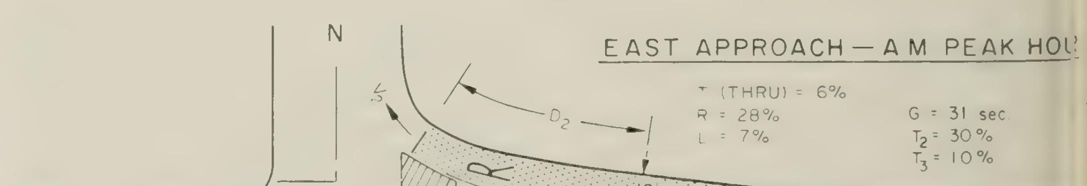


Figure 7.—Problem 17 illustrated.

TABLE A — ADJUSTMENT FACTOR (Y) FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet								
		10	15	20	25	30	35	40	50	60
A No Backlog	0·0	—	—	0·95	0·95	0·95	0·94	0·94	0·94	0·93
B	0·1	—	—	0·97	0·97	0·97	0·96	0·96	0·96	0·95
C Design Capacity	0·3	—	—	1·00	1·00	1·00	1·00	1·00	1·00	1·00
D	0·7	—	—	1·12	1·09	1·07	1·07	1·08	1·11	1·13
E Possible Capacity	0·85	—	—	1·15	1·13	1·12	1·12	1·13	1·15	1·17

*NOTE:
Condition where volume of left turning vehicles can be handled without a separate signal indication; before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5.

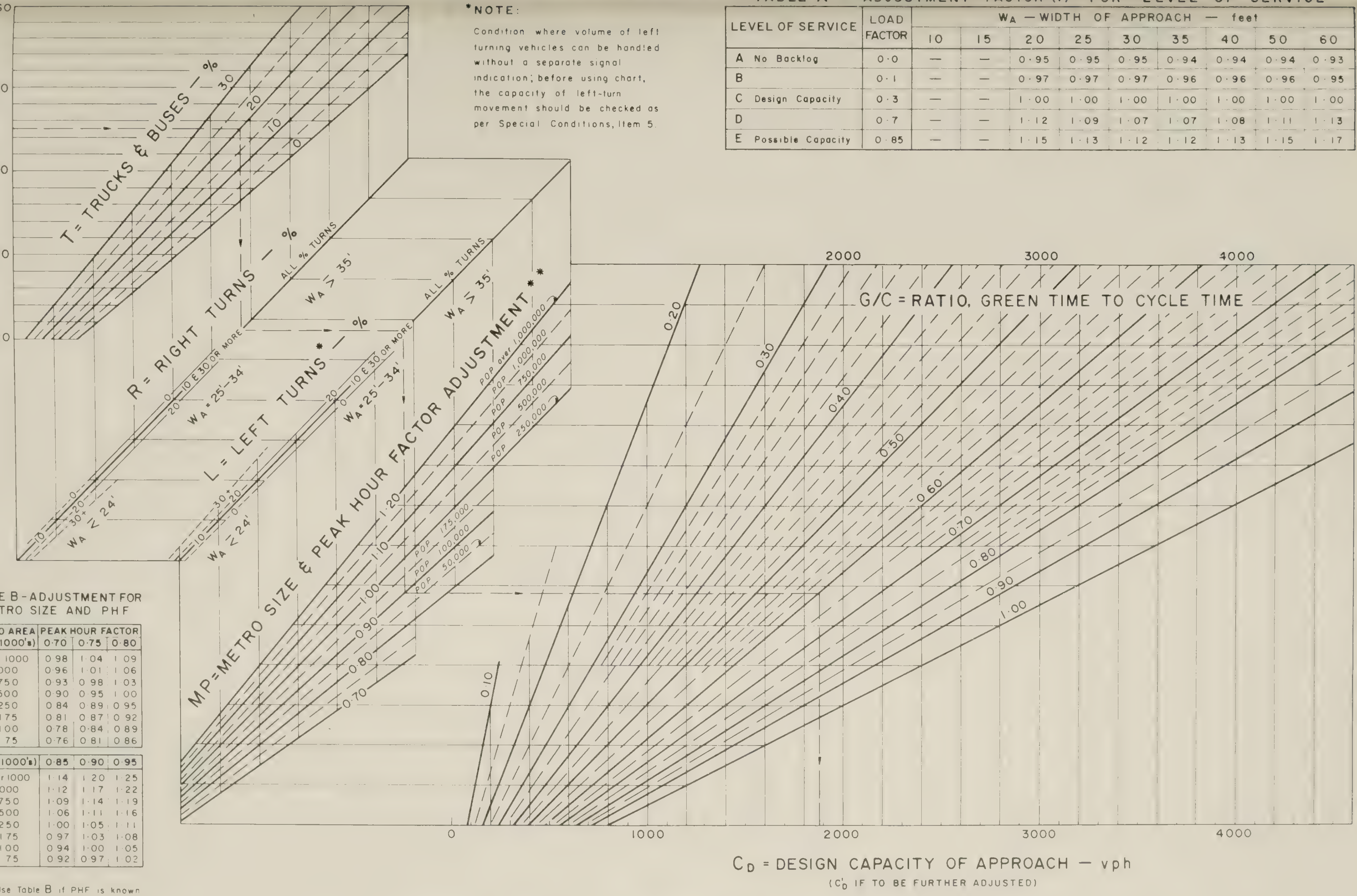


TABLE B - ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP. (1000's)	PEAK HOUR FACTOR 0·70	0·75	0·80
Over 1000	0·98	1·04	1·09
1000	0·96	1·01	1·06
750	0·93	0·98	1·03
500	0·90	0·95	1·00
250	0·84	0·89	0·95
175	0·81	0·87	0·92
100	0·78	0·84	0·89
75	0·76	0·81	0·86

POP. (1000's)	0·85	0·90	0·95
Over 1000	1·14	1·20	1·25
1000	1·12	1·17	1·22
750	1·09	1·14	1·19
500	1·06	1·11	1·16
250	1·00	1·05	1·11
175	0·97	1·03	1·08
100	0·94	1·00	1·05
75	0·92	0·97	1·02

** Use Table B if PHF is known to find adjust factor; otherwise use Population directly

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

ONE-WAY STREET — NO PARKING — FRINGE AREA & O.B.D.

CHART 8

8

FRNG.
O.B.D.

cation, the capacity of the through-plus-right lanes is found by using W_A exclusive of the left-turn lane and $L=0\%$ in charts 3–6 or 15. With a right-turn lane having a separate signal indication, the capacity of the through-plus-left lanes is found by using W_A exclusive of the right-turn lane and $R=0\%$ in charts 3–6 or 15.

$D_2=220$ feet. From Chart 18-B, using $G'/C=0.31$, $a=12$, and $T_2=6\%$ (see arrows) find on the expressway $C_{D2}=350$ v.p.h.; $D_2=400$ feet.

Problem 19

What should be the green interval for each phase on the east approach of the intersection

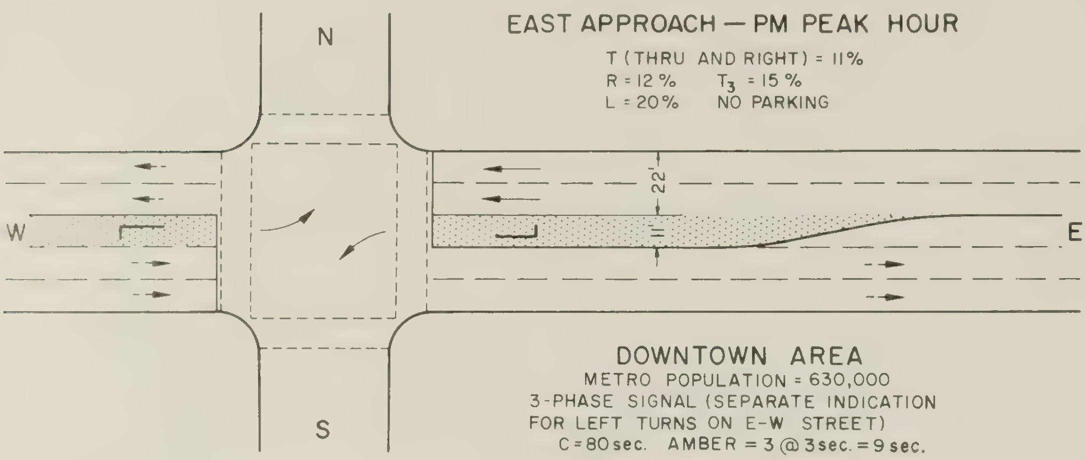


Figure 8.—Problem 19 illustrated.

Problems 18 and 19 demonstrate the use of chart 18.

Problem 18

Determine the design capacity and minimum length of a right-turn lane on a major street, 10 feet wide, normal conditions assumed, for which a separate signal indication of 25 seconds is used out of an 80-second cycle, and on which trucks comprise 6% of the total turning traffic. What would be the design capacity and length required for the same turning volume and signal timing if the turning lane is on an expressway at grade (high-type design)?

Solution: From chart 18-A, using $G'/C=25/80=0.31$, $a=10$, and $T_2=6\%$ (see arrows), find on the major street $C_{D2}=260$ v.p.h.;

shown in figure 8 for operation at design capacity, if a total approach volume of 790 v.p.h. is to be accommodated? What will be the green interval for the movement of traffic on the cross street?

Solution: Volume of left-turning traffic is 20% of 790=160 v.p.h. The volume of through-plus-right movements to be accommodated on a width of 22 feet is 790−160=630 v.p.h. The proportion of green time required for this movement, from chart 3, with $W_A=22$, $T=11\%$, $R=12\div(100-20)=15\%$, $L=0\%$, MP=population of 630,000 and $C_D=630$, is $G/C=0.40$. $G=80\times0.40=32$ seconds.

The proportion of green time required for the separate phase of left-turn lane is obtained from chart 18-A. Using a volume of 160 v.p.h $T_3=15\%$, and $a=11$ feet, obtain $G'/C=0.1$ Hence $G'=80\times0.19=15$ seconds. The lengt of left-turning lane required, from chart 18-(is $D_3=200$ feet. Green time available fo movement of traffic on the cross street 80−32−15−9 (for amber)=24 seconds.

Problem 20

Determine the total length of green indica tion required during the a.m. peak to accon modate the traffic demand at design capacit on the east approach of the intersection show in figure 5.

Solution: Signal timing based on a combin through-plus-right volume is obtained fro chart 4 using $W_A=20$, $T=(20+50)\div(200-560)=9\%$, $R=200/760=26\%$, $L=0\%$, MP= population of 500,000, and approach volume= 200+560=760 v.p.h.; find $G/C=0.43$.

Check in charts 17-A and 17-B whether tl left-turning volume can be handled without separate signal indication. Using in cha 17-A $V_o=550$, $T_o=70/550=13\%$, $G/C=0.$ and $T_3=20/190=11\%$, C_{D3} is found to negligible; thus, chart 17-B governs and sho a design capacity of 80 v.p.h. The 190 v.p. turning left, therefore, must be handled eith with a much larger G/C on the east approac or with a separate signal indication for the l turn—three-phase control.

Re-entering chart 17-A with $V_o=550$, T_o 13% at the upper left, and with $C_{D3}=V_3=1$ v.p.h. and $T_3=11\%$ at the bottom, find t intersection of $G/C=0.70$. Considering a sep rate phase for the left-turning moveme $G'/C=0.23$ is indicated in Chart 18-A, usi $C_{D3}=V_3=190$ v.p.h., $T_3=11\%$, and $a=11$ fe

Thus, without a separate signal indicati for the left turn, 2-phase control, the G required for the east approach is 0.70. With separate signal indication for the left tu 3-phase control, the G/C required for the e approach (total) is equivalent to 0.43+(3/72 0.04 amber)+0.23=0.70. The total G/C r the east approach is the same with eit phasing. However, the west approach and t approaches on the north-south street shou be analyzed before determining the phasi Another alternative for handling traffic on east approach may involve an advance gre with possibly a lesser total G/C. See *Part Special Conditions, Left-Turn Lane on Adva Green Signal.*

TABLE A — ADJUSTMENT FACTOR [illegible] FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet								
		10	15	20	25	30	35	40	50	60
A No Backlog	0·0	—	—	0·95	0·95	0·95	0·94	0·94	0·94	0·93
B	0·1	—	—	0·97	0·97	0·97	0·96	0·96	0·96	0·95
C Design Capacity	0·3	—	—	1·00	1·00	1·00	1·00	1·00	1·00	1·00
D	0·7	—	—	1·12	1·09	1·07	1·07	1·08	1·11	1·13
E Possible Capacity	0·85	—	—	1·15	1·13	1·12	1·12	1·13	1·15	1·17

* NOTE: Condition where volume of left turning vehicles can be handled without a separate signal indication; before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5.

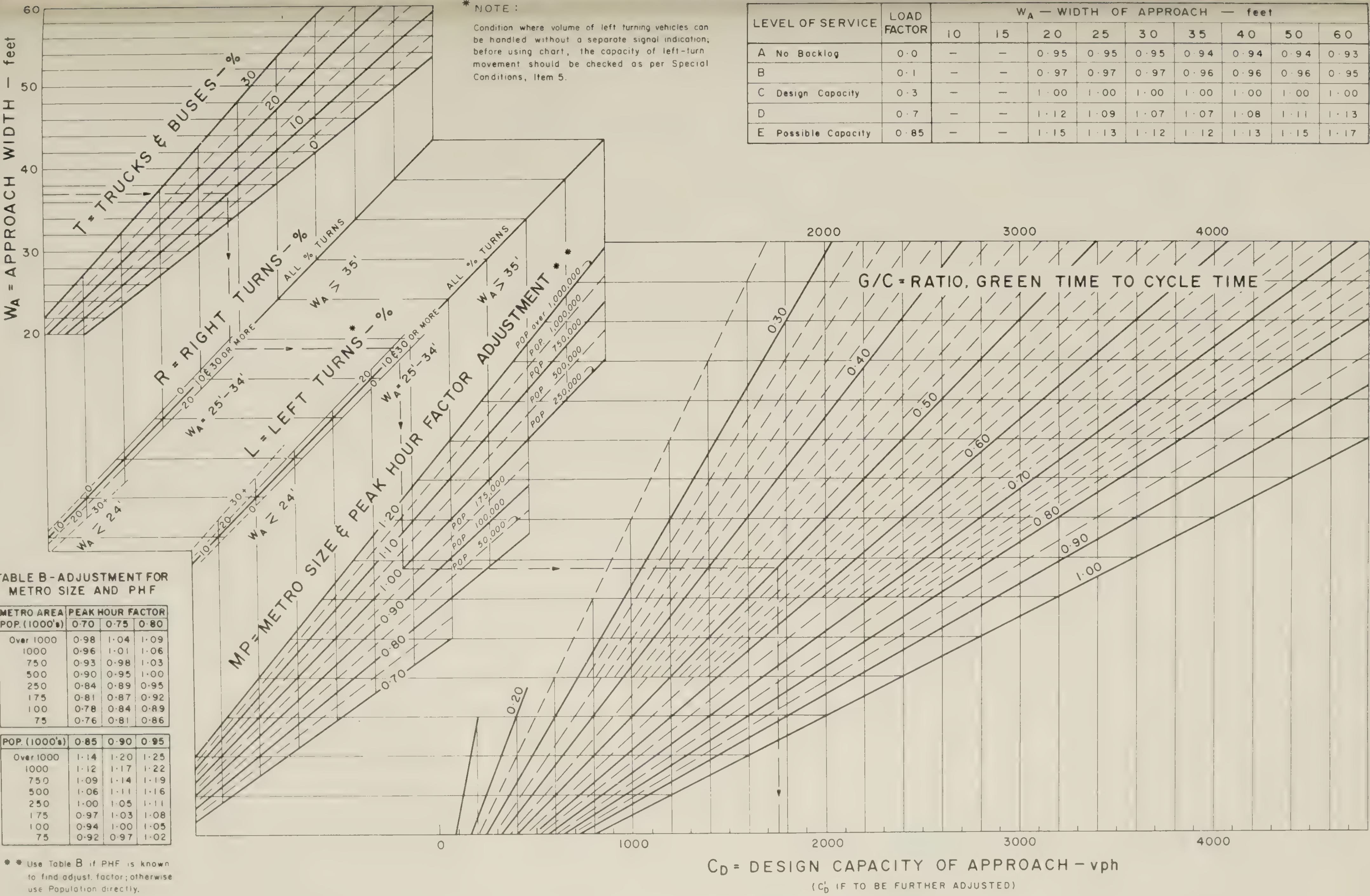


TABLE B - ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP. (1000's)	PEAK HOUR FACTOR 0·70	0·75	0·80
Over 1000	0·98	1·04	1·09
1000	0·96	1·01	1·06
750	0·93	0·98	1·03
500	0·90	0·95	1·00
250	0·84	0·89	0·95
175	0·81	0·87	0·92
100	0·78	0·84	0·89
75	0·76	0·81	0·86

POP. (1000's)	0·85	0·90	0·95
Over 1000	1·14	1·20	1·25
1000	1·12	1·17	1·22
750	1·09	1·14	1·19
500	1·06	1·11	1·16
250	1·00	1·05	1·11
175	0·97	1·03	1·08
100	0·94	1·00	1·05
75	0·92	0·97	1·02

** Use Table B if PHF is known to find adjust. factor; otherwise use Population directly.

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

ONE-WAY STREET — NO PARKING — RESIDENTIAL AREA

CHART 9

9 RES.

Intersections With Average Conditions

As with two-way facilities, charts were developed for evaluation of capacities of 1-way streets. For purposes of planning and preliminary design, chart 2 is presented in simplified form, using average conditions. It allows the determination of design capacity of 1-way street approaches with and without parking for various locations within metropolitan areas of different sizes.

Problem 21

An intersection on a 1-way street, 42 feet wide, with parking on both sides, located in the outlying business district of a metro area having a population of 750,000, is assumed to be operating under average conditions. What is the design capacity if the signal is so timed that $G/C=0.60$? What will be the possible capacity?

Solution: In chart 2, using $W_A=42$, *OBD–PKG BOTH SIDES* curve, and $G/C=0.60$, read for metro size of 750,000, $C_D=1{,}700$ v.p.h. From Table 2, $f=1.28$ and $C_P=1{,}700\times 1.28=2{,}170$ v.p.h.

Streets Without Parking

Charts 7, 8 and 9 include the adjustments for specific intersection conditions on 1-way streets without parking. Chart 7 is applicable to central business districts, chart 8 to fringe areas and outlying business districts, and chart 9 to residential areas.

Chart format and procedural steps in the solution of problems are similar to those for 2-way streets, including tables A and B, which provide adjustments for various levels of service and peak-hour factors. Where bus stops are present, chart 16-A is applicable.

Problem 22

A 41-foot street in a downtown area of a metro area of 245,000 population is converted to 1-way operation with no parking. Other conditions are $T=15\%$, $R=12\%$, $L=20\%$, $B=$ no bus stop, and 35% of the cycle time must be devoted to the cross street. What is the design capacity of the 1-way street if 2-phase signal control is used with $C=60$ seconds and each amber interval=3 seconds? What will be the possible capacity?

Solution: Green time that must be allotted to the cross street is 35% of 60=21 seconds. Green time available for the 1-way street is $60-21-(2\times 3)=33$ seconds, and $G/C=33/60=0.55$.

Using chart 7, with $W_A=41$, $T=15\%$, $R=12\%$, $L=20\%$, $MP=$ population of 245,000 and $G/C=0.55$, obtain $C_D=1{,}680$ v.p.h.

$C_P=1{,}680\times 1.13=1{,}900$ v.p.h.

Streets With Parking—One Side

Charts 10 and 11 show capacity relations for 1-way streets with parking on one side only. The same charts are applicable whether the parking is on the left or on the right side. The only difference to be accounted for is when bus stops are present. With parking on the right, chart 16-B for bus stops, with parking, should be used to adjust design capacity values in charts 10 and 11. With parking on the left, chart 16-A for bus stops, with no parking, should be used to supplement the results in charts 10 and 11.

Problem 23

Determine the design capacity of a 33-foot, 1-way street with parking on the right side at a cross street which requires an 18-second green interval within a 75-second cycle. The intersection is situated in the fringe area of a city having a metro population of 650,000. Other conditions are $T=12\%$, $R=20\%$, $L=10\%$, bus stop on the near side serving 48 buses during the peak hour, and two 3-second amber periods per cycle.

Solution: Enter chart 10 with $W_A=33$ feet and proceed through the chart in accordance with the conditions given, using $G/C=1.00-18/75-6/75=0.68$ (see arrows); read $C_D=1{,}270$ v.p.h. In chart 16-B, using 48 buses per hour, fringe area, and $W_A=33$ feet, find a local bus factor of 1.09. Design capacity is $C_D=1{,}270\times 1.09=1{,}380$ v.p.h.

Problem 24

If, in Problem 23, all conditions remain the same, except that parking is shifted from the right to the left side, what will be the design capacity?

Solution: C_D, as above, is 1,270 v.p.h. Chart 16-A, with no parking, however, is applicable and produces a local bus factor of 0.85. Design capacity is $C_D=1{,}270\times 0.85=1{,}080$ v.p.h.

Streets With Parking—Both Sides

Capacities of 1-way streets with parking on both sides are covered in chart 12 for downtown and fringe areas, chart 13 for outlying business districts, and 14 for residential areas.

Problem 25

Determine the design capacity and possib[le] capacity of a 41-foot, 1-way street with par[k]ing on both sides in the *CBD* of a metro ar[ea] with a population of 175,000. Other conditio[ns] are: $T=15\%$, $R=28\%$, $L=9\%$, $G/C=0.4$[?] and no bus stops.

Solution: Proceed through chart 12 [in] accordance with the given conditions (s[ee] arrows); find $C_D=900$ v.p.h.

$C_P=900\times 1.28=1{,}150$ v.p.h.

Intersections With Separate Turnin[g] Lanes

Separate turning lanes on 1-way stree[ts] have the same operational characterist[ics] and capacities as comparable turning la[nes] on 2-way facilities. Capacities of exclusi[ve] left- and right-turning lanes without [a] separate signal indication on 1-way approach[es] may be found in charts 27-C and 17-D, a[nd] required lengths in chart 17-E. With separa[te] signal indication, the relations in chart 18 a[re] applicable. The procedures for evaluati[ng] service volumes and capacities of 1-way a[p]proaches with turning lanes are the same [a]s for 2-way facilities. A left-turn lane wh[en] separate signal indication is not as adva[n]tageous on a 1-way approach as on a 2-w[a]y approach with opposing traffic. Howev[er], separate signal indications for turning la[nes] on 1-way approaches may be employed [in] some channelization arrangements or in c[on]junction with separate pedestrian phases.

Problem 26

If all the conditions are the same as i[n] problem 22, except that an 11-foot lane wit[hin] the 41-foot street is designated exclusiv[el]y for left turns, what is the design capacity [o]f the approach?

Solution: For the through-plus-right mo[ve]ment from chart 7, using $W_A=41-11=30$ feet, $T=15\%$, $R=12\div(100-20)=15\%$, $L=0\%$, $MP=245{,}000$ pop., and $G/C=0.55$, $C_D=1{,}280$ v.p.h.

Left-turning volume $=(1{,}280\times 0.20)\div(100-20)=320$ v.p.h. Check for des[ig]n capacity in chart 17-C, using $G/C=0.55$ a[n]d $T_3=15\%$, is $C_D=300$ v.p.h.; $C_{P3}=300\times 1.$[?] $=390$ v.p.h.

If the left-turn lane is allowed to oper[ate] slightly under level of service C, operati[on] equivalent to design capacity in the appro[ac]h would be $1{,}280+320=1{,}600$ v.p.h. If [no] movement is to exceed design capacity, [the] approach volume would be $1{,}200+300=1{,}$[?] v.p.h.

TABLE A — ADJUSTMENT FACTOR (Y) FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet 10	15	20	25	30	35	40	50	60
A No Backlog	0·0	—	—	0·90	0·89	0·89	0·89	0·88	0·87	0·86
B	0·1	—	—	0·93	0·93	0·93	0·93	0·92	0·91	0·90
C Design Capacity	0·3	—	—	1·00	1·00	1·00	1·00	1·00	1·00	1·00
D	0·7	—	—	1·07	1·08	1·10	1·12	1·14	1·10	1·22
E Possible Capacity	0·85	—	—	1·10	1·13	1·16	1·18	1·20	1·25	1·30

*NOTE:

Condition where volume of left turning vehicles can be handled without a separate signal indication; before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5.

EXAMPLE

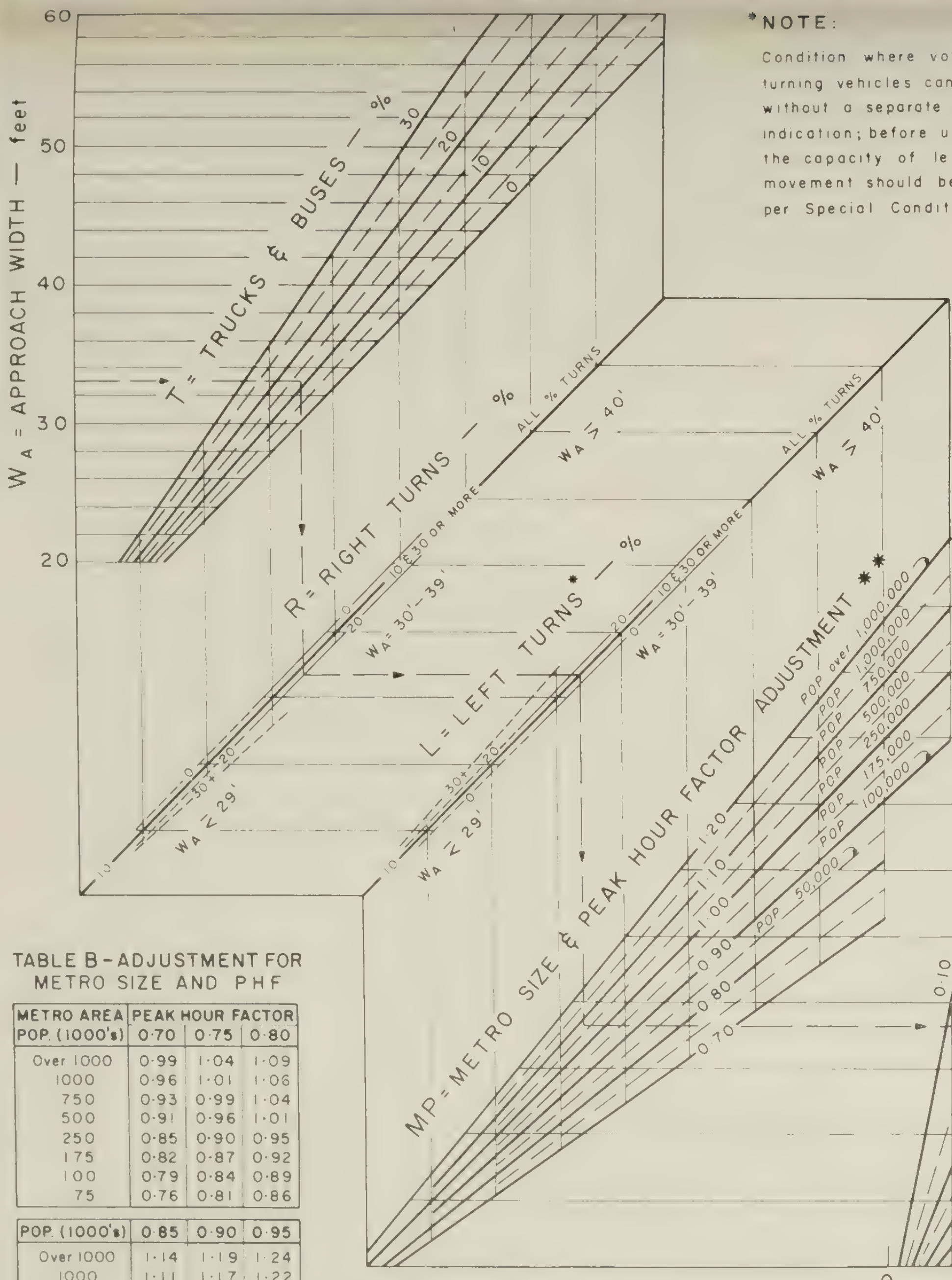
Given

W_A = 33'
T = 12%
R = 20%
L = 10%

MP = 650,000
Fringe Area, Pkg. left side
G/C = 0·68
B = 48/hr., Near-side stop

Solution

C'_D = 1270 vph
C_D = 1270 x 1·09 = 1380
(Factor 1·09 from Chart 16-B; if parking were on left instead, factor from Chart 16-A would apply.)



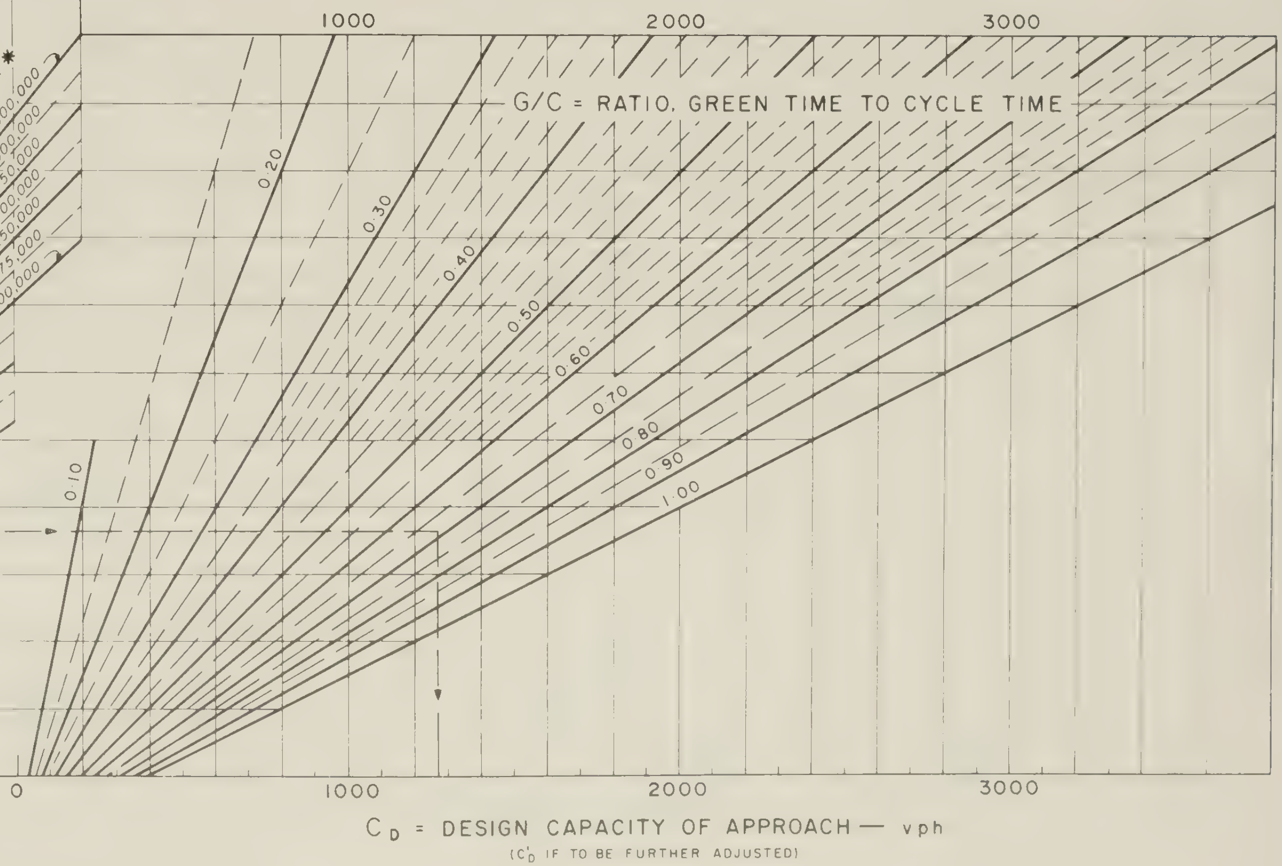


TABLE B – ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP. (1000's)	PEAK HOUR FACTOR 0·70	0·75	0·80
Over 1000	0·99	1·04	1·09
1000	0·96	1·01	1·06
750	0·93	0·99	1·04
500	0·91	0·96	1·01
250	0·85	0·90	0·95
175	0·82	0·87	0·92
100	0·79	0·84	0·89
75	0·76	0·81	0·86

POP. (1000's)	0·85	0·90	0·95
Over 1000	1·14	1·19	1·24
1000	1·11	1·17	1·22
750	1·09	1·14	1·19
500	1·06	1·11	1·16
250	1·00	1·05	1·10
175	0·97	1·02	1·07
100	0·94	0·99	1·04
75	0·91	0·97	1·02

** Use Table B if PHF is known to find adjust. factor; otherwise use Population directly.

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS
ONE-WAY STREET — PARKING ONE SIDE
C.B.D. & FRINGE AREA
CHART 10

10
C.B.D.
FRNG.

PART 3—SPECIAL CONDITIONS

CHARTS 1–15 cover the general conditions found at 2-way and 1-way intersections under traffic signal control. In addition, there are a number of other conditions in the analysis and design of intersections for which certain steps or instructions are to be followed in applying the charts.

Determination of Levels of Service Other Than at the Designated Design Capacity

Levels of service recommended in the *Manual* for design of intersections are C for urban conditions and B for rural conditions. Because of the complexity of intersection operation and the numerous adjustment factors required, some of which are empirical, there seems to be little justification for considering other levels of service in design. Accordingly charts 1–14 and 17–20 are predicated on a design capacity corresponding to level of service C. Chart 15, representing rural conditions, is based on level of service B. Possible capacity, or level of service E, is also considered in the analyses to establish the maximum attainable service volume. Possible capacity is determined by the application of conversion factors in table 2 to the chart design capacity values.

If desired, other levels of service can be evaluated by the application of factors listed in tables 3, 4 and 5, as follows:

(1) To find service volume for some other level of service for a given width of approach and other conditions: find design capacity on chart and multiply result by appropriate factor in table 3, 4 or 5.

(2) To find width of approach or G/C to accommodate a given traffic volume at a desired level of service: divide traffic volume by appropriate factor in table 3, 4 or 5 and enter chart to find the required element.

(3) To determine the level of service for a given traffic volume when all the intersection approach conditions and signal timing are known: find design capacity in chart; divide traffic volume by design capacity; compare result, V/C_D ratio, with factors in table 3, 4 or 5 to determine level of service. An illustration of this procedure may be found in problem 29.

Signal Systems Other Than Fixed Time

The capacity charts and analyses presented thus far are predicated on fixed-time signal control: that is, the signal setting remains unchanged during the analysis period. This also pertains to signal timing which may be varied during certain periods of the day—such as 3-dial control for which a different signal setting is used during the morning peak period, the off-peak periods, and the evening peak period. Utilization of the G/C ratio in capacity analyses is also applicable, with some modification, to actuated control and progressive systems.

Fully actuated control

The proportion of green time during a given hour, or the average G/C, for an intersection approach operated under fully actuated control is the sum of all the green intervals in seconds during the hour divided by 3,600. Because the green interval varies generally in accordance with demand, the actual proportion of green time within the hour cannot be predicted. However, the G/C ratio, based on the average green interval for the hour, would indicate the proportion of green time used. Therefore, since the signal timing varies approximately with the demand flow of vehicles per hour per lane on the several approaches, in accordance with relative capacities of each approach, there is a tendency for the average G/C ratios to approach the G/C ratios of fixed-time control. That is, the average G/C ratio for a given approach under actuated control is approximately equal to the ratio of the demand volume per lane on that approach to the sum of the demand volumes per lane on the two approaches controlling the signal timing during the hour, less the proportion of cycle used for amber periods. This, in essence, is the procedure used for capacity analyses of intersections operated under fixed-time control. Therefore, *for fully actuated signal control the same procedure as for fixed-time control may be employed as a close approximation of the G/C ratios for purposes of planning and design* of intersections. For improvement of existing intersections, field studies to determine average G/C ratios should be undertaken to achieve more precise results. Actually, the fully actuated control is more efficient than fixed-time control because the effect of full actuation is to more nearly fill or load each green interval. Accordingly, the method suggested for analyses in design provides a conservative approach.

Semi-actuated control

The estimation of the average G/C ratio requires consideration of the individual characteristics of the installation, including the green time setting on the minor approach and the minimum cycle length used. For intersection design, the following procedure is suggested:

(1) A trial average G/C for the critical approach on the minor road is determined, using an appropriate chart, as if the intersection were operated under fixed-time control.

(2) Usually, the green time on the minor road is kept to a minimum taking the hourly approach volume and peak-hour factor into consideration, but usually not less than a fixed interval of 15 seconds.

(3) A minimum C value is selected which would yield a G/C ratio approximately equal to or greater than that originally calculated.

(4) Generally, minimum C is selected be not greater than 150 seconds, preferab 120 seconds or less. When there is no demai on the cross road, the green phase on tl major facility continues and C is extende However, when there are repeated calls fro the side road, the signal alternates in accor ance with the minimum C.

(5) The average G/C on the minor road then adjusted to these controls and tl design of the major road approaches is prec cated on the remaining available G/C.

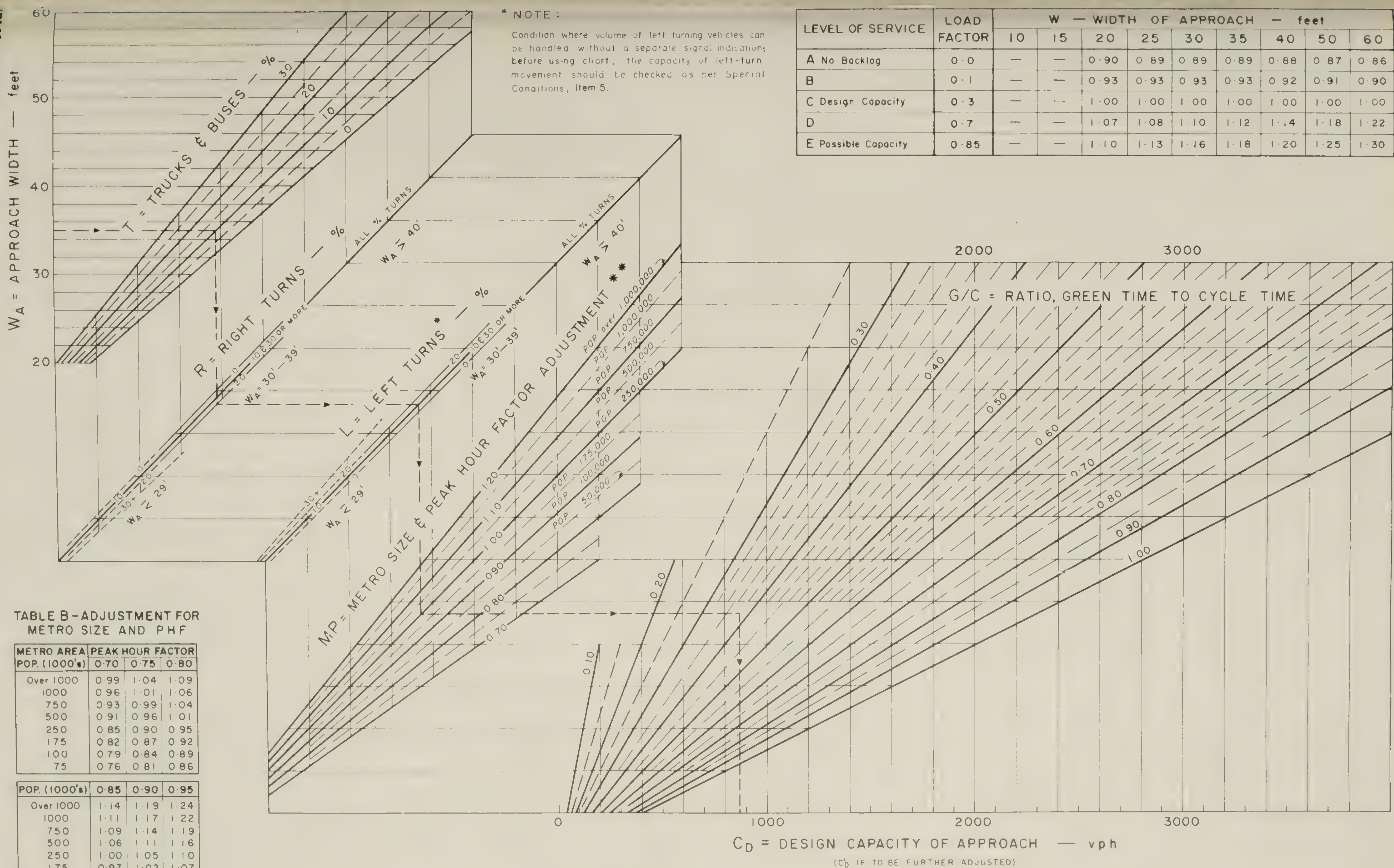
Local practice in the use of the length the green interval on the minor road and tl minimum cycle length may temper the valu given in the above procedure. Caution shou be exercised in determining the initial G/C the first step of the procedure because sen actuated control is used along major highwa in conjunction with cross roads that car light volumes and frequently accommoda peaks of short duration, such as 15 or minutes. In such situations, an *hourly ra of flow* should be used as a basis for dete mining the G/C ratio and the G interval the minor road. The procedure for analyzi approaches with high peaking characteristi is covered in this Part under the headi *Intersections with High Peaking Characteristi*

Progressive signal systems

Signalizations that provide coordinati between successive signals designed to ke platoons of traffic moving continuous through intersections, yield somewhat high capacities than do isolated signals or series signals without coordination. Stated anoth way, for the same volume and G/C, the lev of service provided at an intersection withi progressive signal system would be somewh higher than at an isolated, fixed-time sig controlled, intersection. The increase service volume or capacity is difficult estimate and may not be significant unle near perfect progression is achieved. To safe, the increase usually is not taken i consideration in the design of geomet requirements of intersections. However, 1-way streets with very efficient signal progre sion load factors equal to or approaching 1. might be achieved. The design and possib capacities under such circumstances, therefo may be increased using tables 3 and 4, by t ratio of the f value for a load factor of 1. (extrapolated) to the f value for a load fact of 0.85. Usually, these values calculate to reasonably consistent ratio of 1.05; th design and possible capacities for signaliz intersections within an excellent progressi system may be determined by multiplyi the chart values by 1.05.

Problem 27

A 2-way minor road intersects a maj street in an outlying residential area of a ci having a metro population of ½ million. T major facility requires a G/C of not less th



* NOTE:

Condition where volume of left turning vehicles can be handled without a separate signal indication; before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5.

LEVEL OF SERVICE	LOAD FACTOR	W — WIDTH OF APPROACH — feet								
		10	15	20	25	30	35	40	50	60
A No Backlog	0·0	—	—	0·90	0·89	0·89	0·89	0·88	0·87	0·86
B	0·1	—	—	0·93	0·93	0·93	0·93	0·92	0·91	0·90
C Design Capacity	0·3	—	—	1·00	1·00	1·00	1·00	1·00	1·00	1·00
D	0·7	—	—	1·07	1·08	1·10	1·12	1·14	1·18	1·22
E Possible Capacity	0·85	—	—	1·10	1·13	1·16	1·18	1·20	1·25	1·30

TABLE B–ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP. (1000's)	PEAK HOUR FACTOR 0·70	0·75	0·80
Over 1000	0·99	1·04	1·09
1000	0·96	1·01	1·06
750	0·93	0·99	1·04
500	0·91	0·96	1·01
250	0·85	0·90	0·95
175	0·82	0·87	0·92
100	0·79	0·84	0·89
75	0·76	0·81	0·86

POP. (1000's)	0·85	0·90	0·95
Over 1000	1·14	1·19	1·24
1000	1·11	1·17	1·22
750	1·09	1·14	1·19
500	1·06	1·11	1·16
250	1·00	1·05	1·10
175	0·97	1·02	1·07
100	0·94	0·99	1·04
75	0·91	0·97	1·02

* * Use Table B if PHF is known to find adjust. factor; otherwise use Population directly.

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

ONE-WAY STREET — PARKING ONE SIDE

O.B.D. & RESIDENTIAL AREA

CHART 11

11

O.B.D.
RES.

0.75. The minor road has a demand volume of 200 v.p.h. in the predominant direction; other conditions on the approach are: $T=28\%$, $R=20\%$, $L=35\%$, no parking, and no bus stops. A semi-actuated signal control is needed with a green interval preferably not less than 18 seconds on the crossroad. Determine the number of lanes on the minor approach and the signal timing for operation at design capacity.

Solution: Since the major facility is not to be altered, the number of lanes required on the minor approach may be estimated on the basis of a fixed-time signal. Because the average cycle is apt to be long, assume that only 5% of the cycle would be devoted to amber periods (A). The average G/C for the minor approach equals, tentatively, $1.00-0.75-0.05=0.20$. Using the indicated conditions in chart 4, $W_A=19$ feet, minimum. Employing 11-foot lanes for this facility, a W_A of 22 feet is selected. Using this width and other conditions, as before in chart 4, a G/C of 0.16 is required to handle 200 v.p.h.; or, holding $G/C=0.20$, a volume of 250 v.p.h. can be accommodated.

The minimum cycle length may be calculated as follows: G/C on major street, plus G on minor street$\div C$, plus A(per cycle)$\div C$ equals 1.00: for $G=18$ seconds, and $A=6$ seconds, $0.75+18/C+6/C=1.00$, or $C=95$ seconds. If a longer minimum cycle of, perhaps, 120 seconds is desired, G on the minor road can be increased to $120\times.20=24$ seconds; or, if 20 seconds of green is set with a 120-second cycle, $G/C=20/120=0.17$; then, additional green time can be devoted to the major street, so that $G/C=(1.00-0.17-0.05)=0.78$.

Intersections With High Peaking Characteristics

Intersection capacities vary with city size and peaking characteristics. Normally, peak-hour factors are within a range of 0.80 to 0.90. Where such factors are not available, which often occurs in design, the capacity analysis may proceed with the knowledge only of metro population as provided for in charts 3–14.

Some intersection approaches serve localized traffic generators that produce high peaks within a 15- or 30-minute period, while the other approaches at the same intersection may be operating under normal peaking conditions. The low peak-hour factor on one approach under such circumstances should be accounted for in evaluation of capacities. This may be done, if the peak-hour factor is 0.70 or more, by using an appropriate adjustment factor in charts 3–14, found in table B on the charts for the particular metro size and PHF. Where the PHF is less than 0.70 in urban areas, the following procedure should be used:

(1) Find C_D in charts 3–14, using metro size directly.

(2) Divide C_D by the following average peak-hour factors: 0.80 for metro population of 50,000 or less, 0.85 for metro population of 100,000–750,000 and 0.90 for metro population of 1,000,000 or more.

(3) Multiply above by PHF determined for the approach to establish design capacity.

In rural areas, find C_D in chart 15 using $PHF=1.00$; then, multiply result by PHF relevant to the approach to determine design capacity.

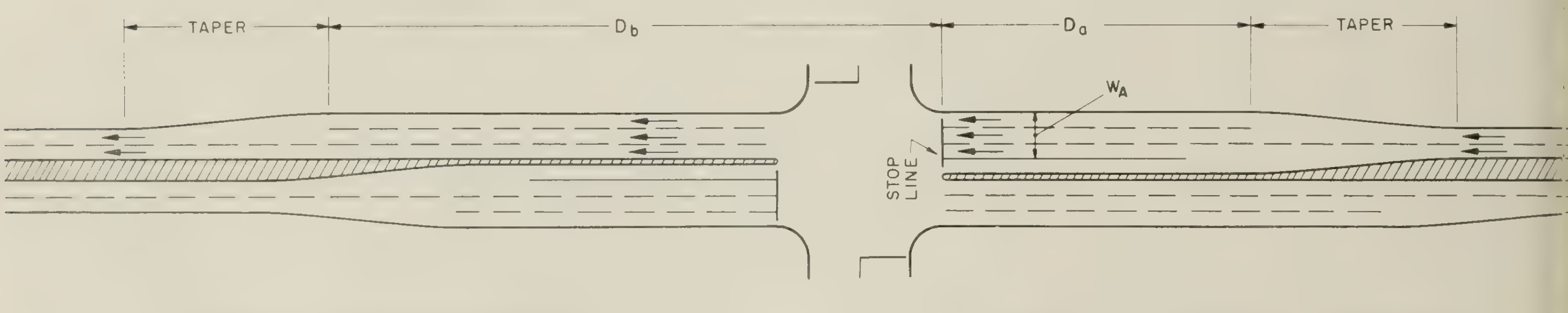
Problem 28

In the outlying district of a metro area of one million population, a road intersecting an arterial facility serves an industrial plant. A G/C of 0.38 is available for moving traffic on the p.m. peak on the crossroad. During this period the volume in one direction on the critical approach is 840 v.p.h.; and, during the highest 15 minutes of the hour, there is a flow of 390 vehicles as determined by field measurements. Other conditions are: $T=5\%$, $R=38\%$, $L=7\%$, no parking, and no bus stops. Determine the number of lanes required on the crossroad approach.

Solution: Because of the high peak within the hour, a PHF of $840\div(4\times390)=0.54$ should be used in the analysis. To find the required width, the chart has to be entered with the demand volume. The demand volume, however, must be adjusted for the special condition of low peak-hour factor. The adjusted volume, as previously explained, in reversed for this problem, is $840\times0.90\div0.54=1{,}400$ v.p.h. Proceed through chart 4 from bottom to upper left using: $C_D=1{,}400$, $G/C=0.38$, MP=population of 1,000,000, $L=7\%$, $R=30\%$ or more, and find $W_A=36.5$ ft. Provide three 12-foot lanes, or $W_A=36$ ft.

Widened Approaches

Intersection capacities can be significantly increased by widening the traveled way through the intersection. This may be accomplished in conjunction with one or more approaches by adding a traffic lane for a certain distance in advance of and beyond the crossroad; or, on streets with parking, an additional lane through the intersection in



LENGTH OF WIDENING BEYOND INTERSECTION

LENGTH REQUIRED FOR: *			TAPER feet
ACCELERATION		MERGING	
DESIGN SPEED mph	D_b — feet		
40	200	D_b = 12 x G	200
50	525	(G, Green interval in seconds)	250
60	900		300

* Use the larger of two values but not less than 300 feet

LENGTH OF WIDENING IN ADVANCE OF INTERSECTION

LENGTH REQUIRED FOR: †			TAPER feet
DECELERATION		STORAGE	
DESIGN SPEED mph	D_a — feet	• Divide approach volume by number of lanes in W_A	
40	150	• Use volume per lane in chart 18-C; find D_2 = D_a on	175
50	200	desirable scale (minimum	225
60	250	scale for restricted conditions)	275

† Use the larger of two values but not less than 200 feet

Figure 9.—Intersection with widened approaches, length requirements.

CHART 12

12

C.B.D. FRNG.

TABLE A — ADJUSTMENT FACTOR (f) FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet								
		10	15	20	25	30	35	40	50	60
A No Backlog	0·0	—	—	—	0·88	0·85	0·84	0·83	0·83	0·82
B	0·1	—	—	—	0·91	0·90	0·90	0·89	0·88	0·88
C Design Capacity	0·3	—	—	—	1·00	1·00	1·00	1·00	1·00	1·00
D	0·7	—	—	—	1·17	1·17	1·17	1·18	1·22	1·25
E Possible Capacity	0·85	—	—	—	1·25	1·25	1·25	1·27	1·32	1·37

*NOTE:
Condition where volume of left turning vehicles can be handled without a separate signal indication; before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5.

EXAMPLE

Given

$W_A = 41'$
T = 15%
R = 28%
L = 9%

MP = 175,000
C.B.D.
G/C = 0·48
B = No bus stops

Solution

C_D = 900 vph
C_P = 900 x 1·28 = 1150 vph

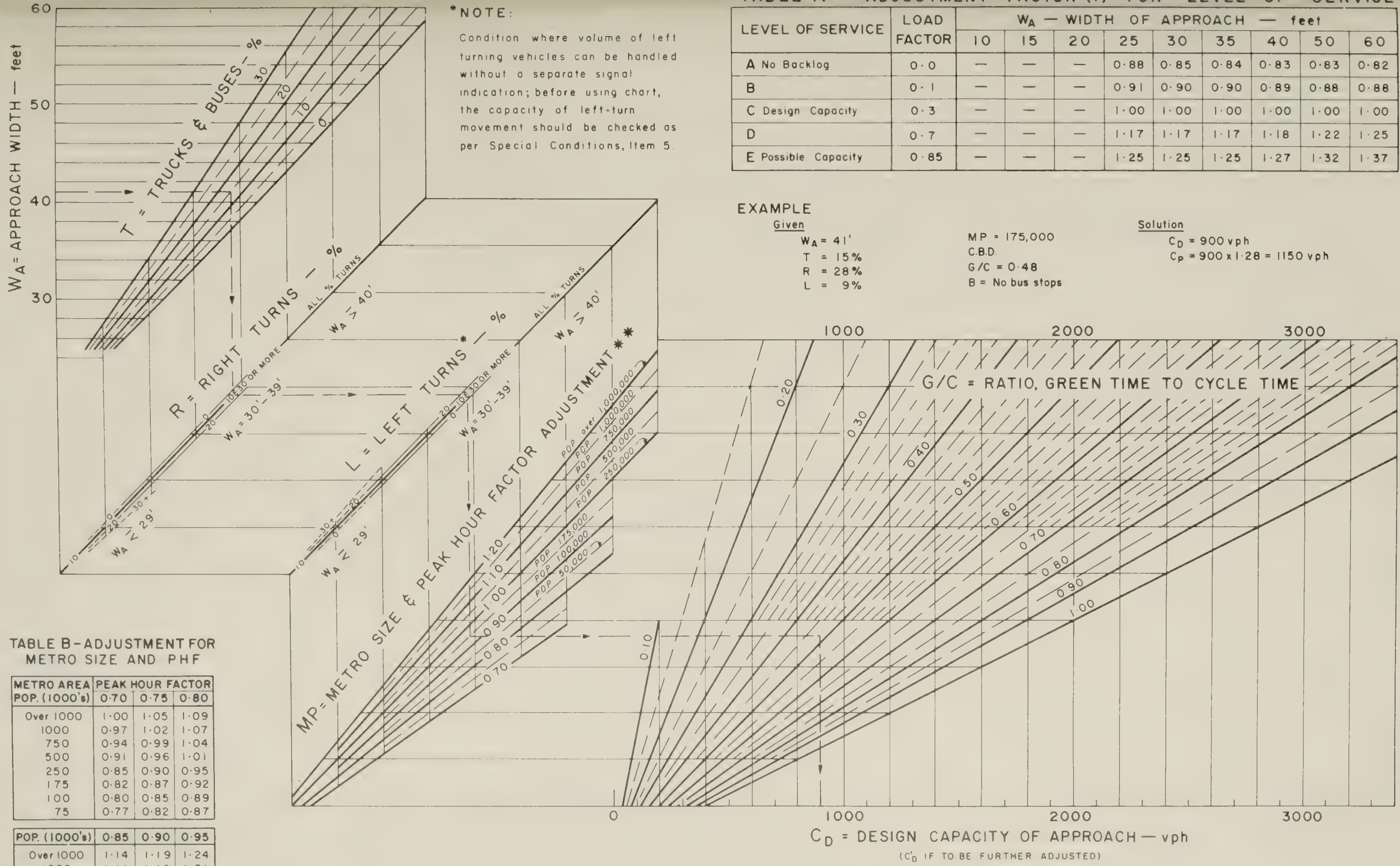


TABLE B-ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP. (1000's)	PEAK HOUR FACTOR 0·70	0·75	0·80
Over 1000	1·00	1·05	1·09
1000	0·97	1·02	1·07
750	0·94	0·99	1·04
500	0·91	0·96	1·01
250	0·85	0·90	0·95
175	0·82	0·87	0·92
100	0·80	0·85	0·89
75	0·77	0·82	0·87

POP. (1000's)	0·85	0·90	0·95
Over 1000	1·14	1·19	1·24
1000	1·11	1·16	1·21
750	1·09	1·14	1·18
500	1·06	1·11	1·16
250	1·00	1·05	1·10
175	0·97	1·02	1·07
100	0·94	0·99	1·04
75	0·92	0·96	1·01

** Use Table B if PHF is known to find adjust. factor; otherwise use Population directly.

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS
ONE-WAY STREET—PARKING BOTH SIDES
C.B.D. & FRINGE AREA
CHART 12

be supplied by partial elimination of parking. Where the extra lane introduced is of sufficient length preceding and following the intersection, the service volumes and capacities will correspond to those for the total width of approach as if the added lane were continuous indefinitely.

Increased width through intersection—no parking

Where the traveled way is widened for a sufficient distance in advance of the intersection and the same widening is continued for a sufficient distance beyond the intersection, enter charts 3–4, 7–9, or 15 with W_A equal to the total width of one approach including the widening on that approach; then use the chart in the normal manner. Required minimum lengths of widening are presented in figure 9. The larger of the two values indicated in the figure for the approach and departure should be used, but not less than 200 and 300 feet, respectively. The widening thus determined would be preceded and followed by appropriate tapers in accordance with design speed.

Elimination of parking through intersection

Where parking on a street approach is eliminated in advance of the intersection and beyond the intersection for a sufficient distance: use chart 3–4 or 7–9, instead of chart 5–6 or 10–14, as if there were no parking on the street. Required minimum distances for elimination of parking to achieve this condition are: First, elimination of parking in advance of the crossroad, measured back from the stop line on the approach, should be for a distance in feet equal to or greater than 8 times the green interval in seconds ($8G$), but not less than 250 feet; and second, elimination of parking beyond the crossroad, measured forward from the stop line on the approach, should be for a distance in feet equal to or greater than 12 times the green interval in seconds ($12G$), but not less than 350 feet.

Problem 29

An isolated intersection at a crossroad in a rural area with normal peaking characteristics is causing congestion on a 4-lane, divided highway. The 2-phase signal control is timed in balance with the demands and capacities of the intersecting facilities and allocates to the major highway a G/C of 0.65 within a 74-second cycle. All lanes are 12 feet wide, and separate left-turn lanes are provided in the median. Design speed is 50 m.p.h. Other conditions on the critical approach are: $T=8\%$, $R=10\%$, $L=5\%$. Determine the level of service on the one approach of the divided highway during the peak hour when the total approach volume is 1,400 v.p.h. Determine to what degree congestion may be alleviated by widening the highway through the intersection, as shown in figure 9, but using the same signal timing. If the uninterrupted flow design capacity of the highway at level of service B is 1,850 v.p.h., how would the signal timing be adjusted—assuming that the crossroad can be improved to operate on a lesser green-time allocation with a G of not less than 18 seconds—to bring the intersection design capacity in balance with the remainder of the divided highway?

Solution: Because the left-turning movement is accommodated in a separate lane, the through-plus-right movement is 95% of 1,400, or 1,330 v.p.h., and R with the left-turning movement removed is $10 \div 0.95 = 11\%$. In chart 15, using $W_A=24$, $T=8\%$, $R=11\%$, $L=0\%$ and $G/C=0.65$, find $C_D=1{,}100$ v.p.h. Ratio of approach volume to design capacity is $V/C_D = 1{,}330 \div 1{,}100 = 1.21$. In table A of chart 15, f for level of service C is 1.11, and for level D is 1.23. Operation, therefore, is at level of service D.

By widening the approach through the intersection by one lane, C_D is increased to 1,530 v.p.h., using $W_A=36$ feet and no change in other conditions. Ratio of approach volume to design capacity is $V/C_D = 1{,}330 \div 1{,}530 = 0.87$, which according to table A in chart 15, operation would be at level of service A. To achieve this design capacity, the lengths of added lane in advance of and beyond the intersection, in each case measured from the stop line on the approach, are as follows in accordance with figure 9:

D_a—for deceleration, length is 200 feet; for storage, using a volume of $1{,}330 \div 3 = 440$ v.p.h. in chart 18–C, length is 480 feet; D_a required is 480 feet plus taper of 225 feet.

D_b—for acceleration, length is 525 feet; for merging, length is $12 \times 0.65 \times 74 = 570$ feet; D_b required is 570 feet plus taper of 250 feet.

To bring the design capacity of the widened intersection in balance with the uninterrupted-flow design capacity of the highway approach, 1,850 v.p.h., the signal timing must be adjusted to accommodate a through-plus-right volume of $1{,}850 \times 0.95 = 1{,}760$ v.p.h. Using chart 15 with the other conditions as before, find $G/C=0.74$. To maintain a green interval of not less than 18 seconds on the crossroad (previously available), assuming an 8-second amber per cycle, the new cycle length is determined from the relation $0.74 + 18/C + 8/C = 1.00$; $C=100$ seconds. The lengths of widening to maintain volumes equivalent to design capacity become $D_a=870$ feet (or minimum of 650 feet), and $D_b=890$ feet.

Check for Capacity of Left Turn

Any intersection approach on a 2-way street that does not involve a separate left-turn lane should be checked for capacity of the left-turn movement. This may be done in the same manner as for a separate left-turn lane as the number of left-turning vehicles that can be accommodated with 2-phase control, whether on a separate lane or not, is governed either by the volume of traffic opposing the left turn or by the length of cycle. Charts 17-A and 17-B should be used to check the capacity of the left-turn movement. Such a check is necessary for every intersection involving 2-way streets. If the volume of left-turning vehicles exceeds the possible capacity as determined in charts 17-A and 17-B, serious congestion may result, the overall capacity of the approach may be materially reduced, and the results determined in charts 3–15 would be erroneous. When this is encountered, the left-turning movement should be prohibited or, if feasible, accommodated on a separate signal indication.

Problem 30

Check whether the left-turn volume in problem 6 can be handled satisfactorily.

Solution: Left-turn volume is 5% of 1,200, or 60 v.p.h. For the conditions given, it is found in chart 17-B that upwards of 80 v.p.h. can be accommodated at design capacity; therefore, the solution in problem 6 is satisfactory.

Problem 31

A 23-foot approach on a 2-way street without parking in the central business district of a metro area of ½ million population operates under the following conditions: approach volume 950 v.p.h., of which 30 v.p.h. turn right and 210 v.p.h. turn left; opposing through volume during the same period is 670 v.p.h.; trucks are generally 8% on all movements; a 36-second green with a 2-phase, 60-second cycle is allotted to this street; there are no bus stops. Determine the level of service on the approach.

Solution: A left-turn check must always be made as a first step in the problem solution on a 2-way facility controlled by a 2-phase signal. In chart 17–A using $V_o=670$, $T_o=8\%$, $G/C=0.60$ and $T_3=8\%$, it is found that few left-turning vehicles can be handled during the given phase. Chart 17–B, therefore, is used and indicates that 96 v.p.h. turning left can be accommodated at design capacity and $96 \times 1.3 = 125$ v.p.h. at possible capacity.

Because the left-turning volume of 210 v.p.h. significantly exceeds the capacity, the left lane on the approach may be largely nullified as a carrier of straight-through traffic during the peak hour. If this condition were ignored in the analysis, an erroneous answer from chart 3 would indicate 860 v.p.h. as the design capacity of the approach—using $W_A=23$, $T=8\%$, $R=14\%$, $L=22\%$, $M=$ population of 500,000 and $G/C=0.60$—and table A on the chart would indicate that, with a factor of $950/860=1.11$, operation is at level of service D. Under these circumstances there is no way of determining the level of service at which the intersection approach would be operating. For example, if the left lane was occupied by vehicles waiting to turn left throughout the hour, capacity would be equivalent to just one lane or C_D of about 600 v.p.h. and C_P of about 720 v.p.h. The latter is approximately equal to the through-plus-right volume of $950-210=740$ v.p.h. Thus, the through-plus-right movement would operate at approximately the possible capacity and the left-turning movement would exceed possible capacity or would operate at level of service F. A solution to this problem will entail either a prohibition of the left turn or an approach widened to include an exclusive left-turn lane using a separate signal phase.

Non-Deterring Turning Movements

On the intersection approach of a 2-way facility where the right-turning path is reasonably direct and pedestrian interference is minor, the right-turning movement may be

13

O.B.D.

TABLE A — ADJUSTMENT FACTOR (f) FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet								
		10	15	20	25	30	35	40	50	60
A No Backlog	0·0	—	—	—	0·88	0·85	0·84	0·83	0·83	0·82
B	0·1	—	—	—	0·91	0·90	0·90	0·89	0·88	0·88
C Design Capacity	0·3	—	—	—	1·00	1·00	1·00	1·00	1·00	1·00
D	0·7	—	—	—	1·17	1·17	1·17	1·18	1·22	1·25
E Possible Capacity	0·85	—	—	—	1·25	1·25	1·25	1·27	1·32	1·37

* NOTE: Condition where volume of left turning vehicles can be handled without a separate signal indication; before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5.

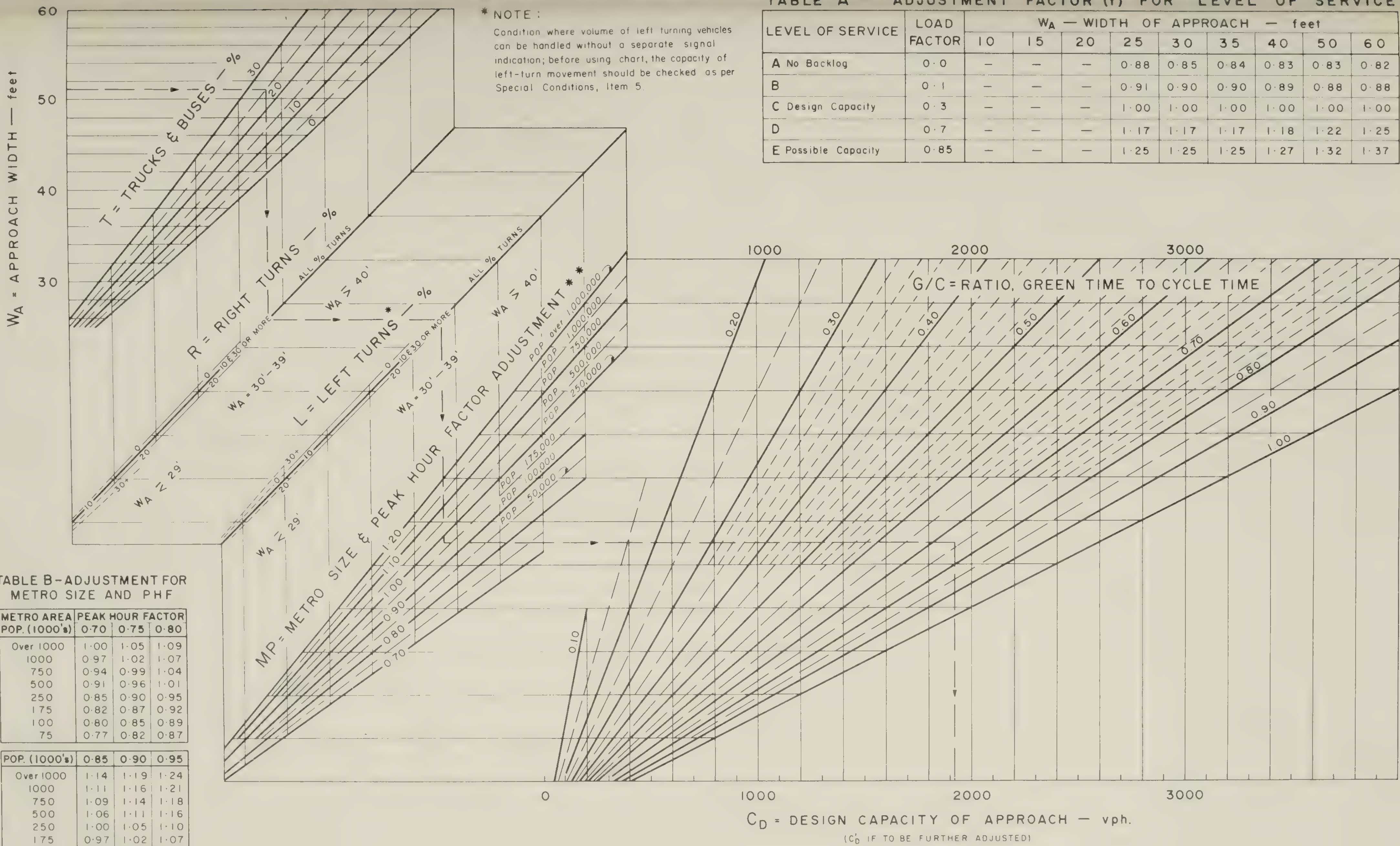


TABLE B — ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP. (1000's)	PEAK HOUR FACTOR 0·70	0·75	0·80
Over 1000	1·00	1·05	1·09
1000	0·97	1·02	1·07
750	0·94	0·99	1·04
500	0·91	0·96	1·01
250	0·85	0·90	0·95
175	0·82	0·87	0·92
100	0·80	0·85	0·89
75	0·77	0·82	0·87

POP. (1000's)	0·85	0·90	0·95
Over 1000	1·14	1·19	1·24
1000	1·11	1·16	1·21
750	1·09	1·14	1·18
500	1·06	1·11	1·16
250	1·00	1·05	1·10
175	0·97	1·02	1·07
100	0·94	0·99	1·04
75	0·92	0·96	1·01

** Use Table B if PHF is known to find adjust. factor; otherwise use Population directly

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

ONE-WAY STREET – PARKING BOTH SIDES – O.B.D.

CHART 13

considered a part of the through movement, in which case $R=0\%$ would be used in the chart solution.

On the intersection approach of a 1-way facility where the turning conditions are as described above, either the right- or left-turning movement, or both, may be considered a part of the through movement. Such conditions are likely to occur at high-type, channelized intersections where turning movements can be accommodated as efficiently as through movements.

left turn is sufficiently light so that it does not require a separate indication, an advance green period may be employed for the first left-turning movement to meet the capacity requirements. The advance green may be in the form of a separate green arrow introduced at the beginning of and simultaneously with the circular green indication for the through movement. The opposing movement is stopped by a red indication while the green arrow is in force. When the green arrow is turned off without using an amber period,

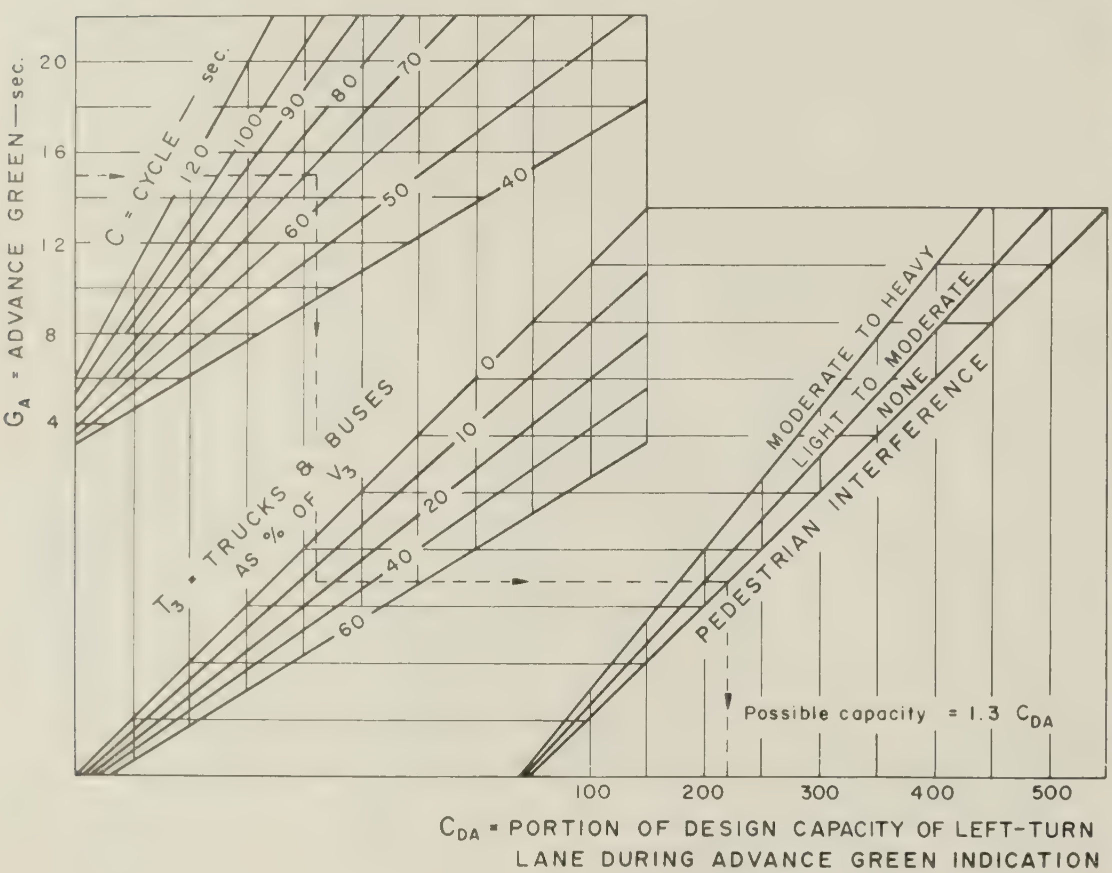


Figure 10.—Left-turn lane capacity on advance green indication.

Left-Turn Lane on Advance Green Indication

Left-turning movements on 2-way facilities that cannot be fully accommodated in the face of opposing traffic under normal 2-phase control require a separate signal indication. The control may be in the form of a third phase devoted to the left-turning movement. This type of signal arrangement is particularly advantageous when the opposing left-turning movement is also relatively high in volume so that both turning movements can be accommodated simultaneously on the same third phase.

When a left-turning movement requires a separate signal indication, but the opposing

the opposing movement is allowed to move and the regular circular green designation on the approach continues unabated. Thus, the left-turning movement is not only accommodated during the green arrow indication, but is allowed to move during the remaining regular green period on the approach as well. The additional left-turning vehicles that can be handled during this latter period depend on the volume of opposing traffic as for any left turn operated without a separate signal indication.

Another form of advance green indication is a flashing green, used effectively in parts of Canada. The initial flashing period on a regular green indication, during which both

through and left turns move on the approach, is equivalent to the green arrow period previously described. When the flashing stops and the green becomes steady, the left turning traffic is under the influence of the opposing through traffic. This form of control, when a full third phase is not required simultaneously to accommodate the opposing left-turning movements, provides considerable efficiency because of the short advance green interval, usually 6 to 12 seconds long, the no amber period at termination of advance green, and the opportunity for left-turning movements to continue during the remaining green phase on the approach. The total number of left-turning vehicles that can be handled by a signal control incorporating an advance green indication may be determined as follows (The *Manual* does not include data or analyses of intersections using advance green indication for left turns; the following material was developed to fill this void in design and operation of arterial streets.)

(1) *Advance Green Period*—Use the nomograph in figure 10 to determine the number of vehicles accommodated per hour at design capacity during the advance green indication.[3]

(2) *Remaining Regular Green Indication*—Use charts 17-A and 17-B in which G/C is exclusive of the advance green to determine the number of vehicles accommodated per hour at design capacity.

(3) *Total Design Capacity of Left-turn Lane*—Sum of design capacities in (1) and (2).

(4) *Total Possible Capacity of Left-turn Lane*—Total design capacity in (3) multiplied by 1.3.

Problem 32

If in problem 20 an advance green indication was used for the left-turning movement, determine the total length of green interval required during the a.m. peak to accommodate the traffic demand at design capacity on the east approach (see figure 5).

Solution: It has already been determined in problem 20 that the number of left-turning vehicles that can be handled at design capacity during the full green interval (2-phase control) on the east approach is 80 v.p.h. with a cycle of 72 seconds (chart 17-B). If an advance green is introduced, the number of vehicles turning left that would have to be accommodated by the advance green is (190−80) = 110 v.p.h. Enter at bottom of nomo-

[3] The nomograph is based on available but limited research. Vehicles accommodated on left-turn lanes under completely loaded conditions (1.00 load factor, representative of possible capacity), as reported from operational studies by F. Capelle and C. Pinnell (5), can be expressed as: $N=0.77$ $(G-1.6)$, where N=number of passenger cars per cycle and G=green interval in seconds. These findings are similar to those of previous research by R. M. Bartle, V. Skoro and D. L. Gerlough (6). Dividing the above expression by the ratio of possible capacity to design capacity previously established and substituting G_A as advance green for G, the formula becomes $N=0.38$ $(G_A-1.6)$. This relation was tested against results of operational studies, carried out by Sanson (1962) and by V. Mitranic (1967) in Toronto, on left-turning movements on advance green indication. The studies by Capelle and Pinnell, when reduced to design capacity, compared favorably with the latter studies which were indicative of normal or average peak-hour operations, particularly for G_A values of 6 to 12 seconds. The nomograph in figure 10 is based on $N=0.38$ $(G_A-1.6)$, with adjustments for trucks and pedestrian interference as previously established.

TABLE A — ADJUSTMENT FACTOR (f) FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet								
		10	15	20	25	30	35	40	50	60
A No Backlog	0·0	—	—	—	0·88	0·85	0·84	0·83	0·83	0·82
B	0·1	—	—	—	0·91	0·90	0·90	0·89	0·88	0·88
C Design Capacity	0·3	—	—	—	1·00	1·00	1·00	1·00	1·00	1·00
D	0·7	—	—	—	1·17	1·17	1·17	1·18	1·22	1·25
E Possible Capacity	0·85	—	—	—	1·25	1·25	1·25	1·27	1·32	1·37

* NOTE:

Condition where volume of left turning vehicles can be handled without a separate signal indication; before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5.

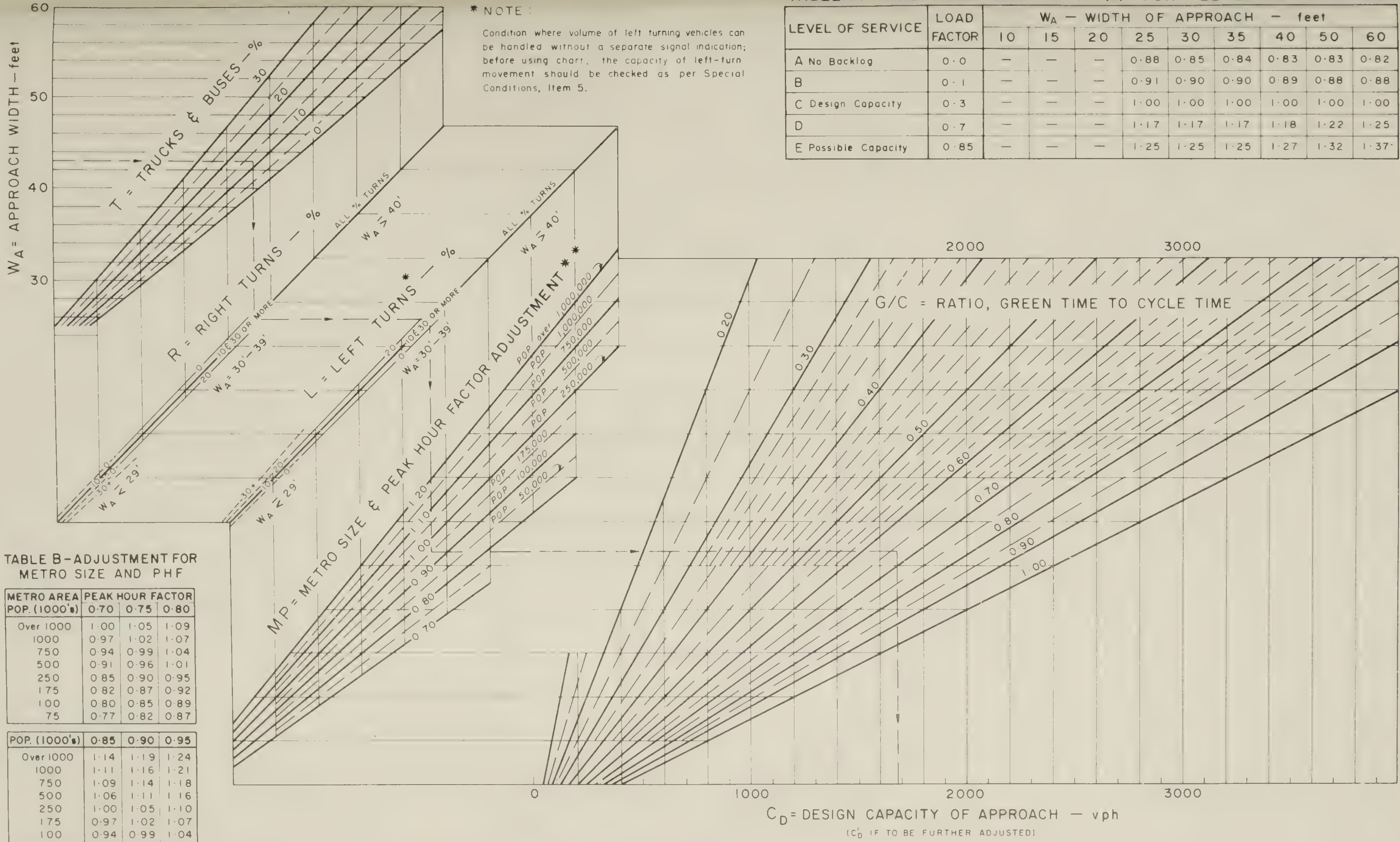


TABLE B—ADJUSTMENT FOR METRO SIZE AND PHF

METRO AREA POP. (1000's)	PEAK HOUR FACTOR 0·70	0·75	0·80
Over 1000	1·00	1·05	1·09
1000	0·97	1·02	1·07
750	0·94	0·99	1·04
500	0·91	0·96	1·01
250	0·85	0·90	0·95
175	0·82	0·87	0·92
100	0·80	0·85	0·89
75	0·77	0·82	0·87

POP. (1000's)	0·85	0·90	0·95
Over 1000	1·14	1·19	1·24
1000	1·11	1·16	1·21
750	1·09	1·14	1·18
500	1·06	1·11	1·16
250	1·00	1·05	1·10
175	0·97	1·02	1·07
100	0·94	0·99	1·04
75	0·92	0·96	1·01

** Use Table B if PHF is known to find adjust. factor; otherwise use Population directly.

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

ONE-WAY STREET—PARKING BOTH SIDES—RESIDENTIAL AREA

CHART 14

14

RES.

graph in figure 10 with a volume of 110 v.p.h. and proceed up and to the left using the conditions: no pedestrians, $T_3=11\%$ and $C=72$; read $G_A=9$ seconds, or $G_A/C=9/72=0.13$.

The G/C required for the through-plus-right movement on the east approach is 0.43, given in chart 4 and shown in problem 20. If $G/C=0.43$ is also required on the west approach, the total green time requirements on the east approach would be $G/C=0.43+0.13=0.56$ in this solution, compared with 0.70 in problem 20. On the other hand, if G/C of 0.30 or less was required on the west

cedures are approximations developed for design purposes only. Right turns controlled by yield sign, and those permitted on red after stop, assume continuously high loading on the street being entered by right turns. The values given, therefore, are considered to be safe for design.

(1) *Continuous right-turning movement on an exclusive lane with an auxiliary lane on the crossroad and turning speeds upwards of 15 m.p.h.*—Design capacity is estimated to be 1,200 passenger cars per hour (p.c.p.h.), or $1{,}200 \div (1+t)$, where t is the proportion of trucks in the turning movement expressed

handled during the green indication on the approach. These merges are considered to take place as a spill-over from the green indication on the approach, plus an occasional entry during the green indication on the crossroad; the several vehicles per cycle are regarded as including an average proportion of trucks. The design capacity, therefore, is as found in chart 17-D for the G/C ratio on the approach as a whole, plus $10{,}800/C$ v.p.h. Possible capacity is 1.3 times design capacity.

(3) *Right-turning movement on exclusive lane controlled by signal with turn permitted on red*—The condition is similar to (2), but less effici

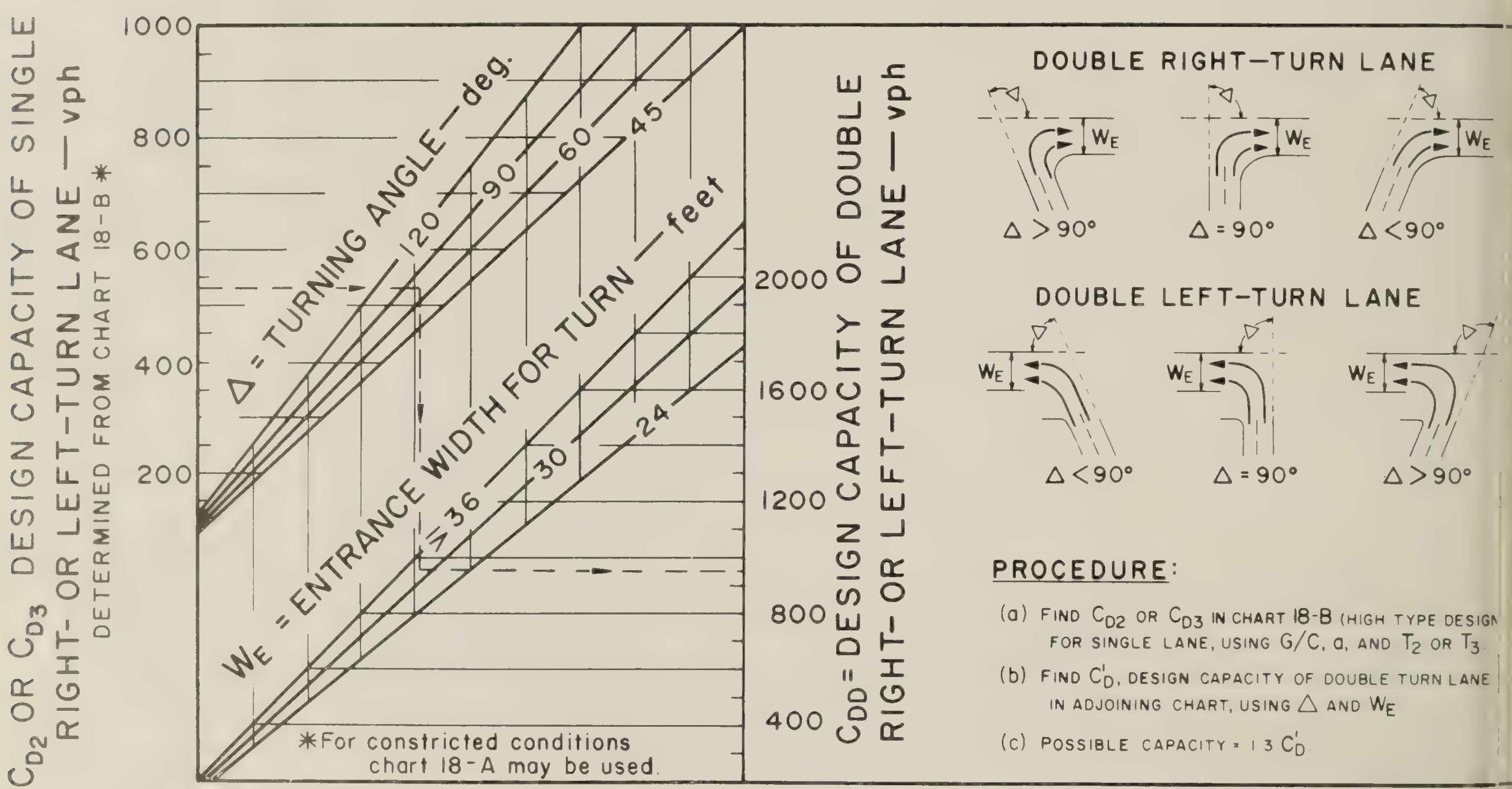


Figure 11.—Capacity of double turning lanes.

approach, the comparison would be 0.43 with 0.70; or the available time for amber and for green on the north-south street would be G/C of 0.57 in this solution and G/C of 0.30 in the solution of problem 20.

Right-Turning Movement—Continuous, Controlled by Yield Sign, or Permitted on Red After Stop

Special right-turning controls for continuous right-turning movement or for right-turning movement either controlled by yield signs or permitted on red after stop, are not covered in the *Manual*. The following suggested pro-

decimally, that is, percentage of trucks divided by 100. Possible capacity is approximately 1.3 times design capacity.

(2) *Right-turning movement controlled by yield sign at the point of crossroad entry, and not by the intersection signal, on an exclusively right-turn lane with channelization*—Capacity depends on the availability of gaps on the crossroad. Because gap availability is difficult to predict, it is assumed that during peak periods there will be little, if any, opportunity for the right-turning movement to merge when the crossroad has a green indication. For design purposes it is assumed that an average of 3 vehicles can merge per cycle in addition to those

To find design capacity, use chart 17- or 17-D, plus 2 vehicles per cycle or $7{,}200/C$ v.p.h. Possible capacity is 1.3 times design capacity.

Capacity Controlled by Intersection Exit

Generally the capacity of the approaches controls the capacity of the intersection. At some locations, however, the width of traveled way beyond the intersection may be restricted or traffic may back up onto the intersection exit from the intersection ahead. The latter situation may be corrected by coordination of signals and by the use of lagging or leading green indications. Where the traveled way

*NOTE:

Condition where volume of left turning vehicles can be handled without a separate signal indication; before using chart, the capacity of left-turn movement should be checked as per Special Conditions, Item 5.

TABLE A — ADJUSTMENT FACTOR (T) FOR LEVEL OF SERVICE

LEVEL OF SERVICE	LOAD FACTOR	W_A — WIDTH OF APPROACH — feet								
		10	15	20	25	30	35	40	50	60
A No Backlog	0·0	0·92	0·95	0·96	0·96	0·97	0·97	0·97	0·97	0·96
B Design Capacity	0·1	1·00	1·00	1·00	1·00	1·00	1·00	1·00	1·00	1·00
C	0·3	1·11	1·11	1·11	1·11	1·11	1·11	1·11	1·11	1·11
D	0·7	1·21	1·21	1·22	1·23	1·25	1·27	1·29	1·31	1·33
E Possible Capacity	0·85	1·28	1·28	1·28	1·30	1·32	1·35	1·38	1·41	1·44

EXAMPLE

Given

$W_A = 22'$ L = 7%

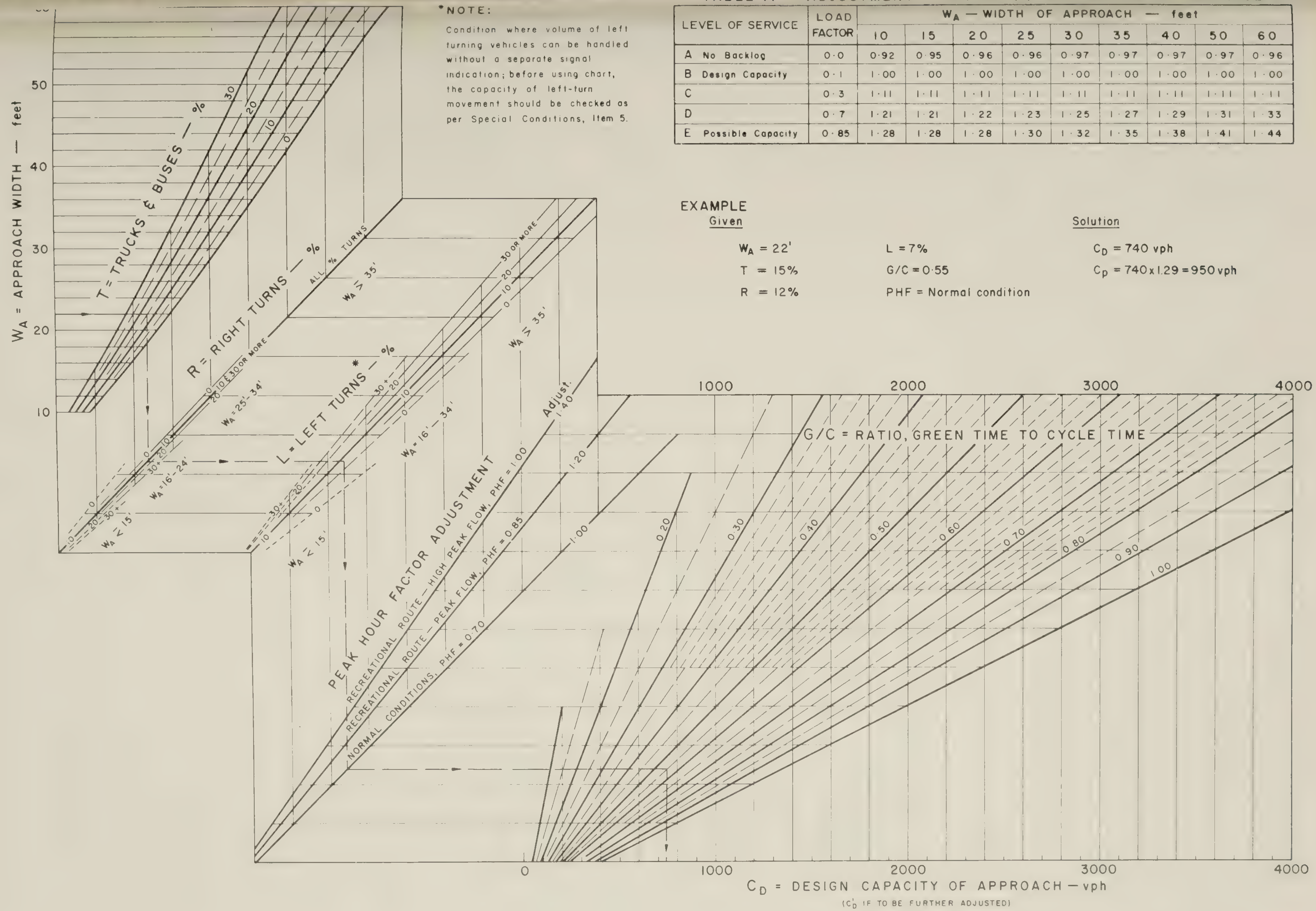
T = 15% G/C = 0·55

R = 12% PHF = Normal condition

Solution

$C_D = 740$ vph

$C_p = 740 \times 1.29 = 950$ vph



DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

RURAL TWO-WAY HIGHWAY — NO PARKING

CHART 15

15

RURAL

available for moving vehicles is narrower on the intersection exit than on the intersection approach, capacities may be estimated as follows: Enter charts 3-14 with W_A equal to the width of the exit. Exit width is measured from division line to curb on 2-way streets, and curb to curb on 1-way streets. Proceed through chart in usual manner, but use T=percentage of trucks and buses in through movement only, $R=0\%$, $L=0\%$, MP=metro population, and G/C equivalent to that used on the approach; adjust for bus stop only if it is on the far side, using $L+R=0\%$.

Double right- or left-turn lane on 2 exclusive lanes

The capacity of a double turning lane may be determined in accordance with the procedure outlined in figure 11 using chart 18-B in combination with the nomograph in figure 11. The procedure is based on the following: According to the *Manual*, at a usual intersection the second or outer lane of a 2-abreast turning movement is capable of handling a service volume or capacity of 0.8 times the value for a turning lane indicated in charts 17 and 18, or an average per lane of 0.9 times the single-lane capacity. This pertains to a turn at or near 90 degrees and with sufficient width available for each line of vehicles throughout the turn. The capacity of the double turning movement is sensitive to the angle of turn, so that for acute-angle turns of more than 90 degrees, the capacity is reduced, and for obtuse-angle turns of less than 90 degrees, the capacity is increased. This sensitivity to angle of turn may be expressed by a factor of less than or more than 0.90.

At some angle of turn flatter than 90 degrees, the full efficiency of the second lane will be achieved so that the 0.90 factor will become 1.00. Moreover, at an extremely flat angle of turn with channelization, the capacity of the movement will approximate the capacity of a usual 2-lane intersection approach as represented in charts 3 and 4 and 7-9. To achieve some correlation between turning lane capacities and regular intersection approach capacities, an adjustment factor of 1.10 has been selected for angles of turn of 45 degrees or less. For an intermediate angle of 60 degrees a factor of 1.00 is designated. On the other hand, for an acute angle of 120 degrees the indicated adjustment factor is 0.80. Because a double-turn movement is usually designed to handle relatively large turning volumes free of pedestrian interference, the capacity for the high-type design, chart 18-B, is generally applicable. The value for a single lane from chart 18-B, therefore, is first determined and then multiplied by a factor of 0.80 to 1.10, depending on the turning angle, to produce the design capacity of a double right- or left-turn lane.

Another characteristic of a 2-abreast turning movement that affects its traffic carrying capabilities is the lateral space available for turning, which determines whether the driver feels a sense of restriction or freedom. At the beginning of the turn the capacity is controlled by the width of approach lanes, 10, 11, or 12 feet. Turning lanes wider than 12 feet on the approach are considered to have the same capacity as 12-foot lanes. Upon completing the turn, the capacity is controlled by the width available for the 2 lanes at the far end of the turn or by the entrance width for the turn on the street receiving the turn, as represented by W_E on the right-hand side of figure 11. Experience shows that approximately a 3-lane width is required at this point to assure the freedom of movement necessary to realize the full capacity of the double turn. An entrance width of 36 feet or more is considered ideal; the adjustment factor therefore is 1.00. Values for W_E of 30 and 24 feet are assumed to yield smaller capacities, expressed by adjustment factors of 0.90 and 0.80 respectively. These factors combine as a product with those given previously for the angle of turn, so that for $\Delta=45°$ and $W_E=36$, the composite factor is $1.10\times1.00=1.10$, and for $\Delta=120°$ and $W_E=24$ the composite factor is $0.80\times0.80=.64$. These factors are applied to twice the design capacity of a single lane and are indicative of the range of values applicable to design of intersections.

Capacity relations as affected by the turning angle, Δ, and the cross-street entrance width, W_E, are shown in the nomograph. Thus, by using the design capacity for a single lane in chart 18-B, the design capacity (or) of the double right- or left-turn lane is found directly in the chart of figure 11.

Problem 33

What is the design capacity and possible capacity of a double left-turning lane 22 feet wide on the approach, two 11-foot lanes, where the angle of turn is 75 degrees and the traveled way, W_E, on the cross street receiving the turn is 33 feet wide? Other conditions are: trucks comprise 17% of the turning movement, and the G/C available for the movement is 0.60.

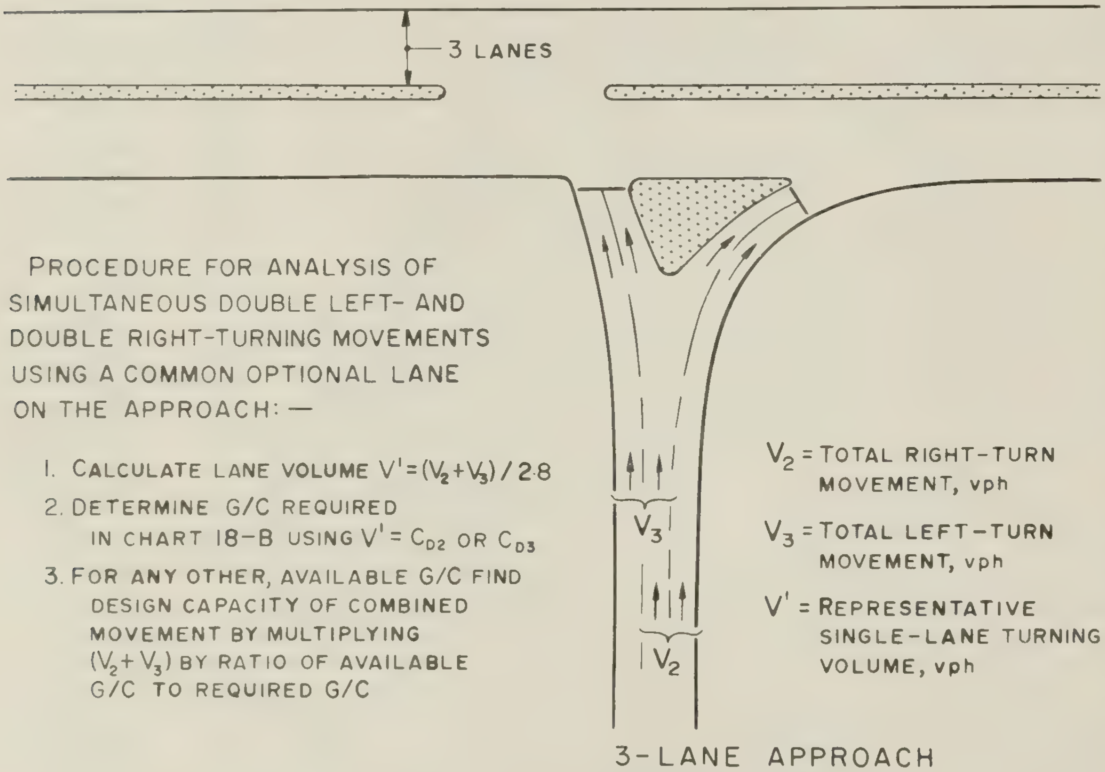


Figure 12.—Combined double left-turning and double right-turning movements with optional lane.

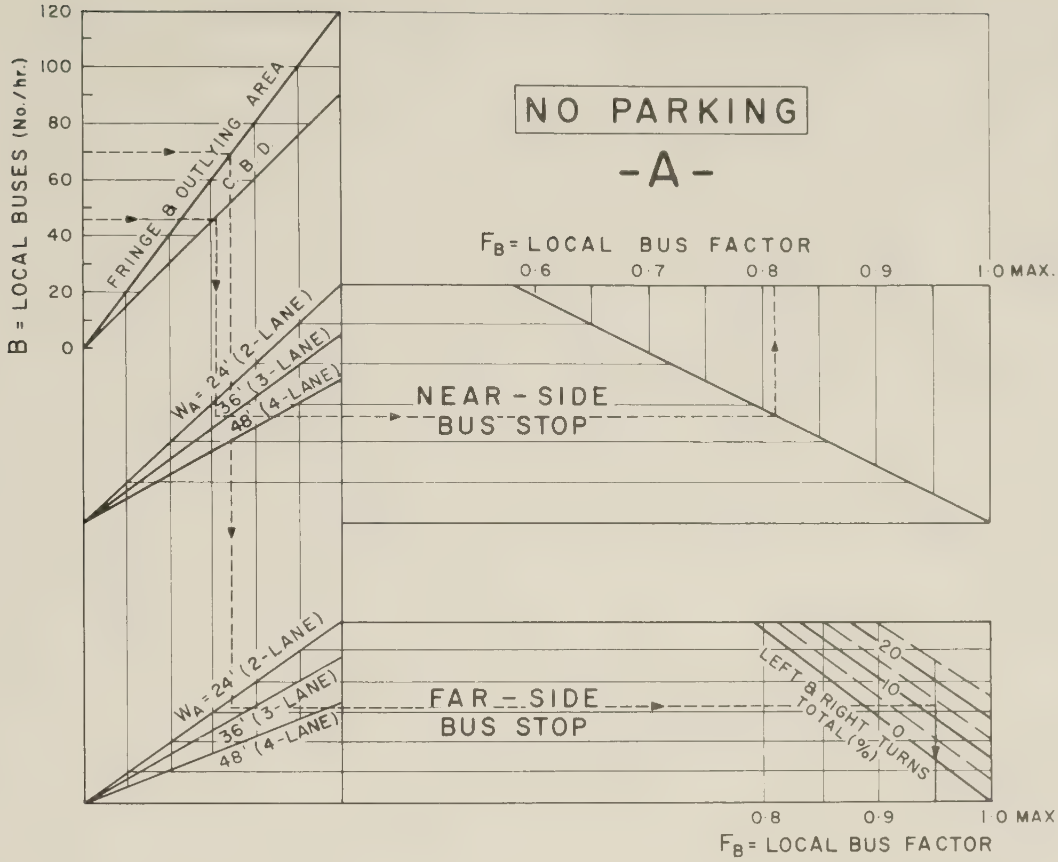
2-Lane Turning Movements

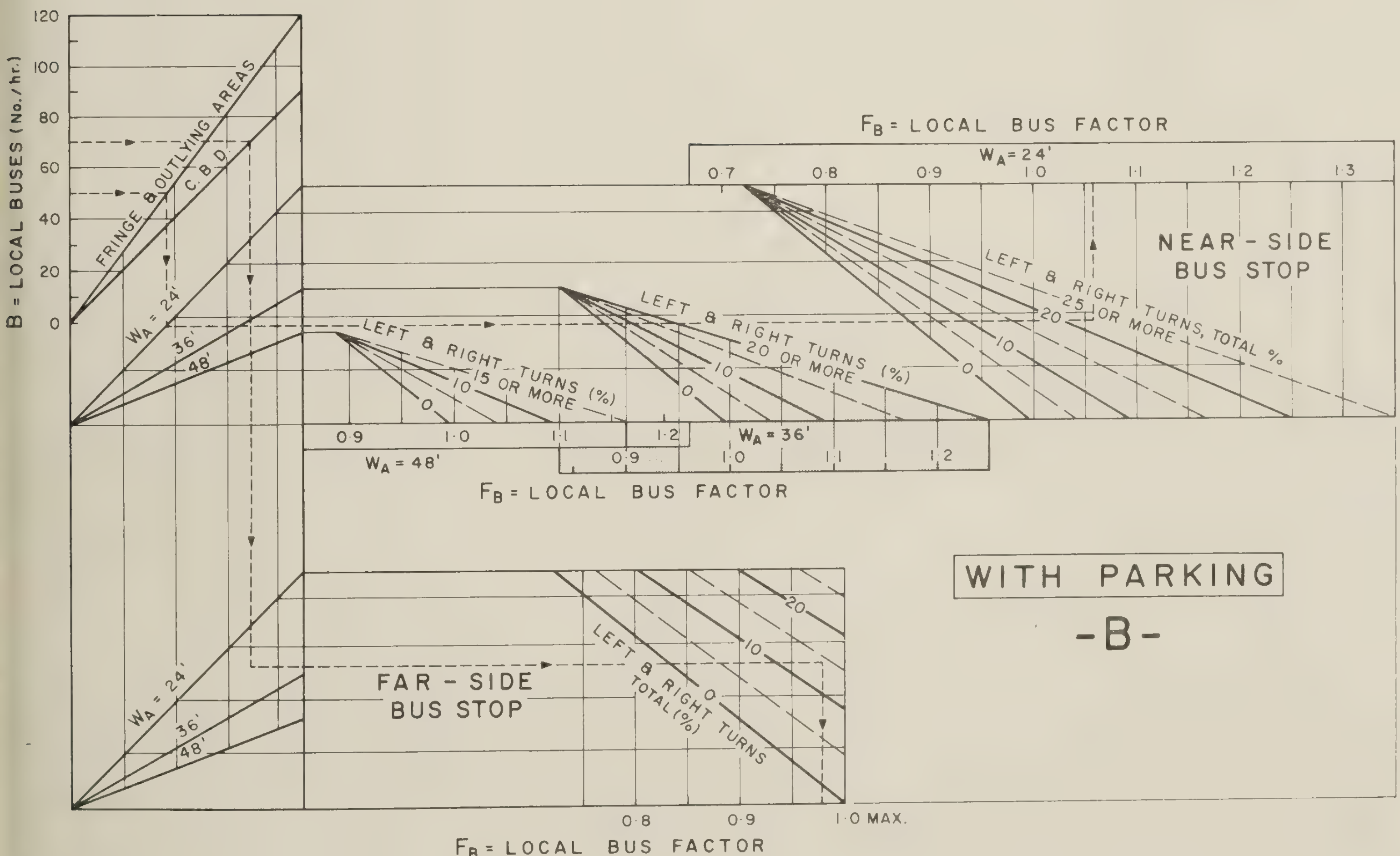
Where the capacity of a right- or left-turn lane is insufficient to handle the demand volume, the use of a double turning lane should be considered. This may be on two exclusive lanes, or on one exclusive lane and a second optional lane; the optional lane simultaneously would be accommodating a through or a right-turning movement. *Only the general case of the double turning lane is covered in the Manual. A rational method for evaluation of capacities of 2-lane turning movements with different angles of turn and entry widths on intersecting road is developed here for design purposes.*

16

BUS STOP

NOTE:
Local buses are to be included in determining overall percentage of trucks and buses (T) on Charts 3 - 14.





DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

LOCAL BUS FACTOR

FOR USE IN CONJUNCTION WITH CHARTS 3 – 14

CHART 16

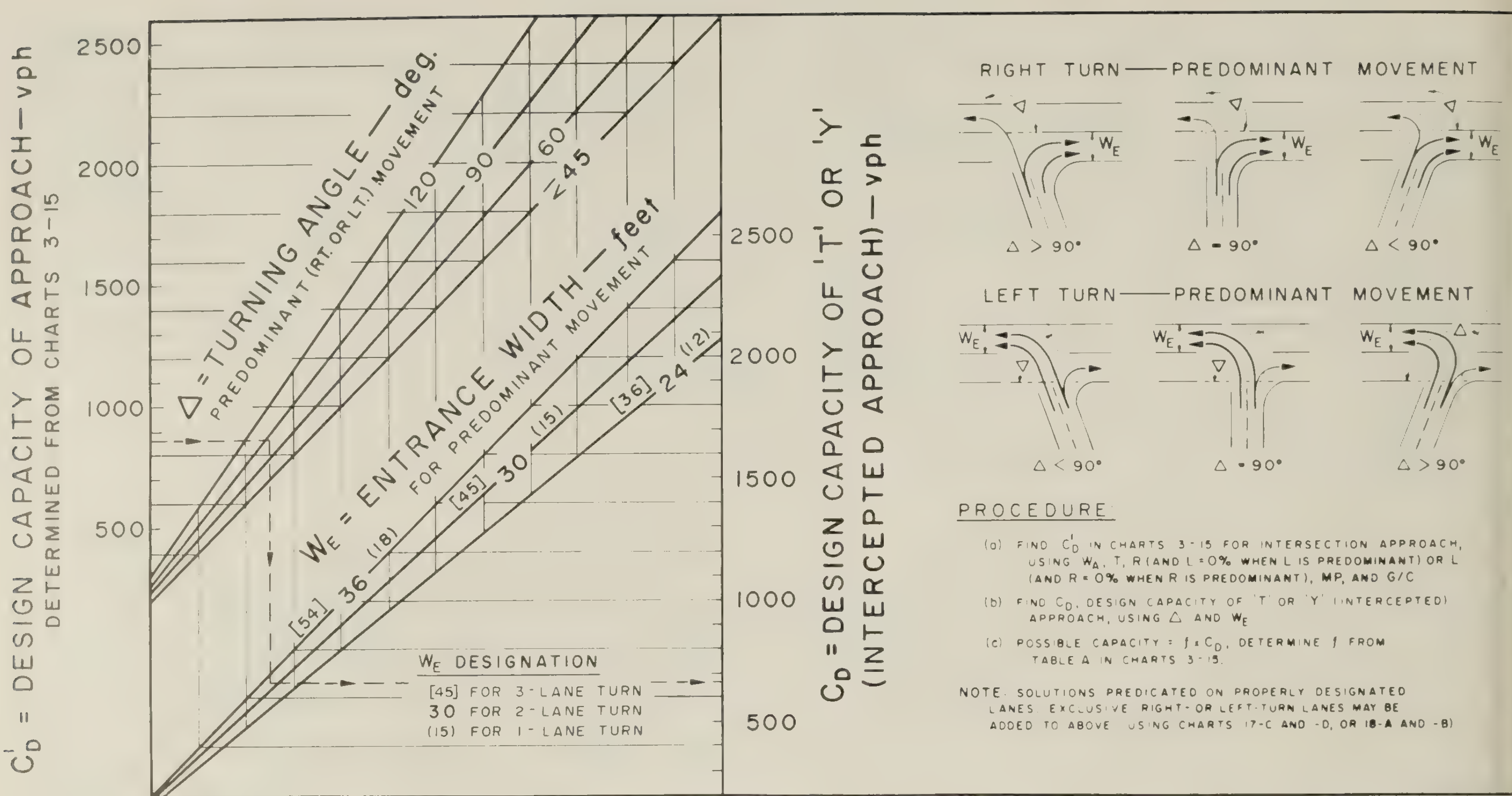


Figure 13.—Capacity of T or Y intersections.

Solution: The design capacity of single left-turn lane in Chart 18-B, using $G/C=0.60$, $a=11$ and $T_3=17\%$, is $C_{D3}=530$ v.p.h. Entering with this value in the chart of figure 11, and applying $\Delta=75°$ and $W_E=33$, the design capacity of the double left-turn lane is found to be $C_{DD}=950$ v.p.h. Possible capacity $C_{PD}=950\times1.30=1{,}240$ v.p.h.

2-abreast turning movement on 1 exclusive lane and on second optional lane

To achieve this, the intended operation should be designated by appropriate pavement marking and signing. Where the optional lane is shared by turning and through traffic, the following procedure should be followed:

(1) Determine the design capacity of the exclusive turning lane (C_{D2} or C_{D3}) in chart 18.

(2) Deduct the design capacity of the turning lane from the total turning volume (V_3-C_{D3}) or (V_2-C_{D2}).

(3) Determine the design capacity of the remainder of the approach roadway (W'_A) of which the optional lane is a part.

(4) The through volume plus (V_3-C_{D3}) or (V_2-C_{D2}) should not exceed the design capacity for approach width W'_A.

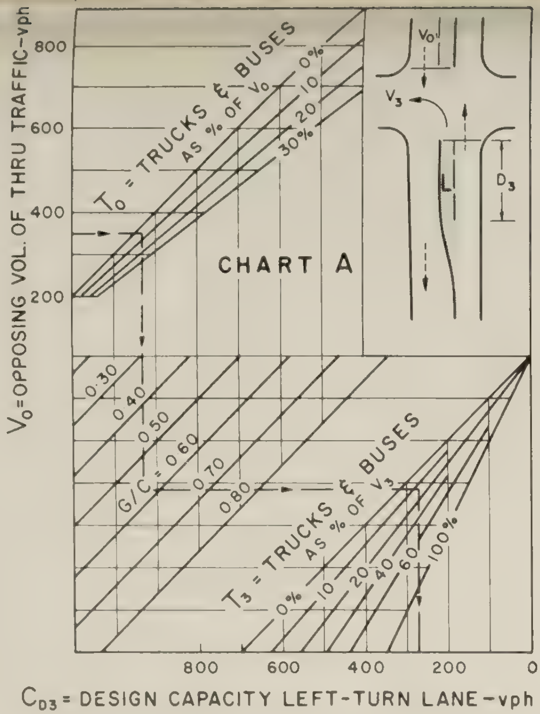
Where the optional lane is shared by a left- and a right-turning movement at a channelized intersection, as is often the case on ramp terminals of diamond and parclo interchanges, the procedure for analysis is outlined in figure 12. The formula shown in the figure allows for determination of the required G/C to accommodate the combined V_2 and V_5 volumes at design capacity by the use of chart 18-B. The formula is predicated on the condition where each of the 2 outer lanes discharges the usual volume of a turning lane at design capacity, while the middle lane discharges its share of the combined remaining left- and right-turning traffic at 0.8 design capacity. This assumes adequate radius and ample width for the double right-turning movement and a turning angle of 90 degrees or less with fully adequate exit width for the double left-turning movement. The design capacity of the combined movement for any other available G/C may be found by multiplying V_2+V_3 by the ratio of available G/C to required G/C.

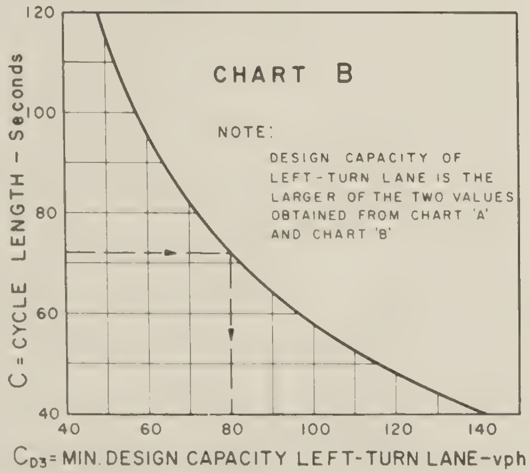
T and Y Intersections

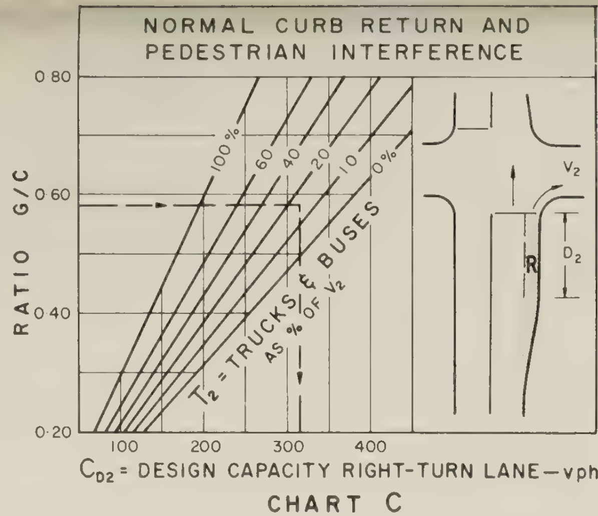
The capacity of the approach of an intercepted street at T and Y intersections, as the east approach in figure 14, may be obtained in accordance with the procedure outlined in figure 13, using charts 3–15 in combination with the nomograph in figure 13. Because the capacity studies reported in the *Manual* do not include direct measurements at T and Y intersections, a rational method was developed using the results from intersection approaches for the usual 4-leg intersections. As for the double turning lanes, the capacity of the intercepted approach at T and Y intersections is affected by the angle of turn and the entrance width for the turn on the street joint. Also, whether the right-turn volume or the left-turn volume is the predominant movement should be considered.

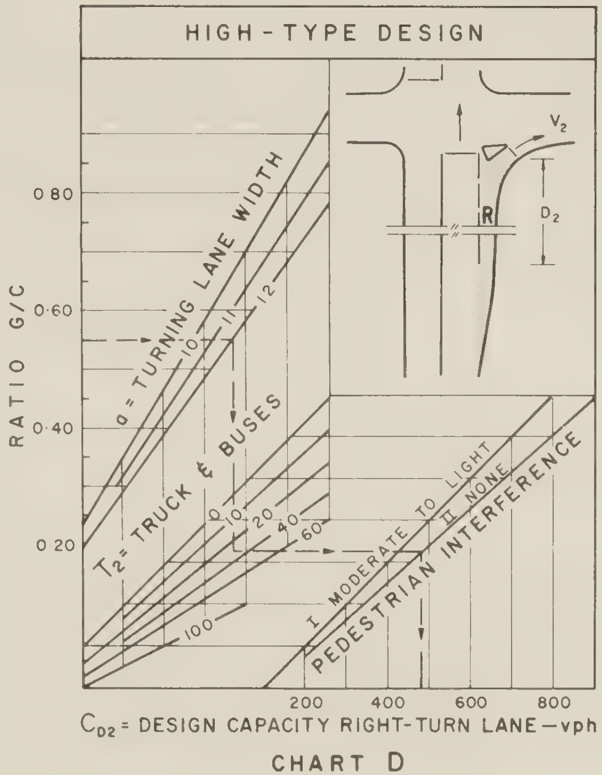
Because all traffic on the intercepted approach is turning, the total discharge from the approach—unless the angle of turn for the predominant movement is very favorable—would be less than for the approaches at regular intersections indicated in charts 3-15. Obviously, a double left-turn lane at a regular intersection and a 2-lane predominant left-turn movement on a T intersection, for the same width and angle controls, would have similar capacities. Results for the two conditions should be correlated, which is accomplished by equating, approximately, the capacities of a double left-turn lane with the capacities of a 2-lane intercepted approach on T intersections with 100% of the traffic turning left.

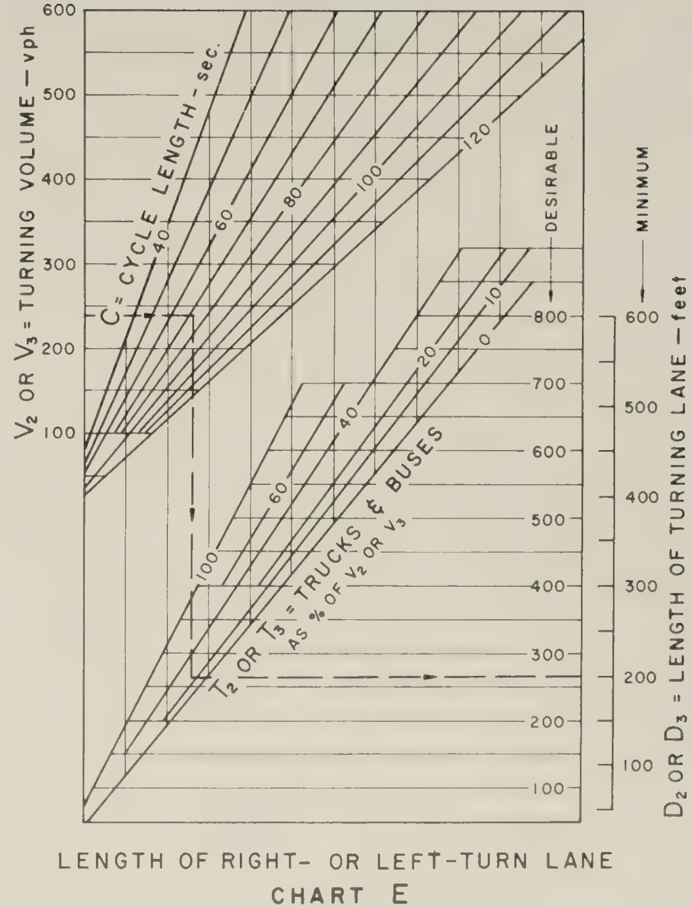
Averages of design and possible capacities of double turning lanes for angles of 120, 90, 60, and 45 degrees were compared, as a base, with averages of design and possible capacities of 2-lane approaches from charts 4 and 9 for an intermediate range of metro area sizes represented by 500,000 population. Ratios of the two sets of values for the different angles of turn, serving as adjustment factors, were found to be as follows:











NOTE:

- CHARTS A AND B APPLY TO TWO-WAY FACILITIES ONLY, IN URBAN AS WELL AS IN RURAL AREAS.
- CHARTS C AND D APPLY TO RIGHT-TURN LANES ON TWO-WAY STREETS AND TO BOTH RIGHT-AND LEFT-TURN LANES ON ONE-WAY STREETS
- CURVE II OF CHART D GENERALLY IS NOT APPLICABLE TO CBD.
- IN RURAL AREAS (FOR SERVICE LEVEL B) MULTIPLY VALUES IN CHART D BY 0.90
- TO DETERMINE POSSIBLE CAPACITY OF RIGHT- OR LEFT-TURN LANE MULTIPLY CHART DESIGN CAPACITY BY 1.30.

DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS

SEPARATE RIGHT- AND LEFT-TURN LANES

NO SEPARATE SIGNAL INDICATION FOR TURNING MOVEMENT

CHART 17

17 TURN'G. LANES

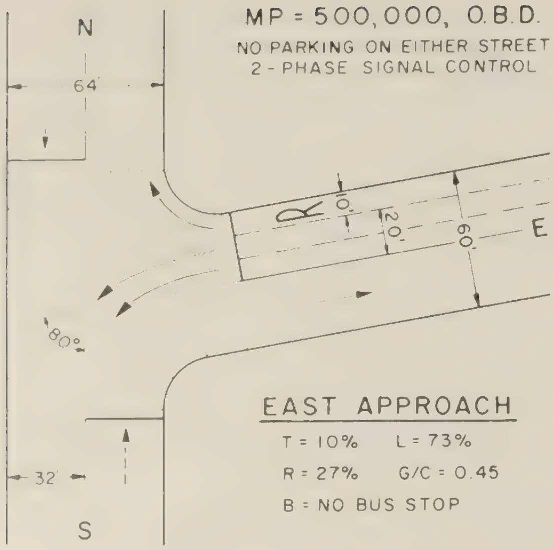


Figure 14.—Problem 34 illustrated.

Δ, degrees, for predominant movement	45	60	90	120
Adjustment factor, rounded	1.00	0.90	0.80	0.70

The adjustment factor of 1.00 indicates a capacity on the intercepted approach (T or Y intersection) equivalent to the full capacity of approach on a regular 4-leg intersection. Thus, for a turning angle of 45 degrees or less for the predominant movement, the directness of the turn is assumed to yield capacities of the order given in charts 3–15. The preceding apply when the intersection approach is not deterred by the intersection exit; that is, the width of traveled way, W_E, receiving a 2-lane predominant movement, is equal to 3 lanes. The same adjustment factors of 1.00, 0.90 and 0.80, used for double turning lanes, are applied here for W_E of 36, 30, and 24 feet, respectively. The factors developed for 2-lane approaches are considered to be applicable to other widths of approach and other conditions including parking. The relations for T and Y intercepted approaches are presented in the nomograph in figure 13.

Problem 34

What are the design and possible capacities of the intercepted street (east approach), from which traffic can turn only left and right into the north-south street, for the conditions indicated in figure 14?

Solution: In chart 4, using $W_A=20$, $T=10\%$, $L=0\%$, $R=0\%$ (because right turn is handled on separate right-turn lane), $MP=500{,}000$ population, and $G/C=0.45$, find $C_D=860$ v.p.h. Entering with this value in the chart of figure 13, and applying $\Delta=80°$ and $W_E=32$, the design capacity of the two lanes handling traffic to the left is $C_D=650$ v.p.h. The corresponding right-turn volume$=650\times 27/73=240$ v.p.h. The design capacity of the right-turn lane from chart 17-C is 260 v.p.h. The volume on the approach that can be accommodated at design capacity$=650+240=890$ v.p.h. The possible capacity of the left-turn movement is $650\times 1.20=780$ v.p.h. The corresponding right-turn volume$=780\times 27/73=290$ v.p.h. Possible capacity of right-turn movement is $260\times 1.30=340$ v.p.h. The volume on the approach that can be handled without exceeding possible capacity$=780+290=1{,}070$ v.p.h.

Multiple-Type Intersections

The capacity of any form of signalized intersection, regardless of the number of approach roads and extent of channelization, can be obtained from the charts by examining each approach road separately. The design of complex intersections, particularly those requiring multi-phase control, may necessitate some study and trial solutions before determining the final plan. Multiple intersections often present several possibilities in the pattern of operation and in the number and arrangement of signal phases. Such alternate arrangements may result in different geometric layouts affecting the shape and location of islands, widths of pavements, size of storage areas, and overall space requirements for the intersection. The geometric layout should be determined jointly with capacity analysis. Care should be taken to check the length and width of those traffic channels where vehicles will store during certain signal phases to preclude the condition of traffic backing up from one intersection point to another. Use of advance or lagging green indications and appropriate offsets to produce progressive movements may be necessary in this regard. Time-space diagrams often are valuable adjuncts to the design procedure.

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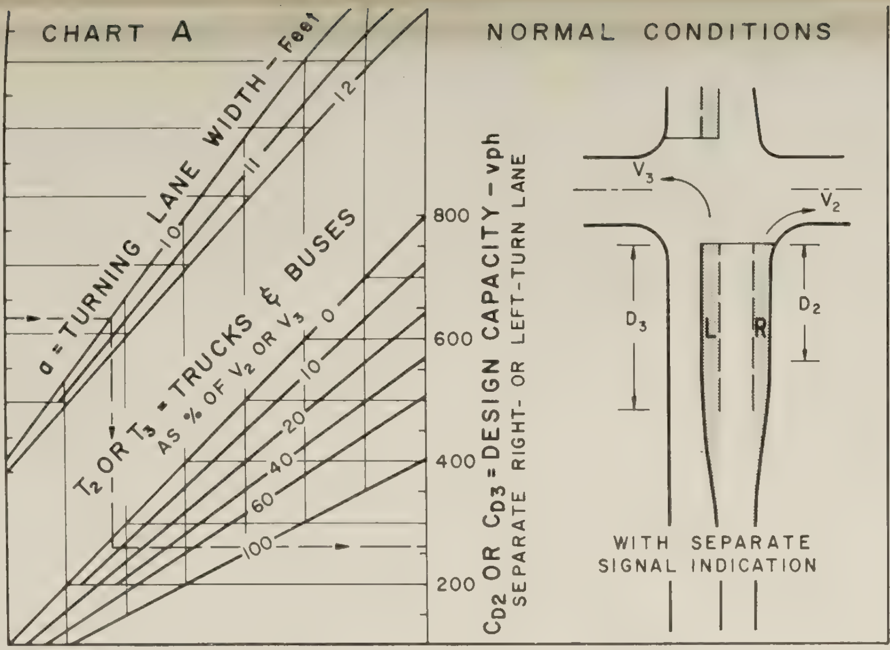
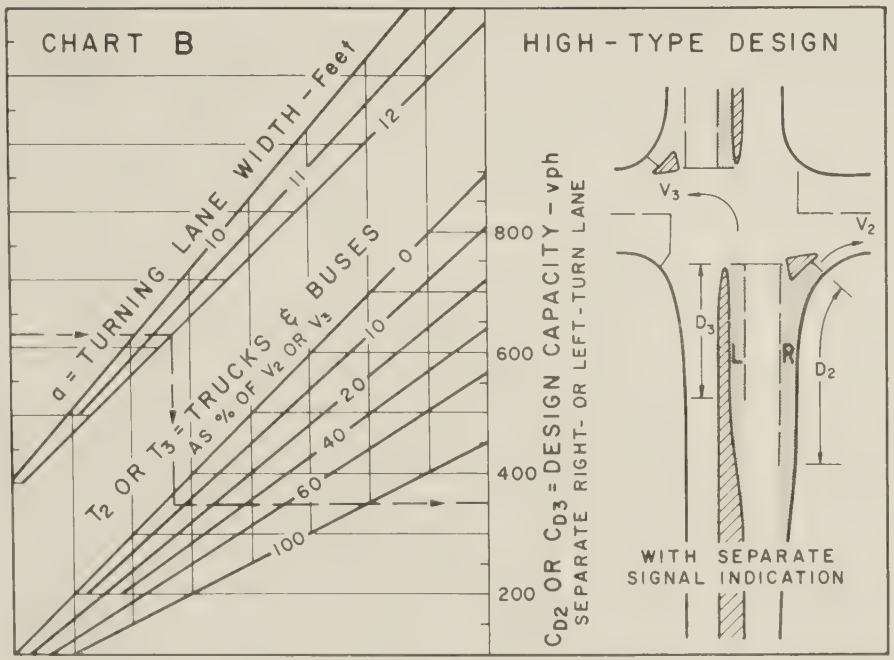
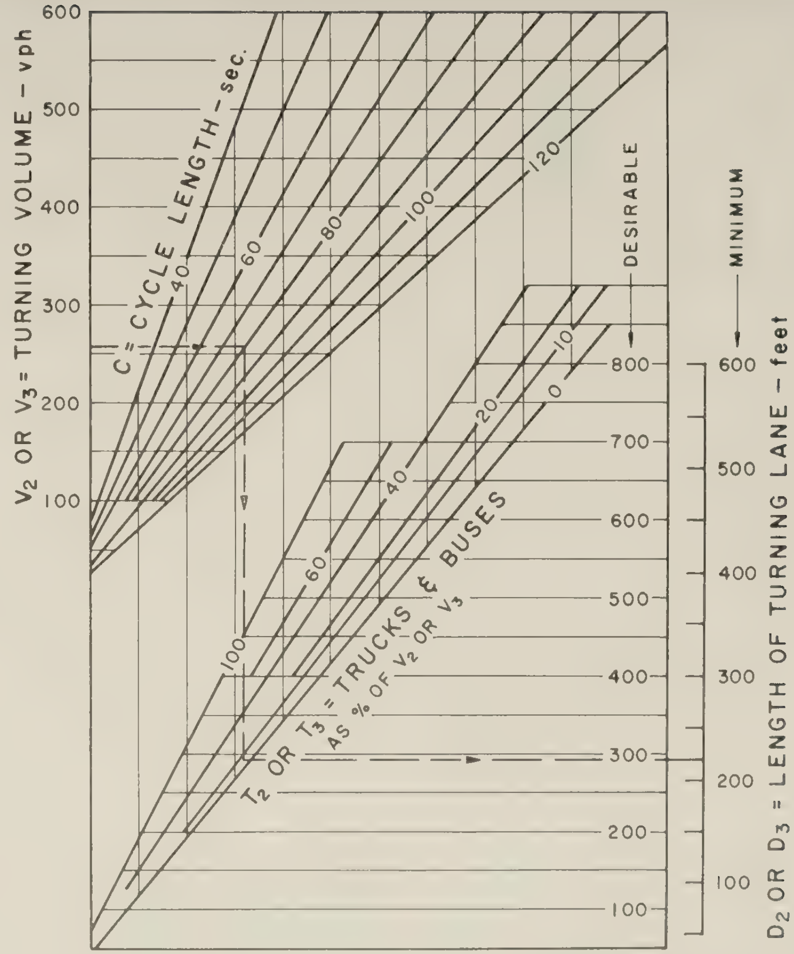






LENGTH OF RIGHT- OR LEFT-TURN LANE
CHART C

NOTE:

- CHARTS A AND B APPLY TO BOTH TWO-WAY AND ONE-WAY STREETS.
- CHART B GENERALLY DOES NOT APPLY TO CBD.
- IN RURAL AREAS (FOR SERVICE LEVEL B) MULTIPLY VALUES IN CHARTS A AND B BY 0·90.
- TO DETERMINE POSSIBLE CAPACITY OF RIGHT- OR LEFT-TURN LANE, MULTIPLY CHART DESIGN CAPACITY BY 1·3.

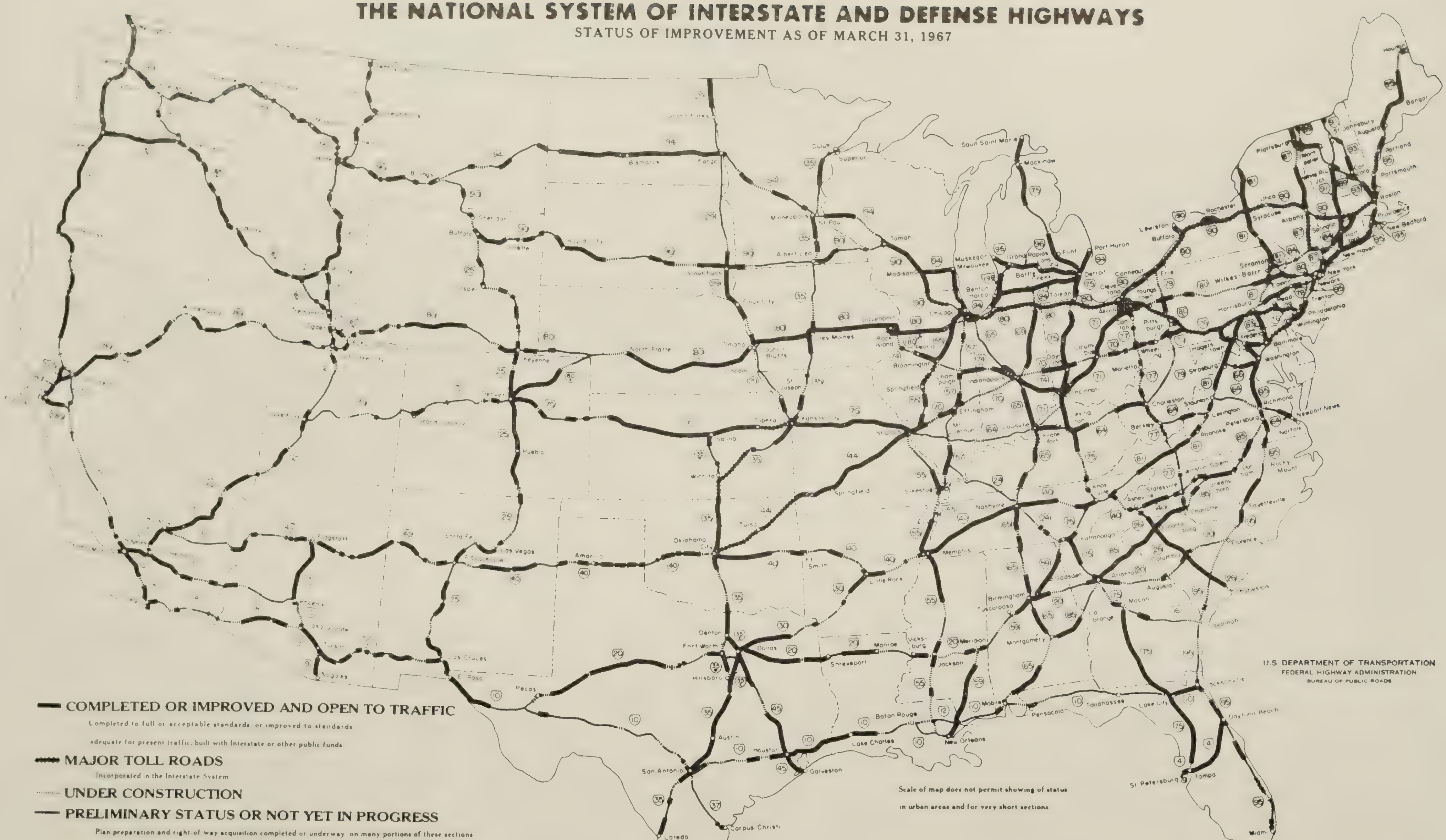
DESIGN CAPACITY OF SIGNALIZED INTERSECTIONS
SEPARATE RIGHT- AND LEFT-TURN LANES
WITH SEPARATE SIGNAL INDICATION FOR TURNING MOVEMENTS

CHART 18

18 TURN'G LANES

THE NATIONAL SYSTEM OF INTERSTATE AND DEFENSE HIGHWAYS

STATUS OF IMPROVEMENT AS OF MARCH 31, 1967

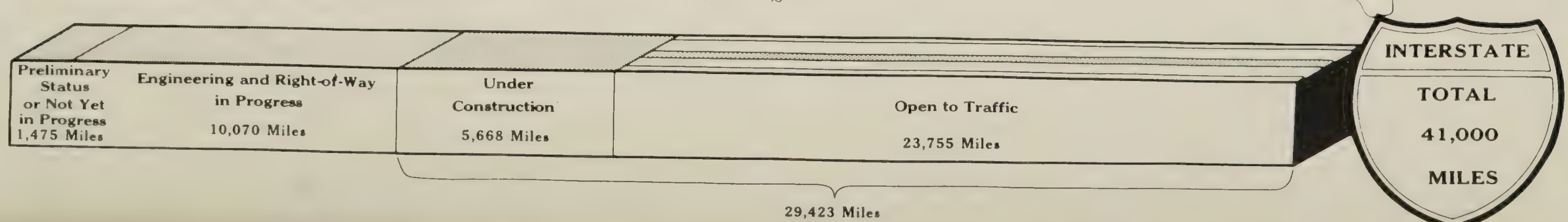


— COMPLETED OR IMPROVED AND OPEN TO TRAFFIC
Completed to full or acceptable standards, or improved to standards adequate for present traffic, built with Interstate or other public funds

MAJOR TOLL ROADS
Incorporated in the Interstate System

UNDER CONSTRUCTION

PRELIMINARY STATUS OR NOT YET IN PROGRESS
Plan preparation and right-of-way acquisition completed or underway on many portions of these sections



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