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Capacity Analysis Techniques for Design and Operation of Freeway Facilities

By Jack E. Leisch

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by

Jack E. Leisch

FEBRUARY 1974

U.S. DEPARTMENT OF TRANSPORTATION
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PREFACE

In 1967 the author presented a procedure for the graphic solution of the capacity of signalized intersections entitled "Capacity Analysis Techniques for Design of Signalized Intersections." The success of this publication prompted the author to develop a similar type of publication for analyzing capacity problems associated with freeway operations.

In this publication the author has presented a procedure for the graphic solution of the capacity of freeway facilities to simplify the work required by the computational procedures in the *Highway Capacity Manual*. Figures and nomographs have been developed to solve the capacity problems on through lanes, exit and entrance ramps, and ramp terminals. A full discussion of the principles and procedures in the application of the figures and nomographs in addition to sample problems has been included in this publication.

The application of these techniques greatly facilitates the design of freeways and is particularly applicable to improving operational efficiency and safety in the rehabilitation of congested and outmoded freeways.

Jack E. Leisch

Evanston, Illinois
February 1974

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List of Abbreviations and Equivalents *

Abbreviation	*Equivalent*
AASHO	American Association of State Highway Officials (in late 1973 changed to American Association of State Highway and Transportation Officials)
AHS	average highway speed
CBD	central business district
C-D	collector-distributor roads adjacent to freeways
DHV	design hourly volume
EN	entrance terminals
EX	exit terminals
G/C	green time for any one signal phase (in seconds) divided by cycle time (in seconds)
lb./hp	pounds per horsepower
MP	metropolitan population
mph	miles per hour
parclo	partial cloverleaf
pcph	passenger cars per hour
PHF	peak hour factor
sta.	station (surveyor's)
SV	service volume
VC	vertical curve
vph	vehicles per hour
VPI	vertical point of intersection

* Omits commonly known units.

Chapter I

INTRODUCTION

The planning and design of freeways cannot be accomplished without consideration of the pattern and volumes of traffic which will utilize them and the ability of such facilities to accommodate this traffic. Accordingly, the geometric features—alinement and profile, interchange configuration and spacing and, most importantly, the number and arrangement of lanes—must be determined largely through capacity analyses.

A procedure based on the data and findings reported in the *Highway Capacity Manual*[1] is presented here as a tool in the planning and design of freeway facilities. A graphical method utilizing special nomographs and procedural steps has been developed to facilitate the understanding of highway capacity and its application to design of freeways. The subject is dealt with by a separate analysis of each component of the freeway, including numerous illustrative samples, and then combining the results to reach a composite solution for design of the complete freeway facility or for its operating system. The analysis techniques and sample solutions of problems are presented both on the basis of the *Manual* approach and in accordance with the design policies of the American Association of State Highway Officials (AASHO).[2] The slight differences in the philosophy and approach are explained and the two methods related.

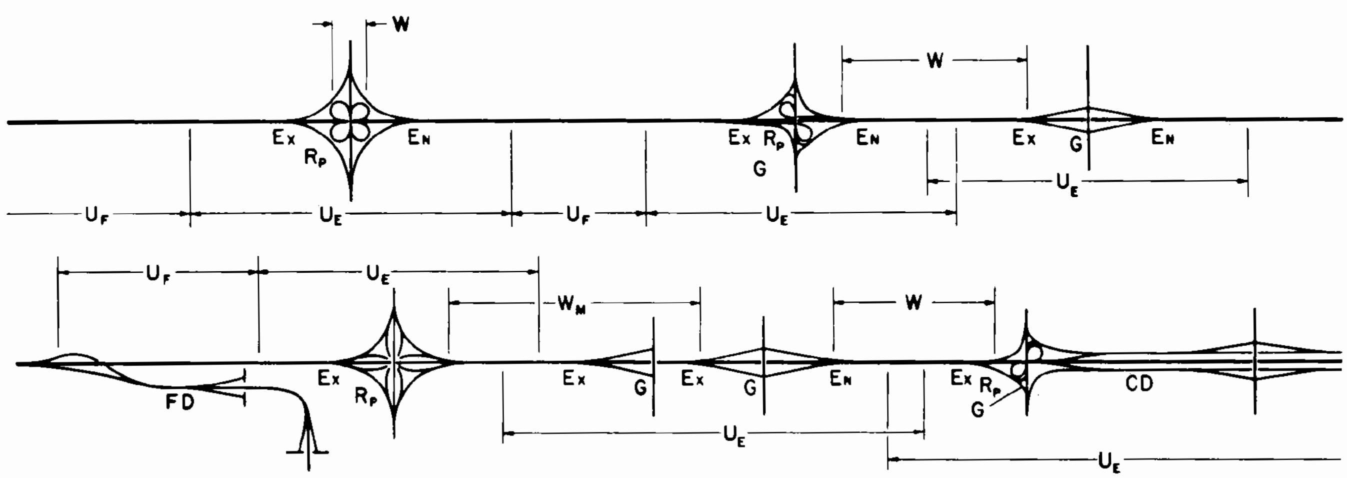
FREEWAY COMPONENTS

As shown in figure 1, capacity analyses in design of freeways are appropriately accomplished by dividing the facility into its basic components, as follows:

a. *Freeway Proper* is that portion of freeway U_F largely unimpeded by merging, diverging, or weaving traffic, char-

[1] *Highway Capacity Manual,* Highway Research Board Special Report 87, 1965; hereafter referred to as the *Manual.*

[2] *A Policy on Geometric Design of Rural Highways,* AASHO, 1965; hereafter referred to as the *AASHO Design Policy.*



U_F — UNINTERRUPTED FLOW SECTION, FREE OF INTERCHANGE INFLUENCE

U_E — UNINTERRUPTED FLOW SECTION, WITHIN INFLUENCE OF ENTERING AND EXITING TRAFFIC

E_X — EXIT TERMINAL (DIVERGING FLOW)

E_N — ENTRANCE TERMINAL (MERGING FLOW)

W — WEAVING SECTION

W_M — MULTIPLE WEAVING SECTION

R_P — RAMP PROPER

G — AT-GRADE RAMP TERMINAL

FD — FREEWAY DISTRIBUTOR

CD — COLLECTOR-DISTRIBUTOR ROAD

FIGURE 1.—Freeway components requiring separate capacity analyses.

acterized by what may be termed *uninterrupted flow.* Other portions of the freeway along which exits and entrances occur are also considered to be generally under uninterrupted flow conditions, but within influence of merging, diverging, and weaving traffic U_E; such sections must be further analyzed with respect to these influences, as represented by the components below.

b. *Ramps* are those elements of interchanges consisting of ingress and egress facilities. Operational characteristics of entrance terminals (EN) and exit terminals (EX) along the freeway not only affect operation on the freeway proper but, in themselves, have specific capabilities of handling merging and diverging traffic. The other ends of the ramps—those joining the intersecting highway—also have influence on the total operation. Some of the terminal designs are operationally comparable to the ramp terminals joining the freeway. Yet, others may be entirely different as, for example, the at-grade terminals of diamond and partial cloverleaf (parclo) interchanges G which have much the same operational characteristics and limitations as normal at-grade intersections. The portion of the ramp proper between its two terminals R_P must also be considered in the analysis to achieve a balanced design.

c. *Weaving Sections* are those portions of freeway on which operation is affected by overlapping movements of the successive entering and exiting traffic. Such arrangements may constitute simple weaving sections W where weaving maneuvers take place between an entering ramp and a following exiting ramp; or, may entail multiple weaving sections W_M where overlapping weaving maneuvers take place in conjunction with several closely spaced ramps. Weaving sections are critical elements on most freeways, particularly in urban areas, and must be analyzed to determine their geometric features.

Initially, each component of the freeway facility is analyzed separately. Then the requirements for each are brought together and harmonized to produce an integrated facility—geometrically and operationally balanced.

CAPACITY AND LEVELS OF SERVICE

In general terms, the "capacity" of a highway refers to its ability to carry traffic. In a strict sense "capacity" indicates the maximum volume of traffic which can be accommodated by a facility. According to the *Manual,* "capacity" is defined as the maximum number of vehicles which have a reasonable expectation of passing over a given section of lane or roadway in one direction during a given time period under prevailing conditions. Unless specified otherwise, capacity is an hourly volume.

The prevailing conditions which affect capacity may be divided into two general groups: (a) prevailing roadway conditions, indicative of the physical features such as roadway alinement, number, and width of lanes; and (b) prevailing traffic conditions, indicative of the nature of traffic, such as peaking characteristics and composition of traffic. Thus, capacity may vary between different freeways depending on the particular conditions. However, a capacity base—a single value—for what may be termed "capacity under ideal conditions," can be established for all freeways. The capacity of a freeway under ideal conditions is considered to be 2,000 passenger vehicles per lane per hour. Ideal conditions for freeways are defined as the state of uninterrupted flow with passenger cars only in the traffic stream, horizontal and vertical alinement adequate for operation at 70 miles per hour (mph) or greater, traffic lanes 12 feet wide, and adequate shoulders and no lateral obstructions within 6 feet of the edge of traveled way. The capacity of 2,000 passenger vehicles per lane per hour is the average lane volume of traffic in all the lanes in one direction of travel for a full hour of operation. Capacity is also indicative of a reasonably uniform flow in which the rate of flow for any 5-minute period during the hour is approximately the same as the total hourly flow. The prevailing conditions which differ from the "ideal" produce a reduction in the capacity base of 2,000 vehicles per hour (vph). The reducing effects of each condition below the ideal have been established in the *Manual* so that the capacity of any freeway under prevailing conditions can be evaluated. Capacity is characterized by high traffic density and relatively low and uniform speed. Maximum flow occurs at what may be termed "optimum speed" and "critical density." At capacity, speeds are in the range of 30 to 35 mph and densities are in the order of 50 to 70 vehicles per mile. A typical traffic condition at capacity on a freeway may be described as an average speed of 30 mph, a density of 65 vehicles per mile, and a mean headway of 1.8 seconds or vehicle spacing of about 80 feet.

As the volume of traffic approaches capacity, the flow becomes unstable. Any mishap which causes traffic to slow down below the optimum speed can produce stoppages of traffic and a breakdown of the facility. Because of this sensitivity and the restrictive operational characteristics, the capacity condition is considered to be inappropriate for planning and design purposes. To insure that a highway provides an acceptable quality of operation to the road user, it is necessary for the volume of traffic to be lower than the capacity of the roadway. To meet this requirement, a level of service concept is introduced. *Level of service,* as stated in the *Manual,* is a qualitative measure of the effect of a number of factors, including speed and travel time, traffic interruptions, freedom to maneuver, safety, driving comfort and convenience, and operating costs. Six levels of service have been selected, A to F, designating operating conditions from excellent to intolerable, including capacity. The various levels of service, although indicative of a combination of several factors, are quantified with respect to two principal parameters—operating speed and service volume. Thus, each level of service is associated with a limiting volume of traffic. Stating it another way, the maximum number of vehicles which can be carried at any selected level of service is referred to as the *service volume* (SV) for that level.

Each level of service actually falls within a volume band ranging between the limiting service volume of the next higher level of service and its own (maximum) service volume. For example, if on a given section of freeway the service volume for level of service C is 4,000 vph and for level of service B it is 3,000 vph, then any volume that the facility would carry just above 3,000 vph and not over 4,000 vph would be considered to be within level of service C.

The level of service concept permits each element of a highway or a system of highways to be designed to produce a balanced facility, with reasonably uniform and consistent quality of operation throughout. This is the main objective in applying the level of service concept. The two levels of service generally associated with design of freeways are level *B* for rural conditions and level *C* (sometimes *D*) for urban environment. The preceding concepts and nomenclature are essentially as presented in the *Manual.* Utilizing the same basic data and general concepts, AASHO has evolved, through its geometric design policies, a somewhat different nomenclature and approach to the application of capacity. The differences are not significant. Understanding both bases, as intended here, will allow the traffic and highway engineer to employ either procedure or a combination of the two. The American Association of State Highway Officials has retained the more comprehensive meaning of the word "capacity" which, incidentally, is the way the term was used in the previous (1950) *Highway Capacity Manual.* In its broader use, "capacity" may relate to other than an absolute value. It may indicate a specified limiting number of vehicles which can be accommodated commensurate with a selected operational quality, or it may be indicative of some measure of comfort and convenience. To differentiate this type of operation and its limiting volume of traffic from the type of operation when an absolute or highest possible volume of traffic is achieved, AASHO refers to the former condition as *design capacity* and the latter condition as *possible capacity.*

Design capacity may be defined as the highest permissible number of vehicles which can be carried under given roadway and traffic conditions without unreasonable delay or restriction to the driver's freedom to maneuver, and is the representative or practical value established in designing the highway to accommodate the design volume. *Possible capacity* represents the maximum number of vehicles which can be accommodated under given or prevailing roadway and traffic conditions, regardless of the effect of delaying drivers and restricting their freedom to maneuver; it is the largest volume of traffic a facility could be expected to carry. Obviously, the AASHO terminology *design capacity* is in essence the same as the *Manual* terminology *maximum service volume* for a selected *level of service.* Also, the AASHO terminology *possible capacity* is identical conceptually and quantitatively to the *Manual* terminology *capacity.*

Both the AASHO Design Policy and the *Manual* select several measures of operational quality, expressed numerically as "design capacities" and "service volumes," respectively. The AASHO, in effect, deals with several levels of service expressed by three design capacity conditions selected to be representative of rural, suburban, and urban conditions. These AASHO design capacity values are numerically blanketed by the service volumes for levels of service *B*, *C*, and *D* as indicated in the *Manual.* The two scales in table 1 compare *levels of service* (and service volumes) in the *Manual* with *design capacities* in the AASHO Design Policy.

The *Manual* relates each level of service to an operating speed. *Operating speed* is the maximum safe speed at which an individual driver can travel under prevailing traffic conditions without exceeding the design speed of the highway at any time. The operating speed of 60 mph or more at level *A* reduces progressively with each lower level of service to about 30 mph at level *E* (capacity). The *Manual* further relates each level of service to a service volume. Although both the operating speed and the service volume are indicators of the levels of service on a highway and are frequently compatible, there are times when the two measures are not in harmony with each other. In these situations either criterion may control the level of service. That is, in accordance with the relationships in table 1, if the operating speed falls below the stipulated value, the next lower level of service pertains, or if the service volume exceeds the indicated amount, the next lower level of service governs. Thus, if either criterion is not satisfied the service is dropped to the next level. On existing facilities, checks for both speed and volume are essential to determine the operating level of service, with the speed frequently being the governing element. Table 1, and more specifically table 2, may be used for this purpose on existing freeways.

On new freeways—being planned or designed—speed cannot be measured, but from known relationships of operating speeds, average running speeds, and highway quality (expressed first in terms of "design speed" and later in more detail as "average highway speed," AHS), speed-service relations can be predicted and hence an approximate level of service. This has been accomplished and reported both in the *Manual* and in the AASHO design policies. Table 2 indicates these relationships with the suggestion that the service volume to capacity ratio v/c be used as the basis for design in combination with AHS.

The *Manual* also relates each level of service to a service volume average per lane, in accordance with the number of lanes on the freeway. For stable flow conditions—levels of service *A*, *B*, and *C*—the service volume increases with successively wider facilities. In other words, given the same average volume of traffic per lane, the freedom to maneuver is greater on 3 lanes than on 2 lanes, and greater on 4 lanes than on 3 lanes. For the same degree of freedom or level of service, wider freeways are capable of handling larger volumes per lane. As shown in table 1, level *A* corresponds to lane service volumes of 700, 800, and 850 vph for 2, 3, and 4 freeway lanes in one direction. Level *B* is represented by volumes of 1,000, 1,150, and 1,250 vph respectively. At level of service *C*, although still in the range of stable flow, higher service volumes of 1,250 to 1,500 vph can be accommodated while at level of service *D*, which approaches unstable flow, volumes up to about 1,650 vph can be carried.

At levels *C* and *D* operation is so critical that (unlike levels *A* and *B*) rates of flow within a period shorter than an hour must be considered. For these conditions the *Manual* suggests that the highest rate of flow for a 5-minute period within the hour, rather than the maximum number of vehicles expected during the full hour, be used to evaluate levels of service. To accomplish this, it is necessary to adjust a uniform hourly flow by an appropriate *peak-hour factor* (PHF). For freeway facilities the PHF has been expressed as the ratio of the whole-hour volume to the highest rate of flow occurring during a 5-minute interval

TABLE 1.—*Capacity and service volume relations on freeways under ideal conditions*

HIGHWAY CAPACITY MANUAL (1)					AASHO DESIGN POLICY (2)		
LEVEL OF SERVICE	OPERATING SPEED MPH	SERVICE VOLUME (3) VPH PER LANE AVR. FOR NO. OF LANES IN ONE DIRECTION OF: 2	3	4	DESIGN CAPACITY VPH PER LANE AVR. FOR NO. OF LANES IN ONE DIRECTION OF: 2, 3 OR 4	AVERAGE RUNNING SPEED MPH	LOCATION
A	≥60						
		700					
			800				
B	≥55			850			
		1000			1000	45 - 50	RURAL
C	≥50		1150		1200	40 - 45	SUBURBAN
		1250		1250			
			1400				
D	≥40			1500	1500	35 - 40	URBAN
				1650			
E (CAPACITY)	30 - 35			2000	2000	30	ALL LOCATIONS (Possible Capacity)

(1) Highway Capacity Manual, 1965, HRB Special Report 87.

(2) A Policy on Geometric Design of Rural Highways, 1965.

(3) At level of service C, peak-hour factor (PHF) of 0.83, 0.87 and 0.91 for 2-, 3- and 4-lane freeway in one direction, assumed (for illustrative purposes) to be representative of small-to-medium, medium-to-large, and large city, respectively. At level of service D a PHF of 0.91 is assumed to be characteristic of this level of service, considering medium and large cities. At levels of services A and B, PHF not applicable; and at level of service E, PHF approaches 1.00. Service volumes, for the peaking conditions shown, are rounded to closest 50 vph.

within the peak hour, that is, the hourly flow divided by 12 times the actual 5-minute flow. For example, if the total hourly volume is 300 vph and the highest 5-minute flow is 30 vehicles, PHF $= 300 \div (30 \times 12) = 0.83$. (Under interrupted flow conditions, involving controlled movements at intersections and on at-grade ramp terminals, the peak-hour factor is based on a 15-minute rate of flow.)

Hourly traffic fluctuations represented by peak-hour factors are largely the product of land uses, trip purposes, and city size. The smaller cities tend to have traffic peaks of shorter duration than one hour, producing sizeable variations in 5-minute flows during the hour and yielding low peak-hour factors. Large cities tend to have extended traffic peaks, frequently of longer duration than one hour. Under these circumstances, the 5-minute flows are more nearly equal during the hour, with resulting high peak-hour factors. Representative peak-hour factors for freeway operations, according to the *Manual*, may be generalized for various metropolitan area sizes as follows:

1. Less than one-half million population—0.77
2. One-half to 1 million population—0.83
3. Over 1 million population—0.91

Actually, there may be a considerable scatter in magnitude of peak-hour factors within any one population group, requiring caution in the application of these relationships. However, where specific PHF's are not available, the values indicated are suggested.

Peaking characteristics may be further influenced by insufficient highway capacity for the traffic demand, which tends to extend the period of peak flow, producing high density or congestion and with it high peak-hour factors.

Thus, there is a tendency generally for the higher peak-hour factors to be associated with the lower levels of service. For example, a PHF of about 0.90 is frequently found at level *D* operation, and at capacity or level *E*, PHF approaches 1.00. Referring to table 1, representative service volumes for level of service *C*—1,250, 1,400, and 1,500 vph for 2, 3, and 4 lanes in one direction of travel—are predicated on the fact that the fewer numbers of lanes are indicative of the smaller-size cities and the larger numbers of lanes are apt to be associated with the larger-size cities. Accordingly, PHF's of 0.83, 0.87, and 0.91, respectively, were assumed for the three widths of freeways, yielding the volumes shown. For level of service *D*, a peak-hour factor of 0.91 was taken to be representative for each width of freeway, producing a single service volume of 1,650 vph. For level of service *E*, the limiting service volume, at capacity, is the 2,000 vph previously established.

The right half of table 1 shows a similar set-up for the AASHO Design Policy and permits direct comparison with the *Manual* values. Each design condition in terms of type of area or environment is designated by a design capacity, expressed in vph per lane. Design conditions are identified as rural, suburban, and urban, for which design capacity values are 1,000, 1,200, and 1,500 vph, respectively. Possible capacity, which pertains to all locations, is 2,000 vph as previously established. Design capacity for each location is further compared to an average running speed, a good indicator of operating conditions (freedom to maneuver). The *average running speed* is the distance over a specified section of highway divided by the average of running times of all vehicles traversing the section during the hour. Thus, it is an average speed of all traffic indicative of operation during the design hour. Generally, 45 to 50 percent of the vehicles exceed the average speed. The faster drivers referred to in the *Manual* as traveling at *operating speed* normally will exceed the average running speed by about 5 to 7 mph. The two speeds, however, tend to become more nearly equal during periods of unstable flow. At capacity (or possible capacity), there is little difference between speeds of individual vehicles, so operating speed and average running speed both approach an optimum value of approximately 30 mph.

The AASHO Design Policy, for any one design condition or location identification (see table 1), assumes an hourly volume per lane which is constant for all widths of freeways. Thus, design capacity value in vph per lane is indicative of a representative width of freeway for the condition—rural, suburban, or urban. Furthermore, each design capacity value may be considered to have a built-in peak-hour factor; that is, the volume indicated by design capacity for each location or environment takes into account the representative peaking characteristics of that environment. For example, for the *rural* condition shown in table 1, a design capacity of 1,000 vph per lane at average running speeds of 45 to 50 mph is representative of the usual 4-lane freeway (two lanes in each direction), and an occasional 6-lane freeway, operating under normal rural peaking characteristics. This is equivalent to level of service *B* designation in the *Manual*, with service volumes of 1,000 and 1,150 vph per lane at operating speeds of 55 mph or more for 4-lane and 6-lane freeways, respectively.

For the *urban* condition shown in table 1, a design capacity of 1,500 vph per lane at average running speeds of 35 to 40 mph is representative of the usual 6- and 8-lane freeways (3 and 4 lanes in one direction), operating under what may be termed average urban peaking conditions—PHF's in the range of 0.85 to 0.90. This is equivalent to the lower limit of level *C* or the higher limit of level *D* designated in the *Manual*. Limiting service volumes for level *C*, as shown, are normally 1,400 and 1,500 vph for 6- and 8-lane facilities at operating speeds of about 50 mph, while for level *D* the limiting service volume is generally 1,650 vph at operating speeds of 40 mph. For the *suburban* environment the AASHO Design Policy designates a design capacity of 1,200 vph at average running speeds of 40 to 45 mph. This is an intermediate value between rural and urban conditions which, for the most part, is equivalent to the *Manual's* level of service *C*. It is evident that the AASHO Design Policy sets out average or representative controlling values for purposes of design. The simplicity of the approach is justified on the basis that the design capacities chosen are applied to projected or assumed conditions 20 years or more in the future. Moreover, the determination of the number of lanes and several other geometric features is not established solely on the basis of capacity analyses. Other controls and criteria, which sometimes are overriding factors, have a bearing on geometric design. The *Highway Capacity Manual* presents a more detailed and thorough evaluation of operational features and their relation to prevailing roadway and traffic conditions. It provides a more precise determination of lane requirements, provided all the conditions are actually known. For this reason, the *Manual* procedures are recommended for "operations," that is, for evaluation of existing operating conditions (for which the various traffic characteristics and factors are known or can be measured) and determination of the design needed to produce a desired improvement for the current conditions, or those which can be projected several years or a short period into the future. With assumed peak-hour factors, the *Manual* can also be used for design of new facilities.

This explanation is intended to assist the highway and traffic engineer in understanding the interrelationship and philosophical approach between the *Manual* and the AASHO procedures, and in properly applying the data and analysis techniques presented.

DEVELOPMENT OF SIMPLIFIED PROCEDURES

The *Highway Capacity Manual* is based on extensive research and field observations and provides a sound basis for evaluating highway capacity. However, it is oriented more toward operations than design. Although procedures in the *Manual* are generally provided for design purposes, they are somewhat cumbersome. Besides, there are areas for which data are not available or where suggested empirical methods are not sufficiently detailed.

The AASHO Design Policy simplifies the general approach to design, as covered under the previous heading, but continues to rely on the *Manual* as a guide for certain details and adjustment factors for specific conditions. Thus, even with the AASHO approach, there is some difficulty in practical application of the available data.

To facilitate the use of the *Manual* and to expand on the AASHO Design Policy procedures, simplified techniques have been developed and are presented later in this report. The research aspects and longhand calculation methods in the *Manual* have been converted into a procedure which is more practical for application to design. Of particular significance are those for analyses of ramp exits and entrances, which in the *Manual* entail two somewhat overlapping and incomplete procedures. In this report only the pertinent data are utilized; where appropriate, the methods have been simplified and streamlined. To achieve a complete analysis technique, several refinements and additional (rational) procedures are presented for conditions not covered in the *Manual*.

A graphical method combined with procedural steps has been developed for design of freeways. The same techniques are applicable to freeway operations. The graphic solution for relating levels of service and capacity to pertinent geometric features and traffic conditions uses design charts and tables. The charts are chiefly stepped nomographs provided separately for each of the major freeway components—freeway proper, freeway exit terminals, freeway entrance terminals, at-grade ramp terminals, and weaving sections.

Chapter II

FREEWAY PROPER—UNINTERRUPTED FLOW

Freeways are designed to provide uninterrupted flow—a condition in which vehicles traveling the facility are not required to stop by any cause external to the traffic stream. A freeway, by the nature of its design and the service it is intended to provide, ideally maintains uninterrupted flow characteristics throughout. However, traffic on the freeway encounters two conditions of uninterrupted flow, as illustrated in figure 1—those free of the influence of entering and exiting traffic and those modified by the influence of merging, diverging and weaving traffic.

The point at which freeway operation may pass from one condition to the other cannot be measured directly. Depending on traffic volume, lane distribution, and geometric features, such influence may range from the actual length of diverging and merging maneuver lengths up to a distance of several thousand feet or more. The exact distance is not important, except for identifying the condition when evaluating levels of service and capacity, and hence determining the number and arrangement of lanes. In general terms, the influence of simple diverging or merging traffic along the freeway is deduced to be in the range of 2,000 to 2,500 feet on each side of the crossroad forming the interchange, or each interchange may be considered to have a total influence of ¾ to 1 mile along the freeway.

Levels of service and capacity for sections free of interchange influence are simply evaluated, as set out below. For sections within the influence of interchanges the same general procedure applies, except that it is modified by analyses required for ramp entrances, ramp exits, and weaving sections.

BASIC VALUES AND FACTORS

Uninterrupted flow on modern freeways (12-foot lanes with full shoulders) may be measured by a service volume for each level of service. This is expressed in the *Manual* as:

$$\mathrm{SV} = 2{,}000\ N\ \frac{v}{c}\ F_T$$

where

- SV = service volume (vph), total one direction of travel
- N = number of lanes, one direction of travel
- $\frac{v}{c}$ = ratio of volume to capacity (values given in table 2); for levels C and D, the ratio also involves peak-hour factor as multiplier
- F_T = truck adjustment factor (values given in table 4)

Table 2 expresses the v/c values with respect to level of service, average highway speed (AHS), number of lanes in one direction N, and the peak-hour factor (PHF).

The average highway speed is a measure of the quality of a highway. It is defined as the weighted average of "potential design speeds" within a highway section when each subsection therein is considered to have a measure of an individual design speed. Each horizontal curve can be calculated for its potential design speed (arbitrarily taken up to 70 mph), while portions of tangents, if long enough (allowing for appropriate acceleration and deceleration), are assumed to have a potential design speed of 70 mph. A "potential" speed profile based on these values can be obtained for the highway section or route (whether it be an existing facility or one under design), and weighted for development of an average highway speed. The area under the speed profile divided by the length of highway produces the average highway speed.

However, for high design speeds such detailed analysis is normally not required. On modern freeways designed for speeds of 60 to 80 mph, average highway speeds do not vary appreciably from design speeds. A freeway designed for a speed of 70 mph would be considered to have an AHS of 70 mph. A freeway designed for a speed of 80 mph would have an AHS of the order of 80 mph. For purposes of evaluating service volumes, according to the *Manual*, values of AHS of "70 mph or more" would be used in table 2 for such facilities. A freeway designed for a speed of 60 mph over some length would tend to have a somewhat higher AHS, probably 65 to 70 mph. As an average, it may be assumed that a freeway designed for a speed of 60 mph would represent an AHS of 65 mph. On the other hand, a gentle alinement with an occasional controlling or minimum curve on the facility would tend to approach an AHS of 70 mph. A 60-mph design freeway with a continuously curvilinear alinement in mountainous terrain would tend toward an AHS of 60 mph.

The significance of the peak-hour factor in determining service volumes was covered previously (pages 3 to 5). At intermediate levels of service C and D, the PHF is used as a multiplier in table 2 in determining the v/c ratio. On existing facilities the peak-hour factors, operating speeds, and volumes can be measured. From these, levels of service can be determined or degree of improvement or redesign indicated in accordance with the desired level of service.

In design of new freeways, which are intended to account for conditions 20 years or so in the future, peak-hour factors based on today's conditions may be meaningless since they

TABLE 2.—*Levels of service and maximum service volumes (expressed as a ratio to capacity) for modern freeways under uninterrupted flow conditions*

Level of Service	Traffic Flow Conditions		Service Volume to Capacity Ratio (v/c)[1] Limiting Value for Average Highway Speed (AHS) of:				
			70 mph or more				60 mph
	Description	Operating Speed[1] mph	4-lane Freeway (2 lanes, one direction)	6-lane Freeway (3 lanes, one direction)	8-lane Freeway (4 lanes, one direction)	10-lane Freeway (5 lanes, one direction)	4- to 10-lane Freeway (any number of lanes, one direction)
A	Free flow	$\geqq$ 60	$\leqq$0.35	$\leqq$0.40	$\leqq$0.43	$\leqq$0.44	—[2]
B	Stable flow (upper speed range)	$\geqq$ 55	$\leqq$0.50	$\leqq$0.58	$\leqq$0.63	$\leqq$0.65	$\leqq$0.25
C	Stable flow	$\geqq$ 50	$\leqq$(0.75 X PHF)[3]	$\leqq$(0.80 X PHF)	$\leqq$(0.83 X PHF)	$\leqq$(0.84 X PHF)	$\leqq$(0.45 X PHF)[3]
D	Approaching unstable flow	$\geqq$ 40	$\leqq$(0.90 X PHF)[3]				$\leqq$(0.80 X PHF)[3]
E	Unstable flow (capacity)	30–35	$\leqq$1.00				
F	Forced flow	$\leqq$30	Not Meaningful				

Adapted from 1965 *Highway Capacity Manual.*

[1] Operating speed and basic v/c ratio are independent measures of level of service; both limits must be satisfied in any determination of level.

[2] Operating speed required for this level is not attainable even at low volumes.

[3] Peak-hour factor (PHF) for freeways is the ratio of the whole-hour volume to the highest rate of flow occurring during a 5-min. interval within the peak hour.

are subject to change with redevelopment. As noted before, representative peak-hour factors may be generalized for several metropolitan area sizes as follows: less than ½ million population—0.77; ½ to 1 million population—0.83; and over 1 million population—0.91. Since there is a tendency for PHF to increase with traffic density, there is indication that at intermediate levels of service peak-hour factors are likely to be in the range of 0.83 to 0.91. Where peak-hour factors are not available, the following relations are suggested for design purposes:

Medium-to-large metropolitan areas

Level *C*—PHF of 0.83
Level *D*—PHF of 0.87
Level *E*—PHF of 0.91 to 1.00 (1.00 at capacity)

Large metropolitan areas (1.5 million or greater population)

Level *C*—PHF of 0.87
Level *D*—PHF of 0.91
Level *E*—PHF of 0.95–1.00 (1.00 at capacity)

The formula for SV indicated above is also fitting to AASHO procedure, except that the v/c values and lane design capacities as shown in table 3 are applicable. The adjustment for trucks F_T described below is the same for either procedure.

The service volume or capacity may be significantly affected by truck traffic. Each truck,[1] with respect to its influence on traffic movement, may be thought of as representing a number or group of passenger vehicles. Increases in the rate and length of grade produce higher passenger-car equivalents per truck. Within a normal range of conditions in alinement geometry and proportion of trucks on freeway facilities, passenger cars equivalent to one truck may be in the range of 2 to 15, as indicated in part A of table 4. The relations here are for measuring the effects on individual or specific grades.

In estimating the effects of trucks over the whole of a freeway or extended sections of it, the equivalents in part B of table 4 are suggested. These permit the evaluation of

[1] Truck—a vehicle with dual tires on one or more axles, excluding two-axle, four-tired vehicles that may be classified as trucks for registration purposes, but which have operating characteristics similar to those of a passenger car. For planning and design purposes, buses with dual tires are included in the truck class.

TABLE 3.—*Design and possible capacity values (AASHO) for modern freeways under uninterrupted flow conditions*

Environment or Operating Condition		Average Running Speed, mph	v/c Ratio	Service Volume pcph per Lane for Ideal Conditions
Design Capacity	Rural	45–50	0.50	1,000
	Suburban	40–45	0.60	1,200
	Urban	35–40	0.75	1,500
Possible Capacity		30	1.00	2,000

overall levels of service of freeways, exclusive of variations in levels of service for shorter lengths of individual grades. The generalized equivalents are 2, 4, and 8 passenger cars per truck for what are termed to be level, rolling, and mountainous terrain conditions. Classification of terrain condition is partly a matter of judgment. The following general guide is suggested:

Level terrain is predominantly flat, less than 2 percent grades.

Rolling terrain is generally undulating, 2 to 3 percent grades, with short lengths of 4 to 5 percent.

Mountainous terrain is long, sustained grades of 3 percent or more, and/or undulating grades of 5 percent or more.

The presence of trucks reduces service volume or capacity. The reduction factor for trucks, or the *truck adjustment factor*, as it is generally referred to, may be expressed as:

$$F_T = \frac{1}{1 + t\,(E_T - 1)}$$

where

F_T = truck adjustment factor

t = percentage of trucks in traffic stream ÷ 100

E_T = number of passenger cars equivalent to one truck

Thus any volume of mixed traffic can be converted to equivalent passenger cars by dividing this volume by F_T. Similarly, any service volume expressed in passenger vehicles can be converted to mixed traffic through multiplication by F_T. Part C of table 4 contains the normal range of truck adjustment factors based on the above equation.

Another aspect of the effect of trucks on levels of service is the reduction in speed of trucks due to grade changes. This is largely accounted for in the application of F_T factors. However, in any design it may be desirable to control the gradeline to keep the speed of trucks from falling below a certain level. Figure 2 shows the effect of length and steepness of grade on speed of average trucks (weight-power ratio of 200 pounds per horsepower (lb./hp)) on modern multilane highways. The solid curves show deceleration based on the assumption that the truck enters the grade at about 50 mph. However, a lower entering speed can be used in the graph to find the speed reduction for any distance upgrade. The dotted curves indicate acceleration performance on a full range of up and down grades.

Where the approach speeds of trucks are estimated to be greater than 50 mph, the relations in table 5 may be used to augment the deceleration distance in figure 2. The distances upgrade in figure 2 and table 5 are based on uniform grades. Where vertical curves are present, a portion of the curves is attributable to the effect of the grade. One-half to one-quarter of the length of vertical curve, as shown in figure 3, may be added to the upgrade (or downgrade) to produce an equivalent uniform grade, L_G, which then can be applied to figure 2 and table 4. Four types of vertical curves, I–IV, corresponding to the *Manual* designation, produce four variations (two shown in figure 3) in the measurement of L_G. Means of determining truck speeds and truck adjustment factors on broken or compound grades utilizing successive L_G values, as L_{G2}, L_{G3}, etc., are covered later in conjunction with illustrative problems.

The *Manual* indicates the sensitivity of the adjustment factor for trucks involving individual upgrades of variable rates and length, particularly as outlined in table 4–A. Although it is known that the effect of trucks on traffic operations and levels of service is in need of further research and refinement, the *Manual* methods are accepted as the best available at this time and are used throughout in the analyses presented here.

It is emphasized again that the effect of trucks on levels of service (other than capacity) are predicated generally on average passenger-car equivalents per truck E_T, either on general profile character (table 4–B)—level, rolling, or mountainous terrain—or on some composite value averaged over a series of grades along the section of highway being tested. However, the determination of E_T values at the end of long grades or along some point of grade normally is necessary for the determination of *capacity* (possible capacity) of the highway. Another purpose for such evaluation is to determine the level of service or capacity of a ramp exit or entrance occurring along an upgrade section of freeway.

As described above, to test the effect of truck traffic with respect to a given point along a profile grade, see table 4–A, which gives the relationship directly. Where the grade is preceded by a sag vertical curve or terminated by a crest vertical curve, figure 3 gives the approximate way for measuring the "length of equivalent uniform grade." The procedure (not covered in the *Manual*) for estimating the effect of trucks along any point of a broken or compound gradeline is fully outlined in a following illustration (problem 4).

TABLE 4.—*Adjustment factors for trucks on freeway facilities*

A. Passenger Car Equivalents on Specific Grades

Up Grade (%)	Length of Grade (miles)	Passenger Car Equivalent, E_T*				
		Levels of Service *A* through *C* for:				
		3% Trucks	5% Trucks	10% Trucks	15% Trucks	20% Trucks
0–1	All	2	2	2	2	2
2	¼–½	5	4	4	3	3
	¾–1	7	5	5	4	4
	1½–2	7	6	6	6	6
	3–4	7	7	8	8	8
3	¼	10	8	5	4	3
	½	10	8	5	4	4
	¾	10	8	6	5	5
	1	10	8	6	5	6
	1½	10	9	7	7	7
	2	10	9	8	8	8
	3	10	10	10	10	10
4	¼	12 ●*	9	5	4	3
	½	12 ●	9	5	5	5
	¾	12 ●	9	7	7	7
	1	12 ●	10	8	8	8
	1½	12 ●	11	10	10	10
	2	12 ●	11	11	11	11
5	¼	13 ●	10	6	4	3
	½	13 ●	11	7	7	7
	¾	13 ●	11	9	8	8
	1	13 ●	12 ●	10	10	10
	1½	13 ●	13 ●	12 ●	12 ●	12 ●
6	Add 1 to E_T values for 5% grade					

* Values with ●, add 1 for levels *D* and *E*.

B. Passenger Car Equivalents on Extended Sections

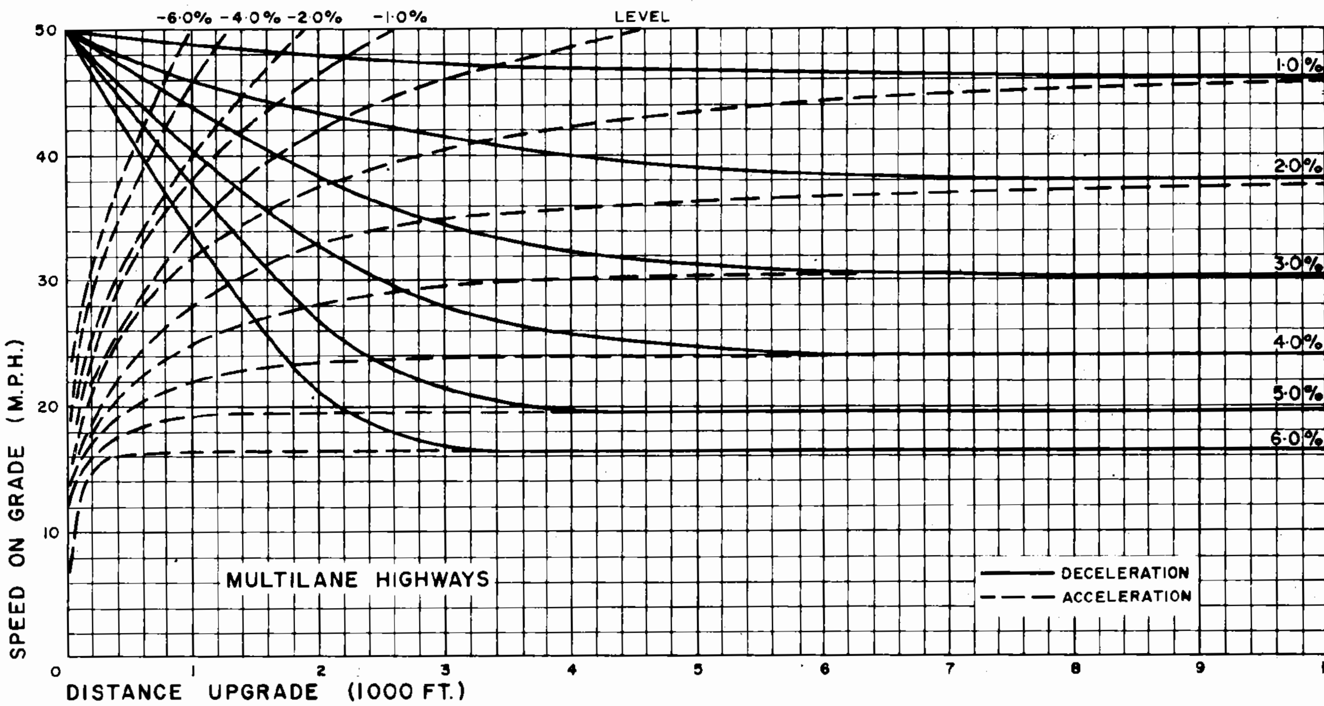
Terrain	E_T* All Levels of Service and Normal Range of Truck (%)
Level	2
Rolling	4
Mountainous	8

* Average, generalized equivalents.

C. Adjustment Factors Based on Truck Percentages and Passenger Car Equivalents

Passenger Car Equivalent E_T	Truck Adjustment Factor F_T for Percentage of Trucks, T, of:														
	1	2	3	4	5	6	7	8	9	10	12	14	16	18	20
2	0.99	0.98	0.97	0.96	0.95	0.94	0.93	0.93	0.92	0.91	0.89	0.88	0.86	0.85	0.83
3	0.98	0.96	0.94	0.93	0.91	0.89	0.88	0.86	0.85	0.83	0.81	0.78	0.76	0.74	0.71
4	0.97	0.94	0.92	0.89	0.87	0.85	0.83	0.81	0.79	0.77	0.74	0.70	0.68	0.65	0.63
5	0.96	0.93	0.89	0.86	0.83	0.81	0.78	0.76	0.74	0.71	0.68	0.64	0.61	0.58	0.56
6	0.95	0.91	0.87	0.83	0.80	0.77	0.74	0.71	0.69	0.67	0.63	0.59	0.56	0.53	0.50
7	0.94	0.89	0.85	0.81	0.77	0.74	0.70	0.68	0.65	0.63	0.58	0.54	0.51	0.48	0.45
8	0.93	0.88	0.83	0.78	0.74	0.70	0.67	0.64	0.61	0.59	0.54	0.51	0.47	0.44	0.42
9	0.93	0.86	0.81	0.76	0.71	0.68	0.64	0.61	0.58	0.56	0.51	0.47	0.44	0.41	0.38
10	0.92	0.85	0.79	0.74	0.69	0.65	0.61	0.58	0.55	0.53	0.48	0.44	0.41	0.38	0.36
11	0.91	0.83	0.77	0.71	0.67	0.63	0.59	0.56	0.53	0.50	0.45	0.42	0.38	0.36	0.33
12	0.90	0.82	0.75	0.69	0.65	0.60	0.57	0.53	0.50	0.48	0.43	0.39	0.36	0.34	0.31
13	0.89	0.81	0.74	0.68	0.63	0.58	0.54	0.51	0.48	0.45	0.41	0.37	0.34	0.32	0.29
14	0.88	0.79	0.72	0.66	0.61	0.56	0.52	0.49	0.46	0.43	0.39	0.35	0.32	0.30	0.28
15	0.88	0.78	0.70	0.64	0.59	0.54	0.51	0.47	0.44	0.42	0.37	0.34	0.31	0.28	0.26

Tables adapted from *Highway Capacity Manual*.



SOURCE: Highway Capacity Manual 1965 – HRB Special Report 87
Based on: Schwender, H.C. et al, "New Methods of Capacity Determination for Rural Roads in Mountainous Terrain" HRB Bulletin 167, pp 10–37 (1957)
Webb, G.M., Traffic Bulletin No. 2, "Truck Speeds on Grades." Cal. Div. of Highways (1961)

FIGURE 2.—Effect of length and grade on speed of average trucks on modern multilane highways.

TABLE 5.—*Speed reduction distance of trucks*[1] *on grade from higher speeds to 50 mph*

Upgrade %	Approximate distance along upgrade, in feet, at which speed is reduced to 50 mph from an initial (approach) speed of:	
	55 mph	60 mph
2	1,000	1,800
3	700	1,300
4	500	1,000
5	400	800
6	300	600

[1] Trucks having a weight-power ratio of 200 lb./hp.

Although the degree of refinement and detail with regard to the calculation of E_T values at points along or at the end of sustained uphill profiles having two or more gradients may be questionable, it is believed that the demonstration of the technique is sufficiently significant for purposes of gaining an advanced knowledge in design of truck operating speeds and their variation along a highway. As part of the procedure (demonstrated in problem 4) a predicted speed profile for the selected truck can be plotted. The plotting of an accompanying passenger-car speed profile can add further to the knowledge of operations, and thus allow the designer to adjust the location or alter the design of the highway in preliminary stages to avoid hazardous speed differential problems, or provide a more favorable or uniform level of service through design modification or lane rearrangements.

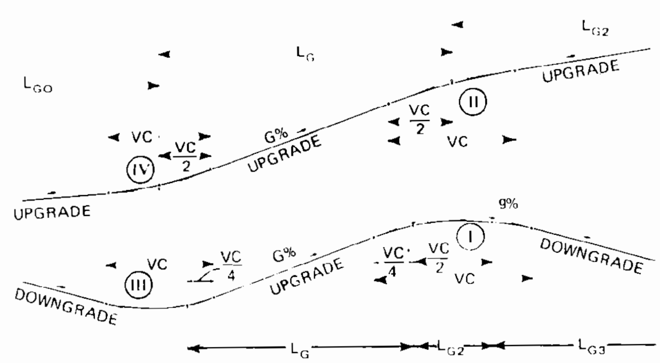


FIGURE 3.—Basis for measuring length of equivalent uniform upgrade in conjunction with truck operations.

ANALYSIS TECHNIQUES

Given the level of service for a specified hour or period of operation, the average highway speed, the number of lanes in each direction, the grade or terrain condition, and the percentage of trucks during the period noted above, the service volume on the freeway can be calculated by using the previously established formula, SV $= 2{,}000\ (N)\ (v/c)$

(F_T). Values for v/c are found in table 2 and for F_T in table 4. If the design hourly volume (DHV) is given instead of the level of service, the latter can be found by substituting DHV for SV in the formula and solving for v/c. Comparing the calculated v/c with values in table 2, the level of service (at which the facility would operate accommodating the DHV) can be identified. The same basic formula and table 4 may be used for the AASHO procedure. Table 3 applies here for the v/c values.

To simplify computation procedures, a set of nomographs has been developed. Three charts are included, figures 4 through 6, corresponding to average highway speeds of 70 mph or more, 65 mph, and 60 mph, respectively. All of the above elements—the basic formula, the table 2 relations, and the E_T and F_T values of table 4—are incorporated in the charts. The relations are based on 12-foot lanes, full shoulders, and adequate clearances. The same solution results are obtained by both the longhand method and the nomographs. The advantage of the nomographs is that any variable can be found directly. The design may be adjusted by adding or removing a traffic lane, increasing the design hourly volume, or changing the design speed or the average highway speed. The effects of these adjustments can be examined visually on the charts.

The three external scales on the nomograph represent the service volume per lane (left), the freeway volume in one direction (bottom), and the percentage of trucks in the freeway volume (upper right). Supplementary scales for service volumes per lane are indicated in conjunction with the left ordinate and are used only when the lane service volumes for a given level of service and a given PHF are required. The charts may be used in any of several ways:

a. Starting at the upper left with a given level of service and number of lanes, proceeding down and to the right to find the permissible freeway service volume on the bottom (V_f) scale, as shown by dotted arrows in figure 4.

b. Entering at the bottom (V_f) scale with a given design hourly volume on the freeway and proceeding up and to the left to find the level of service for a given number of lanes.

c. Again entering at the bottom but this time proceeding to find the fractional number of lanes required for a limiting level of service.

The nomographs can also be applied to the solution of capacity problems using AASHO procedures. In thus using figure 4, the PHF scales of 1.00 are utilized only and no reference is made to the level of service bands. The average highway speed does not enter directly into the solution. (Figures 5 and 6 can also be employed and will give the same answer.) Lane service volumes in this case are significant (chart ordinate on the left) and would be used as follows: rural conditions, 1,000 passenger cars per hour (pcph); suburban conditions, 1,200 pcph; urban conditions, 1,500 pcph; and capacity conditions, 2,000 pcph.

The following example problems illustrate the application of these procedures for interrupted-flow conditions on freeways.

Problem 1

Determine the maximum service volume to maintain level of service *C* on a 4-mile section of eight-lane freeway through an industrial district on the outskirts of a large metropolitan area. Other conditions are: AHS = 70 mph, PHF = 0.83, 12-foot lanes, full shoulders, profile characteristic of a rolling terrain, and trucks make up 15 percent of the predominant flow during peak hours. Use both the longhand and the nomograph procedures in the solution.

Solution In using the longhand method, the service volume is established by the formula, SV = 2,000 (N) (v/c) (F_T). The maximum volume/capacity ratio at level of service *C* for AHS = 70 mph and N = 4 (eight-lane freeway) is found in table 2 to be v/c = (0.83) PHF = 0.83 × 0.83 = 0.69. To determine the truck adjustment factor, first a passenger-car equivalent per truck must be established. In table 4–B, $E_T = 4$ for an extended section of highway in rolling terrain; in table 4–C, using $E_T = 4$ and T = 15 percent, $F_T = 0.69$. Maximum service volume in one direction to maintain level of service *C* is then SV = 2,000 × 4 × 0.69 × 0.69 = 3,800 vph.

For the nomograph solution, use figure 4. The example on the chart, shown by dotted lines and arrows, illustrates the solution of this problem. In the upper left portion of the chart locate the intersection point of "4-lane" line and "maximum level *C*" curve (circled point). From this point, project downward to PHF = 0.83, then proceed right intersecting the truck adjustment factors, F_T. Reenter chart at upper right with T = 15 percent. Proceed horizontally to $E_T = 4$ (value obtained from small table at lower left for rolling terrain), then vertically down to F_T adjustment. At the turning line, project radially to intersect the horizontal line previously drawn. From this point, project down to read $V_f = 3{,}800$. In this case, the freeway volume V_f is equal to the service volume or SV = 3,800 vph in one direction.

Problem 2

Determine the number of lanes required to maintain a directional volume of 2,900 vph at level *C* operation on a one-mile section of 3 percent upgrade. Other conditions are: AHS = 65 mph, PHF = 0.91 and T = 10 percent. With the required number of lanes provided, determine also the maximum volume which can be accommodated by the facility.

Solution Using figure 5, enter chart at bottom with $V_f = 2{,}900$ and proceed vertically to intersect the fan of F_T factors. Reenter chart at upper right with T = 10 percent; proceed left to $E_T = 6$ (value obtained from table at upper left using 3 percent grade, length 1 mile, and 10 percent trucks), and then down to F_T adjustment. At the turning line ($F_T = 0.67$), project radially to intersect vertical line previously drawn. From this point of intersection, proceed left to PHF = 0.91, and up to intersect the "maximum for *C*" curve; read 3.8 lanes.

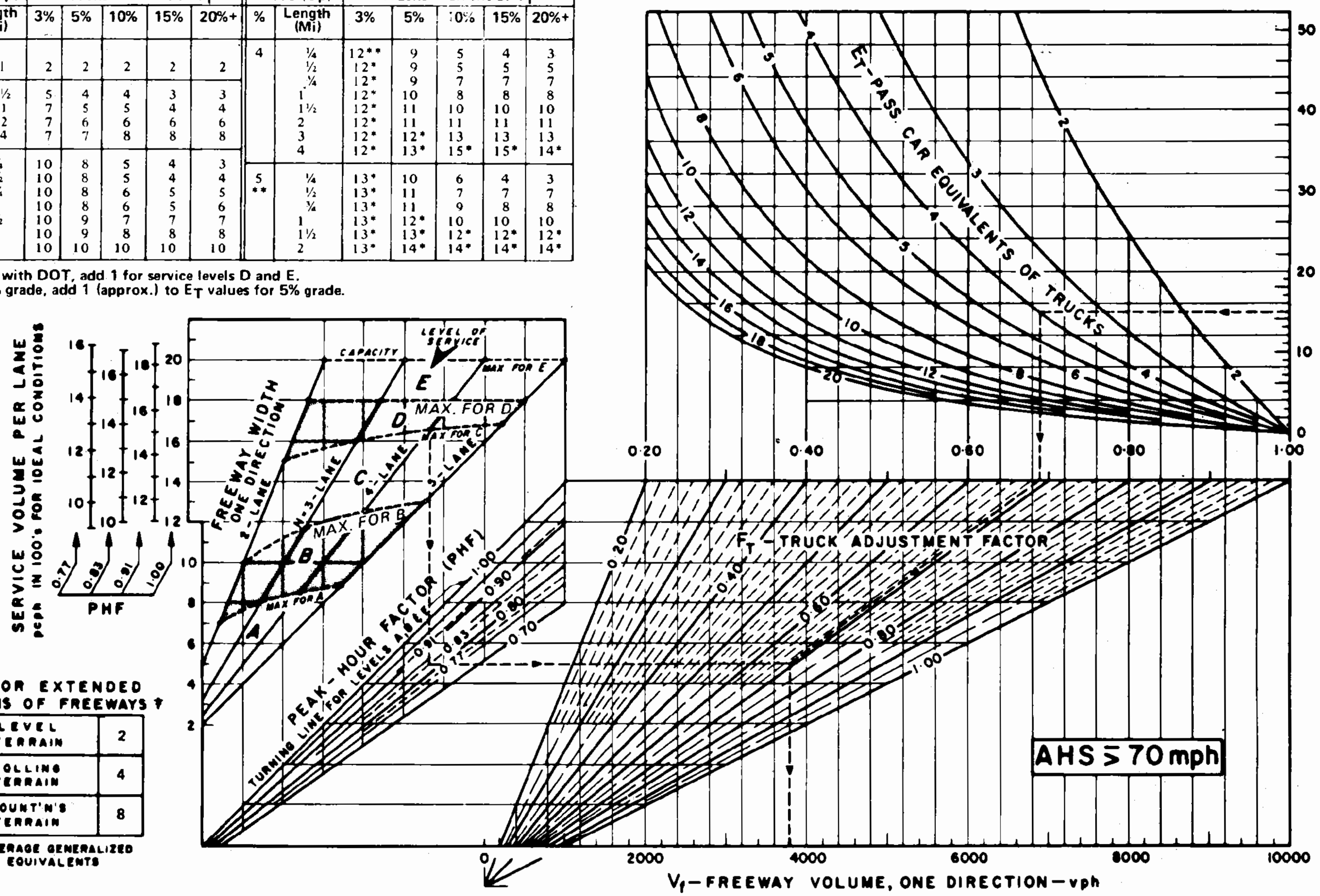
To maintain level *C* operation, the number of lanes is rounded to N = 4. Proceeding left from the point where the last projected line in the chart intersects the "4-lane" line, a service volume per lane may be read on the bar scale for PHF of 0.91; it is found to be approximately 1,080 pcph. This base value can be found also by dividing the total freeway volume by the number of lanes and by the truck adjustment factor, or 2,900 ÷ (4 × 0.67) = 1,080 pcph.

E$_T$—PASSENGER CAR EQUIVALENTS OF TRUCKS ON FREEWAYS

Grade (Up) %	Length (Mi)	Trucks—Percent of V_f 3%	5%	10%	15%	20%+
0-1	All	2	2	2	2	2
2	¼-½	5	4	4	3	3
	¾-1	7	5	5	4	4
	1½-2	7	6	6	6	6
	3-4	7	7	8	8	8
3	¼	10	8	5	4	3
	½	10	8	5	4	4
	¾	10	8	6	5	5
	1	10	8	6	5	6
	1½	10	9	7	7	7
	2	10	9	8	8	8
	3	10	10	10	10	10

Grade (Up) %	Length (Mi)	Trucks—Percent of V_f 3%	5%	10%	15%	20%+
4	¼	12**	9	5	4	3
	½	12*	9	5	5	5
	¾	12*	9	7	7	7
	1	12*	10	8	8	8
	1½	12*	11	10	10	10
	2	12*	11	11	11	11
	3	12*	12*	13	13	13
	4	12*	13*	15*	15*	14*
5 **	¼	13*	10	6	4	3
	½	13*	11	7	7	7
	¾	13*	11	9	8	8
	1	13*	12*	10	10	10
	1½	13*	13*	12*	12*	12*
	2	13*	14*	14*	14*	14*

*Values with DOT, add 1 for service levels D and E.
**For 6% grade, add 1 (approx.) to E$_T$ values for 5% grade.



E$_T$ FOR EXTENDED SECTIONS OF FREEWAYS †

LEVEL TERRAIN	2
ROLLING TERRAIN	4
MOUNT'N'S TERRAIN	8

† AVERAGE GENERALIZED EQUIVALENTS

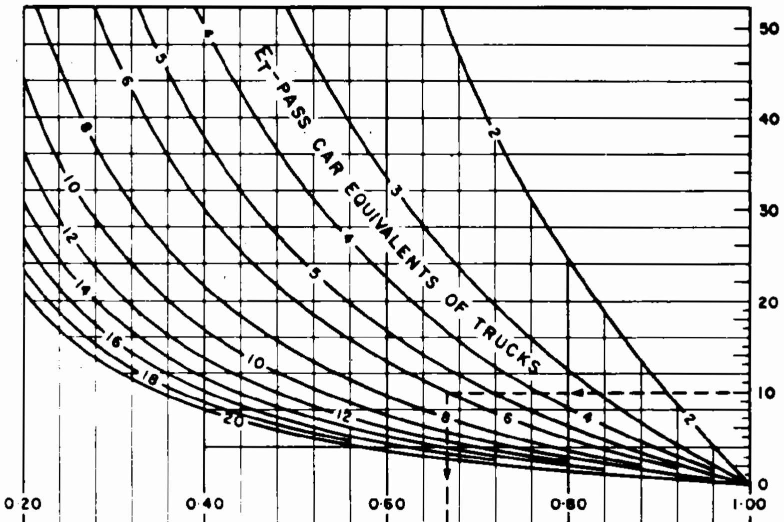
FIGURE 4.—Nomograph for determination of levels of service, service volumes, and capacity on freeways; average highway speed—70 mph.

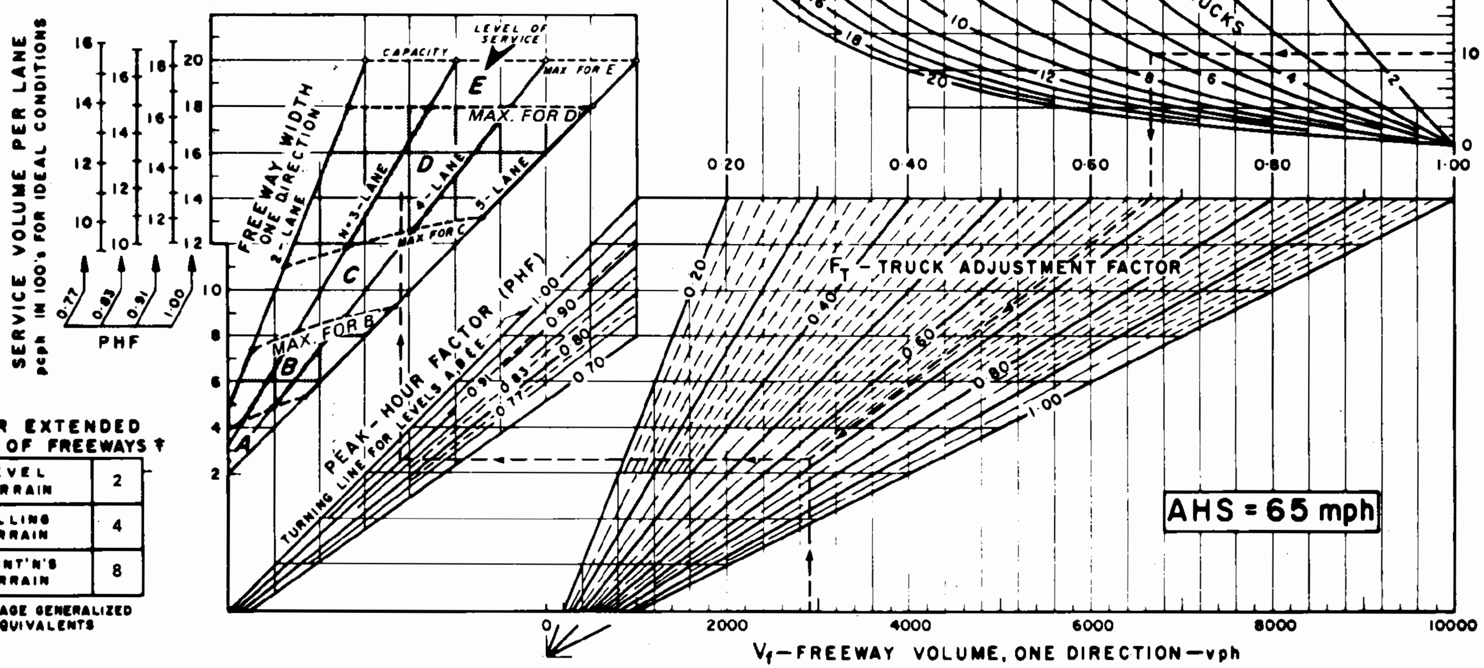
E_T—PASSENGER CAR EQUIVALENTS OF TRUCKS ON FREEWAYS

Grade (Up) %	Grade (Up) Length (Mi)	Trucks—Percent of V_f 3%	5%	10%	15%	20%+
0-1	All	2	2	2	2	2
2	¼-½	5	4	4	3	3
	¾-1	7	5	5	4	4
	1½-2	7	6	6	6	6
	3-4	7	7	8	8	8
3	¼	10	8	5	4	3
	½	10	8	5	4	4
	¾	10	8	6	5	5
	1	10	8	6	5	6
	1½	10	9	7	7	7
	2	10	9	8	8	8
	3	10	10	10	10	10

Grade (Up) %	Grade (Up) Length (Mi)	Trucks—Percent of V_f 3%	5%	10%	15%	20%+
4	¼	12**	9	5	4	3
	½	12*	9	5	5	5
	¾	12*	9	7	7	7
	1	12*	10	8	8	8
	1½	12*	11	10	10	10
	2	12*	11	11	11	11
	3	12*	12*	13	13	13
	4	12*	13*	15*	15*	14*
5 **	¼	13*	10	6	4	3
	½	13*	11	7	7	7
	¾	13*	11	9	8	8
	1	13*	12*	10	10	10
	1½	13*	13*	12*	12*	12*
	2	13*	14*	14*	14*	14*

*Values with DOT, add 1 for service levels D and E.
**For 6% grade, add 1 (approx.) to E_T values for 5% grade.





ET FOR EXTENDED SECTIONS OF FREEWAYS †

LEVEL TERRAIN	2
ROLLING TERRAIN	4
MOUNT'N'S TERRAIN	8

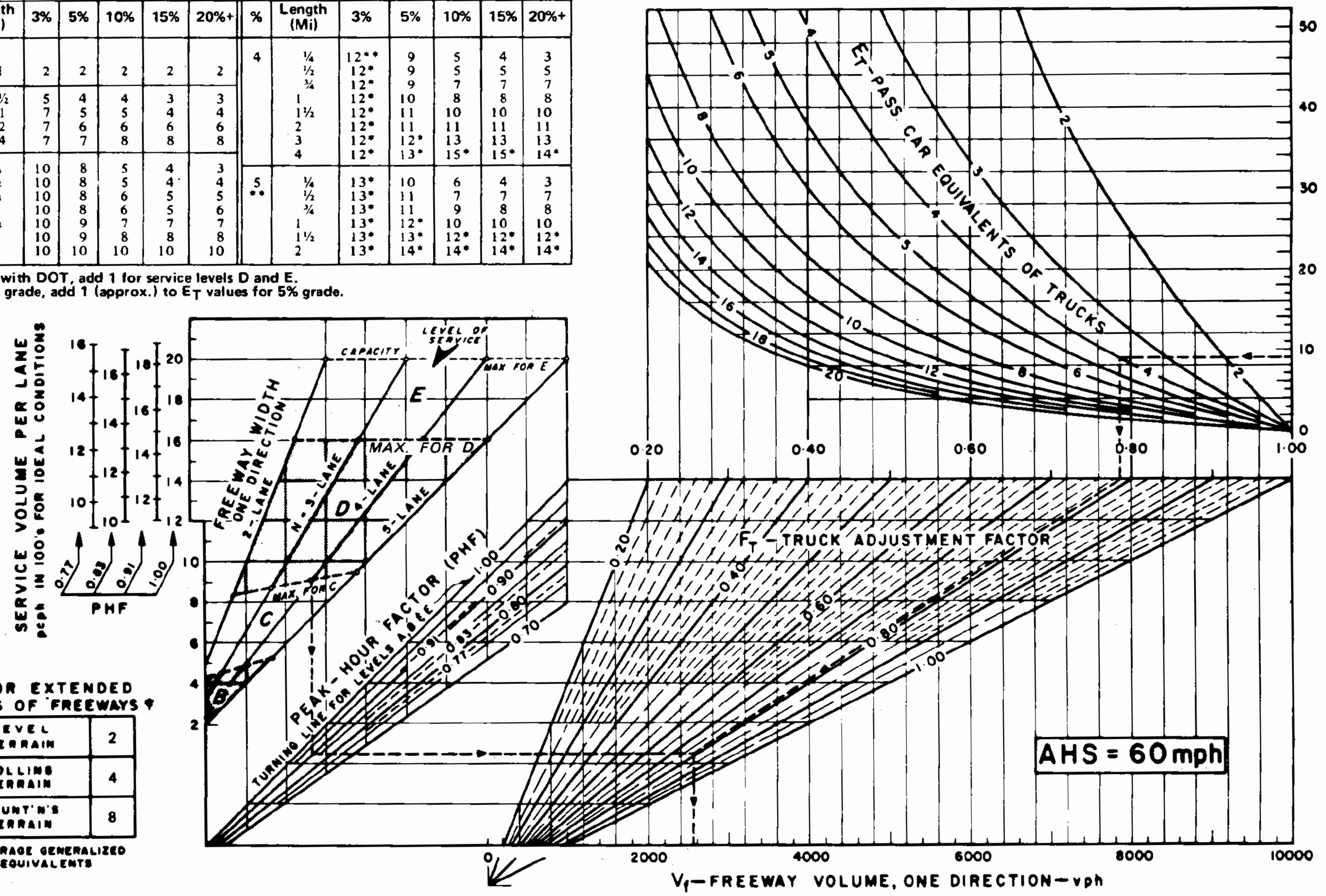
† AVERAGE GENERALIZED EQUIVALENTS

FIGURE 5.—Nomograph for determination of levels of service, service volumes, and capacity on freeways; average highway speed—65 mph.

E_T—PASSENGER CAR EQUIVALENTS OF TRUCKS ON FREEWAYS

Grade (Up)		Trucks—Percent of V_f				
%	Length (Mi)	3%	5%	10%	15%	20%+
0-1	All	2	2	2	2	2
2	¼-½	5	4	4	3	3
	¾-1	7	5	5	4	4
	1½-2	7	6	6	6	6
	3-4	7	7	8	8	8
3	¼	10	8	5	4	3
	½	10	8	5	4	4
	¾	10	8	6	5	5
	1	10	8	6	5	6
	1½	10	9	7	7	7
	2	10	9	8	8	8
	3	10	10	10	10	10
4	¼	12**	9	5	4	3
	½	12*	9	5	5	5
	¾	12*	9	7	7	7
	1	12*	10	8	8	8
	1½	12*	11	10	10	10
	2	12*	11	11	11	11
	3	12*	12*	13	13	13
	4	12*	13*	15*	15*	14*
5 **	¼	13*	10	6	4	3
	½	13*	11	7	7	7
	¾	13*	11	9	8	8
	1	13*	12*	10	10	10
	1½	13*	13*	12*	12*	12*
	2	13*	14*	14*	14*	14*

*Values with DOT, add 1 for service levels D and E.
**For 6% grade, add 1 (approx.) to E_T values for 5% grade.



E_T FOR EXTENDED SECTIONS OF FREEWAYS †

LEVEL TERRAIN	2
ROLLING TERRAIN	4
MOUNT'N'S TERRAIN	8

† AVERAGE GENERALIZED EQUIVALENTS

FIGURE 6.—Nomograph for determination of levels of service, service volumes, and capacity on freeways; average highway speed—60 mph.

To find the capacity of this facility, locate the intersection point of "4-lane" line with the "maximum for E" or capacity line (circled point). From this point, project downward to the upper turning line representing a PHF of 1.00. (PHF approaches 1.00 at capacity, even though at free flow or uncongested conditions it may be considerably lower; thus the unrestrained PHF of 0.91 noted above becomes 1.00 at capacity.) Then, proceed right to intersect the previously established F_T radial of 0.67. From this point project down to V_f (at capacity) = 5,350 vph.

The above solution was accomplished by using the nomograph. The longhand solution was not attempted because of the trial and error procedure which would have been required in using table 2. It may be noted that v/c varies not only with the number of lanes (which is an unknown to start with) but with the AHS. The nomograph is more expedient, providing direct solutions in all cases.

Problem 3

A 5-mile section of freeway is planned within a built-up district of a metropolitan area of 2 million population. Its curvilinear type of alinement conforms to a design speed of 70 mph. The vertical alinement undulates generally between 2 and 2.5 percent, with a critical grade of 3 percent 1,400 feet long and another 2.5 percent 7,500 feet long. Trucks constitute about 8 percent of the one-way peak-hour flow. The design hourly volume over the length of the improvement varies from 4,200 to 4,800 vph in one direction. Determine the basic number of lanes required and the capacity of this facility, based on (a) the level of service concept, and (b) the AASHO procedure.

Solution a. Design of new freeways in urban areas is normally predicated on level of service C, which level will be applied here. The PHF is not known. Hence, for level C operation in a large city, select a factor of 0.87 in accordance with the suggestion on page 8. Since the 70-mph design speed is equivalent to AHS of 70 mph, use figure 4 for the analysis. The vertical alinement described fits into a "rolling terrain" category, per classification outlined on page 9. Thus the passenger-car equivalent of trucks is taken as 4 (table, lower left corner of figure 5).

Enter the chart with DHV = V_f = 4,800 and T = 8 percent; proceed in reverse order, using $E_T = 4$ (yielding F_T of 0.80) and PHF = 0.87. Find 4.2 lanes required where the projected line intersects the "maximum for C" curve. Proceeding similarly through the chart, but using the lower range of DHV (4,200), the number of lanes required for level C operation is found to be 3.8. Thus, in terms of basic number of lanes, $N = 4$ in each direction may be considered appropriate. Each individual upgrade of 2 percent or more, however, should be analyzed to determine whether an extra (auxiliary) lane may be required for trucks. Use of auxiliary lanes is covered in subsequent chapters, with particular emphasis in chapter VI.

In evaluating the capacity of a freeway, the generalized passenger-car equivalents of trucks over extended sections of the facility (which may be used for levels of service under stable-flow conditions) cannot be employed. Instead, the most critical grade within a given section of freeway—as its weakest link—controls the capacity. In this problem two critical grades are indicated. The first, a 3 percent upgrade 1,400 feet long ($\pm\frac{1}{4}$ mile), with 8 percent trucks, yields $E_T = 6$ (table 4–A or upper table in figure 4). The second, a 2.5 percent upgrade 7,500 feet long ($\pm\frac{1}{2}$ mile), yields by interpolation $E_T = 7$. The latter is therefore the controlling grade. To determine capacity of the freeway, enter the chart of figure 4 at the upper left with $N = 4$, and proceed from the uppermost circled point (maximum for E) to the bottom of the chart, using PHF turning line of 1.00, T = 8 percent and $E_T = 7$ ($F_T = 0.68$); find capacity of 5,400 vph in one direction.

b. According to AASHO, the design of freeways in urban areas is predicated on a design capacity base or service volume of 1,500 pcph per lane. Peak-hour factor is not involved since it is considered "built in" for average conditions assumed in the future. Adjustments for trucks and grades apply as before. The chart in figure 4 may be used, ignoring the PHF and the level of service bands. Use the upper left ordinate scale on the chart directly as the design capacity base or service volume per lane and only the upper line (1.00) of the fan for PHF adjustment. In this manner, figure 4 can be adapted for all freeway uninterrupted flow solutions by the AASHO procedure. Enter the chart at bottom with DHV = V_f = 4,800 and intersect radial F_T = 0.80 (previously determined using T = 8 percent and $E_T = 4$). From this point proceed to left and turn on 1.00 line, projecting vertically to intersect a horizontal value of 1,500 pcph per lane. Read 4.0 lanes. A similar procedure for a DHV = 4,200 vph yields a required number of lanes of 3.6. As before, a basic number, 4 lanes, is established. Possible capacity likewise is controlled by the critical grade, which was found to have $E_T = 7$ and $F_T = 0.68$. Using a possible capacity base of 2,000 pcph, the procedure through the chart is identical to that in solution a for capacity. Possible capacity = 5,400 vph.

Problem 4

A section of rural freeway under design has a profile consisting of a combination of several grades as shown in figure 7. Determine passenger-car equivalents and approximate speed of trucks at pertinent points of grade change. Trucks constitute 5 percent of the one-way movement.

Solution Figure 2 shows the effect of length and steepness of grade on speed of average trucks (weight-power ratio of 200 lb./hp) on modern multilane highways. The chart permits a close approximation of speed of trucks upon deceleration or acceleration on various grades. Speeds of up to 50 mph are indicative of truck operations, when not hindered by upgrades, during peak periods at intermediate levels of service. The chart permits estimating speed along a broken or compound grade, as described below.

The effective beginning point of the 2 percent upgrade, point A in figure 7, includes one-quarter of the preceding vertical curve. (See figure 3 for basis of measurement.) At this point (sta. 135), the approach speed is taken to be 50 mph. The equivalent length of 2 percent upgrade is measured to the next vertical point of intersection (VPI) at sta. 175, a distance from A to B of 4,000 feet. A horizontal measurement of 4,000 feet along the 2 percent upgrade in the chart of figure 2 indicates a speed reached of 40 mph. This is shown at point b in figure 8(1), a sche-

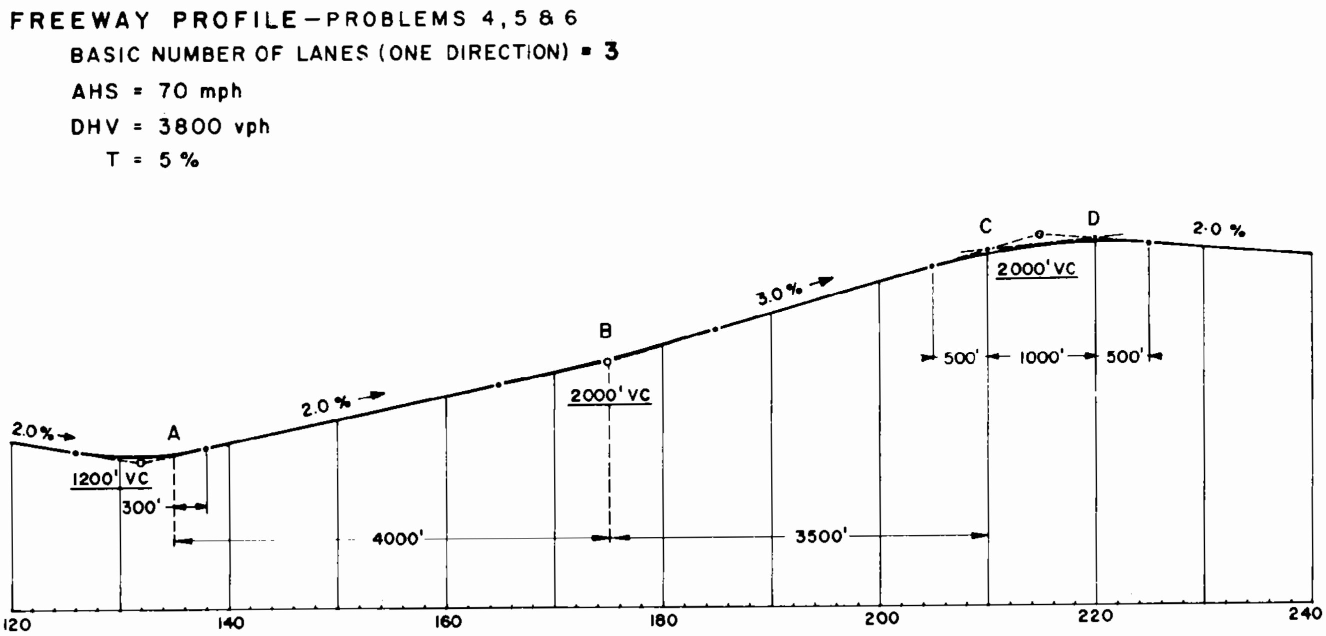


FIGURE 7.—Problems 4, 5, and 6 illustrated.

matic of figure 2 presented to illustrate the procedure in the chart. From point *b*, project the 40-mph speed horizontally on the chart to the intersection with the 3 percent upgrade (deceleration) curve. This point, noted *b′* in figure 8(1), is spotted at 1,700 feet on the horizontal scale of the chart.

The next strategic point on the profile (figure 7) occurs at *C*, the end of equivalent length of 3 percent grade which includes one-quarter of the following vertical curve. Thus, the effective length of 3 percent upgrade *B* to *C* is 3,500 feet, or a measurement on the abscissa of figure 2 of 1,700 + 3,500 = 5,200 feet. The speed read at this distance on the 3 percent grade is 31 mph. The schematic in figure 8(2) identifies this as point *c*.

The speed of trucks is next considered at the other quarter point on the crest curve, point *D*. (See figure 7.) A local tangent grade established at the middle of the vertical curve indicates the average grade between the two quarter points. Its horizontal length is equal to $VC/2$ and its rate of slope is $(3.0 - 2.0) \div 2 = +0.5$ percent. Returning to the chart of figure 2 (and the schematic in figure 8), transfer the 31-mph speed last measured to the left to an upgrade (acceleration) curve of +0.5 percent. This position on figure 2 is represented by point *c′* in figure 8(3), which is spotted at 850 feet on the horizontal scale of the chart. The length along the vertical curve, *C* to *D* (figure 7), is 1,000 feet or a measurement on the abscissa of figure 2 of 850 + 1,000 = 1,850 feet. The speed read at this distance on the +0.5 percent grade is 38.5 mph (point *d* in figure 8(4)).

The 38.5-mph speed in figure 2 is transferred to the left to intercept the −2.0 percent grade, the terminal grade which includes the last quarter of the crest curve. This position in figure 2 is represented by point *d* in figure 8(4), which is spotted at 900 feet on the horizontal scale of the chart. Proceeding along the −2.0 percent curve on the chart to point *e* where a speed of 46 mph is reached (speed at which the truck is assumed to have the same effect as on 0 to 1 percent grade, or a passenger-car equivalent of 2), a distance of 1,500 feet is found on the horizontal scale. A reading of 1,900 feet occurs at point *f* where the speed reached is 50 mph (figure 8(4)).

Truck speeds determined at the various points along the profile, decelerating from 50 mph to a minimum of 31 mph and accelerating again to 50 mph, are indicated in figure 10. If desired, an approximate speed profile for trucks can be plotted along the length of freeway being designed.

When truck speeds higher than 50 mph are to be evaluated (as during off-peak periods in conjunction with extensive flat or sustained downgrade approaches), use the same procedure with figure 2 as described above, except reduce the initial length of grade by the appropriate value in table 5. For example, if in the above problem the approach speed were 60 mph, a truck would travel 1,800 feet up a 2 percent grade before it would reduce speed to 50 mph. Enter the chart of figure 2, then, with 4,000 − 1,800 = 2,200 feet, producing a speed at point *B* of 43 mph, rather than 40 mph shown in figure 10; and at point *C* a speed of 32 mph rather than 31 mph, etc.

After spotting speeds, approximate the passenger-car equivalents at the various changes in grade using figure 2 and table 3. These are predicated on the 50 mph approach speed as the starting point at the bottom of the grade. The values of E_T, also shown in figure 10, may be determined as follows:

At *A*, trucks are unaffected by grade, so that $E_T = 2$ (same as level condition).

At *B*, using 2 percent upgrade, length 4,000 feet (sta. 135 to sta. 175) or ¾ mile, and 5 percent trucks, $E_T = 5$ in table 4.

At *C*, a composite grade must be used, a combination of 2 percent and 3 percent over a total distance, *A* to *C*, of 7,500 feet (sta. 135 to sta. 210) or 1.4 miles. An equivalent

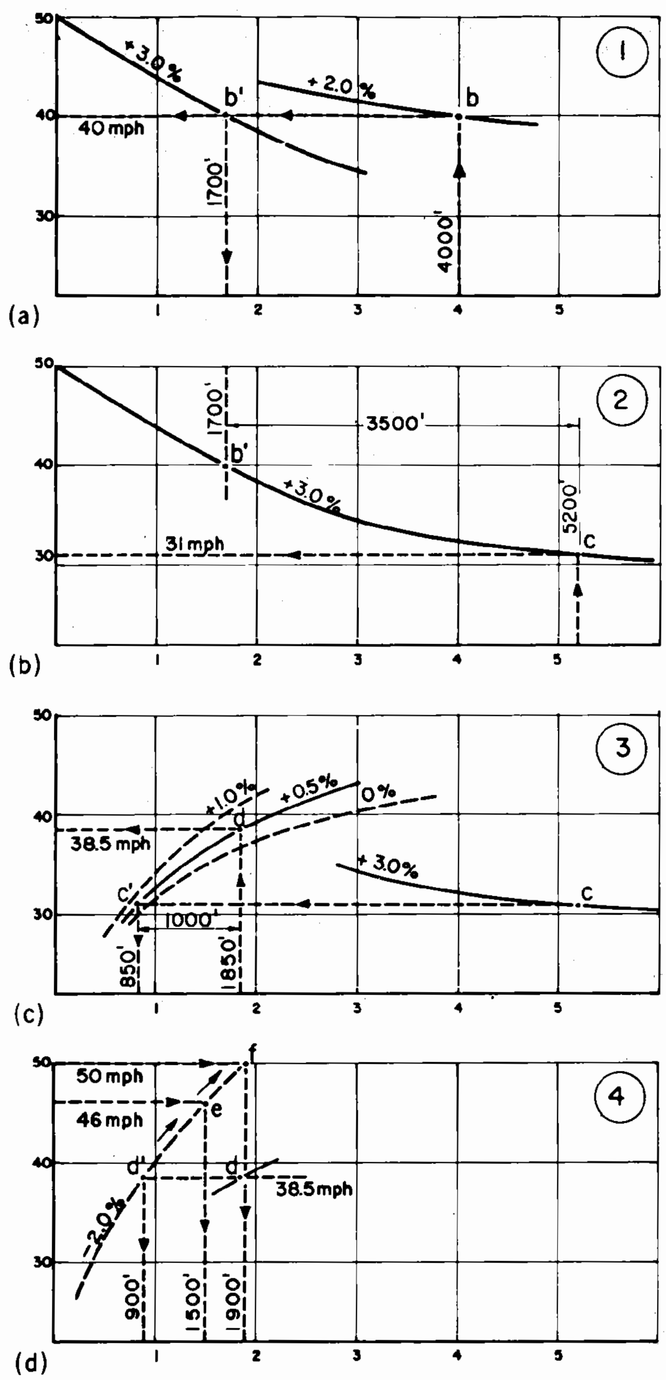


FIGURE 8.—Chart solutions for problem 4—truck speeds.

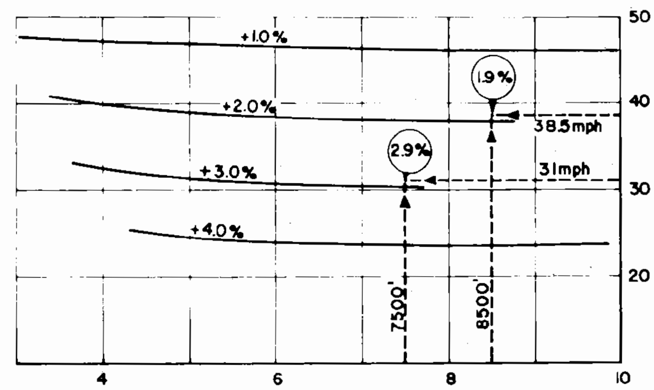


FIGURE 9.—Chart solutions for problem 4—equivalent grade values.

single grade over this distance may be estimated in figure 2. This is done by spotting the truck speed of 31 mph (previously determined at profile point C) on the chart of figure 2 at a distance of 7,500 feet. As shown in schematic of figure 9, the interpolated upgrade at this point is 2.9 percent. By interpolation in table 4, for an upgrade of 2.9 percent, 1.4 miles long, and 5 percent trucks, $E_T = 8$.

At D, the same procedure is followed by spotting the truck speed of 38.5 mph (previously determined) on the chart of figure 2 at a distance of 8,500 feet (sta. 135 to sta. 220). The interpolated upgrade at this point is 1.9 percent, as shown in figure 9. In table 4, using $G = +1.9$ percent, $L = 1.6$ miles and $T = 5$ percent, $E_T = 6$.

At points E and F, truck speeds of 46 and 50 mph are indicative of $E_T = 2$ (same as level condition).

A plot of E_T values can produce a continuous curve (E_T profile) along the length of freeway to facilitate the determination of an average or representative E_T value to establish the overall level of service or capacity for a selected section of highway.

Problem 5

If the freeway in problem 4 is a 6-lane urban facility, carries 3,800 vph in one direction, has an AHS of 70 mph and a general PHF of 0.90, determine the range in the levels of service at which the facility would operate over the indicated length. ($T = 5$ percent, as previously stated.)

Solution The variations in levels of service would be produced by the effect of trucks as reflected by the E_T factors. The highest level of service would occur where $E_T = 2$ (points A and F on the profile) and the lowest where $E_T = 8$ (point C on the profile). Progressing through the nomograph of figure 4 in reverse order, using $V_f = 3{,}800$, $T = 5$ percent, $E_T = 8$, and PHF $= 0.90$, project the vertical line upward to intersect a width of 3 lanes in the level of service zone E. Using the chart similarly but this time with $E_T = 2$, the intersection point in the upper left portion of the nomograph falls in the level of service C zone. Thus, the effect of grade in combination with trucks causes the operating conditions to change from a satisfactory level C to an unsatisfactory (near capacity) level E.

Problem 6

Using the AASHO procedure, determine the design capacity of the freeway in problem 5. What would be the design capacity of the facility if the 3 lanes in one direction are supplemented by a fourth (auxiliary) lane between points A and F to be used as a climbing lane for trucks?

Solution In table 3, the design capacity base is 1,500 pcph per lane in urban areas. (Remember: The peak-hour factor is considered to be built-in by this procedure, so that the value of PHF $= .90$ given previously is not used in the solution.) Enter chart of figure 4 at upper left with 1,500 pcph, proceed right to $N = 3$, down to turning line designated 1.00, and right to intersect the fan of F_T adjustment factors. Reenter chart at upper right with $T = 5$ percent, using the critical E_T value of 8 (previously

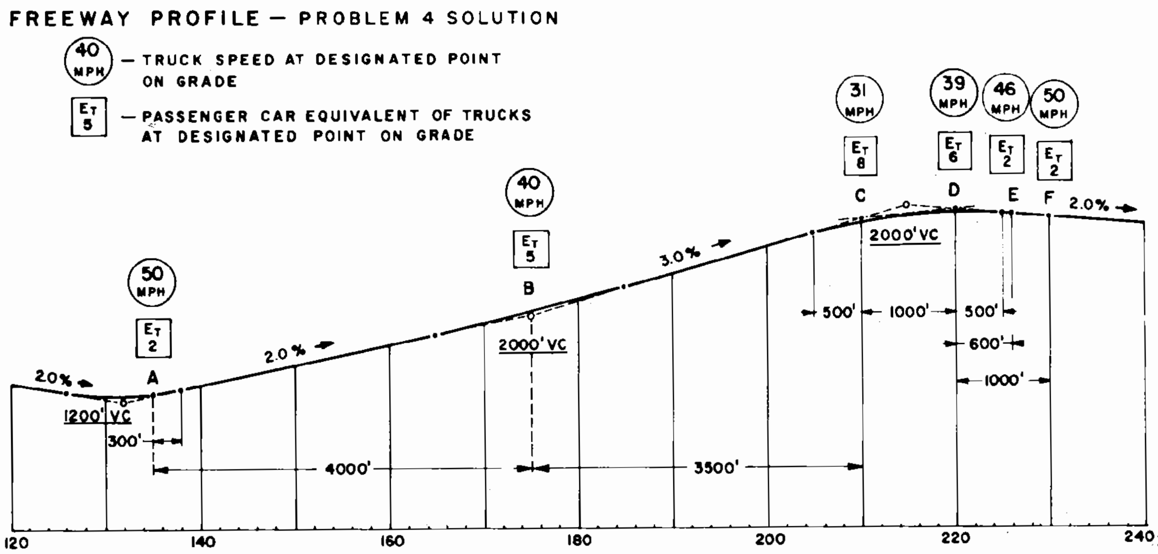


FIGURE 10.—Solution for problem 4 (also related to problems 5 and 6).

determined), proceed radially along F_T (0.74), intersecting the horizontal line (referred to above). From point of intersection, project downward to read V_f or design capacity = 3,350 vph. This is less than the DHV of 3,800 vph.

With an exclusive lane added for truck climbing, the design capacity of the uphill section is predicated on 3 lanes with $T = 0$ percent, plus the number of trucks in the fourth lane (assuming that this lane can accommodate all the trucks). Thus, the design capacity of the 3 basic lanes is $3 \times 1{,}500 = 4{,}500$ vph. The number of trucks using the fourth lane would be $V_t \div (4{,}500 + V_t) = 5$ percent $\div$ 100, or $V_t = 237$. The sum of the two volumes, or 4,737 vph, however, would not be the design capacity at the critical point (C) on the profile, since the 237 trucks having $E_T = 8$ would be equivalent to $237 \times 8 = 1{,}900$ pcph (approximately). The 1,500 vph design capacity base for urban conditions indicates that, to maintain the design capacity level of operation, trucks would use more than one lane Therefore, the design capacity of the critical section in this case would be predicated on a 4-lane section with 5 percent trucks, or, actually, $4 \times 1{,}500 \times 0.74 = 4{,}440$ vph. The design capacity of the 3-lane section in advance of and beyond the grade, on which $E_T = 2$ ($F_T = 0.95$) is $3 \times 1{,}500 \times 0.95 = 4{,}275$ vph. Thus, the auxiliary lane on the grade provides a good balance, and the freeway readily accommodates the DHV of 3,800 vph.

Chapter III

C–D ROADS AND FREEWAY DISTRIBUTORS

Full freeways accommodate the main lines of traffic, providing interchange at selected crossroad facilities. Where ingress and egress of traffic cannot be handled properly by an individual interchange or closely spaced ramps, the freeway must be supplemented by an auxiliary facility which can perform the function of collecting and distributing traffic. Such adjuncts to the freeway may constitute parallel one-way traveled ways, one on each side of the freeway, referred to as *C–D roads,* or, an extension as an off-shoot of the freeway on independent alinement, referred to as a *freeway distributor*. Both varieties are illustrated in figure 1. Some C–D roads do not differ much from an extended ramp. Other C–D roads and freeway distributors have much the same qualities as a freeway. All have control of access. Because such facilities serve as a link between the freeway and arterial streets, the measure of their level of service qualitatively would be intermediate between the two.

The *Highway Capacity Manual* does not include a procedure for evaluating levels of service on freeway distributors and C–D roads. The criterion used for full freeways obviously cannot be applied to distributors. On freeways, for example, the level *C* service volume recommended for urban conditions is 1,200 to 1,500 pcph per lane for an AHS of 70 mph, about 700 pcph per lane for AHS of 60 mph, and not attainable at AHS of 50 mph. Freeway distributors and C–D roads are usually designed for 60 and 50 mph, sometimes for 40 mph. Such facilities would be considered operating satisfactorily in urban areas if they accommodated the same volumes as a full freeway—1,200 to 1,500 pcph per lane—but understandably at lower operating speeds.

TABLE 6.—*Service volumes suggested for design of freeway distributors and C–D roads under uninterrupted flow conditions*

Level of Service	Maximum Service Volume, $\overline{sv}$, in pcph per Lane, for Design Speed of:		
	60 mph	50 mph	40 mph
B	1,200	1,100	1,000
C	**1,500**	**1,350**	**1,200**
D	1,700	1,600	1,500
E (Capacity)	2,000	2,000	1,800

Note: Values in bold type are assumed for urban design condition.

Service volumes of this order also are indicative of AASHO design considerations for expressways at grade and high-type ramps. Recognizing further that these are connecting facilities between freeways and major streets, and taking into account the v/c ratios and operating speeds of each as associated with levels of service in the *Manual,* the following relations (table 6) fall into line and are suggested for design of freeway distributors and C–D roads.

Lane service volumes in table 6 are base values for design purposes. Peak-hour factor is not a consideration with these generalized values. The adverse effects of restricted alinement and overall ability to maintain relatively high-speed operation are reflected in the attainable or designated design speed. The lower the design speed (quality of highway), the lower the service volume which can be achieved for a given level of service. Collector-distributor roads desirably should maintain 12-foot lanes and adequate lateral clearances, resembling full freeways. This, however, may not be feasible and narrower cross-sectional dimensions have to be considered in some cases. Adjustments for lateral restrictions, F_W, are given in table 7. Restrictive elements are lane widths less than 12 feet wide and objects along the roadway less than 6 feet from outer edges of traveled way. Lateral obstructions are such elements as formidable curbs, bridge parapets or rails, guardrails, retaining walls, etc.

TABLE 7.—*Adjustment factors, F_W, for lane width and lateral clearance on freeway distributors and C–D roads (2-, 3-, and 4-lane one-direction roadways)*

Distance Lane Edge to Obstruction, ft.	Obstruction One Side of One-Direction Roadway		Obstruction Both Sides of One-Direction Roadway	
	12-ft. lanes	11-ft. lanes	12-ft. lanes	11-ft. lanes
6	1.00	0.97	1.00	0.97
4	0.99	0.96	0.98	0.95
2	0.97	0.94	0.94	0.91
0	0.90*	0.87*	0.82*	0.79*

Adapted from *Highway Capacity Manual.*

* For zero lateral clearance on 3- and 4-lane roadways, *ADD* to tabular (asterisked) values 0.04 for obstruction one side and 0.08 for obstruction both sides.

Note: For special case with 10-foot lanes, subtract 0.06 from tabular values for 11-foot lanes.

E_T—PASSENGER CAR EQUIVALENTS OF TRUCKS ON FREEWAYS

Grade (Up) %	Grade (Up) Length (Mi)	Truck—Percent of V_f 3%	5%	10%	15%	20%+
0-1	All	2	2	2	2	2
2	¼-½	5	4	4	3	3
	¾-1	7	5	5	4	4
	1½-2	7	6	6	6	6
	3-4	7	7	8	8	8
3	¼	10	8	5	4	3
	½	10	8	5	4	4
	¾	10	8	6	5	5
	1	10	8	6	5	6
	1½	10	9	7	7	7
	2	10	9	8	8	8
	3	10	10	10	10	10

Grade (Up) %	Grade (Up) Length (Mi)	Trucks—Percent of V_f 3%	5%	10%	15%	20%+
4	¼	12**	9	5	4	3
	½	12 *	9	5	5	5
	¾	12 *	9	7	7	7
	1	12 *	10	8	8	8
	1½	12 *	11	10	10	10
	2	12 *	11	11	11	11
	3	12 *	12*	13	13	13
	4	12 *	13*	15*	15*	14*
5 **	¼	13 *	10	6	4	3
	½	13 *	11	7	7	7
	¾	13 *	11	9	8	8
	1	13 *	12*	10	10	10
	1½	13 *	13*	12*	12*	12*
	2	13 *	14*	14*	14*	14*

*Values with DOT, add 1 for service levels D and E.
**For 6% grade, add 1 (approx.) to E_T values for 5% grade.

E_T FOR EXTENDED SECTIONS OF FREEWAYS ♦

LEVEL TERRAIN	2
ROLLING TERRAIN	4
MOUNT'N'S TERRAIN	8

♦ AVERAGE GENERALIZED EQUIVALENTS

VALUES OF F_W

LATERAL CLEARANCE (feet)	ADJUSTMENT FACTOR ON ONE-DIRECTION ROADWAY: OBSTRUCTION ONE SIDE FOR LANE WIDTH OF: 12 ft.	OBSTRUCTION ONE SIDE FOR LANE WIDTH OF: 11 ft.	OBSTRUCTION BOTH SIDES FOR LANE WIDTH OF: 12 ft.	OBSTRUCTION BOTH SIDES FOR LANE WIDTH OF: 11 ft.
6	1·00	0·97	1·00	0·97
4	0·99	0·96	0·98	0·95
2	0·97	0·94	0·94	0·91
0	0·90*	0·87*	0·82*	0·79*

* APPLICABLE TO 2, 3 AND 4 LANES ONE-WAY, EXCEPT FOR ZERO LAT. CLEARANCE ONLY ON 3 AND 4 LANES. ADD TO TABULAR VALUES 0·04 OBSTRUCTION ONE SIDE AND 0·08 OBSTRUCTION BOTH SIDES.

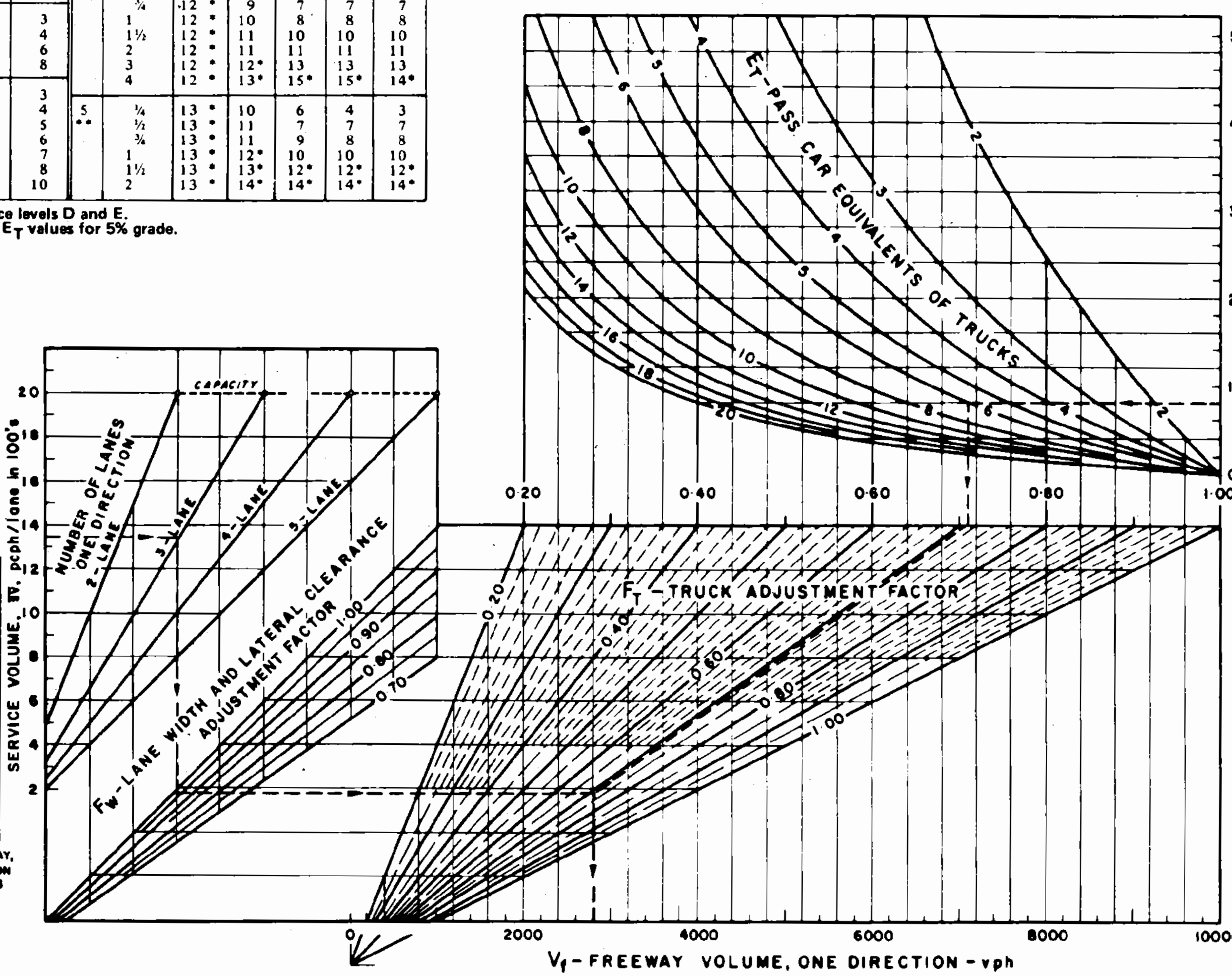


FIGURE 11.—Nomograph for determination of service volumes and capacity on freeway distributors and C–D roads.

Barrier-type curbs, higher than 6 inches with near vertical face, normally are considered to be lateral obstructions. For combinations of curb and parapet, curb and wall, etc., the higher element (providing the two are less than 3 feet apart) constitutes the obstruction, unless the curb is 10 inches or higher (in which case the curb is the controlling feature).

For a given design speed, the maximum lane service volume, $\overline{sv}$ at each level of service, is designated in table 6. The total service volume, SV, in one direction of travel may be calculated using the formula:

$$SV = \overline{sv}\ N\ F_W\ F_T$$

The same results may be achieved directly by the use of the nomograph in figure 11.

Problem 7

A C–D road in an urban area, designed for 50 mph to serve several interchanges, has 3 basic lanes in one direction—12-foot lanes, 8-foot clear shoulder on the right, and 2-foot offset to a barrier-type curb on the left. Trucks make up 8 percent of the one-way flow during peak hours, controlled by a 3 percent upgrade ¼ mile long. Determine the service volume this facility can accommodate (level of service *C* or AASHO "design capacity"), using both the longhand and the nomograph procedures.

Solution In using the longhand method, the service volume is established by the formula, $SV = \overline{sv}\ N\ F_W\ F_T$. In table 6, for level *C* and design speed of 50 mph, $\overline{sv}$ = 1,350. In table 7, $F_W = 0.97$, using 12-foot lanes with 2-foot obstruction on one side. In table 4, for 8 percent trucks and 3 percent upgrade ¼ mile long, $F_T = 0.71$. SV = 1,350 × 3 × 0.97 × 0.71 = 2,800 vph. The nomograph solution is accomplished by the use of figure 11. The example on the chart illustrates, by dotted lines and arrows, the solution of the problem. Enter chart at upper left with sv = 1,350 pcph (value from table 6 for urban condition and design speed of 50 mph). Proceed right to $N = 3$ and turn downward to $F_W = 0.97$ (value obtained from table at lower left for 12-foot lanes with 2-foot obstruction on one side). Again proceed right, intersecting the truck adjustment factors, F_T. Reenter chart at upper right with $T = 8$ percent. Proceed left to $E_T = 6$ (value obtained from table at upper left for 8 percent trucks and ¼ mile upgrade of 3 percent), then vertically down to F_T adjustment (0.71). At turning line, project radially to intersect the horizontal line previously drawn. From this point, project down to read V_f (or SV) = 2,800 vph.

Problem 8

An existing 6-lane viaduct in an urban area is being adapted as a distributor in conjunction with a new freeway. The structure has 11-foot lanes, no shoulders, and a narrow median. The traveled way in each direction has a 10-inch barrier curb on the right and an 8-inch barrier curb on the left, each offset 1 foot from the edge of traveled way. From the face of left curb to the face of median barrier (guardrail) is a further offset of 2 feet. The design speed of the facility is considered to be 60 mph. Longitudinal gradient is less than 1 percent and trucks comprise 7 percent of the peak-hour one-directional volume. Using AASHO terminology, determine the *design* and *possible* capacities of the distributor.

Solution Enter chart of figure 11 at upper left with $\overline{sv}$ = 1,500 (value from table 6 for urban design condition and design speed of 60 mph). Proceed right to $N=3$ and turn downward to $F_W = 0.94$. (The value for F_W is found in the tabulation at lower left of figure 11, for a 1-foot obstruction on the right and a 3-foot obstruction on the left. This is accomplished by averaging the adjustment factors for an assumed obstruction of 1 foot on both sides and an assumed obstruction of 3 feet on both sides; or $F_W = (0.89 + 0.93) \div 2 = 0.91$. Turn right and proceed horizontally to intersect truck adjustment factors, F_T. Reenter chart at upper right with $T = 7$ percent. Proceed left to $E_T = 2$ (value obtained from table at upper left for grades of less than 1 percent), then vertically down to F_T adjustment (0.93). At turning line, project radially to intersect he horizontal line previously drawn. From this point project downward to read V_f (or design capacity) = 3,800 vph in one direction. Possible capacity = 3,800 × (2,000/1,500) = 5,070 vph.

Chapter IV

RAMPS

The term *ramp* generally refers to an interconnecting roadway of an interchange. The designation may also pertain to an individual connection or one of a pair or series of connections, allowing for transfer of traffic between two facilities. Normally a ramp constitutes a one-way roadway. Varieties of situations involving ramps are indicated diagrammatically in figure 1. These include turning roadways at freeway-to-freeway interchanges, turning roadways at freeway-to-noncontrolled access highway or street interchanges, junctions between freeway and freeway distributors, transfer roadways between freeway and C–D roads, and connecting roadways between freeway distributor (or C–D road) and surface streets.

RAMP ELEMENTS

There are three distinctive elements of a ramp which affect the operating characteristics and capacity of the total ramp facility. These are identified, in direction of travel, as: (a) exit terminal, (b) ramp proper, and (c) entrance terminal (see figure 12). For a given level of service, the smallest of the three service volumes achieved controls and designates the level of service or the capacity of the ramp as a whole. There are two varieties of ramp exit terminals, as shown in figure 12: (a) diverging-flow terminals providing for gradual separation of traffic streams and (b) at-grade terminals, accommodating traffic through some extent of intersection control. Likewise there are two forms of ramp entrance terminals: (c) merging flow terminals providing for gradual convergence of traffic streams, and (d) at-grade terminals producing a degree of interrupted flow on the ramp and (with signal control) on the crossroad.

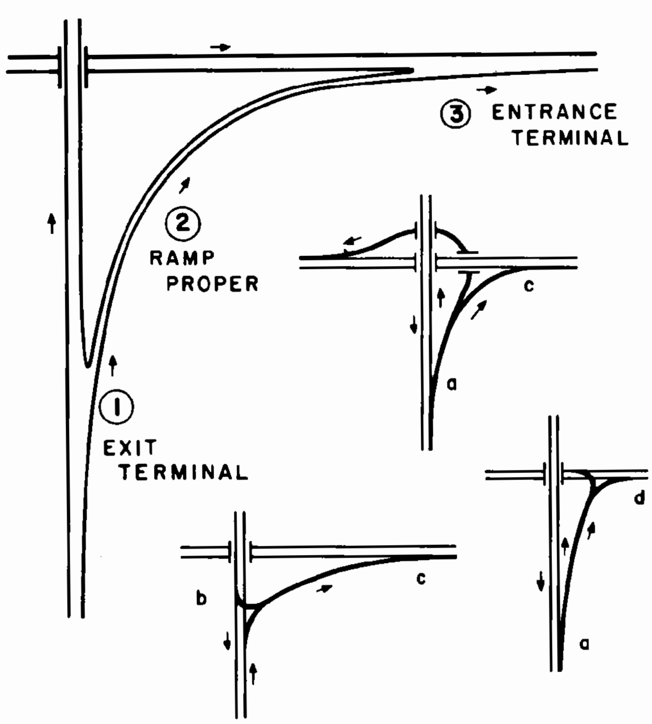


FIGURE 12.—Ramp elements.

The number and arrangement of lanes in the terminal areas also influence operational characteristics and capacity. The spacing and arrangement of entrances and exits where they occur in succession along the freeway further affect capacity. The breakdown and classification of various ramp terminal forms and their combinations allow for analysis procedures to be developed.

BASIC VALUES AND FACTORS

Peak period variations in traffic flow on ramps may significantly affect operating conditions on the freeway. Peaking characteristics within the hour could be critical, requiring that geometrics be based on a rate of flow in a period of time shorter than 1 hour. The application of the peak-hour factor in such cases would tend to enlarge or improve the facility to compensate for this characteristic. Such considerations apply to existing operational problems. For design applications, however, the estimated design hourly volumes coupled with unknown peaking characteristics in localized areas, do not justify analysis refinements in the use of peak-hour factors. Even though a PHF may be estimated for a section of freeway as a whole, it may vary considerably at an interchange associated with various concentrations of development and attendant traffic generation. Thus, for design purposes, average preselected PHF's are introduced and are "built-in" to the maximum service volumes prescribed for the various levels of service. Where it is anticipated, by the nature of the planned development, that certain ramps will be required to accommodate high concentrations of traffic for short periods within the peak hour, special adjustments may be justified in the analysis as described later on.

The capacity of a ramp is generally controlled by one of its terminals. Occasionally the ramp proper determines the capacity, particularly where speeds may be significantly affected by curvature, grades, and truck operations. Of the three ramp elements, the entrance terminal will more often

TABLE 8.—*Service volumes (base values) suggested for design of ramps proper (single-lane operation)*

Design Condition	Maximum Service Volume in pcph for Ramp Design Speed of:			
	$\lessapprox$20 mph	25 mph	30–40 mph	$\gtrapprox$50 mph
Rural Service Level *B*	800	1,000	1,100	1,200
Urban Service Level *C*	1,000	1,250	1,400	1,500
Capacity Service Level *E* (approximately)	1,250	1,600	1,800	1,900

Adapted from AASHO Design Policy.

control the capacity. Deficiencies at either the entrance terminal or the exit terminal can be overcome in design by the introduction of an auxiliary lane beyond the entrance or in advance of the exit. Deficiencies in the ramp proper can be accounted for by extending the auxiliary lane onto the ramp, as a second lane and carrying it throughout the length of the ramp.

Although the need to check the capacity of the ramp proper is recognized in the *Manual*, a specific procedure for doing so is not included. It is suggested here that the basic values presented in the AASHO Design Policy be employed for this purpose. Table 8 shows the basic values for single-lane ramps listed as service level *C*, the approximate operational level (design capacity) for which the volumes were apparently intended in the Policy. These service volumes are adapted to urban design conditions, while a lower set of values is selected for rural conditions indicative of service level *B*. The latter is more nearly in line with lane service volumes in rural areas for uninterrupted flow on freeways to which the ramps connect. The table includes a third group of values for level of service *E* (capacity) based closely upon the "possible capacity" of ramp proper in the AASHO Design Policy.

Properly designed ramps can accommodate two lines of vehicles; however, full efficiency of the second lane cannot be assured except at relatively high design speeds. Consequently, service volumes and capacities of 2-lane ramps approach, but usually are less than, twice the values of single-lane ramps. For design purposes the multiplier to convert the single-lane tabular values to properly designed 2-lane ramps is taken to be in the range of 1.7 to 2.0, for low and high design speeds, respectively.

At entrance ramp terminals the critical feature in evaluation of capacities is the availability of sufficient time-space in lane 1 traffic stream (the outer right freeway lane adjoining the entrance ramp). Hence, the primary factor in ramp service volume and capacity determination is the prediction of lane 1 volume at the merging area. In the case of exit ramp terminals, estimation of lane 1 volume immediately upstream of the exit also is the essential factor in the determination of service volumes and capacity. (See figure 13 for identification of v_1 with respect to merging and diverging.)

In both cases the accumulation of vehicles in lane 1 of the freeway determines the degree of freedom or congestion experienced by entering or exiting traffic, as well as its influence on the flow of traffic on the freeway proper. Thus, both aspects play a part in the degree of driver satisfaction and, consequently, on the measure of the level of service at ramp entrances and exits. Lane 1 service volumes for various levels of service in the vicinity of ramp terminals are presented in table 9.

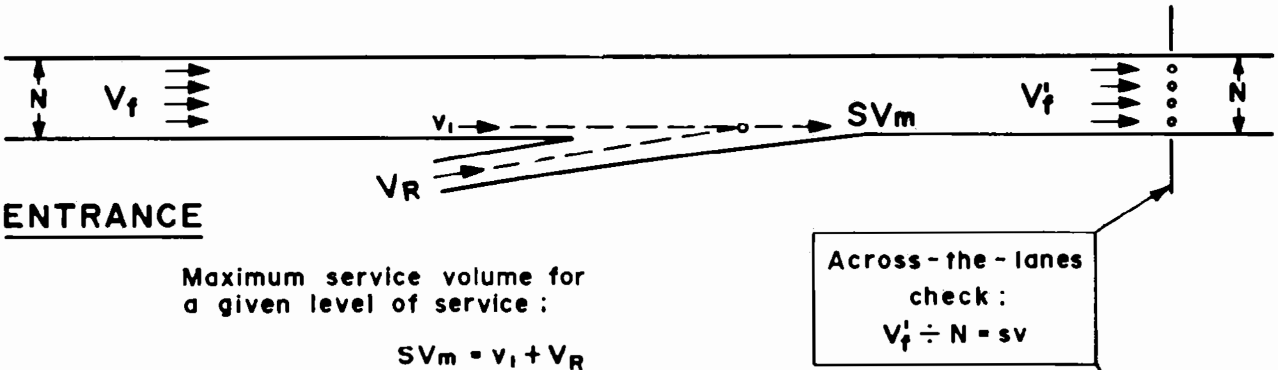
Compared with lane service volumes on the freeway proper, the values in table 9 are larger for the higher levels of service, nearly the same for the intermediate levels of service and slightly lower for capacity. The relatively larger values at higher levels of service are indicative of driver acceptance of somewhat greater densities in vicinity of ramp terminals, providing this condition is short-lived and the quality of operation resumes approximately to what it was prior to the ramp junction. At levels of service *A* and *B*, and to a lesser degree at *C*, reasonable overloading in lane 1 can be dissipated downstream from the ramp

TABLE 9.—*Service volumes for design of ramp terminals*

Level of Service	Maximum Service Volume, Base vph,[1] Occupying Lane 1 on Freeway after Merge (SV_m) or before Diverge (SV_d)	
	Merge SV_m	Diverge SV_d
A	1,000	1,100
B	1,200	1,300
C	1,400	1,500
D	1,600	1,700
E	1,800	1,900

Adapted from *Highway Capacity Manual.*

[1] Base value, in compliance with the *Manual*, for 5 percent trucks and near level conditions. Values for levels *C*, *D*, and *E* are adjusted for representative PHF of 0.83, and 0.90 to 0.95, respectively.



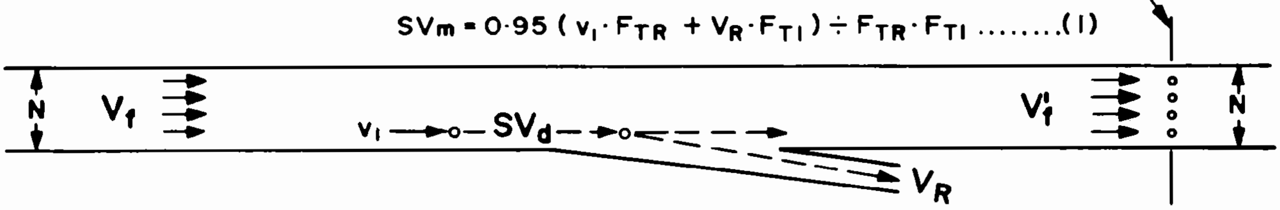
ENTRANCE

Maximum service volume for a given level of service:

$$SV_m = v_1 + V_R$$

Equation adjusted for trucks:

$$SV_m = 0{\cdot}95\,(v_1 \cdot F_{TR} + V_R \cdot F_{T1}) \div F_{TR} \cdot F_{T1} \;........(1)$$



EXIT

Maximum service volume for a given level of service:

$$SV_d = v_1$$

Equation adjusted for trucks:

$$SV_d = 0{\cdot}95 \;\; v_1 \div F_{T1} \;.................. (2)$$

Both for exits and entrances, v_1 is a function of V_f, V_R and N. Values of v_1, based on operational data, are given in the Highway Capacity Manual. Solutions of equations (1) and (2), including built-in v_1 relations are presented in NOMOGRAPH form in figures 17 and 18.

DEFINITION OF TERMS

SV_m = MAXIMUM SERVICE VOLUME AT RAMP ENTRANCE FOR A GIVEN LEVEL OF SERVICE IN LANE 1 OF FREEWAY AFTER MERGE, BASE VPH (5% TRUCKS, NEAR-LEVEL CONDITIONS).

SV_d = MAXIMUM SERVICE VOLUME AT RAMP EXIT FOR A GIVEN LEVEL OF SERVICE IN LANE 1 OF FREEWAY AT POINT OF DIVERGE, BASE VPH (5% TRUCKS, NEAR-LEVEL CONDITIONS).

V_f = TOTAL FREEWAY VOLUME UPSTREAM OF RAMP ENTRANCE OR RAMP EXIT, VPH

V_f' = TOTAL FREEWAY VOLUME DOWNSTREAM FROM RAMP ENTRANCE OR RAMP EXIT, VPH.

v_1 = LANE 1 VOLUME (AS PART OF V_f) JUST UPSTREAM OF RAMP ENTRANCE OR RAMP EXIT, VPH.

sv = MAXIMUM SERVICE VOLUME FOR A GIVEN LEVEL OF SERVICE ON THE FREEWAY PROPER, VPH PER LANE.

V_R or SV_R = RAMP VOLUME (DEMAND), OR RAMP SERVICE VOLUME FOR A GIVEN LEVEL OF SERVICE; THE TWO ARE INTERCHANGEABLE IN ABOVE EQUATIONS AND NOMOGRAPHS.

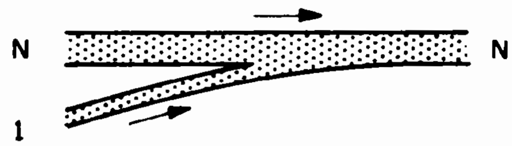
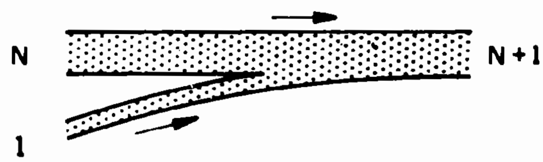
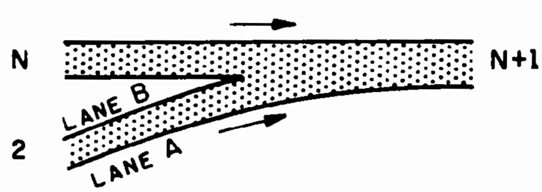
N = NUMBER OF LANES ON THE FREEWAY, BEFORE AND AFTER RAMP JUNCTION.

F_T or F_T' = TRUCK ADJUSTMENT FACTOR FOR PERCENTAGE OF TRUCKS IN FREEWAY VOLUME.

F_{T1} = TRUCK ADJUSTMENT FACTOR FOR PERCENTAGE OF TRUCKS IN LANE 1 VOLUME.

F_{TR} = TRUCK ADJUSTMENT FACTOR FOR PERCENTAGE OF TRUCKS IN RAMP VOLUME.

FIGURE 13.—Service volume relations at entrance and exit terminals.

ENTRANCE RAMP CASE	SERVICE VOLUME OR CAPACITY
 Single-Lane Entrance— Normal Design EN(1)	•Use chart in fig. 17 directly for N = 2, N = 3, or N = 4, utilizing F_{Tl} from fig. 21 and table 4, and F_{TR} from table 11. •Find ramp proper value, table 11. •Smaller of two results governs.
 Single-Lane Entrance— On Exclusive (Added) Lane EN(1 + aux)	•Use chart of fig. 19 utilizing F_{Ta} from table 11. •Find ramp proper value, table 11. •Smaller of two results governs.
 2-Lane Entrance[2] EN(2)	•Lane A—use fig. 19 as for Case EN(1 + aux).[1] •Lane B—use fig. 17 as for Case EN(1). •Add values for Lane A and Lane B. •Find ramp proper value, table 11. •Smaller of two results governs.

[1]Percent trucks in entering volume is assumed to apply equally to Lane A and Lane B; i.e., trucks distributed uniformly in proportion to Lane A and Lane B volumes.

[2]Note that the lane terminology is the reverse of the Manual usage.

FIGURE 14.—Procedure for determining service volumes and capacity of isolated entrance ramps.

junction. The relatively smaller values in table 9, with respect to freeway proper service volumes at lower levels of service, are designated in recognition of the complexity and difficulty of operation at conditions approaching capacity. Thus, at the less stable flows, service levels *D* and *E*, short-term fluctuations in ramp volume are subject to "breakdown" of lane 1 and ramp terminal, with likely serious operational consequences on the freeway. The values selected are indicative of maximum volumes which can reasonably be expected to merge and diverge on modern freeways.

The return to the designated or desired level of service downstream from the ramp junction is assured in design by the "across-the-lanes" check on the freeway. At this point, the freeway volume divided by the number of lanes on the freeway should be equal to or less than the permissible service volume per lane determined for the uninterrupted flow condition (see figure 13). This type of check is *essential* in conjunction with ramp terminal design. Utilizing the SV_m and SV_d values from table 9 in combination with demand volumes on the freeway and ramp, service volumes of isolated single-lane ramp exits and entrances can be evaluated as detailed in figure 13. The v_1 values required to complete the solution of the equations shown can be obtained from relations developed in the *Manual* based on operational research. These relations are incorporated in the design nomographs presented later on.

The *Manual* further includes data and procedures for handling combinations of ramp exits and entrances in sequence, entailing additional variables such as distances between ramps and upstream or downstream ramp volumes. Because these are not complete and generally entail cumbersome calculations, the need for simplified procedures is indicated. As covered under the next heading, this is accomplished largely through organization of specific cases of isolated ramps and ramp sequences, particularly in combination with auxiliary lanes. The additional width and maneuvering space provided by auxiliary lanes is an essential feature in modern design, providing the key to maintenance of a balanced level of service along the freeway.

At-grade ramp terminals are treated as signalized intersections. Appropriate design charts, developed in a previous publication,[1] are presented here with minor modifications. Truck adjustments as for other elements of the freeway pertain to ramps and their terminals. The adjustment factors in table 4 generally apply to evaluation of service volumes and capacities on ramps. Special adaptations of table 4 values, together with other relations from the *Manual*, are incorporated in the design charts and tables which follow.

ANALYSIS TECHNIQUES

Using the basic values and factors described, a complete analysis procedure for design of ramp facilities is presented. The procedure utilizes nomographs and tables especially prepared for design purposes. The *Manual* data were utilized for the most part, but a number of adjustments and extension of methods in the *Manual* were required to produce a fully workable procedure for design. Complicated and sometimes confusing analysis steps in the *Manual* were replaced with more streamlined and simplified procedures, once it was determined that essentially the same solution results could be achieved. A number of gaps in data or in essential procedures were filled by extending and further detailing certain empirical methods in the *Manual* and introducing others. Further use of material from the AASHO Design Policy, coupled with some rationalization, finally resulted in a composite procedure for design of ramps and interchanges.

Classification and Procedural Steps for Ramp Entrances and Exits

The key to the proposed analysis procedure is a basic relationship worked out for an isolated single-lane entrance ramp and an isolated single-lane exit ramp for 2-, 3-, and 4-lane freeway traveled ways in one direction of travel. The rest of the procedure, for other or more complicated and expanded situations, is built upon these basic relations. Step-by-step procedures for analysis of isolated entrance and exit ramps are outlined in figures 14 and 15. Three basic cases are included for entrance ramps and three for exit ramps: (a) single-lane ramps—normal design; (b) single-lane ramps—on exclusive lane; and (c) 2-lane ramps. These are identified for ready reference as EN(1), EN(1 + aux), and EN(2) for ramp entrances, and as EX(1), EX(1 + aux), and EX(2) for ramp exits, respectively. Step-by-step procedures for analysis of successive ramp arrangements are outlined in figure 16. Four basic cases are included: (a) successive entrances, EN–EN; (b) successive exits, EX–EX; (c) entrance followed by an exit, EN–EX; and (d) exit followed by an entrance, EX–EN.

The total of six cases for isolated ramps and the four cases for combinations of successive ramps allows for every possible arrangement to be analyzed. Combinations of more than two successive ramps, within influence of each other, can be handled by analysis of two ramps at a time, with overlaps, along the freeway. The procedural steps in figures 14 through 16 are keyed in to a series of nomographs and tables which permit complete analyses to be carried out as part of the design process.

Application of Nomographs and Design Tables

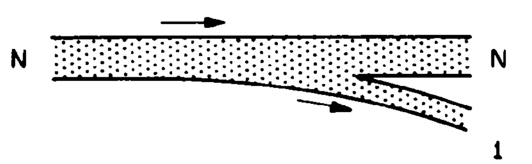
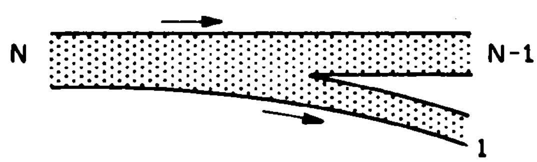
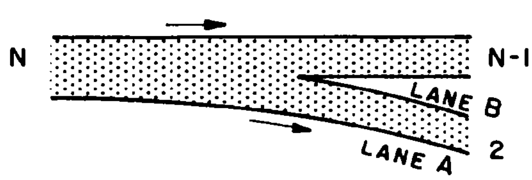
Analyses of ramp problems are accomplished by the use of nomographs and procedures set out in figures 17 through 21 and 25 through 35, and the use of tables 10 through 12. These are designed for rapid solution of problems associated with interchanges without further reference to the *Manual*. Having these relations in graphic form, the nomograph can be used as a testing device to determine the effects of changes in traffic and geometric features. For example: changes in the level of service are immediately seen with increases in either the freeway or ramp traffic; changes in the amount of traffic that can be handled by a ramp are made apparent directly by excluding truck traffic from the ramp, or by directing all trucks from the freeway to the ramp as may be required for stage construction; advantages or disadvantages in changing freeway or ramp grades, as in the case of deciding whether the freeway should be over or under the crossroad, determined largely by the effects of trucks on service volumes, is made obvious on the nomograph; or, comparative operational features of ramp entrances and exits of alternative interchange forms being considered at a given location can be quickly established by inspection of the chart.

Entrance Terminals

The chart in figure 17 provides for the determination of level of service, service volume, and capacity of a single-lane entrance. The ramp is isolated and has a normal or standard terminal for acceleration and merging. The number of lanes N on the freeway before and after the junction are unchanged. This represents the basic condition, referred to as EN(1) in figure 14.

The nomograph provides for a graphic solution of equation (1) in figure 13, coupled with equations or relations in the *Manual* of v_1 in terms of V_f and V_R. The interrelations cover the normal ramp entrance on 4-, 6-, and 8-lane

[1] Jack E. Leisch, "Capacity Analysis Techniques for Design of Signalized Intersections." *Public Roads*, Vol. 34, Nos. 9 and 10, August and October 1967.

EXIT RAMP CASE	SERVICE VOLUME OR CAPACITY
 Single-Lane Exit — Normal Design EX(1)	• Use chart of fig. 18 directly for $N = 2$, $N = 3$, or $N = 4$, utilizing F_{T1} adjustment from fig. 21 and table 4. • Find ramp proper value, table 11. • Smaller of two results governs.
 Single-Lane Exit — On Exclusive Lane EX(1+aux)	• Use chart of fig. 20 utilizing F_{Ta} adjustment for exiting trucks from table 4. • Find ramp proper value, table 11. • Smaller of two results governs.
 2-Lane Exit[3] EX(2)	• Lane A – use fig. 20 as for Case EX(1 + aux).[1] • Lane B – use fig. 18 as for Case EX(1).[2] • Add values for Lane A and Lane B. • Find ramp proper value, table 11. • Smaller of two results governs.

[1]Percent of exiting trucks assumed to apply equally to Lane A and Lane B; i.e. trucks distributed uniformly in proportion to Lane A and Lane B volumes.

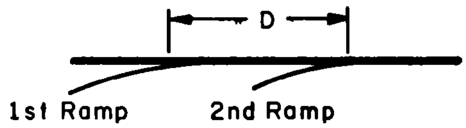
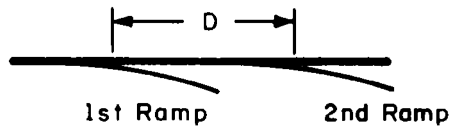
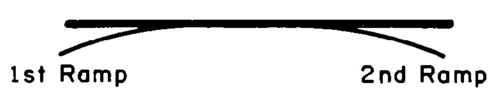
[2]Utilize freeway approach volume to total approach volume less traffic exiting on exclusive Lane A, or $V_f' = V_f - V_a$; also % trucks (T_f') in remaining freeway volume (V_f') is:

$$T_f' = \frac{(V_f)(T_f) - (V_a)(T_a)}{(V_f - V_a)}$$

where T_f and T_a are percentages of trucks in V_f and V_a, respectively.

Note that lane terminology is the reverse of Manual usage.

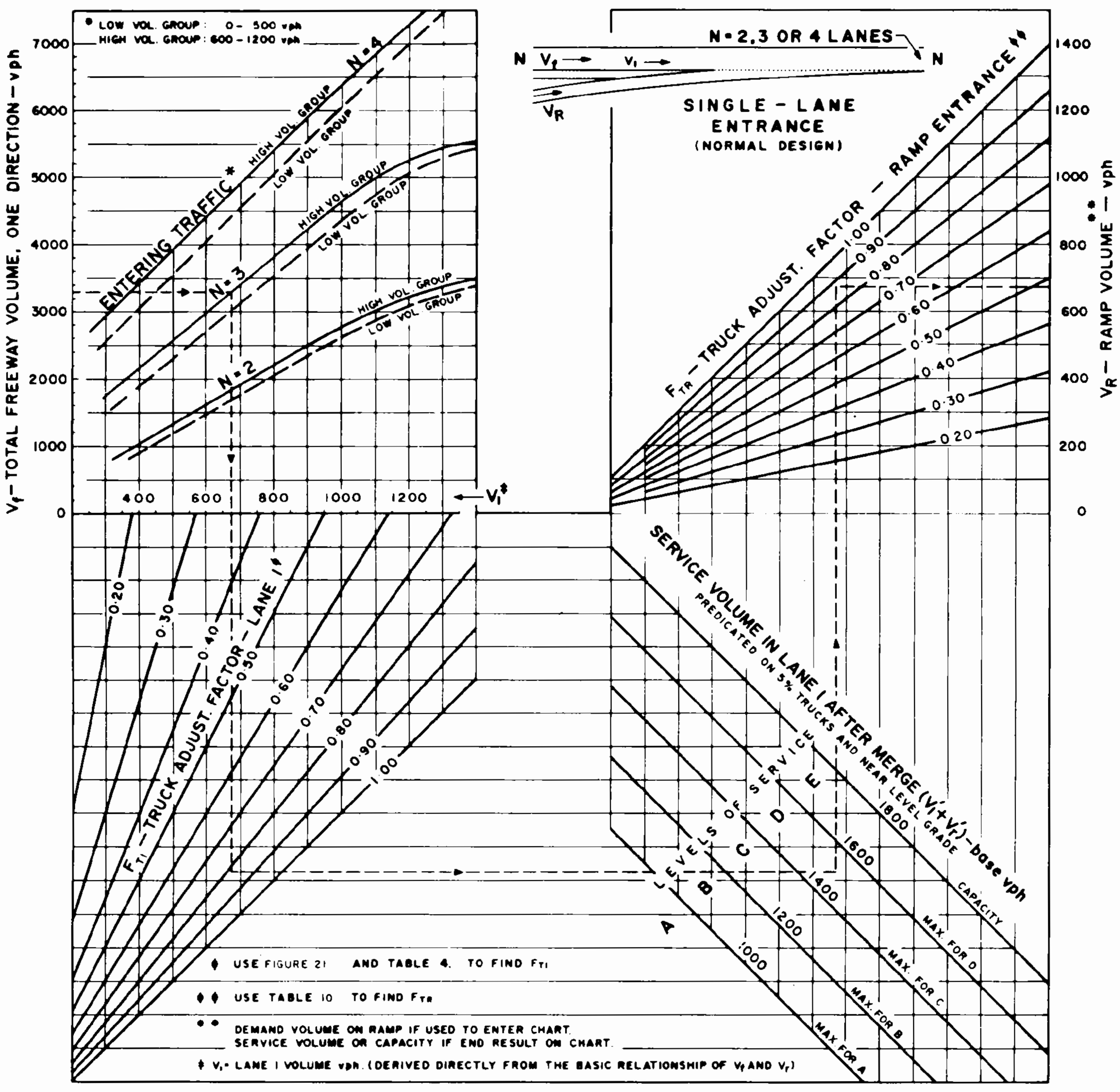
FIGURE 15.—Procedure for determining service volumes and capacity of isolated exit ramps.

ENTRANCE/EXIT RAMP CASE	SERVICE VOLUME OR CAPACITY
 Successive Entrances EN-EN	• <u>1st Ramp</u> - Use appropriate procedure given for isolated entrance ramp, fig. 14. • <u>2nd Ramp</u> - If $D \geq 3000'$ use appropriate procedure given for isolated entrance ramp, fig. 14. If $D \leq 3000'$ use table 12 with instructions for application; except for single-lane entrance with auxiliary lane at 2nd ramp, treat as isolated entrance-case EN (1+aux), fig. 14. *
 Successive Exits EX-EX	• <u>1st Ramp</u> Case EX(1) - Use table 13 with instructions for application. (If $D \geq 4000'$, treat as isolated exit ramp, fig. 15. Case EX(1+aux) - Use procedure given for Case EX(1+aux), fig. 15. Case EX(2) - Use procedure given for Case EX(2), fig. 15 except for Lane A apply table 13 where $D < 4000'$. • <u>2nd Ramp</u> - Use appropriate procedure given for isolated exit ramp, fig. 15.
 Entrance Followed by Exit EN-EX	• Apply WEAVING CHART (fig. 41 and table 15) to find level of service or to determine geometric design requirements. • Check 1st Ramp using appropriate procedure given for isolated entrance ramp, fig. 14. (General check for design purposes only). • Check 2nd Ramp using appropriate procedure given for isolated exit ramp, fig. 15. (General check for design purposes only.)
 Exit Followed by Entrance EX-EN	• <u>1st Ramp</u> - Use appropriate procedure given for isolated ramp, fig. 15. • <u>2nd Ramp</u> - Use appropriate procedure given for isolated entrance ramp, fig. 14.

* For the special case where an <u>auxiliary lane</u> is introduced on the freeway as a <u>continuation of the 1st ramp entrance</u>, and such lane continues through and beyond the 2nd ramp entrance, data are not available for determining the volume remaining in the auxiliary lane. The traffic occupying this lane apparently would be somewhere between the V_u volume and V_e volume in table 12. For design purposes (only!), v_1 at the 2nd ramp is assumed to be equal to V_e for D_u of 500 feet, and equal to 50 percent greater than the tabular value of V_e for D_u of 1000 to 3000 feet; then use figure 17 and, if required, figure 19.

FIGURE 16.—Procedure for determining service volumes and capacity of successive entrance and exit ramps.

freeways, that is, 2-, 3-, and 4-lane traveled ways in one direction; adaptation to solutions in conjunction with 5-lane traveled ways is presented under a following heading. The nomograph in figure 17 allows for rapid solutions for two primary situations: given the freeway approach volume V_f and the ramp entrance volume V_R, to find the level of service at the ramp entrance; and, given the freeway volume and the desired level of service, to find the permissible service volume SV_R or V_R on the ramp entrance. In the first case, with V_f and V_R given, the chart is entered simultaneously at upper left and upper right, and the level of service (and with it, v_1) is found by the intersection of the projected lines in the lower right portion of the chart. In the latter case, with V_f and the level of service given, the



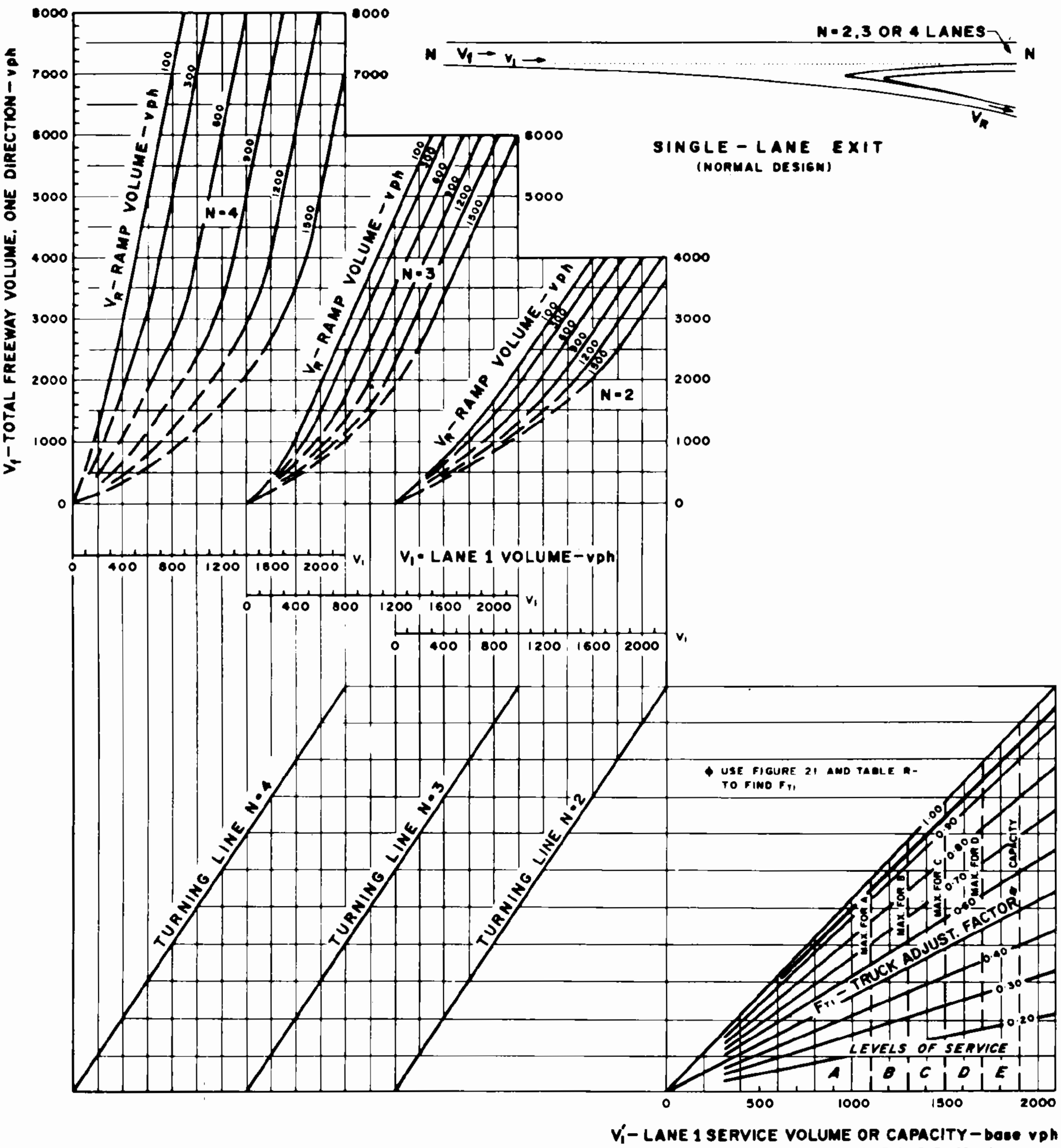
NOTE:

FOR AASHO PROCEDURE ENTER CHART AT UPPER LEFT WITH V_f AND TURN AT LOWER RIGHT V_m' = 1140 OR V_m' = 1420 BASE VPH* FOR RURAL OR URBAN CONDITIONS, RESPECTIVELY; FIND V_R, THE DESIGN CAPACITY OF RAMP ENTRANCE, AT UPPER RIGHT. TURN AT V_m' = 1800 BASE VPH TO FIND POSSIBLE CAPACITY OF RAMP ENTRANCE.

MAXIMUM SERVICE VOLUMES OF 1400, 1600 AND 1800 BASE VPH FOR LEVELS C, D AND E RESPECTIVELY, BASED ON A PHF OF 0.83 TO 0.91 ARE CONSIDERED TO BE REPRESENTATIVE.

*BASED ON AASHO VALUES OF 1200 AND 1500 PCPH; CONVERTED TO BASE VPH (WHICH INCLUDES 5% TRUCKS) FOR CHART APPLICATION: 1200X0.95 = 1140 AND 1500X0.95 = 1420.

FIGURE 17.—Single-lane entrance ramps—nomograph for determination of levels of service, service volumes, and capacity.



NOTE:

FOR ASSHO PROCEDURE ENTER CHART AT UPPER LEFT WITH V_f AND AT LOWER RIGHT WITH $V_1' = 1230$ OR $V_1' = 1520$ BASE VPH* FOR RURAL OR URBAN CONDITIONS RESPECTIVELY; INTERSECT LINES IN UPPER PART OF CHART TO FIND V_R, THE DESIGN CAPACITY OF THE RAMP ENTRANCE, ENTER WITH $V_1' = 1900$ BASE VPH TO FIND POSSIBLE CAPACITY OF RAMP ENTRANCE.

MAXIMUM SERVICE VOLUMES OF 1500, 1700 AND 1900 BASE VPH FOR LEVELS C, D AND E RESPECTIVELY, BASED ON A PHF OF 0.83 TO 0.95 ARE CONSIDERED TO BE REPRESENTATIVE.

*BASED ON AASHO VALUES OF 1300 AND 1600 PCPH; CONVERTED TO BASE VPH (INCLUDING 5% TRUCKS) FOR CHART APPLICATION-- 1300X0.95 = 1230 AND 1600X0.95 = 1520.

FIGURE 18.—Single-lane exit ramps—nomograph for determination of levels of service, service volumes, and capacity.

chart is entered at upper left and followed through continuously to find V_R at upper right. In conjunction with these solutions, appropriate truck adjustment factors are obtained by reference to figure 21 and tables 4 and 10. Table 10 is a rational approach developed to provide adjustments for effect of trucks for a representative ramp approach and merging condition.

The following examples illustrate the various uses of figures 17, 19, and 21, and table 10.

Problem 9

Determine the level of service at which an isolated ramp entrance (normal design) will operate on a 4-lane suburban freeway having a peak-hour factor of 0.85. The

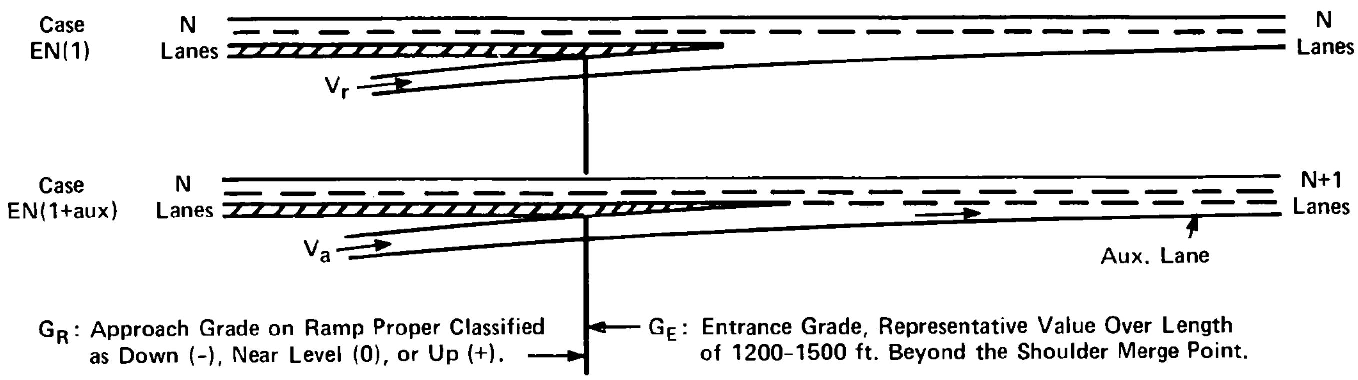


TABLE 10.—*Adjustment factor (F_{TR} or F_{Ta}) for entering trucks*
(To be Used in Conjunction with Figures 14 and 16)

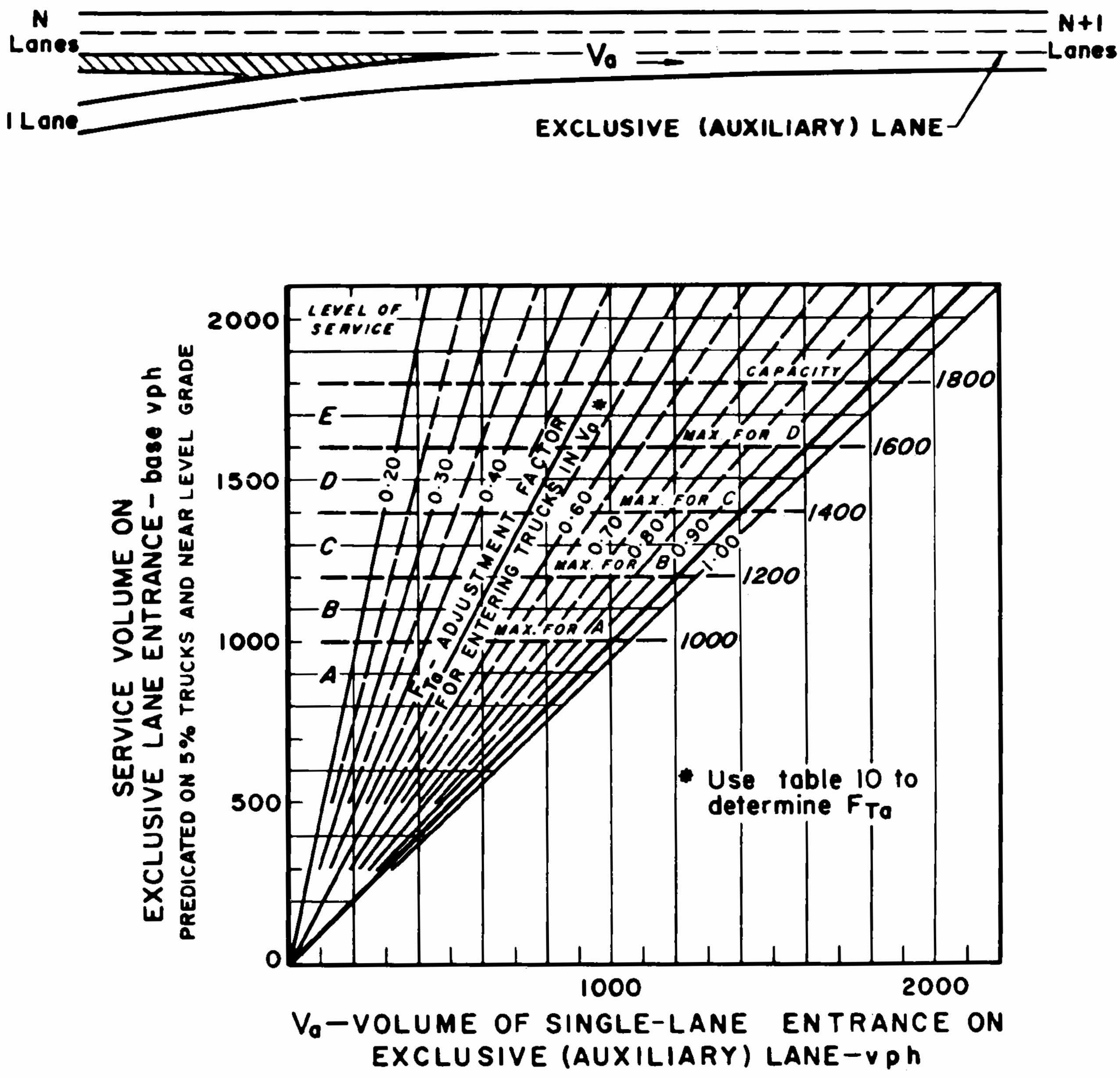
G_E	G_R	Trucks as a Percentage of Entering Traffic																
		1	2	3	4	5	6	8	10	12	14	16	18	20	25	30	40	50
Downgrade or 0–1% Upgrade	–	.99	.98	.97	.96	.95	.94	.93	.91	.89	.88	.86	.85	.83	.80	.77	.72	.67
	0	.99	.98	.97	.96	.95	.94	.93	.91	.89	.88	.86	.85	.83	.80	.77	.72	.67
	+	.97	.94	.92	.89	.87	.85	.81	.77	.74	.70	.68	.65	.63	.57	.53	.45	.40
+2%	–	.99	.98	.97	.96	.95	.94	.93	.91	.89	.88	.86	.85	.83	.80	.77	.72	.67
	0	.98	.96	.94	.93	.91	.89	.86	.83	.81	.78	.76	.74	.71	.67	.62	.56	.50
	+	.97	.94	.92	.89	.87	.85	.81	.77	.74	.70	.68	.65	.63	.57	.53	.45	.40
+3%	–	.98	.96	.94	.93	.91	.89	.86	.83	.81	.78	.76	.74	.71	.67	.62	.56	.50
	0	.97	.94	.92	.89	.87	.85	.81	.77	.74	.70	.68	.65	.63	.57	.53	.45	.40
	+	.95	.91	.87	.83	.80	.77	.71	.67	.62	.59	.56	.53	.50	.44	.40	.33	.29
+4%	–	.97	.94	.92	.89	.87	.85	.81	.77	.74	.70	.68	.65	.63	.57	.53	.45	.40
	0	.96	.93	.89	.86	.83	.81	.76	.71	.68	.64	.61	.58	.56	.50	.45	.38	.33
	+	.94	.89	.85	.81	.77	.74	.68	.63	.58	.54	.51	.48	.45	.40	.36	.29	.25
+5%	–	.96	.93	.89	.86	.83	.81	.76	.71	.68	.64	.61	.58	.56	.50	.45	.38	.33
	0	.95	.91	.87	.83	.80	.77	.71	.67	.63	.59	.56	.53	.50	.44	.40	.33	.29
	+	.93	.88	.83	.78	.74	.70	.64	.59	.54	.51	.47	.44	.42	.36	.32	.26	.22
+6%	–	.95	.91	.87	.83	.80	.77	.71	.67	.63	.59	.56	.53	.50	.44	.40	.33	.29
	0	.94	.89	.85	.81	.77	.74	.68	.63	.58	.54	.51	.48	.45	.40	.36	.29	.25
	+	.93	.86	.81	.76	.71	.68	.61	.56	.51	.47	.44	.41	.38	.33	.29	.24	.20

ramp junction is situated on a 3 percent upgrade which extends back 2,100 feet (measured from the midpoint of the merging maneuver length) where it joins a 1,200-foot long type III sag vertical curve. Geometrics are commensurate with a full freeway. Design speed is 70 mph on the freeway and 35 mph on the ramp. Traffic volumes and other features are as outlined in figure 22.

Solution Identify case EN(1) in figure 14 as that fitting the conditions given. Following the procedure outlined, enter chart of figure 17 at upper left with $V_f = 1{,}500$ and proceed right to $N = 2$ curve for low volume ramp group (ramp volume less than 500), then down to read $v_1 = 610$. Calculate $v_1/V_f = 610/1{,}500 = 0.40$. In figure 21 using $V_f = 1{,}500$, $N = 2$, $T_f = 10$ percent and $v_1/V_f = 0.40$, read $T_1 = 17$ percent. In table 4–A, using 3 percent upgrade ½ mile long (2,100 + ¼ of 1,200-ft vertical curve, as per figure 3), and $T = 17$ percent, find $E_T = 4$. In table 4–C, for $E_T = 4$ and $T = 17$ percent, truck factor $F_{T1} = 0.66$. The adjustment for entering trucks on the ramp is found to be $F_{TR} = 0.83$ in table 10 for a merge on a 3 percent upgrade coupled with a preceding upgrade on the ramp accommodating 4 percent trucks.

Use figure 17 again, picking up at $v_1 = 610$ previously established on the chart. From this point proceed vertically down to $F_{T1} = 0.66$, then horizontally to intersect the levels of service lines. Reenter chart at upper right with $V_R = 350$. Proceed left to $F_{TR} = 0.83$, then down to levels of service bands to intersect the horizontal line previously drawn; read $(v'_1 + V'_R) = 1{,}280$ base vph which falls within level of service *C*.

With every entrance there must be a downstream across-the-lanes check (see figure 13). This may be accomplished one of two ways—by using table 2 or figure 4. The volume on the freeway upon completion of the merge is 1,500 + 350 = 1,850 vph; T_f = (1,500 × 0.10 + 350 × 0.04) ÷ 1,850 = 9 percent; for approximately ½ mile of preceding 3 percent grade (which crests over near the end of the merge), E_T = 5 and F_T = 0.74 in table 4. The resulting volume per lane on the freeway at the check point is (1,850 ÷ 0.74) ÷ 2 = 1,250 pcph; to check on whether



NOTE:

FOR AASHO PROCEDURE ENTER AT LEFT WITH SERVICE VOLUME OF 1140 OR 1420 BASE VPH* FOR RURAL OR URBAN CONDITIONS RESPECTIVELY. FIND Va, THE DESIGN CAPACITY OF THE EXCLUSIVE LANE, ON HORIZONTAL AXIS. ENTER WITH SERVICE VOLUME OF 1800 BASE VPH TO FIND POSSIBLE CAPACITY OF EXCLUSIVE LANE.

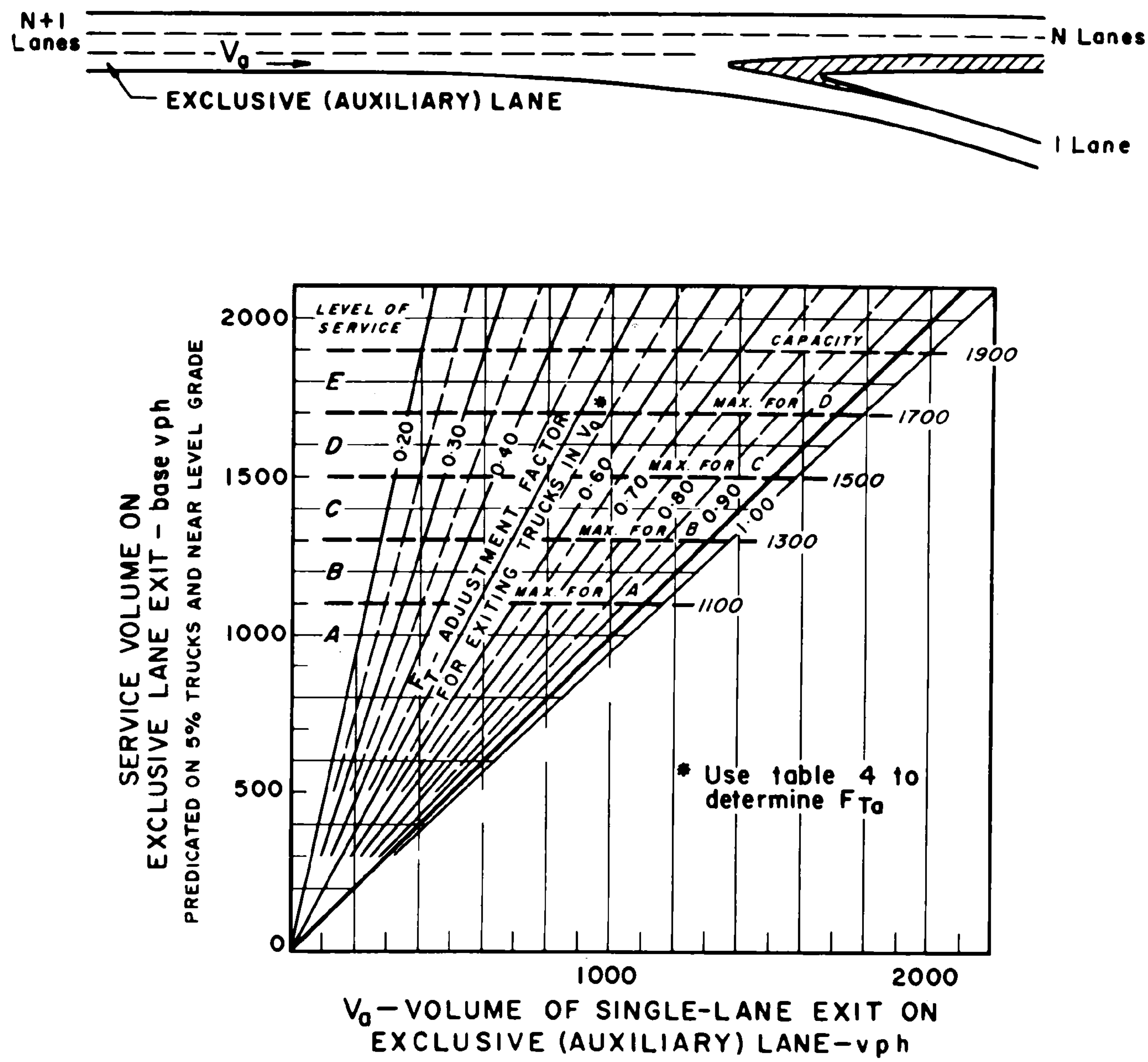
MAXIMUM SERVICE VOLUMES OF 1400, 1600 AND 1800 BASE VPH FOR LEVELS C, D AND E RESPECTIVELY, BASED ON A PHF OF 0.83 TO 0.91 ARE CONSIDERED TO BE REPRESENTATIVE.

*BASED ON AASHO VALUES OF 1200 AND 1500 PCPH; CONVERTED TO BASE VPH (INCLUDING 5% TRUCKS) FOR CHART APPLICATION.

FIGURE 19.—Single-lane entrance on exclusive (auxiliary) lane—nomograph for determination of levels of service, service volumes, and capacity.

the level of service *C* achieved by entering traffic is maintained on the freeway beyond the merge, is done so by reference to table 2. For level of service *C* on a 4-lane freeway with 70-mph average highway speed, $v/c = 0.75 \times$ PHF or the maximum service volume $\overline{sv} = 0.75 \times 0.85 \times 2{,}000 = 1{,}275$ pcph. This is greater than the load per lane of 1,250 pcph, verifying level of service *C* operation.

The alternative across-the-lanes check is made by entering the chart of figure 4 at bottom right with $V_f = 1{,}850$; proceed in reverse order using $F_T = 0.74$, PHF $= 0.85$ and



NOTE:
FOR ASSHO PROCEDURE ENTER AT LEFT WITH SERVICE VOLUME OF 1230 OR 1520 BASE VPH* FOR RURAL OR URBAN CONDITIONS RESPECTIVELY. FIND Va, THE DESIGN CAPACITY OF THE EXCLUSIVE LANE, ON HORIZONTAL AXIS. ENTER WITH SERVICE VOLUME OF 1900 BASE VPH TO FIND POSSIBLE CAPACITY OF EXCLUSIVE LANE.

MAXIMUM SERVICE VOLUMES OF 1500, 1700 AND 1900 BASE VPH FOR LEVELS C, D AND E RESPECTIVELY, BASED ON A PHF OF 0.83 TO 0.95 ARE CONSIDERED TO BE REPRESENTATIVE.

*BASED ON ASSHO VALUES OF 1300 AND 1600 PCPH; CONVERTED TO BASE VPH (INCLUDING 5% TRUCKS) FOR CHART APPLICATION.

FIGURE 20.—Single-lane exit on exclusive (auxiliary) lane—nomograph for determination of levels of service, service volumes, and capacity.

$N = 2$; the intersection with $N = 2$ line falls within level of service C zone.

The procedure in figure 14 further indicates that the merge may be controlled by the service volume attainable on the ramp proper. This check is presented later on in problem 15(a).

Problem 10

If, in problem 9, an auxiliary lane is introduced on the freeway beyond the entrance, what maximum service volume could be handled by the ramp entrance at service level C operation?

Solution In this case the entrance is not directly affected by the traffic on the freeway. As before, the in-

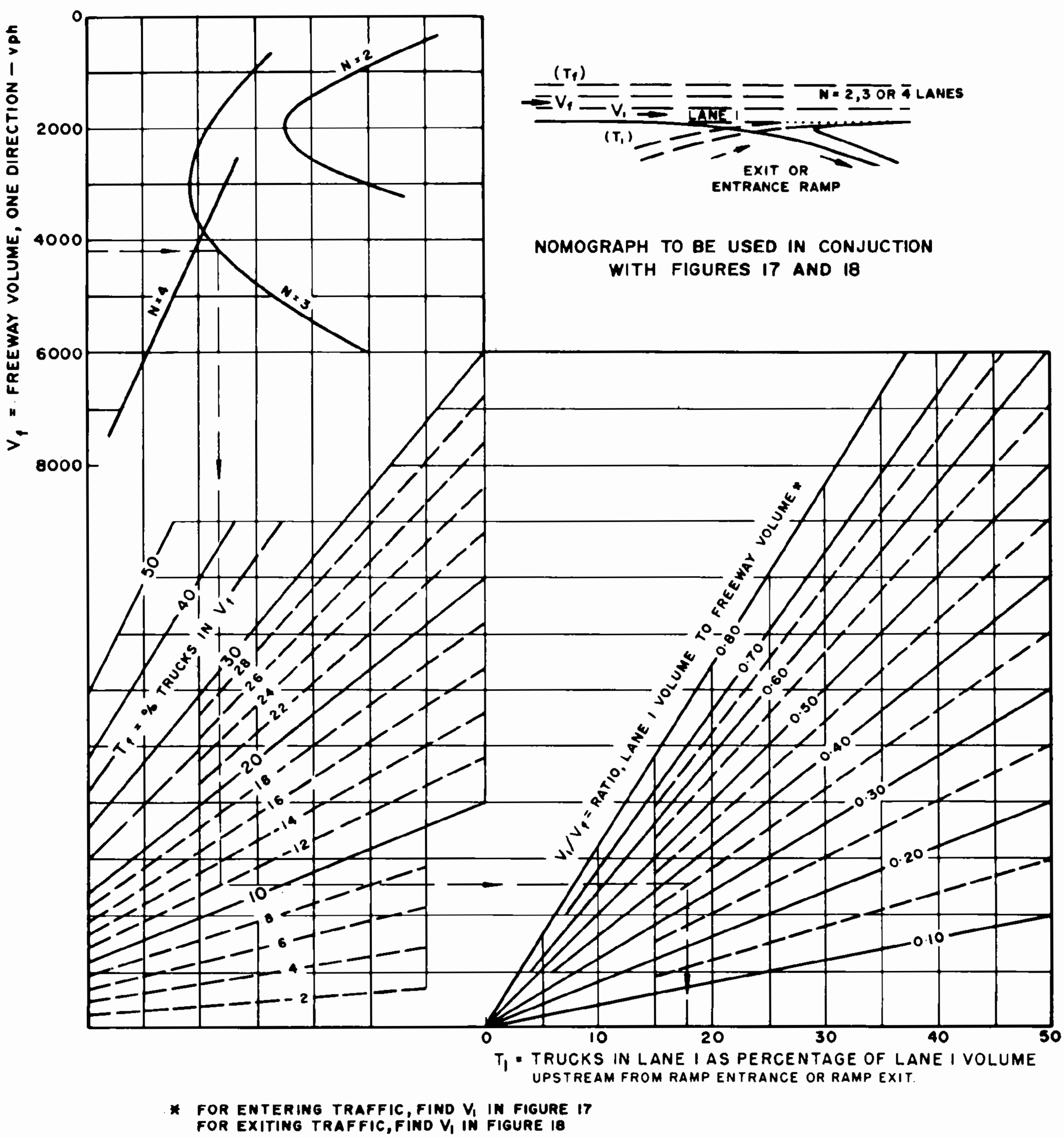


FIGURE 21.—Relationship between overall percentage of trucks in freeway approach volume and percentage of trucks in Lane 1 volume.

fluence of trucks on the ramp is represented by the adjustment factor $F_{Ta} = 0.83$ taken from table 10. Identify ramp entrance case EN(1 + aux) and follow procedure therewith. Using in figure 19 a maximum service volume of 1,400 base vph for service level *C*, and $F_{Ta} = 0.83$, the volume of traffic which can be handled by this design is found to be $V_a = 1{,}220$ vph. The across-the-lanes check downstream, including the auxiliary lane, or 3 lanes in one direction, shows in the chart of figure 4 that operation would be well within level *C* operation for 1,500 + 1,220 = 2,720 vph at this point. The ramp proper check, which also is necessary, is presented later on in problem 15(c).

Problem 11

Determine the design capacity (per AASHO) of an isolated single-lane entrance (normal design) on an 8-lane urban freeway. Other conditions are: design speed of freeway, 60 mph; DHV on 4 lanes upstream of entrance is 4,700; design speed of ramp is 25 mph; truck traffic is nil.

Solution In figure 14 select case EN(1) and follow instructions employing chart in figure 17. Enter chart with $V_f = 4{,}700$, using $N = 4$ (high-volume group ramp assumed), $F_{T1} = 1.00$, $v'_1 + V'_R = 1{,}420$ base vph (see footnote in chart for urban condition, AASHO procedure), and $F_{TR} = 1.00$, find $V_R = 850$ vph, the design capacity of ramp entrance.

The downstream across-the-lanes check is accomplished in figure 4, as follows: proceed through chart from upper left to lower right, using service volume base (design capacity base, according to table 3) of 1,500 pcph per lane, PHF turning line of 1.00, and $F_T = 1.00$; find design capacity of freeway, $V_f = 6{,}000$ vph. The condition is satisfied since the freeway volume downstream, with the ramp entrance operating at design capacity, would be 4,700 + 850 = 5,550 vph.

Problem 12

An existing 6-lane freeway is operating at extremely congested conditions in the easterly direction at a ramp entrance during the morning peak. To break the bottleneck at this point and to the east, the ramp is to be widened to 2 lanes, and 1 lane from the ramp entrance is to be brought on to the freeway as an extra continuous lane, providing 4 lanes away on the freeway. Based on traffic studies of the facility and considering an improvement for a 10-year period, the design hourly volumes and other pertinent conditions are indicated for the improvement in figure 23.

Determine whether the ramp volume shown can be provided for on the improved 2-lane ramp entrance without exceeding level of service *D*, or, if not, what is the maximum ramp volume which can be accommodated at this level of service?

Solution The level of service on the freeway approach is checked before analyzing the ramp entrance. Proceeding through the chart of figure 4 (in reverse order) with $V_f = 3{,}300$ vph, $T_f = 225/3{,}300 = 7$ percent, $E_T = 7$ (per table 4 for $T = 7$ and $G = 4$ percent, ¼- to ½-mile long), PHF = 0.90, and $N = 3$, read level of service *D*. Approach condition, therefore, is satisfactory for the designated service. Identify ramp entrance type as EN(2) in figure 14 and use instructions given to find solution. For lane *A* (outer lane) apply figure 19; using service volume base = 1,600 for level *D* and $F_{Ta} = 0.63$ (per table 10 for $G_E = 4$ percent, $G_R = 5$ percent (or + grade), and $T_R = 10$ percent), find $V_a = 1{,}070$ vph.

For lane *B* (inner lane) apply figure 17. A preliminary run through the chart indicates ramp entrance in lane *B* to be in "low volume group." Using $V_f = 3{,}300$, $N = 3$ and low volume ramp group, $v_1 = 580$ vph; then $T_1 = 115/580 = 20$ percent. $E_{T1} = 5$ (per table 4–A for $G = 4$, ½-mile long, and $T_1 = 20$ percent), and $F_{T1} = 0.56$ (per table 4–C). $F_{TR} = 0.63$ (per table 10 for $G_E = 4$ percent, $G_R = 5$ percent (or + grade), and $T_R = 10$ percent), find $V_a = 1{,}070$ vph.

For lane *B* (inner lane) apply figure 17. A preliminary run through the chart indicates ramp entrance in lane *B* to be in "low volume group." Using $V_f = 3{,}300$, $N = 3$ and low volume ramp group, $v_1 = 580$ vph; then $T_1 = 115/580 = 20$ percent. $E_{T1} = 5$ (per table 4–A for $G = 4$, ½ mile long, and $T_1 = 20$ percent), and $F_{T1} = 0.56$ (per table 4–C). $F_{TR} = 0.63$ (per table 10 for $G_E = 4$ percent, $G_R = 5$ percent (or + grade), and $T_R = 10$ percent). Using in figure 17, $V_f = 3{,}300$, $N = 3$, low volume ramp group, $F_{T1} = 0.56$, $v'_1 + V'_R = 1{,}600$ for level *D*, and $F_{TR} = 0.63$, read $V_R = 400$ vph.

Maximum service volume of 2-lane ramp entrance at level $D = 1{,}070 + 400 = 1{,}470$ vph. Therefore, ramp volume of 1,200 vph can be accommodated without exceed-

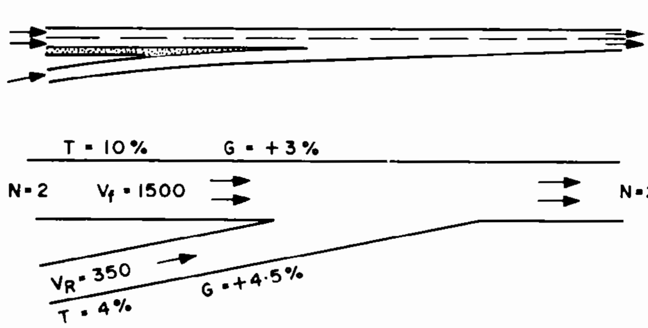


FIGURE 22.—Problem 9 illustrated.

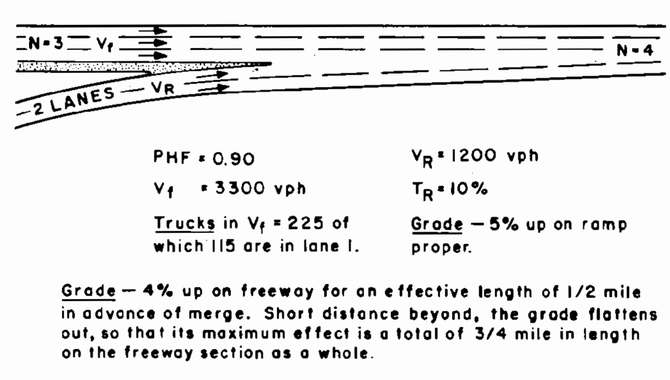


FIGURE 23.—Problem 12 illustrated.

ing level D, providing the ramp proper can handle 1,200 vph and the freeway downstream can receive 1,200 vph without exceeding level D. The ramp proper service volume check is given in the solution of Problem 15(d), showing that there is ample reserve for the demand volume.

The across-the-lanes, downstream check on the freeway is accomplished in figure 4. Proceeding through the chart of figure 4 in reverse order with V_f = 3,300 + 1,200 = 4,500 vph, T_f = (225 + 1,200 × 0.10) ÷ (3,300 + 1,200) = 8 percent, E_T = 8 (based on G = +4 percent, ¾-mile long), PHF = 0.90 and N = 4, find level of service E. Or, entering chart with N = 4 and maximum for level D, and proceeding forward through chart with PHF, T_f, and E_T as before, find V_f = 4,100 vph. Thus, not to exceed level D, entering traffic on ramp would have to be limited to a combined volume of 4,100 vph. Assuming that the pattern of traffic (or the proportion of entering traffic to freeway traffic) remains unaltered, the volumes which can be accommodated at level of service D, therefore, are:

Freeway approach	= 3,300 × 4,100/4,500 = 3,000
Ramp entrance	= 1,200 × 4,100/4,500 = 1,100
Total on freeway at level D downstream	= 4,100 vph

Should this design be accepted, it means that probably in 7 or 8 years, rather than in 10 years, the facility would reach the limit of level of service D. In later years, level of service E would prevail. If the freeway past this critical point is relatively flat or downgrade, operation on the freeway would be restored to level D or better at a volume of 4,500 vph. Figure 4 indicates that with a near level grade traffic would recover to level C. Beyond 8 years or so, however, a bottleneck would again be created—this time caused by the freeway upgrade rather than the ramp entrance. To avoid this, the design may be arranged to carry a 5-lane section after the merge for a distance of a half mile or more until the effect of the grade is sufficiently overcome; then the outer lane can be dropped, resulting in a continuing 4-lane section.

Exit Terminals

The chart in figure 18 is designed to provide solutions for level of service, service volume, and capacity of a single-lane exit. The condition represented is for an isolated ramp, having a normal or standard terminal for deceleration and diverging with the number of lanes N on the freeway remaining the same before and after the exit. This represents the basic exit, referred to as EX(1) in figure 15. The nomograph provides for a graphic solution of equation (2) in figure 13, coupled with equations or relations in the Manual of v_1 in terms of V_f and V_R. Solutions for 4-, 6-, and 8-lane freeways are included. Adaptation to solutions in conjunction with 5-lane traveled ways in one direction is presented under a following heading.

The nomograph in figure 18 facilitates the solution for two basic situations: (a) given the freeway approach volume V_f and the ramp exit volume V_R, to find the level of service at the ramp exit; and (b) given the freeway volume and the desired level of service, to find the maximum service volume SV_R or V_R on the ramp exit. In the first case, the chart is entered at upper left and followed through continuously to find level of service at lower right. In the second case, the chart is entered simultaneously at upper left and lower right and V_R is found by the intersection of projected lines in the upper portion of the graph. In the use of figure 18, appropriate truck adjustment factors are obtained by reference to figure 21 and table 4. In using figure 21, which determines T_1 (percent of truck traffic in freeway lane 1 just in advance of the ramp exit), the value of v_1 is first obtained from figure 18. For the condition where V_R is given and the solution calls for the determination of level of service on the ramp exit, figure 21 is employed to provide T_1 directly. In the case where the level of service is given and the service volume or design capacity of the ramp exit V_R is to be determined, T_1 must first be approximated in order to find a tentative value of v_1. This is done arbitrarily by assuming the percent of trucks in lane 1 volume is equal to one and one-half times the percent of trucks in the total freeway volume, or T_1 = 1.5 T_f. Using the truck adjustment factor F_{T1}, based on the approximate T_1 in figure 18, produces a new v_1; this, then, is used in figure 21 to find a more accurate T_1, which finally is applied in figure 18 to find V_R.

In addition to the standard, single-lane ramp exit, solutions for single-lane exit on exclusive lane, case EX(1 + aux) and for the 2-lane exit, case EX(2), are included in the following example problems employing figures 18, 20, and 21.

Problem 13

Determine the level of service at which an isolated single-lane ramp exit of standard design would operate on a 6-lane urban freeway in a large city, having the following characteristics: DHV on freeway approach—4,100 vph including 6 percent trucks; DHV on ramp—700 vph including 11 percent trucks; approach grade on freeway—1.5 percent down, and predominant grade on ramp—3.0 percent up; and design speed on freeway—70 mph, on ramp—30 mph. Also, find the maximum exit volume which the ramp can accommodate at level C operation while V_f of 4,100 vph is retained; and at level E operation (capacity) while V_f of 4,100 vph is retained, and when V_f approaches capacity volume.

Solution Identify ramp exit case EX(1) and follow instructions indicated in figure 15. Using in figure 18, V_f = 4,100 vph, V_R = 700 vph and N = 3, find v_1 = 1,350. Applying in figure 21, V_f = 4,100 vph, N = 3, T_f = 6 percent and v_1/V_f = 1,350/4,100 = 0.33, find T_1 = 8 percent; and in table 4–C, for E_T = 2, establish F_{T1} = 0.93. Proceeding forward through the chart of figure 18, using V_f = 4,100 vph, V_R=700 vph, N = 3 and F_{T1} = 0.93, locate at lower right level of service C (v'_1 = 1,420, whereas 1,500 is maximum permissible). The ramp exit therefore would operate at service level C.

To find the *maximum* service volume on the ramp exit for service level C operation, it is necessary to estimate T_1 (since v_1 cannot be found directly). As previously suggested, T_1 = 1.5 T_f = 1.5 × 6 = 9 percent; tentative F_{T1} = 0.92 from table 4–C. Proceeding through the chart in figure 18 in reverse order, with maximum service volume = 1,500, F_{T1} = 0.92 and N = 3, find v_1 = 1,430. Using in figure 21, v_1/V_f = 1,430/4,100 = 0.35 along with V_f = 4,100, N = 3 and T_f = 6 percent, find T_1 = 8 percent. Corresponding F_{T1} = 0.93 from table 4–C. Enter again the chart of figure 18 with a service volume base of 1,500 vph (maximum for level *C*) and, proceeding in reverse order with F_{T1} = 0.93 and N = 3, project vertically across the V_R family of curves; reenter chart at left with V_f = 4,100 vph intersect vertical line previously set, and read V_R = 850 vph. This is the maximum ramp volume which can be handled without exceeding level of service *C* operation. To find the capacity of the ramp exit with V_f remaining at 4,100 vph, a new value of T_1 must be obtained. Using the procedure as before, T_1 again is 8 percent and F_{T1} = 0.93. Utilizing in chart 18 a capacity service volume of 1,900 base vph, F_{T1} = 0.93, N = 3, and V_f = 4,100, ramp exit capacity V_R is found to be upwards of 1,500 vph. This is indicative of a situation where if the freeway downstream (beyond the ramp) were to become congested or blocked, the ramp would serve to divert traffic.

Should the freeway itself become fully loaded, according to figure 4, it would yield a capacity volume of 5,000 vph in one direction. In figure 21 and table 4, T_1 = 10 percent and F_{T1} = 0.91. Using these values in figure 18, the capacity of the ramp exit is found to be V_R = 1,100 vph. It is interesting to note that less traffic can be discharged by the ramp when the freeway upstream is operating at capacity than when it is operating at level of service *C*.

The check for maximum service volume at levels *C* and *E* on the ramp proper is necessary to assure that the above values can be achieved. This is done in problem 16(a).

Problem 14

A section of freeway including a 2-lane exit in an urban area is under design. The geometric features and design hourly volumes are indicated in figure 24. Determine, on the basis of AASHO procedure, whether the 2-lane exit can accommodate the DHV's shown without exceeding design capacity. Check the design capacity at the critical points past the exit, at stations 325 and 366.

Solution To determine whether the volume of 1,500 vph can be accommodated, reference is made to the analysis instructions in figure 15 for the ramp exit case EX(2). Lane *A* and lane *B* are analyzed separately and the results for each are added to find the total capacity for the ramp exit as follows:

Lane *A:* In table 4, using length of 2 percent upgrade of 5,300 feet (sta. 300–sta. 247) or 1 mile and trucks in the exclusive lane of 14 percent, find E_{TR} = 4 and F_{TR} = 0.70. Using in chart of figure 20, service volume on exclusive exit lane of 1,520 base vph (AASHO design capacity base for urban conditions, per footnote on chart), and F_{TR} = 0.70, find V_a = 1,120 vph (design capacity of lane *A*).

Lane *B:* In accordance with note 2 in figure 15, adjusted truck percentage T'_f on freeway approach (with lane *A* traffic removed) is determined as follows:

$V'_f = V_f - V_a$ = 4,700 − 1,120 = 3,580 vph
$T'_f = (V_fT_f - V_aT_a) \div V'_f$
= (4,700 × 0.08 − 1,120 × 0.14) ÷ 3,580
= 6 percent.

The percentage of trucks in lane 1 volume is estimated by the expression T'_1 = 1.5 T'_f (relationship previously set up where the service volume for a given level of service or

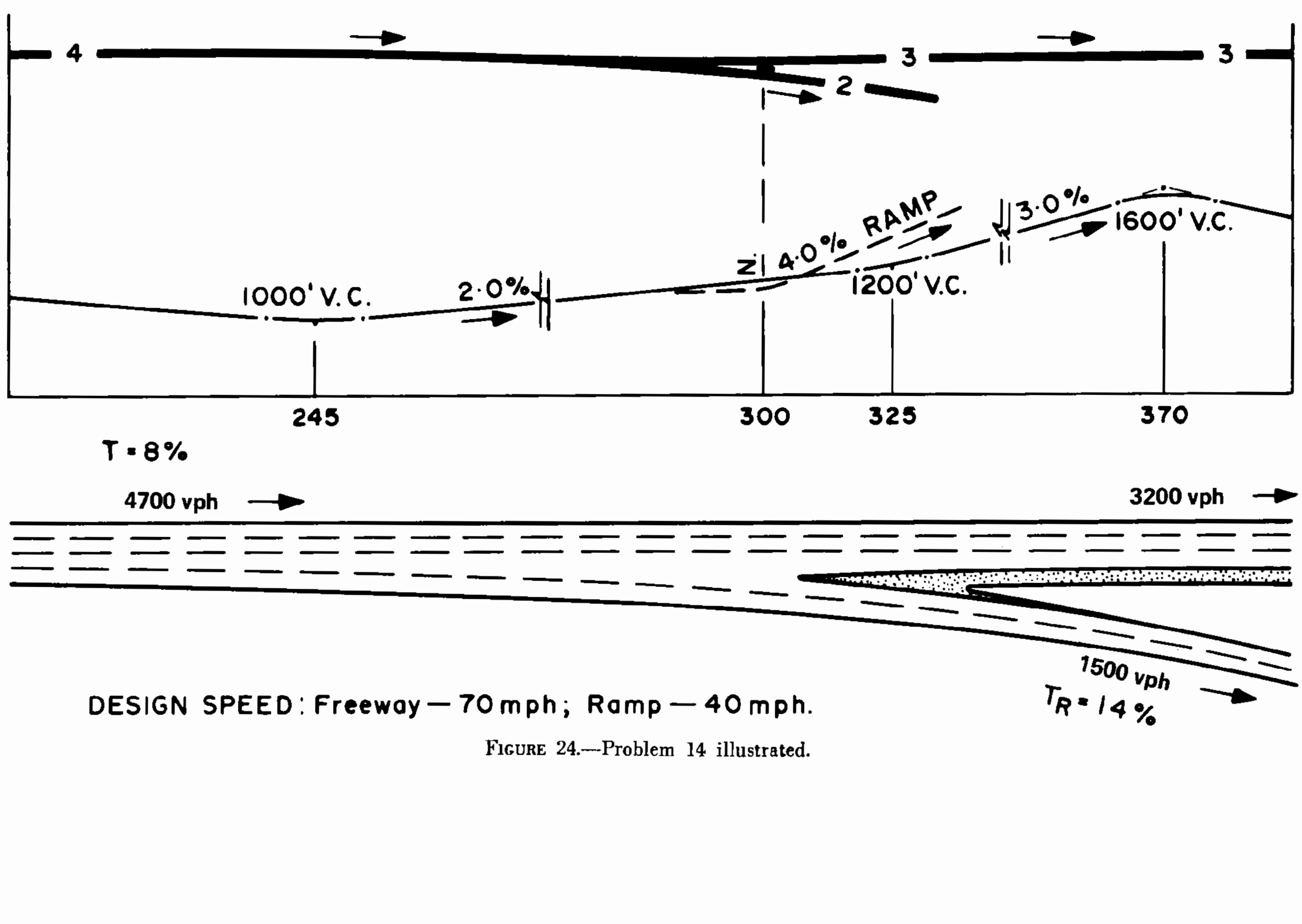


FIGURE 24.—Problem 14 illustrated.

the design capacity of ramp exit is to be found); $T'_1 = 1.5 \times 6 = 9$ percent; tentative $E_{T1} = 5$ and $F_{T1} = 0.74$ (from table 4 for $T = 9$ percent and 1-mile long 2 percent upgrade). Proceeding through the chart of figure 18 in reverse order, with maximum service volume of 1,520 (AASHO base for urban conditions), $F_{T1} = 0.74$ and $N = 3$, find $V_1 = 1{,}170$. Using in figure 21, $V_1/V_f = 1{,}170/3{,}580 = 0.33$ along with $V_f = 3{,}580$, $N = 3$ and $T_f = 6$ percent, find $T_1 = 8$ percent. Corresponding $F_{T1} = 0.76$. Enter again the chart of figure 18 with a service volume base of 1,520 vph and, proceeding in reverse order with $F_{T1} = 0.76$ and $N = 3$, project vertically across the V_R family of curves. Reenter chart at left with $V_f = 3{,}580$ vph, intersect vertical line previously set, and read $V_R = 600$ vph (design capacity of lane *B*).

Lane *A* + Lane *B*: Design capacity of ramp exit = 1,120 + 600 = 1,720 vph. This is more than the demand volume of 1,500 vph. Therefore, the design is adequate, providing the ramp proper is capable of handling 1,500 vph. This is checked in problem 16(b), where it is indicated that the ramp proper can handle 2,070 vph at design capacity. The design capacity of the ramp exit, therefore = 1,720 vph.

Design capacity (unlike possible capacity) is not determined by a situation at any one point but over an appropriate length of grade. However, across-the-lane checks at what may be critical points are made to give some idea of operational quality and to allow for judgment of overall adequacy of the facility, as well as to assure that possible capacity is not exceeded. Accordingly, design capacity (service volume) is determined for this purpose as follows:

At sta. 325 (end of effective 2 percent upgrade)— $E_T = 6$ for 1.5 miles of 2 percent grade and $T = (4{,}700 \times 0.08 - 1{,}500 \times 0.14) \div 3{,}200 = 5$ percent; $F_T = 0.80$.

At sta. 366 (approximate end of effective 3 percent upgrade)—$E_T = 9$, based on $T = 5$ percent, and an equivalent grade of 2.9 percent and 11,900 feet (2¼ miles) long from figure 2 (per procedure in problem 4); $F_T = 0.71$.

Design capacity, check points using figure 4:

At sta. 325 is 3,600 vph
At sta. 366 is 3,200 vph

Sufficient design capacity is available at these control points since the DHV is 3,200 vph.

Ramps Proper

The base service volumes suggested for design of ramps proper in table 8 have been expanded to include various percentages of trucks over a range of profile grades. The expanded set of volumes is given in table 11, predicated on passenger car equivalents per truck (E_T) of 2, 3, and 4 for upgrades (percent) of 0–2, 3–4, and 5 and over, respectively. (The E_T values are those used for ramps proper in the AASHO Design Policy.)

The table is set up to give direct service volumes on single-lane ramps for rural condition (service level *B*) and for urban condition (service level *C*). Values for 2-lane ramps and capacity operations are provided through the use of conversion factors. Although service volumes and capacities of ramps are generally governed by either the exit or the entrance terminal, the ramp itself should be checked just the same. Occasionally, a low design speed or a high percentage of trucks on an upgrade may produce a situation where the ramp proper controls the capacity.

The following examples, relating to problems 9 through 14, illustrate the use of table 11.

Problem 15

Determine the maximum service volume of the ramp proper for the conditions noted.

a. Single-lane ramp associated with the entrance in problem 9—design speed = 35 mph, $G_R = 4.5$ percent up, $T_R = 4$ percent, and operation at level of service *C*.
b. Single-lane ramp associated with the entrance on exclusive lane in problem 10—other characteristics as in (a).
c. Single-lane ramp associated with the entrance in problem 11—design speed = 25 mph, truck traffic nil, and operation at design capacity in urban area.
d. Two-lane ramp associated with the entrance in problem 12—design speed = 30 mph, $G_R = 5$ pecent up, $T_R = 10$ percent, and operation at level of service *D*.

Solution Using the characteristics noted above in table 11, the following volumes can be accommodated at the indicated levels of service:

a. 1,275 vph at service level *C*;
b. 1,275 vph at service level *C*;
c. 1,250 vph at design capacity (urban);
d. 1,080 × 1.9 = 2,050 vph at service level *C*, and 2,050 × 1.25 = 2,560 vph at service level *E* (capacity); level of service *D* is not detailed in the table, but the service volume for it may be considered between 2,050 and about 2,300 vph (about halfway between *C* and *E*).

Problem 16

Determine the maximum service volume of the ramp proper for the conditions noted.

a. Single-lane ramp associated with the exit in problem 13—design speed = 30 mph, $G_R = 3$ percent up, $T_R = 11$ percent, and operation at level of service *C* and at capacity.
b. Two-lane ramp associated with the exit in problem 14—design speed 40 mph, $G_R = 4$ percent up, $T_R = 14$ percent, and operation at design capacity (urban).

Solution Using the characteristics noted above in table 11, the following volumes can be accommodated at the indicated levels of service:

a. 1,150 vph at service level *C*; 1,150 × 1.25 = 1,440 vph at service level *E* (capacity).
b. 1,090 × 1.9 = 2,070 vph at design capacity (urban).

TABLE 11.—*Service volumes for design of ramps proper*

DESIGN CONDITION	T % Trucks During Peak Hour	RAMP SERVICE VOLUME (vph) Single-Lane Operation											
		Design Speed ≦20 mph			Design Speed 25 mph			Design Speed 30–40 mph			Design Speed ≧ 50 mph		
		Rate of Upgrade–%			Rate of Upgrade–%			Rate of Upgrade–%			Rate of Upgrade–%		
		0–2	3–4	≧ 5	0–2	3–4	≧ 5	0–2	3–4	≧ 5	0–2	3–4	≧ 5
	0	800	800	800	1000	1000	1000	1100	1100	1100	1220	1200	1200
	2	780	770	750	980	960	940	1080	1060	1040	1180	1150	1130
	4	770	740	710	960	920	890	1060	1020	980	1150	1110	1070
Rural	6	750	710	680	940	890	850	1040	980	930	1130	1070	1020
	8	740	690	650	920	860	810	1020	950	890	1110	1040	970
Service	10	720	670	610	910	830	770	1000	920	850	1090	1000	920
Level	12	710	650	590	890	800	730	980	890	810	1070	970	880
B	14	700	630	560	880	780	700	900	860	770	1050	940	840
	16	690	610	540	860	760	670	960	840	740	1040	910	810
	18	680	590	520	850	740	650	930	810	710	1020	880	780
	20	670	570	500	830	720	620	920	780	690	1000	860	750
	25	640	530	460	800	670	570	880	730	630	960	800	690
	30	610	500	420	770	620	530	850	690	580	920	750	630
	0	1000	1000	1000	1250	1250	1250	1400	1400	1400	1500	1500	1500
	2	980	960	940	1220	1200	1180	1370	1340	1320	1470	1440	1410
	4	960	920	890	1200	1160	1120	1340	1300	1250	1440	1390	1340
Urban	6	940	890	850	1180	1120	1060	1320	1250	1190	1410	1340	1270
	8	920	860	810	1160	1080	1010	1300	1210	1130	1390	1290	1210
Service	10	910	830	770	1140	1040	960	1270	1170	1080	1360	1250	1150
Level	12	890	800	730	1120	1010	920	1250	1130	1030	1340	1210	1100
C	14	880	780	700	1100	980	880	1230	1090	990	1310	1170	1050
	16	860	760	670	1080	950	840	1210	1060	950	1290	1140	1010
	18	850	740	650	1060	920	810	1190	1030	910	1270	1100	970
	20	830	720	620	1040	890	780	1170	1000	870	1250	1070	940
	25	800	670	570	1000	830	720	1120	940	800	1200	1000	860
	30	770	620	530	960	780	660	1080	880	740	1150	940	790

Adapted from *A Policy on Geometric Design of Rural Highways*, AASHO, 1965—table II–16.

Notes: 1. For *2-lane* ramps multiply tabular values by the following factors: 1.7 for ≦20 mph; 1.8 for 25 mph; 1.9 for 30–40 mph; 2.0 for ≧50 mph.

2. To approximate *capacity* (service level *E*) multiply above values for urban conditions by 1.25.

At-grade Ramp Terminals

Ramp terminals on the crossroad in conjunction with diamond and parclo type interchanges constitute level or at-grade crossing situations for turning traffic. The design and operation of such terminals follow the basic rules for at-grade intersections. In most instances, particularly in urban areas, these points of intersection require signalization, to which the following nomographs and procedures (based largely on the *Manual* data) apply directly. Where the crossroad and ramp volumes are so light that apparently no signalization is needed, the procedure is still the same; that is, a signal is assumed as a device (only) for purposes of analysis and design. This provides the means for determining the appropriate number and arrangement of lanes.

The intersection involving the ramp terminal constitutes one- or two-way approaches on the crossroad and a T- or Y-junction on the ramp. Nomographs for determination of service volumes or number of lanes required to sustain a given level of service on crossroads in urban and in rural areas are presented in figures 25 through 27. In urban areas, the service volumes are affected by the PHF, which for intersection conditions is based on a 15-minute period

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	0·35	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	1·15	1·15	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 indicated in Figures 29-A and 29-B.

EXAMPLE

Given

W_A = 25′ CBD · area
T = 8% MP = 500,000
R = 25% G = 36 sec.,
L = 10% G/C = 0.60

Solution
Design capacity
C_D = 1040 vph

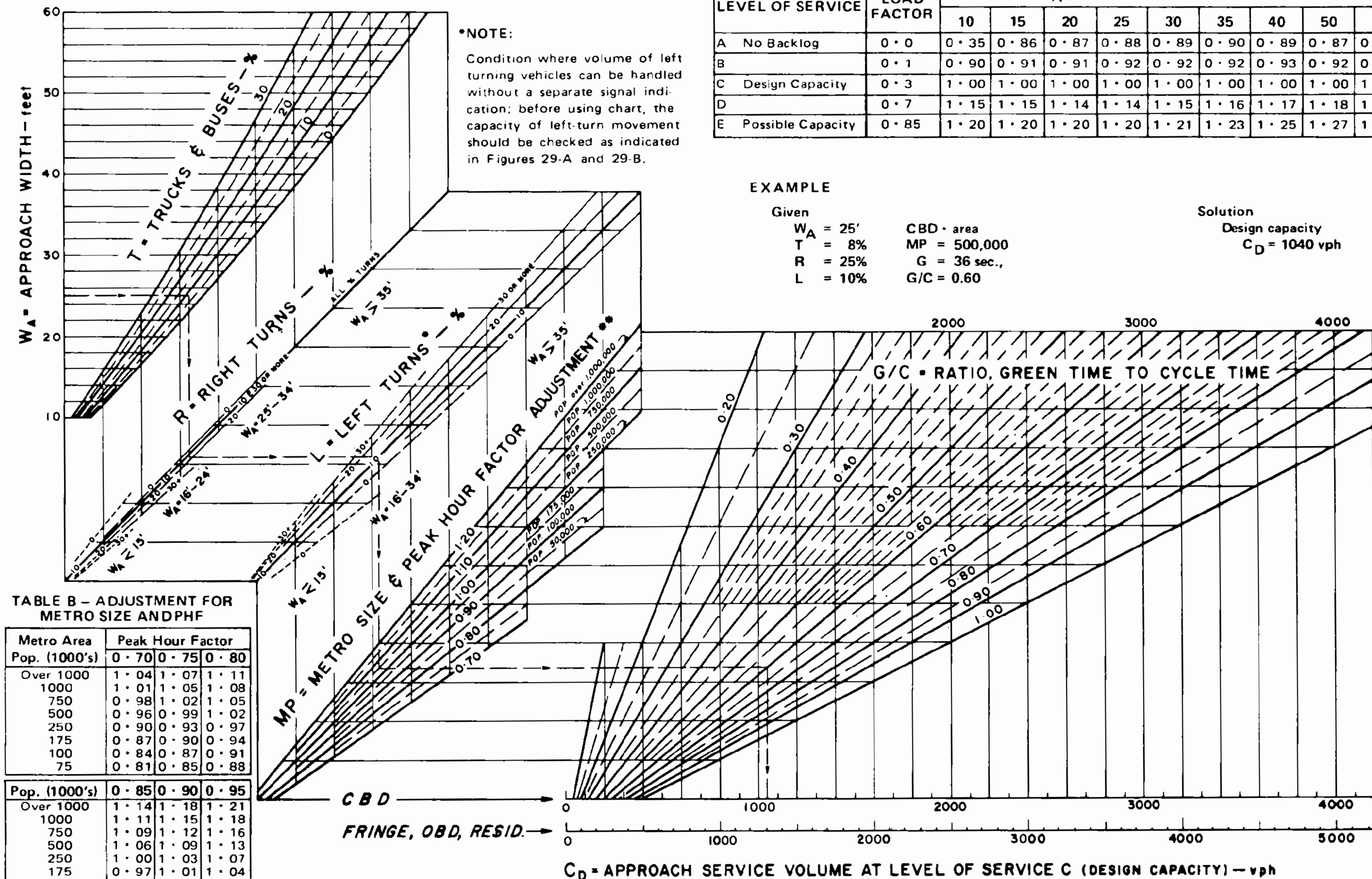


TABLE B – ADJUSTMENT FOR METRO SIZE AND PHF

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.

FIGURE 25.—Service volumes for design of signalized intersections and ramp terminals—two-way facilities, urban area.

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	0·95	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
CBD
G/C = 33/60 = 0·55
B = No bus stops

Solution

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

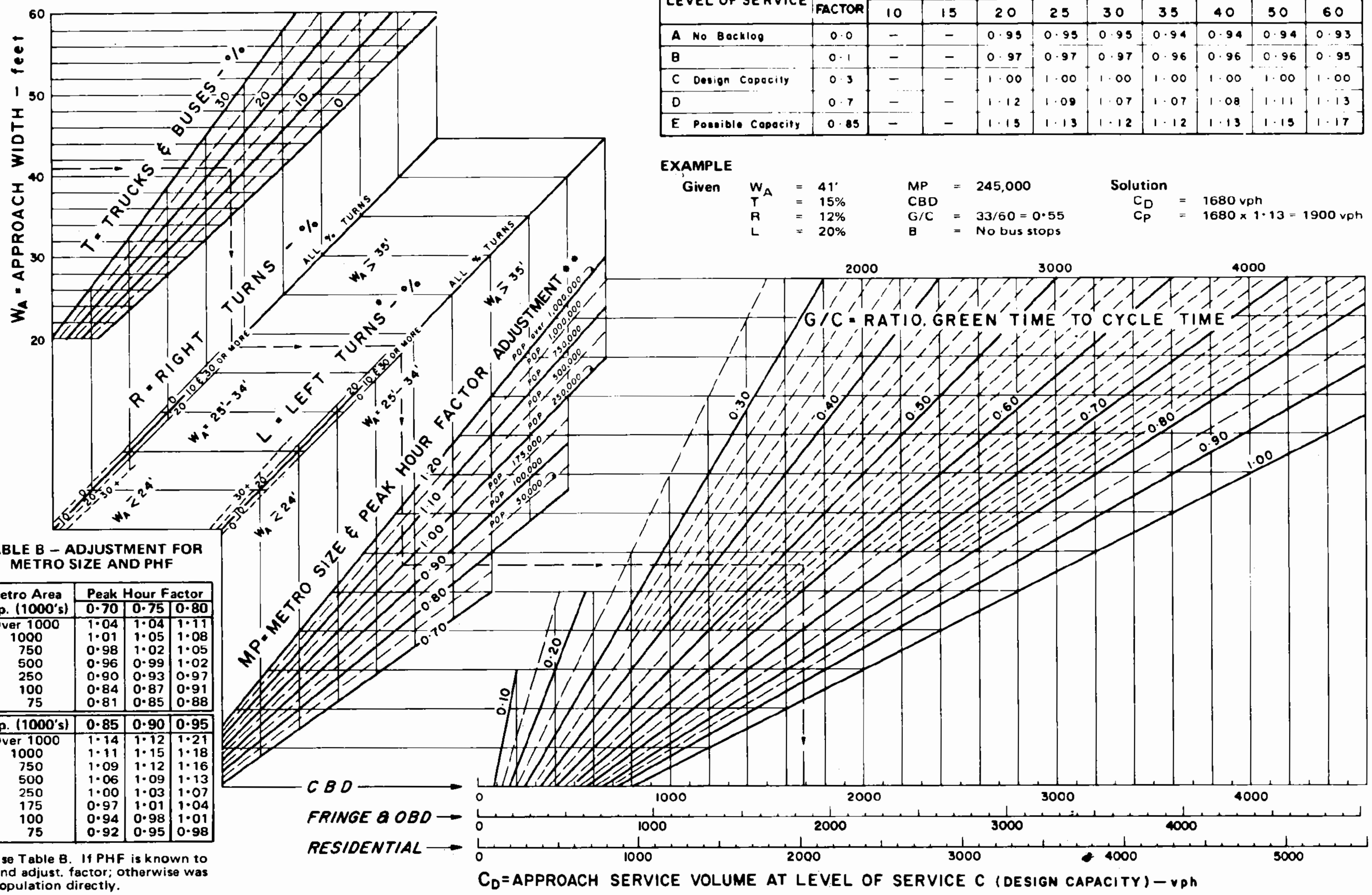


TABLE B – ADJUSTMENT FOR METRO SIZE AND PHF

Metro Area Pop. (1000's)	Peak Hour Factor 0·70	0·75	0·80
Over 1000	1·04	1·04	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
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	1·12	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 was Population directly.

FIGURE 26.—Service volumes for design of signalized intersections and ramp terminals—one-way facilities, urban area.

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	0·92	0·95	0·96	0·96	0·97	0·97	0·97	0·97	0·96
B Design Capacity	0·1	1·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00	0·00
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

***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 note on Chart B in Figure 29.

EXAMPLE

Given

W_A = 22′ L = 7%

T = 15% G/C = 0·55

R = 12% PHF = Normal condition

RURAL AREA, NORMAL CONDITIONS

Solution

C_D = 740 vph

C_P = 740 x 1.29 = 950 vph

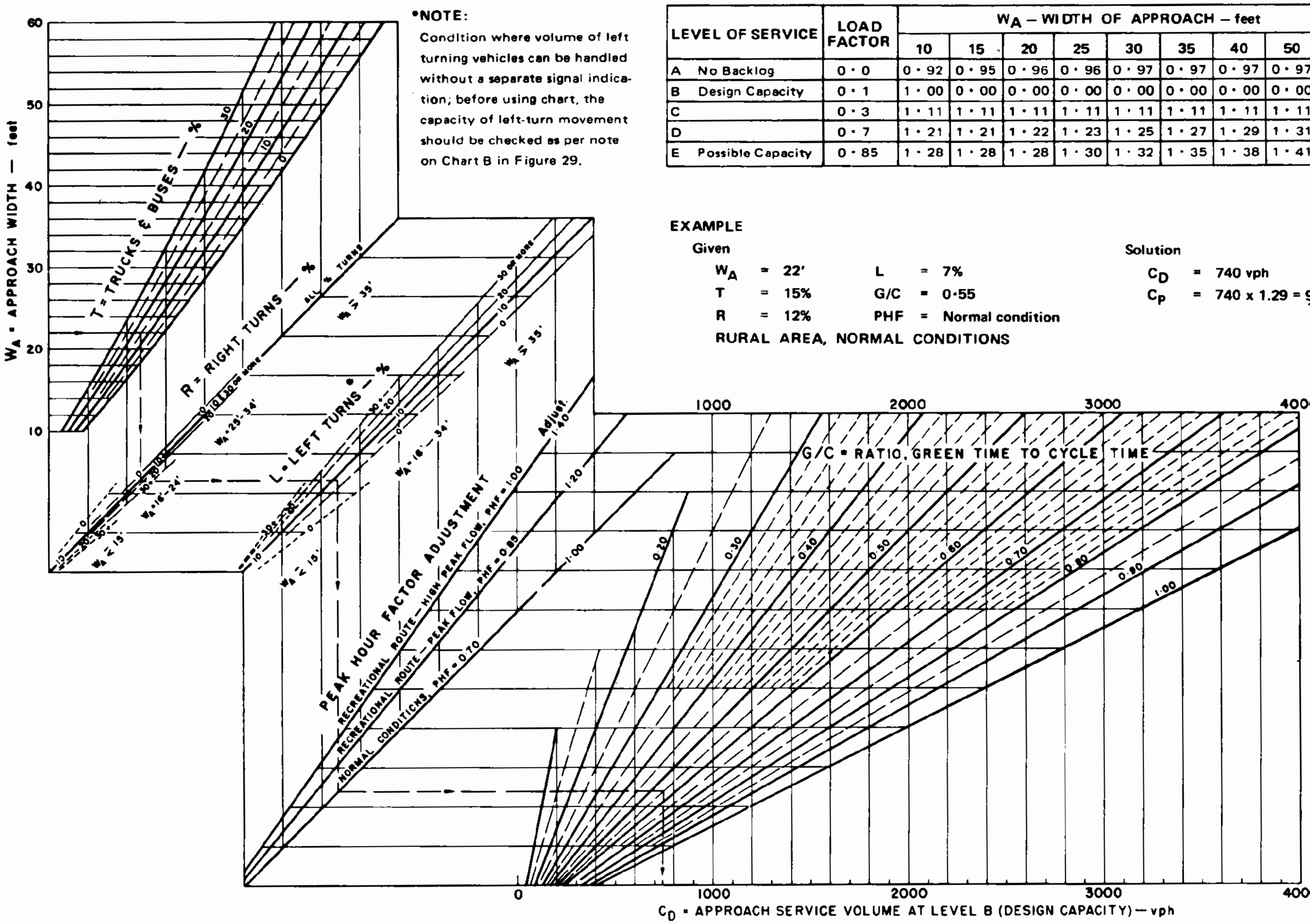


FIGURE 27.—Service volumes for design of signalized intersections and ramp terminals—rural area.

(rather than on a 5-minute period as for uninterrupted flow conditions); or the analysis may be based directly on the metropolitan area population, provided for in the nomographs. Analyses of turning lanes in conjunction with such approaches, with and without separate signal indications, including double turning lanes, are covered by nomographs in figures 29 through 31. Supplementary information is given in figures 32 and 33. Analyses pertaining to the ramp junction itself are accomplished by the use of nomographs and tables outlined in figures 28 through 33. The various types of at-grade *entrance* terminals are categorized and indexed in figures 34 and 35. Four varieties of terminals for single-lane ramps, 1–A through 1–D, and six varieties for 2-lane ramps, 2–A through 2–F, are described including steps for analysis.

At-grade *exit* terminals are not similarly categorized since these facilities normally comprise a combination of a right exit and a separate left exit from the crossroad. The analysis of the two opposing approaches on the crossroad, with the service volume or capacity of the right turn on one approach plus that of the left turn on the other approach, determines the service volume or capacity of the exit terminal as a whole. The charts and procedures presented here are adaptations from the previous work reported in *Public Roads*.[2] Reference may be made to this publication for the more fundamental aspects.

[2] Jack E. Leisch, "Capacity Analysis Techniques for Design of Signalized Intersections." *Public Roads*, Vol. 34, Nos. 9 and 10, August and October 1967.

The following examples illustrate the analysis techniques for at-grade ramp terminals. Of significance is the allocation of the signal time to the several approaches to the intersection. Signal timing is generally expressed in terms of G/C ratios, green time for any one signal phase in seconds divided by the cycle time in seconds. At-grade ramp terminals usually fit into 2- and 3-phase control. Occasionally, an advance green indication may be utilized for a left turn movement to avoid a full 3-phase control. For any one cycle, the sum of the G/C values plus A/C values (amber periods divided by cycle time) equals 1.00. The total of the two amber periods for a 2-phase signal for design purposes usually is taken to be 0.10 (two 3-second intervals within a cycle of about 60 seconds). For a 3-phase signal, it may be in the range of 0.10 to 0.12 (9 to 11 seconds within 80- to 100-second cycle). Thus, the total G/C available for moving traffic each cycle is assumed to be 0.90 for 2-phase control and 0.88 to 0.90 for 3-phase control.

Problem 17

A 2-lane crossroad over a freeway accommodating a diamond interchange is widened to a 4-lane facility with a narrow median through the interchange. Peak-hour traffic on the crossroad is of the order of 200 vph in each direction. Traffic on the single-lane entrance ramp of the design shown for case 1–A in figure 34 is 120 vph. Traffic movements constitute about 15 percent trucks during peak hours. Determine whether the single-lane ramp terminal is adequate.

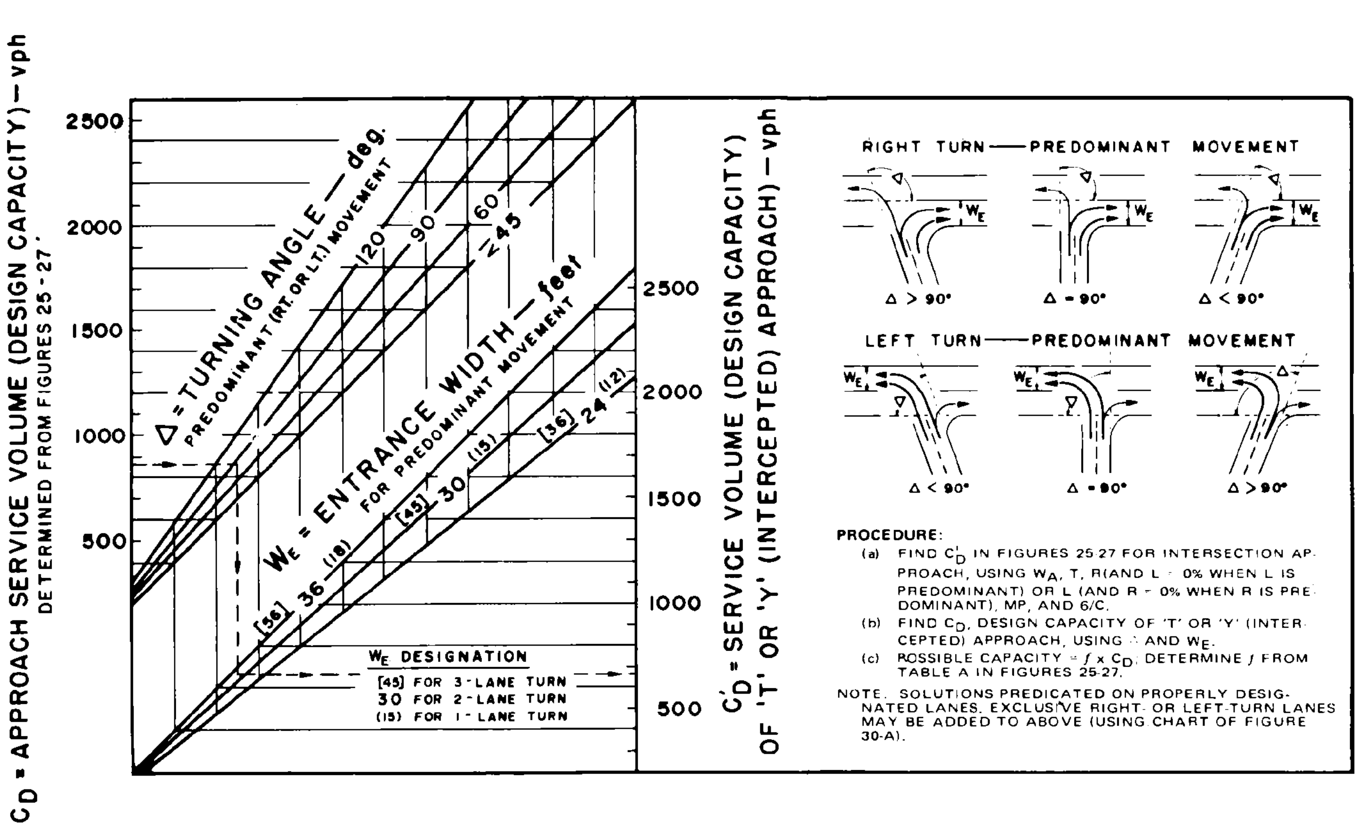
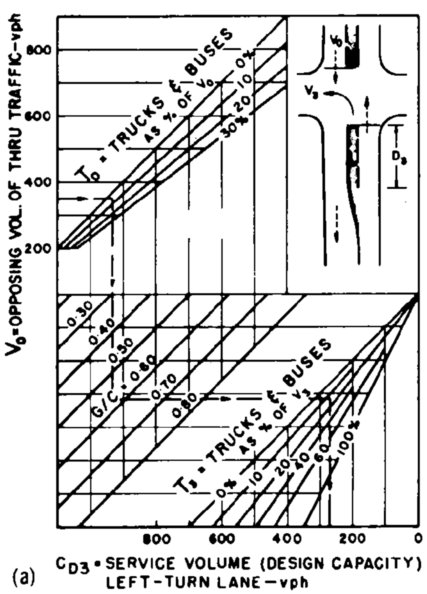
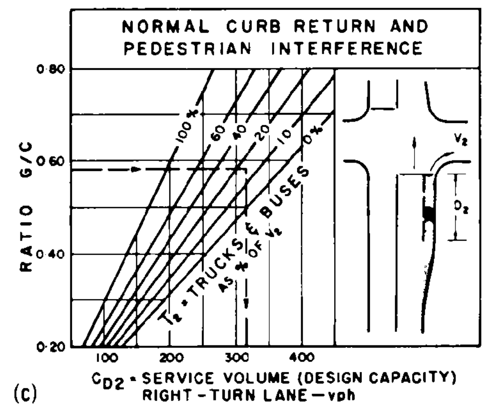
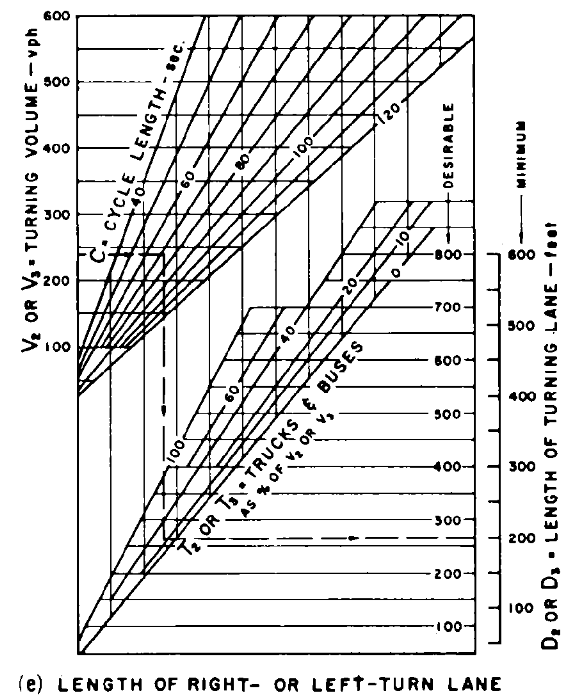
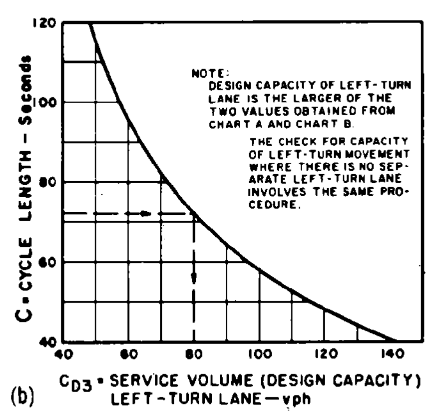


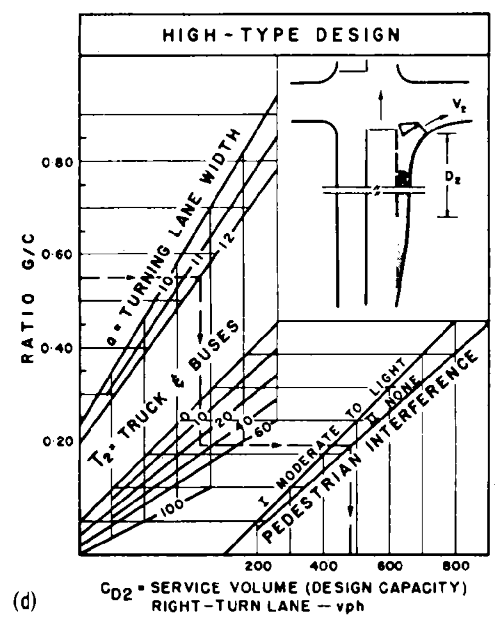
FIGURE 28.—Service volumes for design of signalized T or Y ramp junctions.





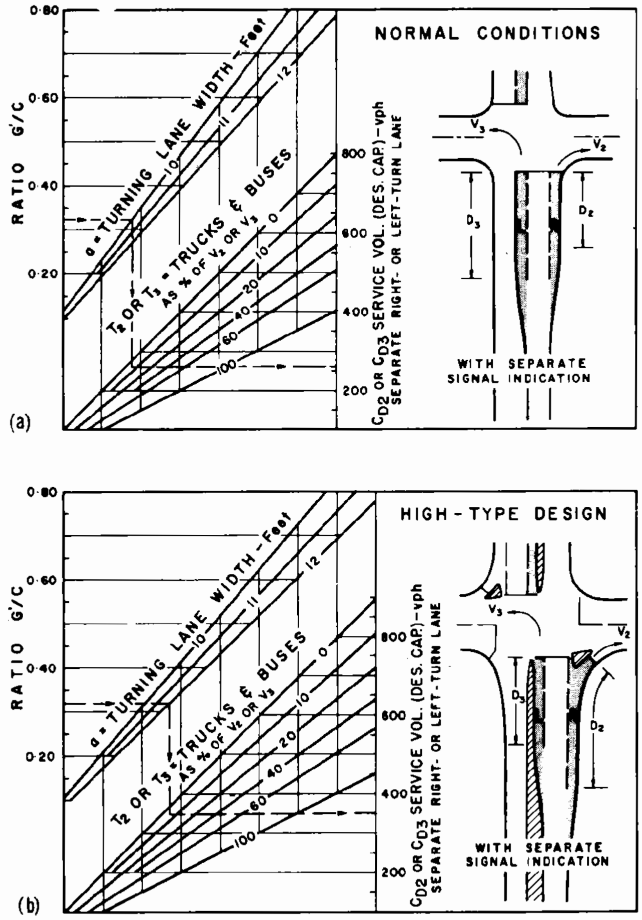


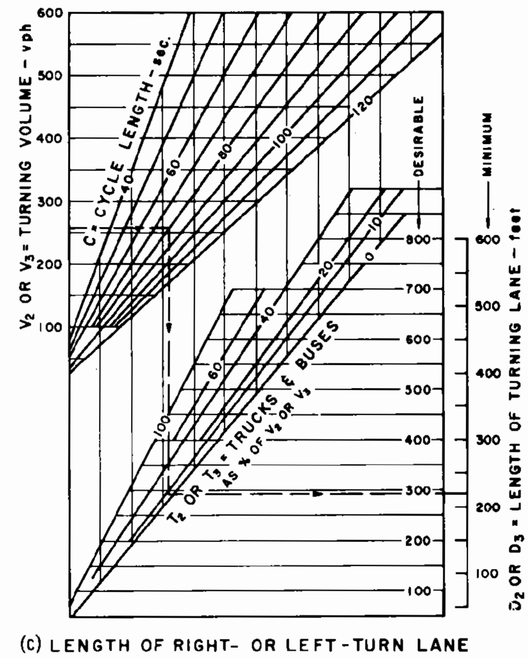




NOTE:
- CHARTS A AND B APPLY TO TWO-WAY FACILITIES ONLY, IN URBAN AS WELL AS IN RURAL AREAS.
- CHARTS C AND D APPLY TO RIGHT-TURN LANES ON TWO-WAY FACILITIES AND TO BOTH RIGHT- AND LEFT-TURN LANES ON ONE-WAY FACILITIES.
- CURVE II OF CHART D GENERALLY IS NOT APPLICABLE TO CBD.
- IN RURAL AREAS (FOR SERVICE LEVELS A AND B) MULTIPLY VALUES IN CHART D BY 0·90, AND FOR SERVICE LEVEL D BY 1·20.
- TO DETERMINE POSSIBLE CAPACITY OF RIGHT- OR LEFT-TURN LANE MULTIPLY CHART DESIGN CAPACITY BY 1·30 (SERVICE LEVEL E).

FIGURE 29.—Service volumes for design of signalized intersections and ramp terminals—SEPARATE RIGHT- AND LEFT-TURN LANES, with **no separate signal indication** for turning movement.





NOTE:

- CHARTS A AND B APPLY TO BOTH TWO-WAY AND ONE-WAY FACILITIES.
- CHART B GENERALLY DOES NOT APPLY TO CBD.
- IN RURAL AREAS (FOR SERVICE LEVELS A AND B) MULTIPLY VALUES IN CHARTS A AND B BY 0·90, AND FOR SERVICE LEVEL D BY 1·20.
- TO DETERMINE POSSIBLE CAPACITY OF RIGHT- OR LEFT-TURN LANE, MULTIPLY CHART DESIGN CAPACITY BY 1·30 (SERVICE LEVEL E).

FIGURE 30.—Service volumes for design of signalized intersections and ramp terminals—SEPARATE RIGHT- AND LEFT-TURN LANES, with **separate signal indication** for turning movement.

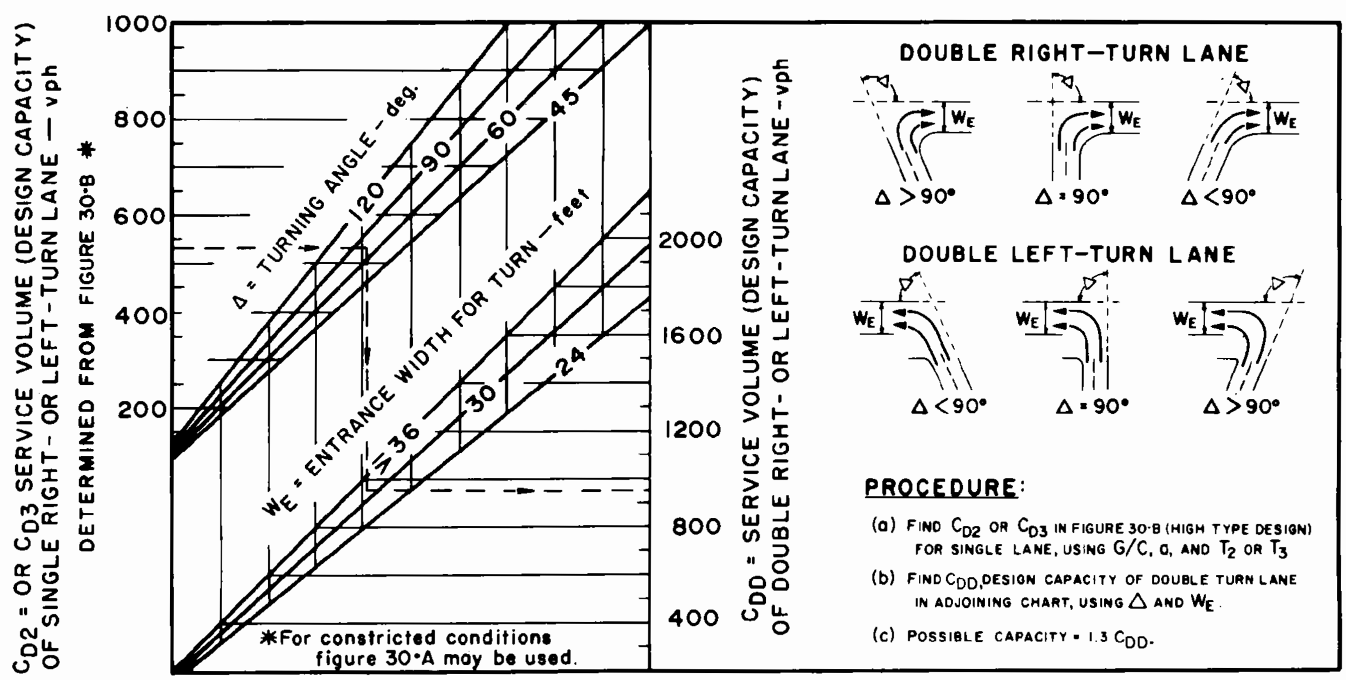


FIGURE 31.—Service volumes for design of signalized intersections and ramp terminals—DOUBLE TURNING LANES.

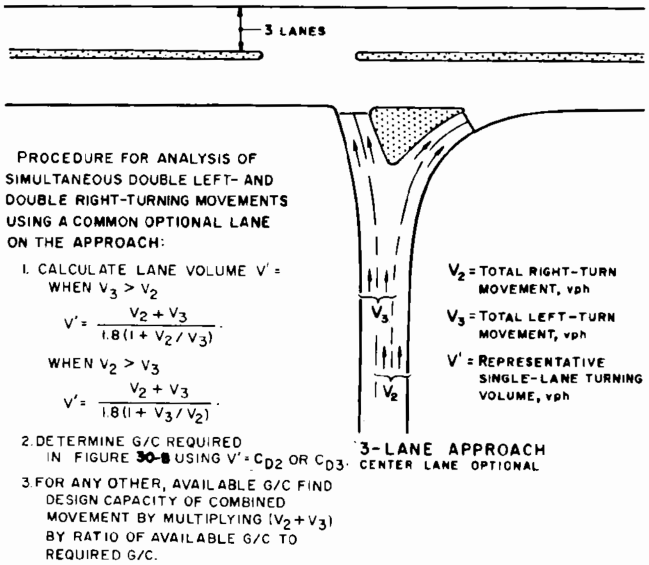


FIGURE 32.—Procedure for analysis of combined double left-turning and double right-turning movements with optional lane.

Solution For the situation indicated, signalization normally would not be warranted but, as previously suggested, signalization is assumed as an analysis device for geometric design purposes. Under such relatively light traffic conditions, a green time allocation for the entrance ramp of about one-third of the cycle is considered normal or G/C of 0.35; for the cross-street movements, then, $G/C = 1.00 - 0.35 - 0.10 = 0.55$, which is more than adequate. According to figure 34, case 1–A, the service volume capability is determined in figure 30, chart B. Using $G/C = 0.35$, $a = 12$, and $T = 15$ percent, $C_{D2} + C_{D3} = 300$ vph (level C). For rural conditions (level B), the entrance service volume $= 0.90 \times 300 = 270$ vph (see note lower right portion of figure 30), compared with the estimated demand volume of 120 vph. Thus, the ramp entrance design is adequate and has the potential to accommodate larger volumes in the future.

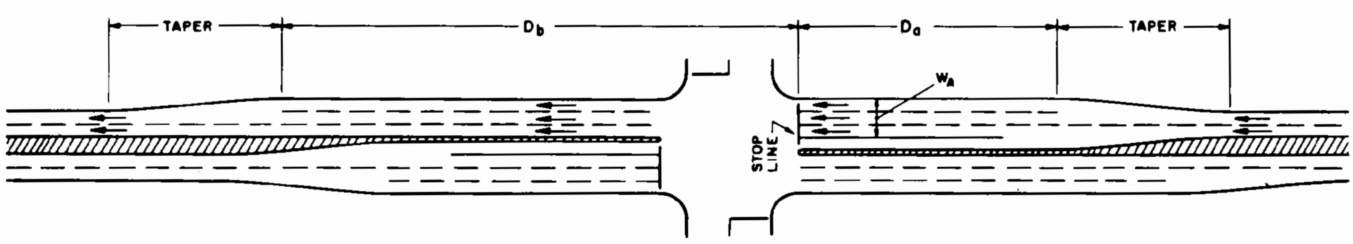
Problem 18

A diamond interchange in a suburban area of a large city accommodates at one of the ramp terminals on the cross street an entering volume of 420 vph including 8 percent trucks. Of this volume, 260 vph turn left and 160 vph turn right. The single-lane ramp is on a 3.2 percent maximum grade and is considered to have a representative design speed of 30 mph. Signalization requirements leave a G/C of 0.28 available within 3-phase control of 80-second cycle for moving ramp entrance traffic. The existing design, which operates under considerable congestion, is of the 1–A variety in figure 34. Determine what form of ramp terminal design (referenced to figure 34) is required to allow for level of service C operation. Both left- and right-turn movements must be stopped in the same phase to allow for a pedestrian crossing.

Solution The next higher form of ramp terminal, 1–B in figure 34, is tested to determine its adequacy for the conditions. In accordance with instructions in figure 34, the service volume for both left- and right-turning movements is determined in figure 30, chart B. Using $G/C = 0.28$, $a = 12$, and $T = 8$ percent, find $C_{D2} = 320$ vph and $C_{D3} = 320$ vph, compared with approach volumes of 260 and 160 vph. The length of widened approach for 2 lanes is governed by the (larger) left-turning movement. In chart C of figure 30, $D_3 = 300$ feet (desirable) using $V_3 = 260$, $C = 80$ seconds and $T_3 = 8$ percent. Check for ramp proper in table 11 shows a service volume of 1,210 vph.

Problem 19

A 2-lane (24-foot) ramp from a freeway in the central business district (CBD) with a metropolitan population (MP) of 500,000 enters a major street under signal control. The G/C available for moving traffic on the ramp is 0.40. The ramp terminal is of the 2–A variety in figure 35. Of the approach volume, 70 percent of the traffic turns left. Trucks and buses make up 25 percent of the entering volume. The angle of turn for the predominant (left) turning movement is 100 degrees and the width of cross street in each direction of travel is 32 feet. Determine the



LENGTH OF WIDENING BEYOND INTERSECTION

LENGTH REQUIRED FOR: *			TAPER feet
ACCELERATION		MERGING	
DESIGN SPEED mph	D_b — feet		
40	200	$D_b = 12 \times G$ (G, Green interval in seconds)	200
50	525		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 • Use volume per lane in Figure 30-C; find $D_2 = D_a$ on desirable scale (minimum scale for restricted conditions)	
40	150		175
50	200		225
60	250		275

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

FIGURE 33.—Procedure for analysis of widened intersection and ramp approaches—length requirements.

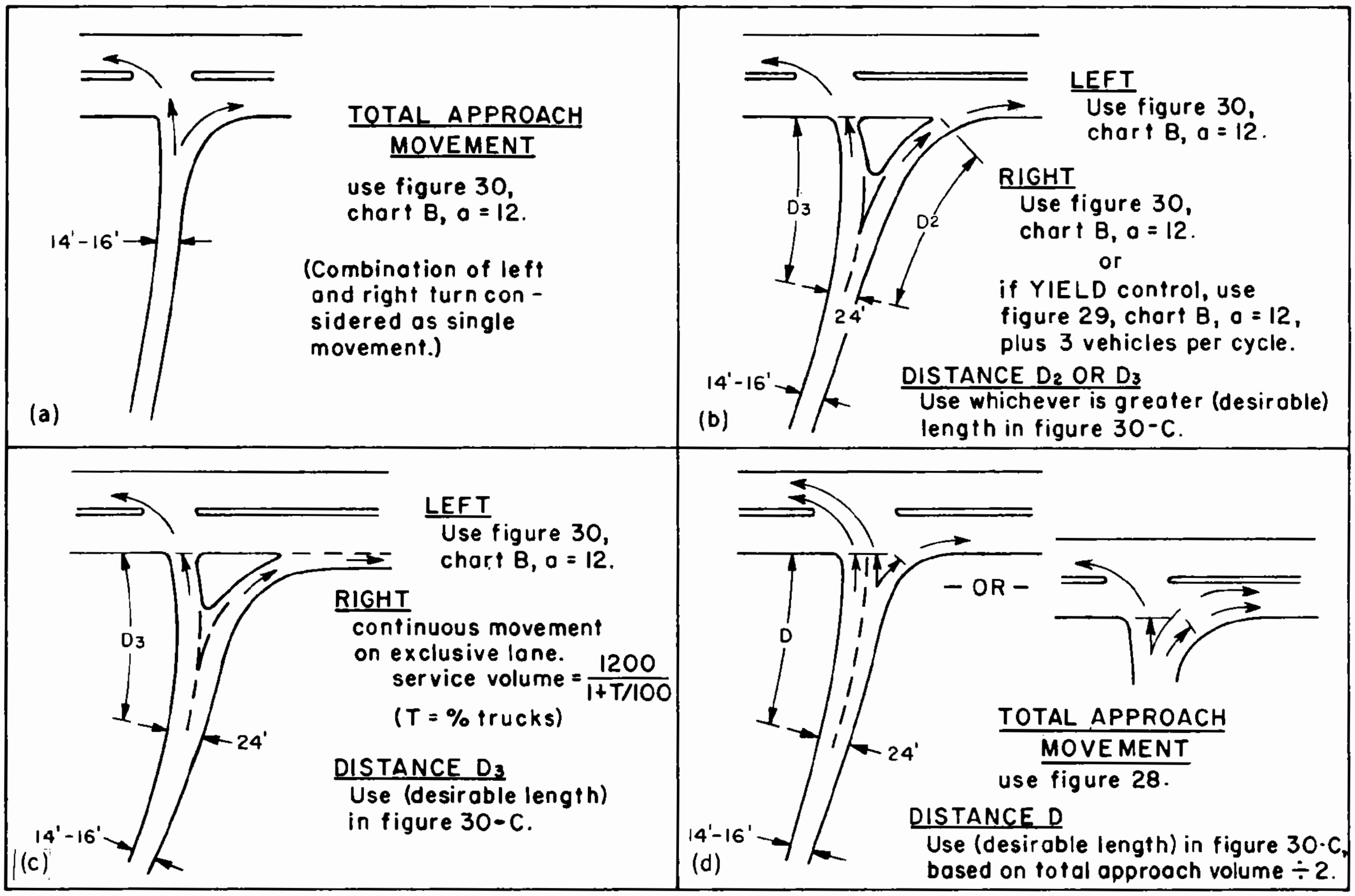


FIGURE 34.—At-grade ramp terminals, single-lane ramps—index to analysis procedures.

ramp volume which can be accommodated at level of service *C*.

Solution In accordance with instructions in figure 35, the procedure in figure 28 must be followed. The chart of figure 26 is first entered, using W_A = 24 feet, T = 25 percent, R = 30 percent, L = 0 percent, MP = 500,000 population, and G/C = 0.40; read C_D = 620 vph. The chart of figure 28 is then entered with C_D = 620 vph and, using $\triangle$ = 100 degrees and W_E = 32 feet, the service volume (design capacity) of the 2-lane ramp approach is found to be C'_D = 430 vph.

Problem 20

At a diamond interchange a ramp entrance to a major street is similar to form 2–C in figure 35. The angle of turn for the predominant (left-turning) movement from the ramp is 75 degrees and the width of traveled way on the street, in one direction, receiving the turn is 36 feet. The approach volume is 680 vph, of which 180 vph turn right. The signal is to be set for 3-phase 100-seconds cycle operation. Truck traffic on the ramp is 22 percent. Determine the G/C required for the ramp entrance to maintain level C operation, and the length of right-turn lane.

Solution Following the analysis steps in figure 35, form 2–C and the chart of figure 31, in combination with charts B and C of figure 30, are utilized. Since G/C is to be determined, rather than service volume, the procedure in figure 31 is followed in reverse order; that is, the chart in figure 31 is used first to find a representative (single-lane) service volume for the 2-abreast left-turning movement, and then the G/C is determined in figure 30–B. Entering the chart of figure 31 with approach volume C_{DD} = 680 − 180 = 500 vph, and using W_E = 36 feet and $\triangle$ = 75 degrees, find C_{D3} = 330 vph. Applying this volume in figure 30–B along with T = 22 percent and a = 12, the required G/C = 0.36. Since the left-turning volume is greater than the right-turning volume, the latter can also be handled at the same time on the exclusive lane within G/C of 0.36. To accomplish this, the length of right-turn lane must be of adequate length, and is determined by the larger of the two storage lengths required, D_2 or D_3. The left-turning movement controls, so that the length of right-turn lane is predicated, in figure 30–C, on V_3 = 500/2 = 250 vph, C = 100 seconds, and T = 22 percent; D_3 or D_2 is found to be 380 feet.

Problem 21

Determine the signal timing and the level of service at which traffic would be accommodated for the conditions shown in figure 36.

Solution The G/C requirements for left-turning movements are usually the first tested where 3-phase control

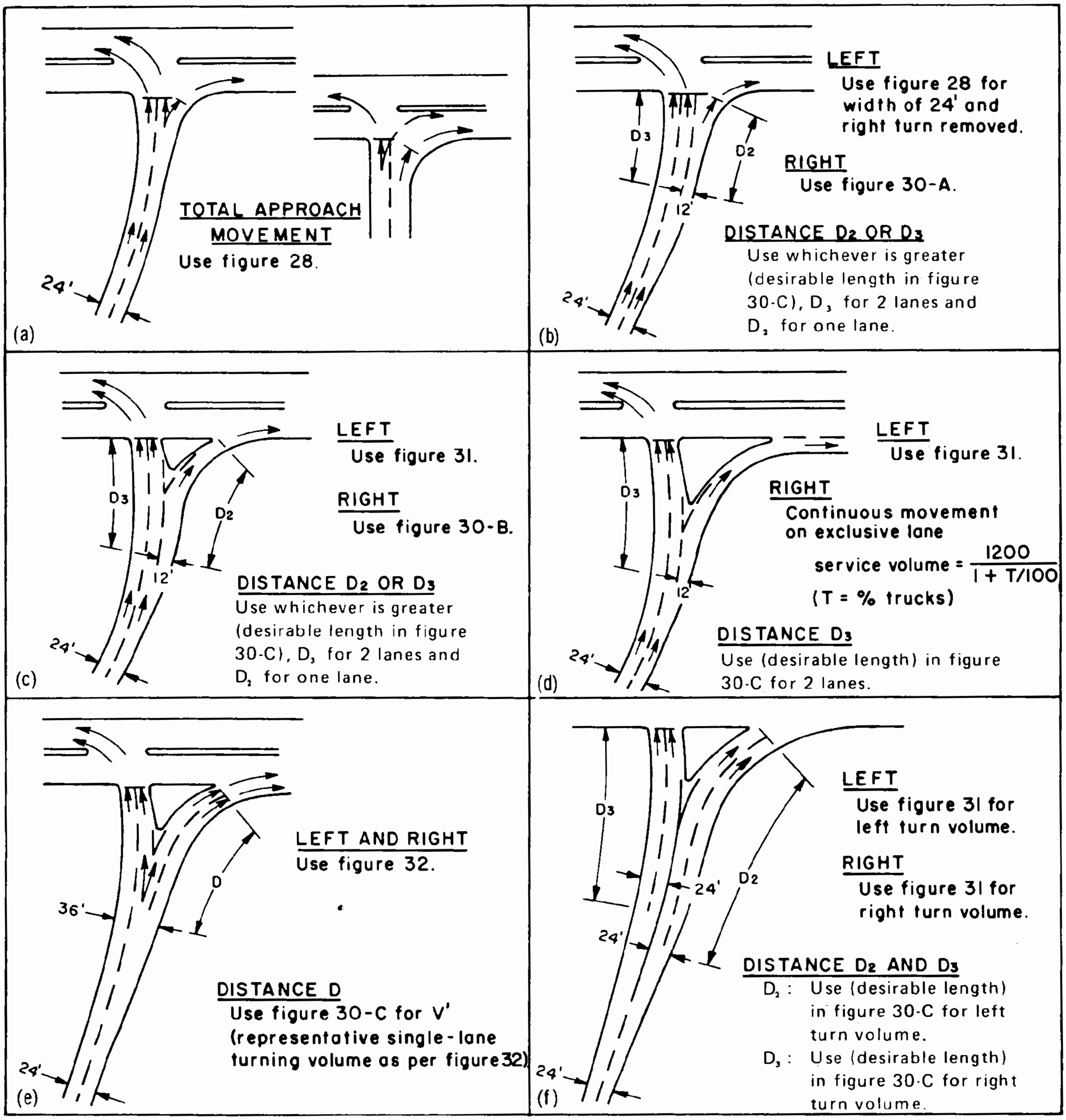


FIGURE 35.—At-grade ramp terminals, 2-lane ramps—index to analysis procedures.

is involved, then the other movements are accounted for in the analysis. Approach *C*, accommodating movements *CA* and *CB*, is identified as form 2–E in figure 35. The procedure for analysis is given in figure 32, in combination with charts B and C in figure 30. The representative single-lane volume for design purposes is $V' = (V_2 + V_3) \div 1.8 (1 + V_2/V_3) = (240 + 380) \div 1.8 (1 + 240/380) = 210$ vph. In figure 30–B, using $V' = C_{D3} = 210$ vph, $T = 15$ percent and $a = 12$ feet, the G/C requirement (for phase 3) is found to be 0.22. The companion right-turning movement *CB* can be accommodated within this G/C since the volume is smaller than for movement *CA*. For the left-turning movement *AD*, figure 30–B is also employed, using in the chart $V_3 = C_{D3} = 300$ vph, $T = 8$ percent and $a = 12$ feet; $G/C = 0.32$ (phase 2). Green time available for handling traffic on approach *BA*

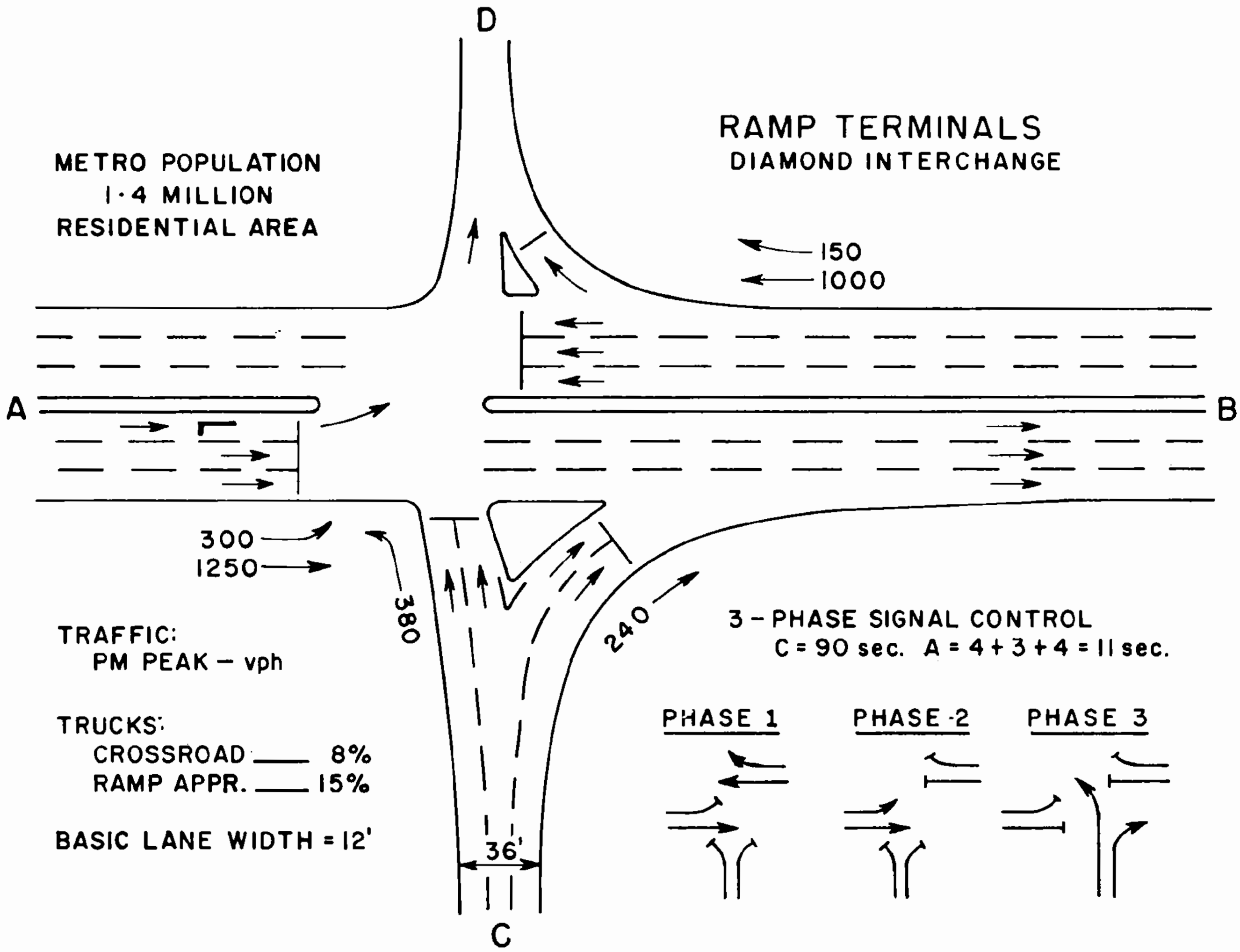


FIGURE 36.—Problem 21 illustrated.

is 90 − (0.22 + 0.32) 90 − 11 = 30 seconds or G/C = 30/90 = 0.33. The green time required for this movement is determined in figure 25. Enter chart with W_A = 36 and, using T = 8 percent, R = 150/1150 = 13 percent, L = 0 percent, MP = population over 1 million, project horizontally across G/C fan of values. Then, enter at bottom on residential area scale with approach volume equal to C_D of 1,150 vph, and extend vertically to intersect the horizontal line previously projected; read G/C = 0.30 (phase 1).

A check on movement AB, which proceeds continuously during phases 1 and 2, is necessary. Available G/C = 0.30 (phase 1) + 3/90 (amber) + 0.32 (phase 2) = 0.65. The green time required is found in figure 25 using W_A = 24, T = 8 percent, R = 0 percent, L = 0 percent, MP = population over 1 million and approach volume or C_D (residential area) = 1,250 vph; G/C = 0.46. If the available G/C of 0.65 were utilized, the chart shows that 1,760 vph could be handled. The total G/C required for level of service C (design capacity) operation is 0.30 + 0.32 + 0.22 = 0.84. The total G/C available for level of service C operation is 1.00 − 11/90 (amber) = 0.88. The ratio of 0.84 to 0.88, or 0.95 is the same as the ratio of the total of approach volumes to the total of service volumes (design capacities); that is, $V/C_D = f$ = 0.95, where f is the adjustment factor for level of service in table A of figure 25. Use of widest approach (36 feet) as the entry into the table, shows a factor of 0.92 as the limit of service level B. Therefore, the intersection as a whole (when the G/C's of the various movements are readjusted and balanced) would still be within level of service C (f = 0.95) but close to level B operation. Movement AB, however, would operate at a higher level of service because of the excess G/C provided. Factor f or V/C_D = 1,250/1,760 = 0.71 and, according to table A in figure 25, values of 0.88 or less provide service level A operation.

Problem 22

At a parclo-A interchange shown in figure 37, the 2-lane two-way crossroad is widened through the interchange in compliance with good design practice. In this case, the widening is also essential in order to satisfy design capacity or service level requirements. Determine the form and length of widening necessary for the traffic conditions given.

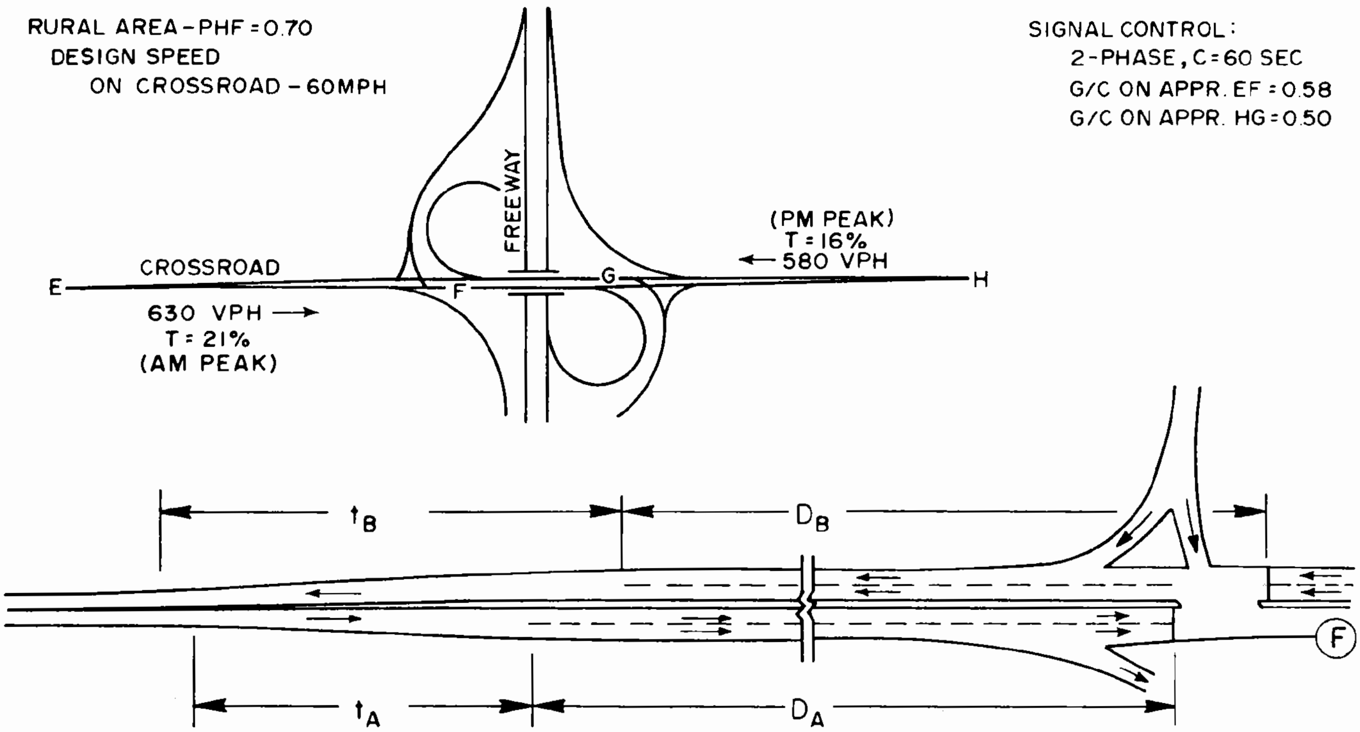


FIGURE 37.—Problem 22 illustrated.

Solution Figure 33 is used to establish minimum dimensions required for widening the crossroad in advance and beyond the two points of intersection, F and G. Figure 27 is used to check the design capacity (level of service B) for rural conditions. For intersection F these are found to be 870 vph for approach EF (the right-turn movement taken to be non-deterring) and 820 vph for approach GF. The service volumes are higher than needed to handle 630 vph and 580 vph, respectively; this, however, would be expected due to adding a whole lane in each direction. Actually, level of service A is achieved. The length of widening (D_a) in advance of intersection F is controlled by storage requirements (figure 33). In figure 30–C, for a lane volume of $630/2 = 315$ vph, $C = 60$ sec. and $T = 21$ percent, and $D_a = 300$ feet. The length of taper t is a minimum of 275 feet predicated on a design speed of 60 mph. The length of widening (D_b) beyond intersection F is determined by acceleration requirements. According to figure 33, $D_b = 900$ feet and $t = 300$ feet.

The actual length of widening and taper for the case illustrated normally would be of more liberal dimensions, as shown in figure 37, to achieve smooth geometrics in consideration of safety and appropriate operational characteristics.

Ramps in Sequence

Since ramps in close sequence may have operational influence one upon another, it is necessary to check this aspect by grouping consecutive ramps along a freeway into pairs. As shown in figure 16, such groupings produce four basic configurations: (a) successive entrances, EN–EN; (b) successive exits, EX–EX; (c) entrance followed by an exit, EN–EX; and (d) exit followed by an entrance, EX–EN.

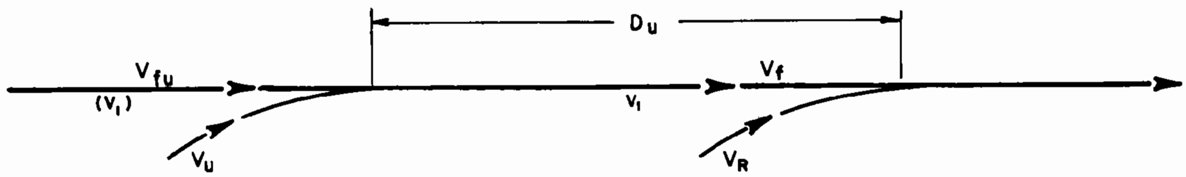
For the EN–EN case where the distance between entrances is 3,000 feet or more, ramps may be analyzed as "isolated," using the charts of figures 17 and 19; where the distance between entrances is less than 3,000 feet, the procedure in table 12 should be applied to the second ramp entrance. For the EX–EX case where the distance between exits is 4,000 feet or more, ramps may be analyzed as "isolated," using the charts of figures 16 and 18; where the distance between exits is less than 4,000 feet, the procedure in table 13 should be applied to the first ramp exit.

For the EN–EX case, the composite section should be analyzed as a weaving section using figure 41 and table 15. The entrance ramp and the exit ramp, however, should be checked individually using the "isolated" ramp condition. The isolated condition for design purposes (only) is considered appropriate as a check. The length/width relations determined by the weaving chart, particularly where ramp entrances and exits are developed with two lanes for ramp volumes exceeding 1,000 vph, automatically provide a balance of elements within the weaving section. Ramp sequences producing weaving sections are analyzed in the following chapter. For the EX–EN case, both terminals may be analyzed as isolated ramps, utilizing the charts of figures 17 through 20.

Problem 23

The sequence of ramps along eastbound roadway of a freeway under design is shown in figure 38–A, together

TABLE 12.—*Volume adjustment for second ramp of successive ramp entrances (single-lane entrance with number of freeway lanes maintained beyond entrance)*



V_u Entering Volume of First Ramp (vph)	V_e—Portion of V_u occupying lane 1 in addition to normal v_1 volume at second ramp entrance—vph						
	D_u—Distance Between Ramps—Feet						
	500	1,000	1,500	2,000	2,500	3,000	3,500
200	200	120	60	40	30	20	20
400	400	240	120	80	60	40	40
600	600	360	180	110	80	70	60
800	800	480	240	150	110	90	80
1,000	1,000	600	300	190	140	110	100
1,200	1,200	720	360	230	170	130	120
1,400	1,400	840	420	270	200	150	140

Compiled from *Manual* figure 8.24A.

INSTRUCTIONS FOR APPLICATION

1. *Given level of service:* to find service volume or capacity on second ramp entrance. Enter figure 17 with V_{fu}, the freeway volume upstream of the first entrance ramp, turn at appropriate curve (pertaining to volume V_u) and find (v_1) on chart at first entrance; add tabular value V_e to find adjusted v_1 at second entrance. As a check, reenter figure 17 with $V_f = V_{fu} \pm V_u$, turn on the ramp volume curve pertaining to V_R and find v_1. The larger of the two values of v_1 thus obtained governs. Continue through the chart, turning at the appropriate level of service to find V_R.
2. *Given ramp entrance volume:* to find level of service at second ramp terminal. The procedure is identical to that followed above, except that figure 17 is entered with V_R as well as V_1 and the level of service found by the intersection of construction lines.
3. For solution of first (upstream) ramp, use figure 17 in the conventional manner.

with distances between ramps, numbers of lanes, and pertinent traffic information. Determine the service volume at each exit and entrance for level *C* operation, and the level of service achieved at each ramp junction with the demand volumes indicated.

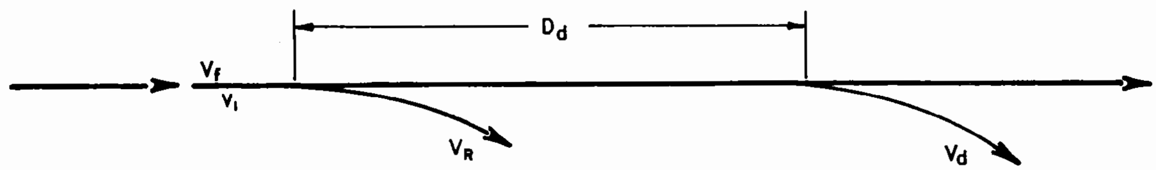
Solution The first step is to group the ramp junctions into overlapping pairs, *a* through *d*, as indicated in figure 38–B. The charts and tables used in the analysis (excluding weaving at this time) are tables 12 and 13 and figures 14–20. Ramp pair *a* (case EX–EX) is dealt with initially in figure 16 and table 13. In determining the service volume for the first ramp, instruction 1 in table 13 is followed. Enter figure 18 at bottom right with service level *C*)$v_1' = 1{,}500$) and, using $F_T = 1.00$ and $N = 3$, read $v_1 = 1{,}580$ vph. In table 13, find $V_e = 380$ vph for V_d and D_d of 600 vph and 2,000 feet, respectively. Continuing through the chart with adjusted $v_1 = 1{,}580 - 380 = 1{,}200$ vph and $V_f = 4{,}000$, determine V_R (service volume) $= 370$ vph. Level of service *C*, therefore, can be maintained since the demand volume on the ramp exit is 300 vph.

For ramp exit (2), figure 18 is used in the conventional manner. Applying $v_1' = 1{,}500$, $F_{T1} = 1.00$, $N = 3$ and $V_f = 3{,}700$, find V_R (service volume) $= 1{,}300$ vph. Level of service *C*, therefore, can be maintained since the demand volume on the ramp exit is 600 vph; in fact, level of service *B* operation is indicated in the chart for V_R of 600 vph.

Ramp pair *b* (case EX–EN) is dealt with initially in figure 16, where it is indicated that both junctions (2) and (3) are considered as isolated ramps. Exit (2) has been evaluated above as an isolated ramp and need not be considered further. Entrance (3), as part of pair *b*, functions as an isolated ramp; also, as part of pair *c* (case EX–EX) it is treated as an isolated ramp according to instructions in figure 16. Following the procedural steps for case EN(2) in figure 14, the service volume of entrance (3) is determined by referring to figures 17 and 19 in two steps for lane *A* and lane *B*. In figure 19, the service volume (level *C*) for lane *A* is found to be 1,470 vph. In figure 17, using $V_f = 3{,}100$, $N = 3$ (high volume ramp lane entrance), $F_{T1} = 1.00$, $v'_1 = 1{,}400$ (maximum for level *C*) and $F_{TR} = 1.00$, find $V_R = 850$ vph the service volume in lane *B*. The total service volume (at level *C*) which can be handled by ramp entrance (3), therefore, is $1{,}470 + 850 = 2{,}320$ vph, which far exceeds the demand volume of 1,100 vph. Entrance (3) as here analyzed satisfies the requirements for both ramp pairs, *b* and *c*.

To determine the level of service for ramp entrance (3), the approximate distribution of the demand volume

TABLE 13.—*Volume adjustment for first ramp of successive ramp exits (single-lane exit with number of freeway lanes maintained beyond exit)*



V_d Exit Volume of Second Ramp (vph)	V_e—Portion of V_d occupying lane 1 in addition to the normal v_1 volume at first ramp exit—vph						
	D_d—Distance Between Ramps—Feet						
	1,000	1,500	2,000	2,500	3,000	3,500	4,000
200	190	160	130	90	60	20	0
400	380	320	250	190	120	50	0
600	560	480	380	280	180	70	0
800	750	640	500	380	240	100	0
1,000	940	800	630	470	300	120	0
1,200	1,130	960	750	560	360	140	0
1,400	1,320	1,210	880	660	420	170	0

Compiled from *Manual* figure 8.24B.

INSTRUCTIONS FOR APPLICATION

1. *Given level of service:* to find service volume or capacity on first ramp exit. Enter figure 18 with the service volume for the given level of service, and find v_1 for the appropriate number of lanes. Subtract tabular value (V_e) from v_1 and continue through chart to find V_R.
2. *Given ramp exit volume:* to find level of service at first ramp terminal. Enter figure 18 with the given V_R and find v_1 for the appropriate number of lanes. Add tabular value (V_e) to v_1 and continue through chart to find the level of service.
3. For solution of second (downstream) ramp, use figure 18 in the conventional manner.

of 1,100 vph between the two lanes on the ramp is taken to be proportional to the service volumes determined for level C; or for lane A it is 1,100 × 1,470/2,320 = 700 vph and for lane B it is 1,100 × 850/2,320 = 400 vph. Using these as V_a and V_R volumes in figures 19 and 17, an average level of service A is indicated. Ramp pair c (case EN–EN), with respect to the second ramp entrance, is dealt with initially in figure 16. Here it is designated that table 12 must be applied if the distance between the pair of ramps is less than 3,000 feet. However, since at the second ramp entrance (4) an auxiliary lane is added to the freeway (increasing the number from 4 to 5), v_1 value is of no concern and the ramp entrance case EN (1 + aux) is analyzed in figure 19, yielding a service volume (at level C) of 1,470 vph. The demand volume of 550 vph is much less, indicating service level A at the point of entrance.

Ramp pair d (case EN–EX), according to figure 16, is analyzed (for design purposes) on the basis of isolated ramps. (In complete analysis the EN–EX configuration is also checked for the effects of weaving traffic between the successive ramps; this aspect is accounted for in chapter V.) For ramp entrance (4) the above result applies in this case. For ramp exit (5) instructions in figure 15 are followed. In figure 20 the service volume (level C) for lane A is found to be 1,580 vph. In figure 18, using $v_1' = 1{,}500$, $N = 4$, and $V_f = 4{,}750 - 1{,}580 = 3{,}170$ vph, the service volume in lane B is found to be $V_R = 1{,}350$ vph. The total service volume which can be handled by ramp exit (5), therefore, is 1,580 + 1,350 = 2,930 vph, which far exceeds the demand volume of 1,600 vph. Assuming the demand volumes to be proportional to the service volumes for level C in each lane on the ramp, $V_a = 1{,}600 \times 1{,}580/2{,}930 = 900$ vph in lane A and $V_R = 1{,}600 - 900 = 700$ vph in lane B. Using these values in figures 20 and 18 respectively, level of service A is found for both lanes. In summary figure 38–C, the analysis shows that the spacing of ramps and the number and arrangement of lanes meet the requirements for service level C operation, or better, at all of the ramp junctions.

Problem 24

Parclo-A interchanges produce two entrances on the freeway in close sequence. The distance between the successive merging ends of the loop and outer ramp normally should be not less than 1,000 feet on a high-speed freeway. The preferable range is 1,000 to 1,500 feet. There are several ways of arranging lanes through the parclo-A interchange along the freeway. The most adaptable arrangement can be determined by a capacity analysis, as illustrated below. In figure 39 three possible lane arrangements for a parclo-A interchange are shown, with two sets of ramp volumes. Using AASHO criteria, determine the design capacities of the two ramps for the six variations indicated.

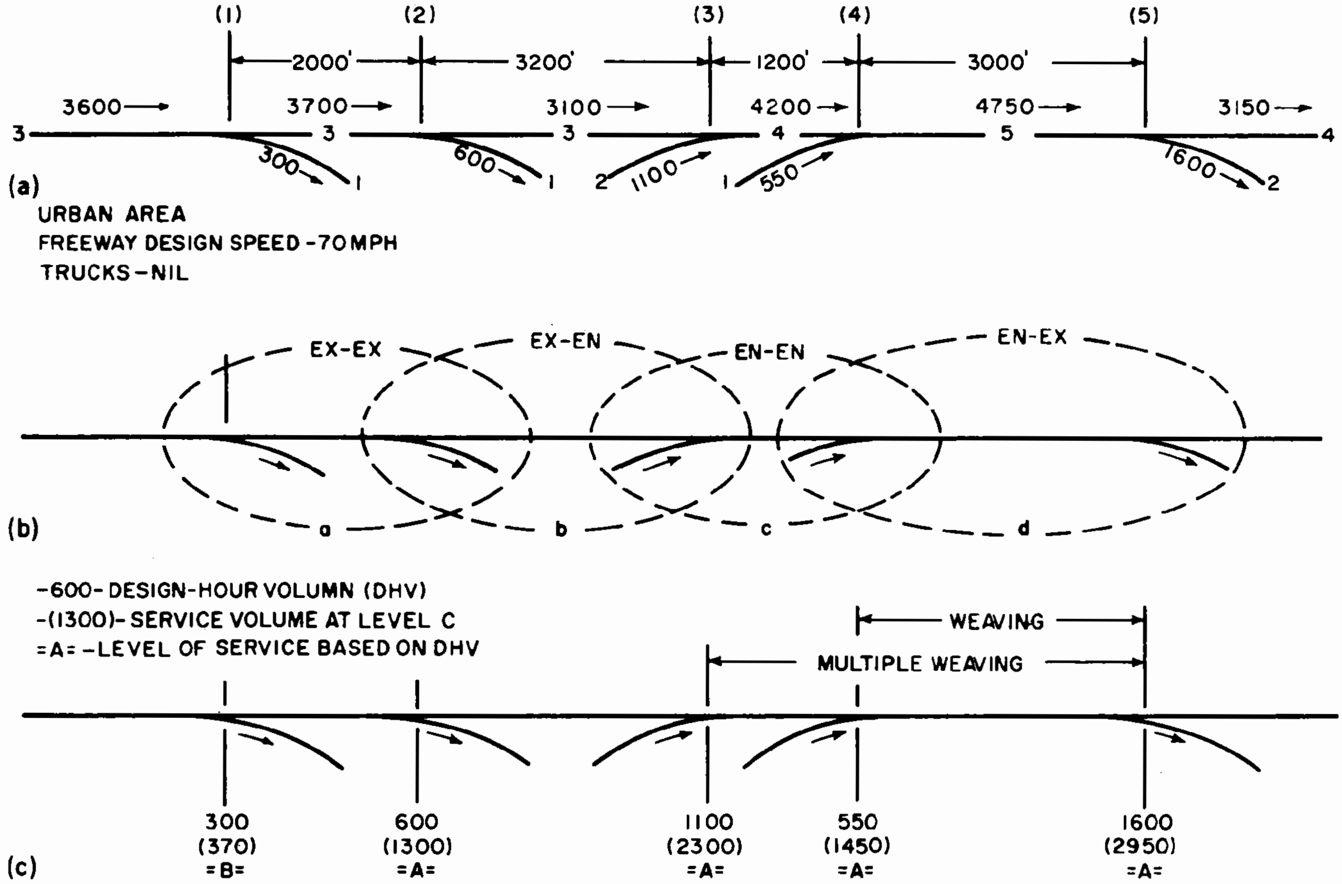


FIGURE 38.—Problem 23 illustrated.

Establish the best lane arrangement for each volume situation; also for the condition where both sets of volumes occur on the same facility, one during the a.m. and the other during the p.m. peak.

Solution The ramp configuration produces case EN–EN which is dealt with initially in figure 16 and table 12. For all conditions the loop ramp, being the first entrance, is treated as an isolated ramp, using figure 17 or 19. The outer ramp, as the second entrance, must be analyzed in accordance with the appropriate instruction in figure 16 for $D \leq 3{,}000$ feet.

Arrangement A(1) For the first entrance, use in figure 17, $V_f = 2{,}600$, $N = 3$, high volume group curve, $F_{T1} = 1.00$ (read $v_1 = 500$ vph in the process), 1,420 service level base, $F_{TR} = 1.00$; find V_R (design capacity) $=$ 1,000 vph. For the second entrance, instruction 1 is used in table 12. For $V_u = 600$ vph, $D = 1{,}250$ feet, and $V_e = 270$ vph; $(v_1) = 500$ vph as determined above for the first ramp. Adjusted v_1 at second ramp equals $(v_1) + V_e = 500 + 270 = 770$ vph. Also v_1 is checked at the second ramp using the full volume on the freeway of 3,200 vph in figure 17. This yields a volume of 650 vph. The larger of the two ($v_1 = 770$ vph), therefore, is reapplied in figure 17 yielding V_R (design capacity) $= 730$ vph. The result is that the first ramp has a good capacity reserve, while the second ramp volume exceeds design capacity. Comparative values are shown in the figure.

Arrangement A(2) For the first ramp the same variables as in the first arrangement apply in figure 17, producing the same design capacity of 1,000 vph. For the second ramp, case EN (1 + aux) is utilized in the conventional manner. Using in figure 19, a service volume base of 1,420 (AASHO, urban condition), V_a (design capacity) $= 1{,}500$ vph. In this case a good capacity reserve is provided at each ramp.

Arrangement A(3) For the first ramp, which joins the freeway with an added (auxiliary) lane, figure 19 applies directly, producing V_a (design capacity) $= 1{,}500$ vph. The second ramp, which joins the continuation of the same auxiliary lane, requires the application of the asterisked note in figure 16. Accordingly, $v_1 = 1.5\ V_e$ in table 12, or $v_1 = 1.5 \times 270 = 400$ vph (approximately). Enter figure 17 with $v_1 = 400$ and proceed through the chart as before; find V_R (design capacity) $= 1{,}100$ vph. In this arrangement there is a sizable reserve for the first ramp, while the volume on the second ramp is just within design capacity. Examining the three alternatives in figure 39 for the 600–1,000 ramp volume combination, arrangement A(2)

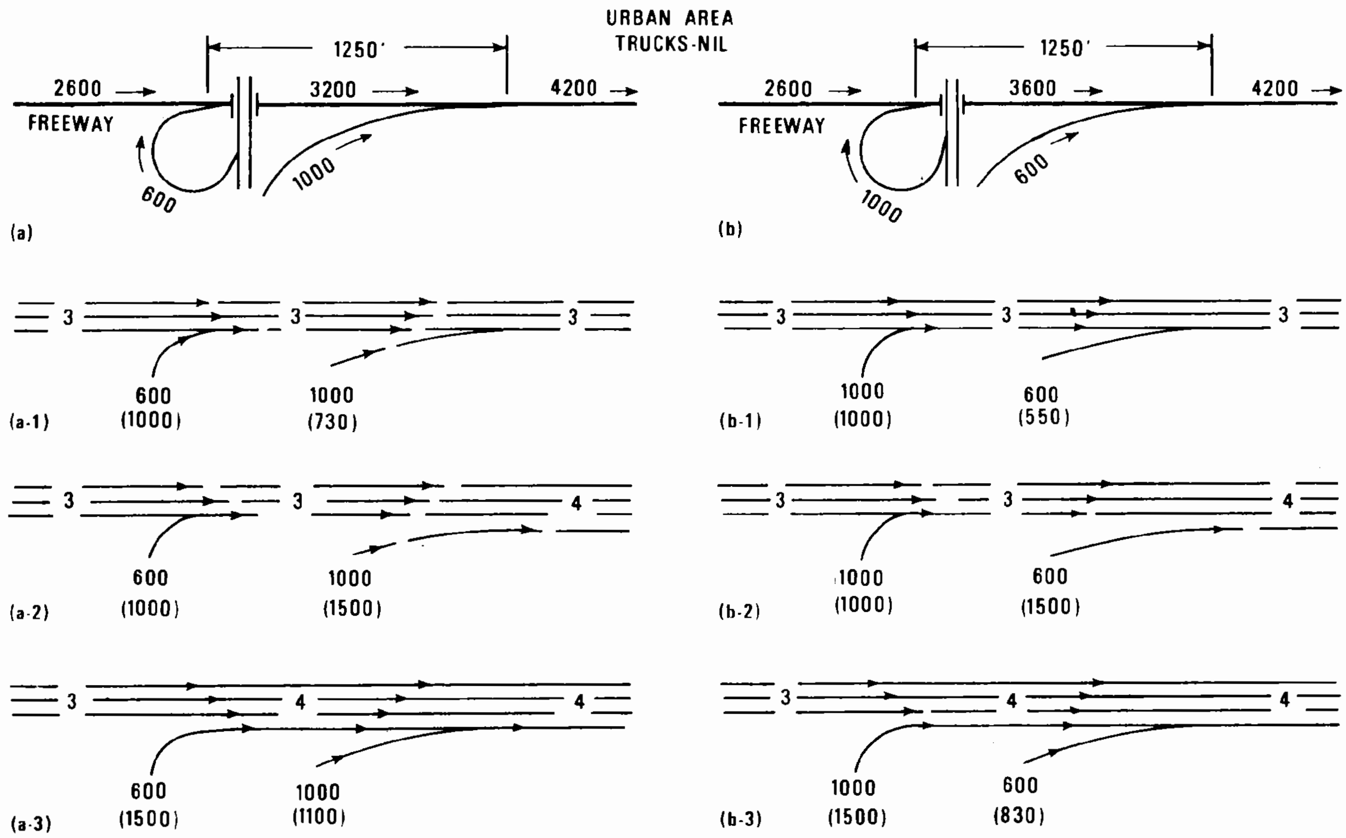


Figure 39.—Problem 24 illustrated.

obviously provides the best solution, indicating a good reserve and balance of design capacities with respect to demand volumes.

Arrangements B(1), B(2), and B(3) This series differs from the first in that the two ramp volumes are reversed, the first being 1,000 vph and the second 600 vph. The analysis here is parallel to that above, utilizing the same steps, charts, and table. The results for these arrangements are summarized on the right in figure 39.

Summarizing For the 600–1,000 ramp volume combination, arrangement A(2) obviously provides the best solution, indicating a good reserve and balance of design capacities and demand volumes. For the 1,000–600 ramp volume combination, arrangement B(3) provides the best result, also maintaining a reasonable balance between capacities and volumes. For the condition where the first volume combination represents an a.m. peak and the second a p.m. peak, the choice is more difficult. Everything being equal, arrangement (3) appears to be the most favorable, although arrangement (2) also may be used to advantage. The situation downstream with respect to exiting and possible weaving maneuvers may affect the choice.

SPECIAL CONDITIONS

The material covered thus far is based largely on the *Manual* data, with some rationalization and adjustment of values to produce a readily workable procedure for analysis in design. Several areas are practically untouched in the *Manual* and are in need of determination in order to round out the necessary capacity analysis techniques. Three critical areas are noted and developed here by piecing together bits of data, operational experience and engineering judgment. The following deal with (a) ramps having high-peaking characteristics, (b) ramp junctions on 5-lane (one direction) traveled ways, and (c) left-hand ramps. The procedures set out are strictly for design purposes.

Ramps with High Peaking Characteristics

Some ramps serve localized traffic generators that produce high peaks within 15- to 30-minute periods, while the other freeway approaches at the same interchange may be operating under normal peaking conditions. For more or less normal conditions, average or representative peak-hour factors have been built into the service volumes, as presented in previous chapters. Departure from the procedures outlined will rarely be required, but where it is anticipated that unduly high peaking characteristics are apt to occur, producing peak-hour factors well below normal, adjustments should be applied in the analysis.

Merging and Diverging—Entrance and Exit Ramp Terminals

Divide the service volume or design capacity (AASHO), determined under normal circumstances in figures 17–20, by: 0.75 for operation at service levels *A* and *B*, or at rural design capacity; 0.85 for operation at service levels *C* and

D, or at urban design capacity. Then, multiply above by specific PHF[3], known or estimated to be applicable to the ramp terminal condition, to establish the adjusted service volume or design capacity.

At-grade Ramp Terminals

Divide the service volume or design capacity, determined under normal circumstances in figures 25 and 26, by average peak-hour factors of 0.80, 0.85 and 0.90 for metro population areas of 50,000 or less, 100,000–750,000, and 1,000,000 or more, respectively; then, multiply above by specific PHF[3] applicable to the intersection approach to establish the adjusted service volume or design capacity. In conjunction with figure 27, rural conditions, find service volume or design capacity using PHF=1.00; then, multiply result by PHF relevant to the particular intersection approach. For exclusive turning lanes (figures 29 and 30), divide chart values of service volumes or design capacity by 0.85, then multiply by specific PHF applicable to the turning lane under consideration.

Weaving Sections

Produced by merging and diverging traffic to and from ramp junctions, weaving sections may also require adjustment in analysis as a result of high peaking characteristics on one or more of the ramp movements. The procedure is presented in the next chapter, along with other aspects of weaving operations.

Problem 25

A single-lane ramp entrance, along a freeway slated for widening, is operating under congestion during the p.m. peak period. The ramp discharges traffic onto the freeway under an intense peaking condition during one-half hour from an industrial complex. Repeated field measurements of traffic movement on the ramp entrance show a representative highest 5-minute flow equal to 130 vehicles within an hourly volume of 1,060 vph. The resulting PHF of 1,060 ÷ (130 × 12) = 0.68 is judged to be indicative of future conditions when the demand volume is estimated to be 1,800 vph. If the analysis for the contemplated design of a 2-lane entrance shows a service volume for level *C* of 2,450 vph (under normal peaking conditions) in the charts of figures 17 and 19, determine whether the demand volume can be accommodated without exceeding level of service *C* operation.

Solution Using the procedure outlined above, the adjusted service volume reflecting the high peaking characteristic of this movement is (2,450/0.85) × 0.68 = 1,960 vph. The new 2-lane ramp, therefore, will accommodate the 1,800 vph entering the freeway at level of service *C*.

Ramp Junctions on 5-Lane Traveled Ways

Data are not available on 5-lane (one-direction) freeways for evaluating operations at ramp entrances and exits. Since 5-lane traveled ways are frequently encountered on urban freeways, a procedure was developed and is presented here to permit analysis of the wider freeway sections. The approximate results thus obtained are predicated on extrapolation of lane distribution data at ramp entrances and ramp exits of narrower facilities. For convenience of application, the procedure entails an equivalent 4-lane (one-direction) freeway volume (just upstream of ramp terminal) that would produce approximately the same lane 1 volume as on the 5-lane (one-direction) freeway. The method thus converts the 5-lane freeway section to a 4-lane freeway section and allows the available nomographs and tables to be used directly. Table 14 provides for this conversion.

Problem 26

A 5-lane section of urban freeway is carrying 6,300 vph upstream of a ramp entrance. The 70-mph design conforms to appropriate standards. Truck traffic is nil. Determine the maximum service volume that can be accommodated on the entrance ramp without exceeding level of service *C* operation.

Solution The 5-lane section of freeway with 6,300 vph, according to table 14, is equivalent for analysis purposes to a 4-lane section carrying 6,300 × 0.78 = 4,900 vph. Using, in figure 17, V_f = 4,900, N = 4 (high volume group), F_{T1} = 1.00, level of service *D* (1,400 vph base), and F_{TR} = 1.00; find V_R = 790 vph.

Problem 27

Using AASHO criteria, determine the design capacity of a 2-lane ramp entrance joining a 5-lane section of freeway in an urban area. Traffic upstream of the ramp junction is 6,050 vph including 8 percent trucks. The 70-mph freeway is on a slight downgrade at this point, while the joining ramp, carrying 14 percent trucks, is on a 2 percent upgrade preceding the junction.

TABLE 14.—*Conversion factors for analysis of ramp entrances and exits on 5-lane (one-direction) freeways*

Ramp junction	Freeway volume on 5-lane section upstream of ramp terminal (vph)	Factor to convert volume on 5-lane to equivalent volume on 4-lane section of freeway[1]
Entrance	All volumes	0.78
Exit	≦4,000	1.00
	5,000	0.90
	6,000	0.85
	≧7,000	0.80

[1] The equivalent volume on 4-lane section of freeway is that which produces approximately the same lane 1 volume for the indicated volume on 5-lane section, allowing nomographs and tables for 4-lane sections to be used directly in the analysis.

[3] For merging and diverging maneuvers, the peak-hour factor is predicated on a 5-minute rate of flow. For controlled movements in conjunction with at-grade intersections, the peak-hour factor is based on a 15-minute rate of flow. See pages 3 and 4.

Solution According to table 14, the 5-lane section of freeway is equivalent to a 4-lane section carrying $6{,}050 \times 0.78 = 4{,}700$ vph. As outlined in figure 14, the ramp capacity is evaluated using the charts of figures 19 and 17 for lanes A and B, respectively. In figure 19, using design capacity base of 1,420 vph (see footnote on chart) and $F_{Ta} = 0.70$ (from table 10), design capacity of lane $A = 1{,}050$. For lane B, using in figure 17, $V_f = 4{,}700$ and $N = 2$ (low vol. group, determined by preliminary run through chart); find $v_1 = 730$. At this point the percentage of trucks in lane 1 must be determined. Using in figure 21, $V_f = 4{,}700$ vph, $N = 4$, $T_f = 8$ percent and $v_1/V_f = 730/4{,}700 = 0.15$; find $T_1 = 25$ percent. For near level conditions $E_{T1} = 2$ (table 4) and $F_{T1} = 1/[1 + 0.25\ (2 - 1)] = 0.80$. Reentering figure 17 with $v_1 = 730$ and proceeding through the chart using $F_{T1} = 0.80$, service volume base of 1,420 and $F_R = 0.70$; find V_R or design capacity of lane $B = 400$ vph. Design capacity of ramp entrance $= 1{,}050 + 400 = 1{,}450$ vph.

Problem 28

A 2-lane ramp discharges traffic from a 6-lane freeway section. The approach volume is 7,300 vph on which truck traffic is considered to be nil. The freeway continues beyond the ramp as a 5-lane section. Determine the volume which the ramp can discharge without exceeding level of service C.

Solution According to figure 15, the charts of figures 20 and 18 are to be used in the analysis for lanes A and B, respectively. The service volume which can be accommodated at level of service C in lane $A = 1{,}580$ vph (figure 20). In conjunction with lane B, the discharge is made from a 5-lane freeway section on which the adjusted approach volume is $V_f = 7{,}300 - 1{,}580 = 5{,}720$ vph. An equivalent 4-lane section, for analysis purposes, would accommodate $5{,}720 \times 0.84 = 4{,}800$ vph (see table 14). Using this as V_f and $N = 4$ in the chart of figure 18, find $V_R = 1{,}100$ vph, or service volume accommodated by lane B at level C. The 2-lane ramp, therefore, can discharge $1{,}580 + 1{,}100 = 2{,}680$ vph at level C operation.

Left-hand Ramps

Left-hand ramps are not recommended and normally are not used on modern freeways. They are employed, however, in conjunction with freeway distributors, including freeway sections functioning as C–D roads. For these special cases, the following rational procedure is offered as a basis for design.

Limited studies show that the volume of traffic in the freeway lane adjacent to the ramp in conjunction with left-hand ramps is higher than that in conjunction with right-hand ramps. For entrances there is approximately 25 percent more traffic volume in the freeway lane adjacent to a left-hand entrance than in the freeway lane adjacent to a right-hand entrance for similar conditions;[4] that is, $v_i/v_1 = 1.25$, where v_i is the volume in the extreme left lane of the freeway upstream of a left-hand ramp junction, and v_1 is the volume in the extreme right lane (lane 1) of the freeway upstream of a right-hand ramp junction. For exits there is approximately 10 percent more traffic volume in the freeway lane adjacent to a left-hand exit than in the freeway lane adjacent to a right-hand exit for similar conditions;[4] that is, $v_i/v_1 = 1.10$. Utilizing these ratios, the procedure for analysis of left-hand ramps is as follows:

Left-hand Entrances To find level of service on ramp, or ramp service volume V_R, enter chart of figure 17 with V_f, using appropriate N and ramp volume group, and read v_1; multiply v_1 by 1.25 to find new v_1 (or v_i); shift over on v_1 scale to new value and proceed through chart in normal manner. In determination of F_{T2} (or F_{T1}) assume for $N = 2$ that 25 percent and for $N = 3$ or more that 0 percent of through trucks on the freeway are in the left lane adjacent to the ramp entrance (do not use figure 21). For left-hand entrances on exclusive (auxiliary) lane or for lane A of 2-lane left-hand entrances, use chart of figure 19 directly.

Left-hand Exits To find level of service on ramp, enter chart of figure 18 with V_f, proceed to given V_R for appropriate N, and read v_1; multiply v_1 by 1.10 to find new v_1 (or v_i); shift left on v_1 scale to new value and proceed through chart in normal manner. In determination of F_{T1} (or F_{Ti}), in addition to exiting trucks, assume for $N = 2$ that 25 percent and for $N = 3$ or more that 0 percent of through trucks on the freeway are in the left lane adjacent to the ramp exit (do not use figure 21). To find ramp service volume (V_R) for a given level of service, enter chart of figure 18 with desired level of service at lower right, proceed in reverse order and read v_1 (or v_i); shift right on v_1 scale to new value and continue through chart to find V_R. For left-hand exits on exclusive (auxiliary) lane or for lane A of 2-lane left-hand exit, use chart of figure 20 directly.

Problem 29

A 2-lane transfer roadway from the freeway (inner core) joins a C–D road (freeway distributor) as shown in figure 40. Determine, for the traffic situation indicated, whether level of service C operation can be maintained. All grades are 1 percent or less.

Solution As a first step, an across-the-lanes check on the C–D road is made downstream and upstream of the entrance. Using in the chart of figure 11 a freeway volume of 2,300 vph with 10 percent trucks on the 2-lane section, and a volume of 4,000 vph with 12 percent trucks on the 3-lane section, lane service volumes of 1,200 pcph and 1,480 pcph are found. In table 6, lane service volumes of 1,200 pcph for level B and 1,500 pcph for level C are indicated for a 60-mph C–D road. Thus level B operation downstream and level C operation upstream can be maintained on the C–D road.

The truck adjustment factor for each lane on the entrance is found in table 10 to be $F_{TR} = 0.87$. Figure 19 indicates that lane A can handle 1,270 vph at level C operation. Using, in figure 17, $V_f = 2{,}300$ and $N = 2$ (low volume group), v_1 is found to be 880. For the left-hand

[4] From data (Northwestern University) of studies reported in Highway Research Record 99.

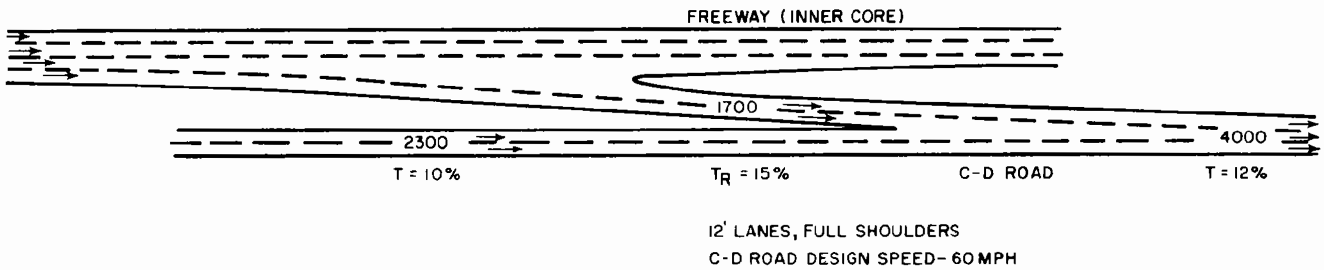


FIGURE 40.—Problem 29 illustrated.

entrance, the adjusted v_1 (or v_i) $= 880 \times 1.25 = 1{,}100$. Percentage of trucks in left lane of C–D road is $(2{,}300 \times 0.10 \times 0.25) \div 1{,}100 = 5$ percent, and F_{T1} of 0.95 (table 4). Reentering figure 17 with $v_1 = 1{,}100$ and using $F_{T1} = 0.95$, level of service C, and $F_{TR} = 0.87$, V_R is found to be 280 vph. At level C the ramp can handle $1{,}270 + 280 = 1{,}550$ vph, which is less than the demand volume of 1,700 vph.

Using figures 19 and 17 again, service volumes for level *D* operation are found to be 1,450 for lane *A* and 450 for lane *B*, or a total service volume of 1,900 vph. Thus the operation of the ramp falls within level of service *D*. As shown by the downstream check, level of service *C* operation would be restored on the C–D road beyond the merge.

Chapter V

WEAVING SECTIONS

BASIC VALUES AND FACTORS

In the course of designing freeways and expressways, interchanges are often spaced in such a manner that exiting and entering patterns have an effect on freeway flow in excess of that for normal ramp merging and diverging. This effect is referred to as "weaving." The *Highway Capacity Manual—1965* defines "weaving" as the crossing of traffic streams moving in the same general direction accomplished by successive merging and diverging.

Weaving can occur along freeways within an interchange or between interchanges as shown in figure 1. The presence of weaving on freeways thus requires a check for capacity and level of service purposes beyond that conducted with respect to v_1 at exits and entrances. Weaving sections are classified as simple or multiple. A simple weaving section is created by an entrance ramp followed by an exit ramp. Multiple weaving is created by an entrance ramp followed by two or three successive exit ramps (such as EN–2EX or EN–3EX), or by two or three entrances followed by one or two exits (such as 2EN–EX, 2EN–2EX, or 3EN–EX). The ramp groupings along the freeway which create multiple weaving sections are demonstrated in figures 1, 38, and 54B. Simple weaving sections are of two types: (a) single purpose, where all vehicles weave, and (b) dual purpose, where both weaving and non-weaving traffic (outer flow) are served.

Weaving sections can also be classified as one- and two-sided. Two-sided weaving occurs when a weaving volume must cross the through traffic flow, such as occurs along a freeway with a right-hand entrance followed by a left-hand exit. One-sided weaving avoids the direct effect on mainline through traffic. Modern freeways avoid two-sided weaving except as it may occur on C–D roads.

Weaving requires the crossing of vehicular paths. The capacity of a basic simple weaving section (two lanes) is thus limited to the number of vehicles that can use a single lane. If the volume exceeds this value, additional lanes are required thus creating a compound weaving section. But as this occurs, longer distances are often required to complete the weaving maneuver. It is noted in the *Manual*, for instance, that a volume equal to double the capacity of a traffic lane theoretically requires three times as much length as for a weaving volume equivalent to a single-lane capacity. At below-capacity situations where desired operating speeds approach 60 to 70 mph, greater vehicular headways are required for crossing vehicular paths than for the slower speeds at capacity. In addition, the distance used (during the time required to complete the maneuver) is greatly increased as well.

The general approach to the EN–EX case (which creates simple weaving) is demonstrated in figure 16. The analysis of weaving is discussed in further detail below. Both length and width requirements must be met in designing for a weaving section. The two are interrelated. The interrelationship is reflected in the design process through the use of separate checks for length and width which are linked by the definition of a weaving intensity factor k. For simple weaving, the length required is based solely on the weaving volumes and the desired level of service. Once a length is established, it is used to define the weaving intensity factor k. The presence of weaving intensity within a given section of roadway is the reason that, for equivalent volumes, more width is required than for uninterrupted flow. As stated in the *Manual*, "The k-factor, in effect, is an equivalency factor expanding the influence of the smaller flow up to a maximum of three times its actual size in number of vehicles."

The weaving intensity factor, determined for the length chosen, is included in the formula for determining the number of lanes in a weaving section, as follows:

$$N = \frac{W + kW' + F + F'}{\text{SV}}$$

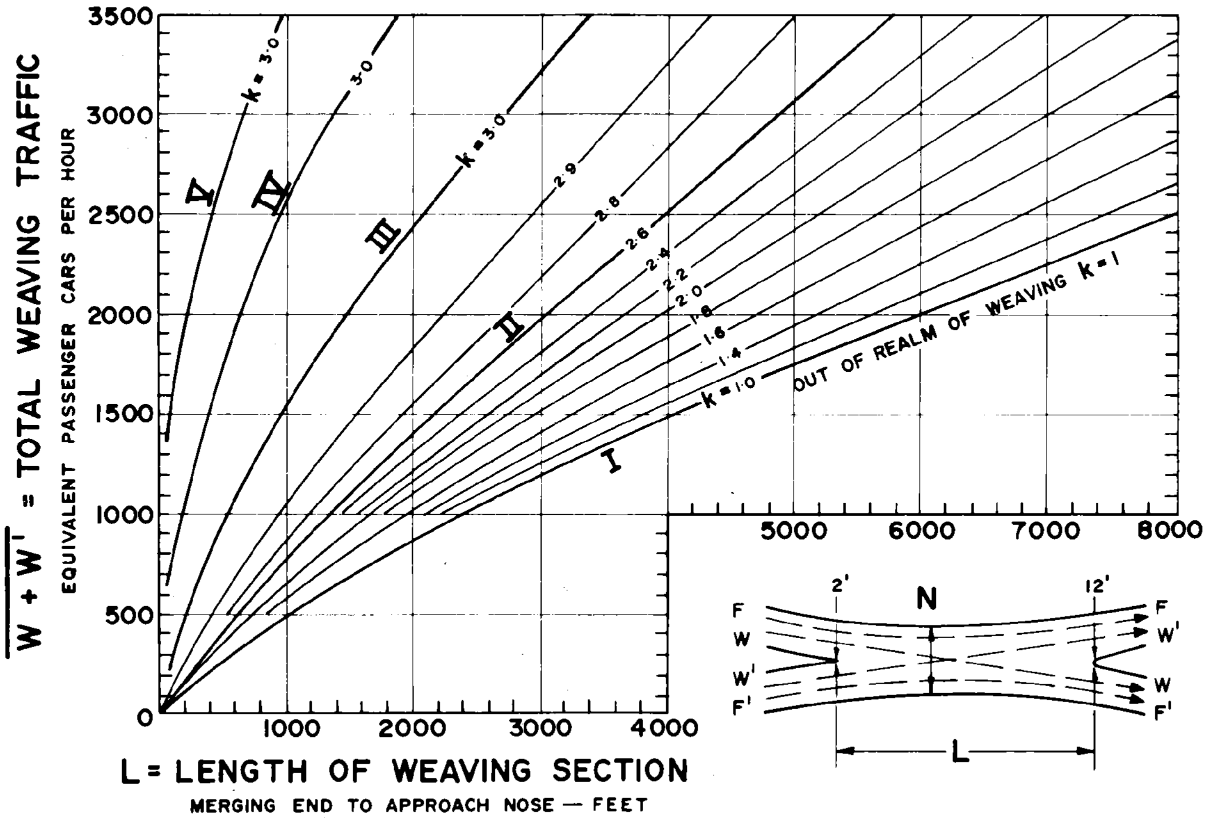
The terms are described pictorially in the diagram on figure 41. The term W' refers to the smaller of the two weaving volumes. The term SV refers to the service volume or capacity per lane on approach and exit roadways. It is predicated on the basic number of lanes approaching the weaving section under study. N is the required number of lanes and k is the weaving intensity factor.

The equation may be simplified to the form:

$$N = \frac{V + (k - 1)\,W'}{\text{SV}}$$

where $V = F + F' + W + W'$.

The operating characteristics of weaving sections are plotted in figure 41 in a format for use in design. The relationship between length, weaving volume and width can be more clearly seen with the use of this diagram. The graph indicates the k value associated with a given length of weaving section and weaving volume. The formulas which use the k value to determine lane width requirements are also shown. The set of curves defining the k values range from the out of the realm of weaving (curve I) to



$$N = \frac{W + kW' + F + F'}{SV}$$

$$N = \frac{V + (k-1)\,W'}{SV}$$

N = NUMBER OF LANES
W = LARGER WEAVING VOLUME, VPH
W' = SMALLER WEAVING VOLUME, VPH
F, F' = OUTER FLOWS, VPH
V = TOTAL VOLUME, VPH
k = WEAVING (INTENSITY) FACTOR
SV = SERVICE VOLUME OR CAPACITY PER LANE ON APPROACH AND EXIT ROADWAYS, VPH
$\overline{W + W'}$ = TOTAL WEAVING TRAFFIC EQIV. PASSENGER CARS PER HOUR

LEVEL OF SERVICE FOR SERVICE VOLUME, $\overline{SV}$, SEE TABLE W-1	WEAVING VOLUME-LENGTH RELATION MINIMUM DESIGN DESIGNATED BY CHART CURVES	
	FREEWAY PROPER	C-D ROADS & INTERCHANGES
A	I-II	II-III
B	II	III
C	II-III	III-IV
D	III-IV	IV
E	IV-V	V

FIGURE 41.—Design chart for weaving sections.

capacity (curve V). The operating speed associated with curve V is about 20 to 30 mph, whereas that associated with curve III is about 40 to 45 mph. Note that the weaving volume $(\overline{W} + \overline{W'})$ is in terms of passenger cars per hour (pcph). The table in the lower part of the figure indicates the curves to be used in design for freeway facilities to attain a desired level of service. Where two curves are used to define a range, the lower roman numeral is the desirable level and the higher one the minimum. In the case of two-sided weaves, however, the lower roman numeral should be used as a minimum to reflect the added disruption to flow caused by the crossing of the mainline traffic.

TABLE 15.—*Per lane service volumes for weaving sections on freeways*

Level of service	$\overline{sv}$—Maximum service volume (ideal conditions)—pcph per lane for number of *basic* lanes (N_b) on major approach roadway		
	$N_b=2$	$N_b=3$	$N_b=4$ or more
A	700	800	850
B	1,000	1,150	1,250
C	1,250	1,350	1,400
D	1,600	1,600	1,600
E	(Varies between 2,000 and 1,600)[1]		

[1] Lane capacities (level *E*) vary with the intensity (frictional effects) of weaving, as related to quality of flow curves I–V in figure W–1. The following relationship (from *Manual* table 7.1) applies:

Weaving Intensity k Factor	*Curve*	*Capacity pcph*
1.0	I	2,000
2.6	II	1,900
3.0	III	1,800
3.0	IV	1,700
3.0	V	1,600

Notes: 1. Table compiled from *Manual* table 9.1.
2. $\overline{sv}$ for level *C* predicated on PHF 0.83, for level *D* on PHF between 0.83 and 0.91, and for level *E* on PHF between 0.91 and 1.00.

Table 15 is a supplement to figure 41. It provides a guide to the selection of the maximum service volume ($\overline{sv}$) for each level of service to insert into the width determination formula. The values shown were derived by using table 9.1 of the *Manual* and applying average peak-hour factors as noted. This level of accuracy seemed compatible with that of the weaving analysis methodology.

Multiple weaving analysis represents a more complex operation. Two-segment multiple weaving sections are a common occurrence in modern freeway design. Three-segment multiple weaving sections are not common and the emphasis here is placed, therefore, on two-segment sections.

The major differences in the analysis of multiple weaving sections, as compared to simple weaving sections, are (a) the check to determine if multiple weaving is present, and if it is, (b) the proportion of the weaving which takes place over the whole section (long weave) which should be assigned to each segment for the purpose of analysis. This is done by assuming that the portion occurring in each segment is proportioned to the length of each segment compared to the total length of the section. The check for multiple weaving is made by determining if the "long-weave," taken over the entire length of the section, is within the realm of weaving. If it is, then the procedure is to analyze the section for multiple weaving. If it is not found to be within the realm of weaving, then in the case of 2EN–EX the second segment is analyzed as a simple weave. The upstream entrance volume which proceeds through on the freeway becomes part of the outer flow and that portion of the freeway volume departing at the downstream exit is included as part of the weaving volume in the second segment. In the case of EN–2EX the weaving in segment one is analyzed as simple weaving in a similar manner.

As an aid in multiple weaving analysis computation, work sheets have been prepared for the two cases which can occur under two-segment multiple weaving. These are 2EN–EX and EN–2EX as shown in figures 46 and 48, respectively. The top of the worksheet provides room for project identification, problem summary, derivation of truck adjustment and *k* factors, and proportioning of the "long-weave." The two segments are separated and provided with segment weaving diagrams so that each segment may be analyzed separately. First, length requirements are determined by selecting the appropriate curves from the tabulation on figure 41 and entering the graphs in figure 41 with the weaving volumes established from the segment weaving diagram. Finally, width requirements are calculated using the *k* value appropriate to the weave (that is, "*k*" for the entire section for the "long weave" and "*k*" for the segment in which the primary weave occurs). Service volumes to be inserted are selected from table 15.

In either multiple-weaving case, the only significant difference from the simple weave concerns the handling of the "long weave." Where a segment is handling the portion of the "long weave" along with the primary weave, they are shown separately in the segment diagram. Since a portion of one of the long weave volumes is presented in both types of weaving volumes, it must be shown as a dashed arrow in one case so as to remind the designer not to double count the effect of that volume. Use of the charts and worksheets for 2-segment weaving sections are demonstrated in the following problems.

The rationale for a 3-segment weaving section is the same as for a 2-segment weaving section; that is, the weaving movements are proportioned over three rather than two segments, utilizing three *k* factors.

In the case of multiple weaving sections embodying 2-sided weaves, as may be the situation on collector/distributor roads (C–D roads), the included work forms, figures 46 and 48, may be adapted for such conditions by orienting the weaving configuration to the diagram on the forms.

ANALYSIS TECHNIQUES

Problem 30

An initial design layout of a section of modern freeway with three basic lanes has resulted in a one-sided weaving section 1,800 feet long. The design hourly volumes of the various movements are $F = 3{,}600$, $W = 1{,}050$, $W' = 950$, and $F' = 100$. Truck traffic is negligible. The number of lanes required to achieve level of service *C* must be determined. In addition, an alternative design would shift the ramp 2,700 feet downstream ($L = 4{,}500$ feet) leaving all other conditions the same. The designer must determine the effect this shift would have on the lane requirements at level of service *C*.

Solution For the first case $L = 1{,}800$ feet. No correction to pcph is necessary because truck traffic is assumed negligible. From table 15 read for $N_b = 3$, level of service *C*, and $\overline{sv} = 1{,}350$ pcph. Read $k = 3.0$. Apply the simple weaving formula for $F = 3{,}600$, $W = 1{,}050$, $W' = 950$, and $F' = 100$. Thus $V = 5{,}700$. The result is $N = 5.6$ lanes. Thus 6 lanes would be required. The shift of the ramp downstream results in an increased weaving distance of $L = 4{,}500$ feet. Enter figure 41 with $L = 4{,}500$ feet and $\overline{W + W'} = 2{,}000$ pcph; read $k = 1.7$ (interpolating between curves). Using the same formula, *N* is calculated as 4.7 lanes. Thus, 5 lanes would be required. An increase of 2,700 feet in the length of the weaving section has allowed the reduction by one of the number of lanes required due to weaving.

Problem 31

Alternative designs are being considered for a high-type freeway in a rural area on which it is intended to maintain a level of service *A*. One design would result in a one-sided weaving section while the other would create two-sided weaving. Grades are near level and trucks comprise 8 percent of the traffic. The alternative arrangements under study are shown in figure 42 along with the pertinent data. The designer must determine the length and width required for each alternative.

Solution From table 4 read $F_T = 0.93$. For the two-sided weaving section $\overline{W + W'} = (850 + 200)/0.93 = 1{,}130$ pcph. From figure 41 determine the appropriate curves to use to achieve level of service *A*. Although curve II is shown as a minimum and I as desirable, the higher curve (curve I, in this case) should be used as the minimum for a two-sided weaving section. Enter figure 41 with 1,130 pcph, intersect curve I and read $L = 2{,}800$ feet. Apply the simple weaving formula with $k = 1.0$. Determine $sv = 700$ pcph from figure 15. Translating SV to vph yields 650 vph. $V = 1{,}850$ vph and $W' = 200$ vph. *N* is calculated as 2.8 lanes. Thus, 3 lanes are required.

For the alternative one-sided weaving case $\overline{W + W'} = (450 + 350)/0.93 = 860$ pcph. Enter figure 41 with 860 pcph and intersect curves I (desirable) and II (minimum), reading $L_{minimum} = 1{,}200$ feet and $L_{desirable} = 2{,}100$ feet. Although a length of 1,200 feet is indicated as sufficient on the basis of these relationships to maintain level *A* operation, a length approaching the desirable should

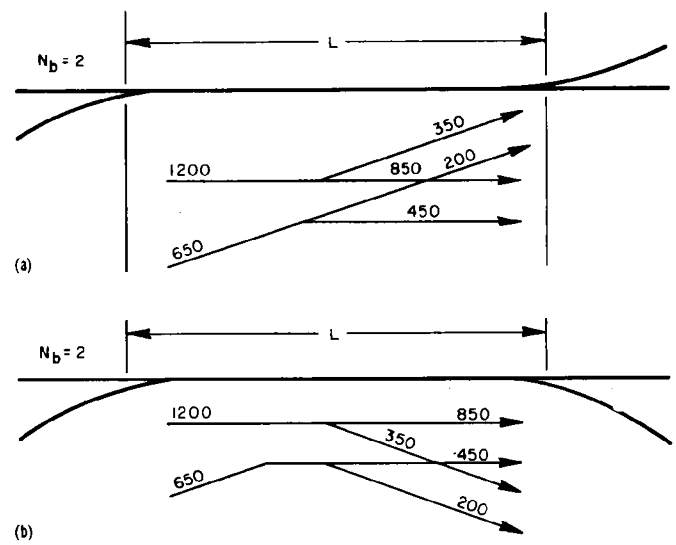


FIGURE 42.—Problem 31 illustrated.

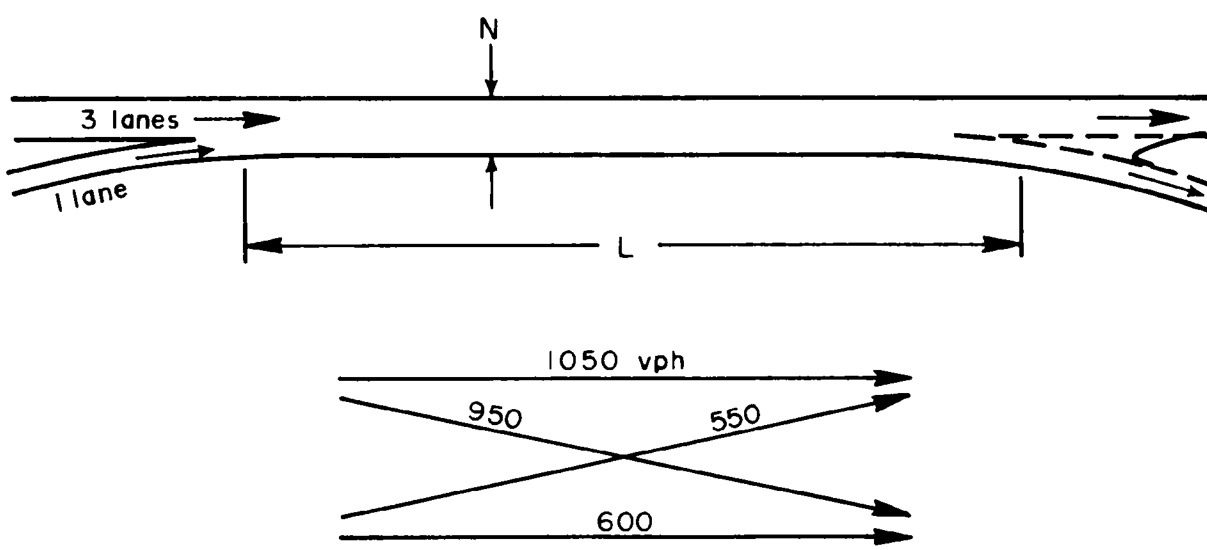


FIGURE 43.—Problem 32 illustrated.

be provided normally on a modern, high-speed and/or high-volume freeway. This control is predicated on the principle that merging and diverging maneuver areas on high-type facilities desirably should not be overlapped. On such a facility, a merging or diverging maneuver area is considered to be 900 or 1,000 feet long. Thus, two maneuver lengths placed end-to-end produce a preferable minimum length for the EN–EX ramp sequence of 1,800 to 2,000 feet. Using 1,800 feet as the length, the k factor is read from figure 41 as 1.3. The number of lanes is then calculated as 3.0.

Problem 32

A one-sided weaving section is formed on a freeway by an EN–EX ramp pattern. The section is preceded by approximately $1\frac{1}{4}$ miles of 3 percent upgrade. Trucks are 4 percent of the traffic throughout. The design uses 12-foot traffic lanes and 10-foot shoulders. Traffic volumes are indicated in figure 43. The minimum length and width requirements must be determined for level of service C. As a further check the width required at capacity should be determined for the calculated length.

Solution The length of section used in determining the truck factor includes the length of the weaving section. Since the length of the weaving section has not been determined a preliminary calculation must be made. The volumes involved are $F = 1{,}050$ vph, $W = 950$ vph, $W' = 550$ vph, and $F' = 600$ vph. $\overline{W + W'} = 1{,}500$ vph. Table 4 indicates that for a $1\frac{1}{4}$ mile, 3 percent grade, $F_T = 0.76$ and $\overline{W + W'}$ becomes 1,975 pcph. Figure 41 indicates that for level of service C a length of about between 1,500 and 3,000 feet is required. Thus assume the length of grade affecting the weaving section is $1\frac{3}{4}$ miles. From table 4, $E_T = 9.5$ and $F_T = 0.75$. Thus, $\overline{W + W'} = 1{,}500/0.75 = 2{,}000$ pcph. Entering figure 41 with $\overline{W + W'} = 2{,}000$ pcph and intersecting curves II and III, read $L_{\text{minimum}} = 1{,}500$ feet and $L_{\text{desirable}} = 3{,}050$ feet. Thus, in this case (lengthy grade preceding the section) the addition of the length of the weaving section had little offset in determining length requirements.

The width requirements are calculated using the simple weaving formula and $k_{\text{minimum}} = 3$ or $k_{\text{desirable}} = 2.6$. SV $= 1{,}350 \times 0.75 = 1{,}015$. Calculations are N (minimum length) $= \dfrac{3{,}150 + 2(550)}{1{,}015} = 4.2$ lanes; and N (desirable length) $= \dfrac{3{,}150 + 1.6(550)}{1{,}015} = 4.0$ lanes. At capacity, $k = 3.0$ (curve III). The $\overline{sv}$ is read from the footnote of table 15 at 1,800 pcph; SV $= 1{,}800 \times 0.75 = 1{,}350$ vph. N is calculated as 3.2 lanes.

Problem 33

A section of modern freeway including two successive entrances followed by an exit is under design as shown in figure 44. The design hourly volumes shown in the figure include 5 percent trucks. The first segment of the section is level. The second segment has a 1 percent downgrade. The basic number of lanes on the freeway has been designated as 3. Level of service C is to be provided. The designer must determine:

a. Whether simple or multiple weaving is present;
b. Whether the length or lengths of weaving section are adequate; and
c. The number of lanes required on the freeway segments to provide level of service C.

Solution The potential multiple weaving section is of the type 2EN–EX. To check for multiple weaving, determine whether the "long weave" (that is, the volumes which would weave over both segments of the section) is so large as to actually be within the realm of weaving. The total length ($L_1 + L_2$) is 4,000 feet. Figure 44 indicates the long weave to be 150 + 450 vph = 600 vph. The truck equivalent is 2 and $F_T = 0.95$ (determined from table 4 for level roadway and $T = 5$ percent). Thus, $\overline{W + W'} = 600/0.95 = 635$ pcph. Enter figure 41 with $\overline{W + W'} =$

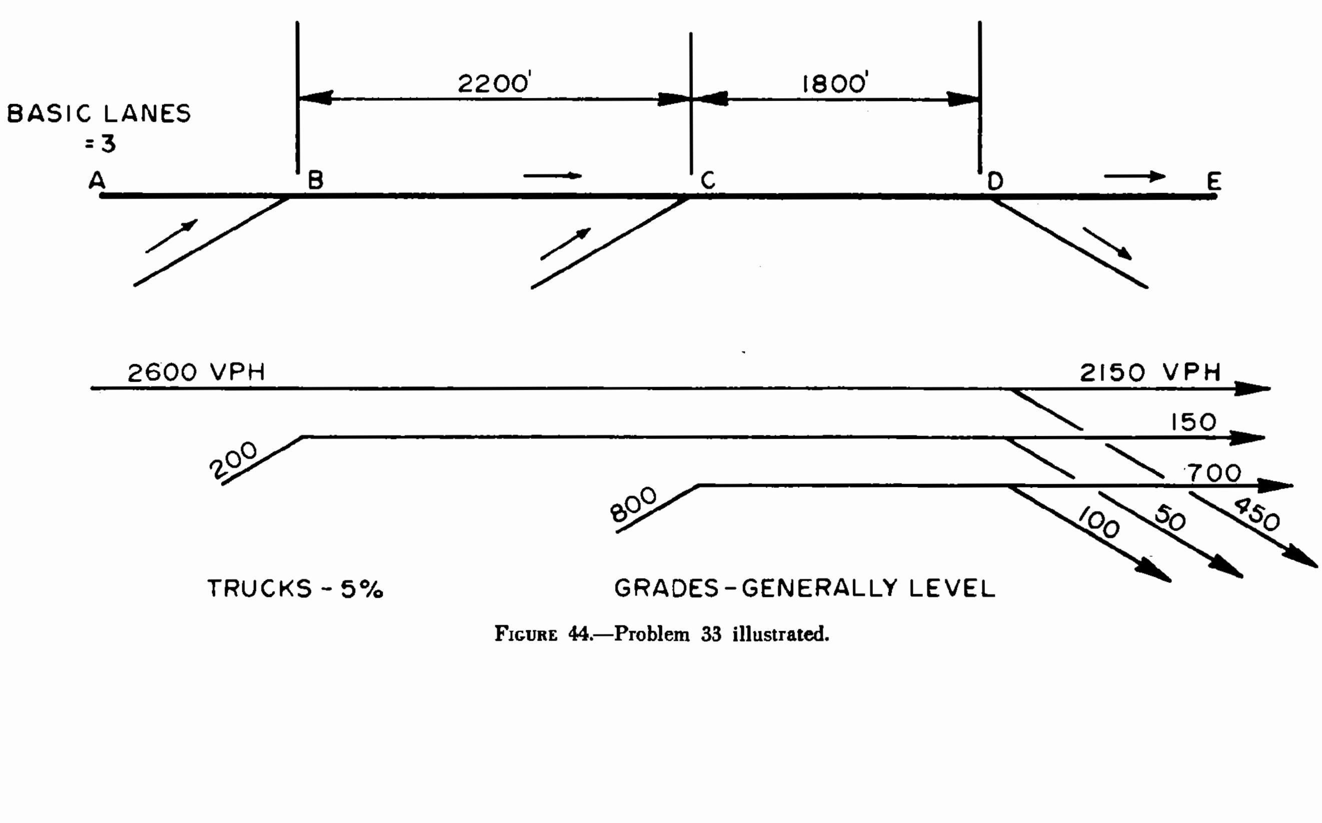
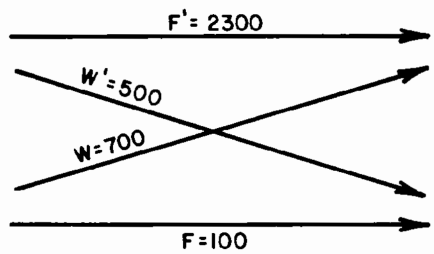


FIGURE 44.—Problem 33 illustrated.

635 pcph and L = 4,000 feet. The intersection of the lines falls outside the realm of weaving. Conclude that multiple weaving would not be present and that the second segment should be analyzed as a simple weaving problem. The weaving diagram for the second segment then resolves to the following:



$\overline{W + W'}$ = 1,200 vph and $\overline{W + W'}$ = 1,200/0.95 = 1,265 pcph. The table in figure 41 indicates that at level of service C, a length corresponding to a value between curves II (desirable) and III (minimum), should be used. The chart in figure 41 is entered with $\overline{W + W'}$ = 1,265 pcph and curves I, II and III are intersected to read:

$L_{minimum}$ (III) = 800 feet;
$L_{desirable}$ (II) = 1,800 feet; and
$L_{out\ of\ realm}$ (I) = 3,300 feet.

The k value for L = 1,800 is read as 2.6. $\overline{sv}$ is read from table 15 for N_b = 3 and level of service C as $\overline{sv}$ = 1,350 pcph. Converting to vph gives $\overline{sv}$ = 1,350 × 0.95 = 1,285 vph. N is calculated for W' = 500 vph, V = 3,600 vph, k = 2.6, and sv = 1,285 vph as 3.4 lanes. Thus 4 lanes would be required in this section.

Problem 34

A section of modern freeway including two successive entrances followed by an exit is under design as shown in figure 45. The design hourly volumes shown in the figure include 5 percent trucks. The section is on a 2-percent upgrade which is preceded by a 2-percent downgrade. The basic number of lanes on the freeway approach has been designated as 3. Level of service C is to be provided. The designer must determine:

(a) whether simple or multiple weaving is present;
(b) whether the length or lengths of weaving section are adequate; and
(c) the number of lanes required on the freeway segments to provide level of service C.

Solution As in problem 33 this is a potential multiple weaving section of the type 2EN–EX. The check for multiple weaving is over a length of 4,000 feet. E_T = 5 and F_T = 0.83 for a +2.5-percent grade of 2,500 feet, as determined from table 4. The "long weave" is 1,600 vph/0.83 = 1,930 pcph. Enter figure 41 with $\overline{W + W'}$ = 1,930 pcph and L = 4,000 feet. The k value is read as 1.4. Thus the section is to be analyzed for multiple weaving. Worksheet 2EN–EX is employed as shown in figure 46. Truck equivalents and factors are determined for each segment as well as the overall section and entered in the appropriate place. The k_2 value is determined to be 2.9 from figure 41 with $\overline{W}$ = 2,050 pcph and L = 2,500 feet. The "long weave" is proportioned between each segment and then each segment is analyzed separately as indicated on the worksheet:

$$AD_1 = 850\frac{(1{,}500)}{(4{,}000)} = 320\text{ vph}; AD_2 = 850 - 320 = 530\text{ vph}.$$

$$BE_1 = 500\frac{(1{,}500)}{(4{,}000)} = 190\text{ vph}; BE_2 = 500 - 190 = 310\text{ vph}.$$

Enter figure 41 with $\overline{W_a + W'_a}$ = 586 pcph and intersect curves I, II and III to read:

$L_{minimum}$ (III) = 300 feet
$L_{desirable}$ (II) = 750 feet
$L_{out\ of\ realm}$ (I) = 1,200 feet

Thus L = 1,500 is sufficient. $\overline{sv}$ for level of service C and N_b = 3 is read from the table in figure 41 as 1,350

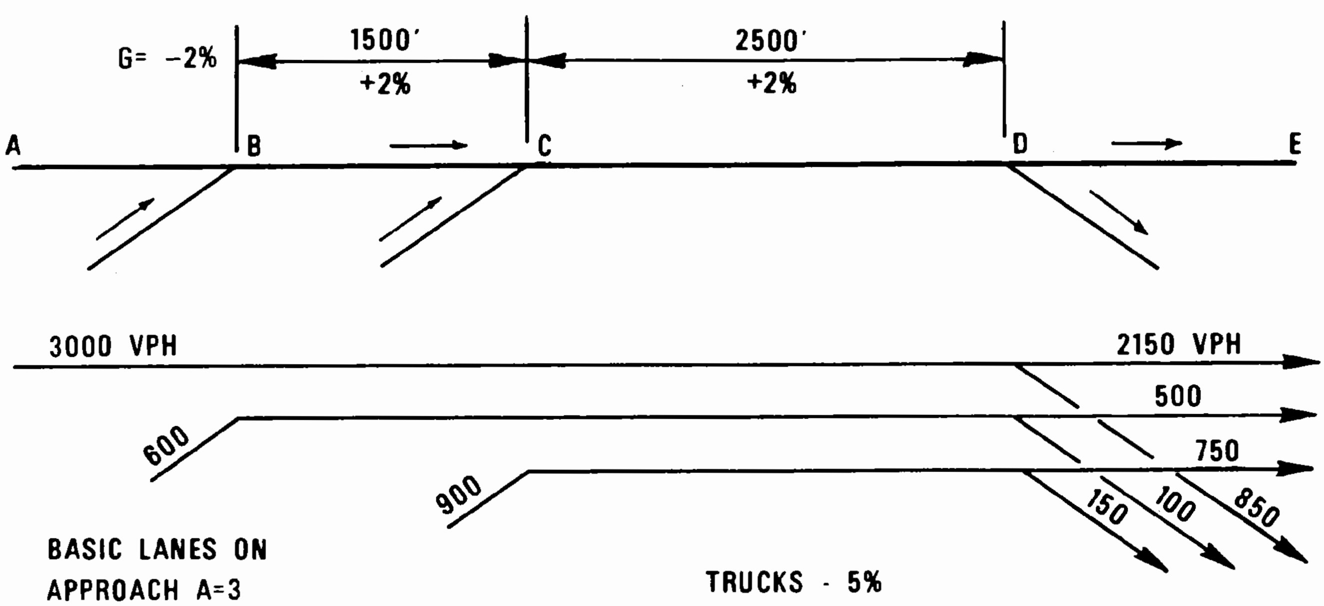
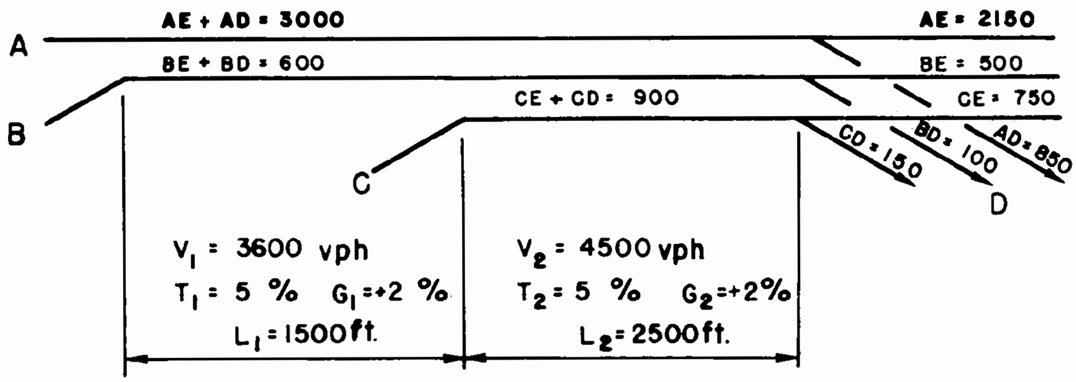


FIGURE 45.—Problem 34 illustrated.

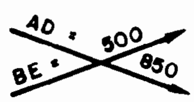
PROJECT BELTWAY PROBLEM 34 BY VFS DATE 7/13/74
LOCATION FREMONT AVENUE INTERCHANGE CHECKED CLK

$N_b = 3$
L of S = C

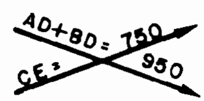


$V_1 = 3600$ vph
$T_1 = 5$ % $G_1 = +2$ %
$L_1 = 1500$ ft.

$V_2 = 4500$ vph
$T_2 = 5$ % $G_2 = +2$%
$L_2 = 2500$ ft.



$E_{T1} = 4$ $F_{T1} = 0.87$
$W_{12} = 1350$ vph
$\overline{W}_{12} = \frac{W_{12}}{F_{T12}} = 1630$ pcph
$k_{12} = 1.4$

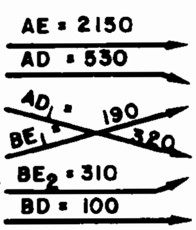


$E_{T2} = 5$ $F_{T2} = 0.83$

$E_{T12} = 5$ $F_{T12} = 0.83$
$W_2 = 1700$ vph
$\overline{W}_2 = \frac{W_2}{F_{T2}} = 2050$ pcph
$k_2 = 2.9$

$AD_1 = AD \frac{L_1}{L_1 + L_2} = 320$

$BE_1 = BE \frac{L_1}{L_1 + L_2} = 190$



$W_a + W'_a = 510$ vph

$\overline{W_a + W'_a} = \frac{W_a + W'_a}{F_{T1}} = 586$ pcph

LENGTH REQUIREMENTS (MANUAL FIG. 7.4)

$\Sigma(\overline{W + W'}) = \overline{W_a + W'_a} = 586$ pcph

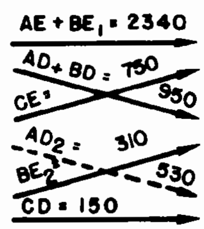
L_1 minimum = 300 ft (curve III)
desirable = 750 ft (curve II)
out-of-realm = 1200 ft (curve I)

WIDTH REQUIREMENTS

$\overline{SV}_1 = 1350$ pcph $SV_1 = \overline{SV}_1 \times F_{T1} = 1175$ vph

$$N = \frac{V_1 + (k_{12} - 1) W'_a}{SV_1} = 3.1 \text{ lanes}$$

$AD_2 = AD - AD_1 = 530$
$BE_2 = BE - BE_1 = 310$



$W_b + W'_b = 1700$ vph

$\overline{W_b + W'_b} = \frac{W_b + W'_b}{F_{T2}} = 2050$ pcph

$W'_c = BE_2 = 310$ vph

$\overline{W}'_c = \frac{W'_c}{F_{T2}} = 375$ pcph

LENGTH REQUIREMENTS (MAN. FIG. 7.4)

$\Sigma(\overline{W + W'}) = \overline{W_b + W'_b} + \overline{W'_c} = 2425$ pcph

L_2 minimum = 2050 ft (curve III)
desirable = 4000 ft (curve II)
out-of-realm = 8000 ft (curve I)

WIDTH REQUIREMENTS

$\overline{SV}_2 = 1350$ pcph $SV_2 = \overline{SV}_2 \times F_{T2} = 1120$ vph

$$N = \frac{V_2 + (k_2 - 1) W'_b + (k_{12} - 1) W'_c}{SV_2} = 5.4 \text{ lanes}$$

NOTES: (1) W_a, W_b, W_c = LARGER OF TWO WEAVING MOVEMENTS IN EACH WEAVING PAIR
(2) W'_a, W'_b, W'_c = SMALLER OF TWO WEAVING MOVEMENTS IN EACH WEAVING PAIR
(3) BAR VALUES $\overline{SV}$, $\overline{W}_a$ ETC ALL IN pcph
(4) GENERAL FORMULA FOR $N = \frac{V + \Sigma(k-1)W'}{SV}$

FIGURE 46.—Analysis worksheet for problem 34.

pcph. This becomes 1,350 × 0.83 = 1,175 vph. Solving for N with V_1 = 3,600 vph, k_{12} = 1.4, $\overline{W}_2'$ = 190 vph and SV = 1,175 vph gives 3.1 lanes Thus a 3-lane section should be sufficient.

The second segment is handled in a similar manner. The additional item worthy of note is that two weaving flows are shown in the segment weaving diagram. The first represents that resulting from ramp C volumes. The second is that portion of the "long weave" remaining to be completed in the second segment. The movement AD_2 is shown dashed so that it is not double counted (it is already included in AD) during the calculations. The length requirements are determined in the same manner as described for the first segment. The total weaving volume is the sum of AD, BD, and BE_2. The resulting minimum, desirable, and "out-of-realm" lengths are 2,050 feet, 4,000 feet, and 8,000 feet, respectively.

The width requirements are determined using V_2 = 4,500 vph, k_2 = 2.9, k_{12} = 1.4, $\overline{W}_b'$ = 750 vph, $\overline{W}_c'$ = 310 and SV = 1,285 vph. The calculation shows N = 5.4 lanes. Thus 6 lanes would be required. Further analysis of the total freeway involved, as well as the merging and diverging ramp analysis, would result in guidelines to follow in the apparent lack of lane balance in proceeding from the first segment with 3 lanes to the second segment with 6 lanes required.

Problem 35

A section of modern freeway including an entrance followed by two successive exits is under design as shown in figure 47. The design hourly volumes shown in the figure include 5-percent trucks. The first segment of the section is level. The second segment is along a 2.5-percent upgrade. The basic number of lanes on the freeway approach has been designated as 4. Level of service C is to be provided. The designer must determine:

a. Whether simple or multiple weaving is present;
b. Whether the length or lengths of weaving section are adequate; and
c. The number of lanes required on the freeway segments to provide level of service C.

Solution This section has a potential multiple weaving situation of the type EN–2EX. The check is again made for multiple weaving by testing to determine if the long weave is "out-of-realm." In this case the volumes (400 + 1,300 = 1,700 vph) are just sufficient to be within the realm of weaving, thus the section is to be analyzed as for multiple weaving. The completed work sheet is shown as figure 48.

The "long weave" is shown distributed between the two segments in proportion to their lengths. In this case the first segment is shown handling two types of weaving maneuvers. The first is that which enters at the first ramp and continues through, crossing that approaching the section on the freeway and exiting at the first exit. The second weaving maneuver is that portion of the "long weave" from the freeway to the second exit which is assumed to occur in the first segment. The volume BE_1 is shown as a dashed line to avoid double counting during calculations (it is included in BE). The length requirements determined from figure 41 with values of $\overline{W} + \overline{W'}$ = 2,165 pcph for the first segment are:

$L_{1,\ \text{minimum}}$ (III) = 1,700 feet
$L_{1,\ \text{desirable}}$ (II) = 3,400 feet
$L_{1,\ \text{out of realm}}$ (I) = 6,700 feet

Since this section of the freeway has been designated as 4 basic lanes the service volume $(\overline{sv})$ is read from table 15 as 1,400 pcph. The lanes required are calculated as 4.3. Thus 4 lanes would be required. The weaving in segment two consists of the remaining portion of the long weave. The resulting length requirements determined from figure 41 with $\overline{W} + \overline{W'}$ = 970 pcph are:

$L_{2,\ \text{minimum}}$ (III) = 500 feet
$L_{2,\ \text{desirable}}$ (II) = 1,150 feet
$L_{2,\ \text{out of realm}}$ (I) = 2,100 feet

The corresponding width requirement was calculated as 3.9 lanes. Thus 4 lanes would be required.

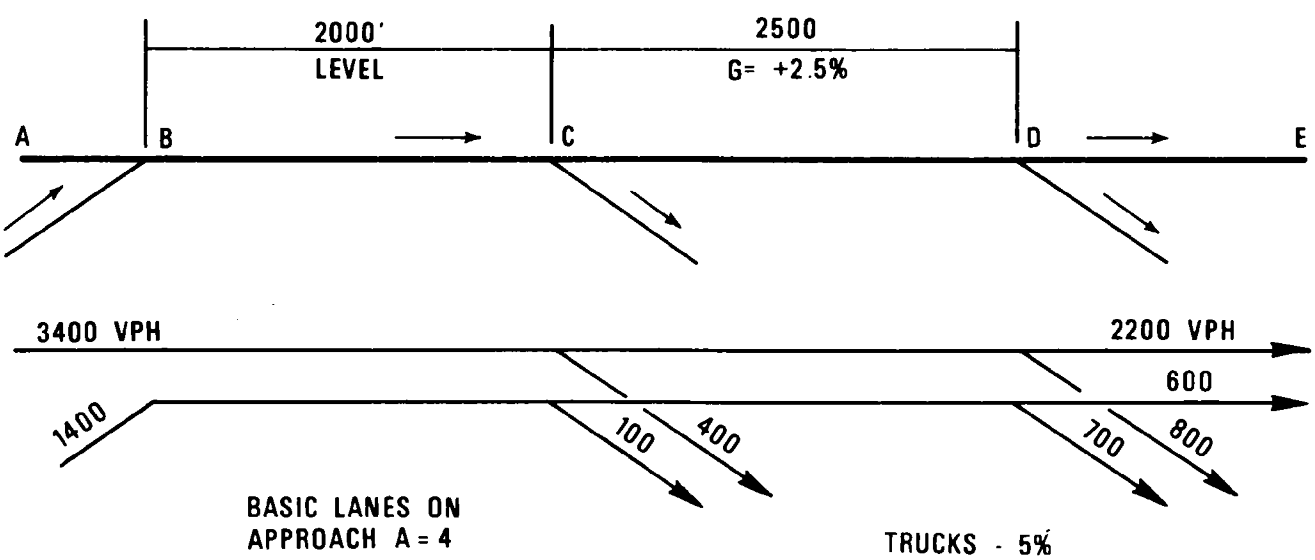
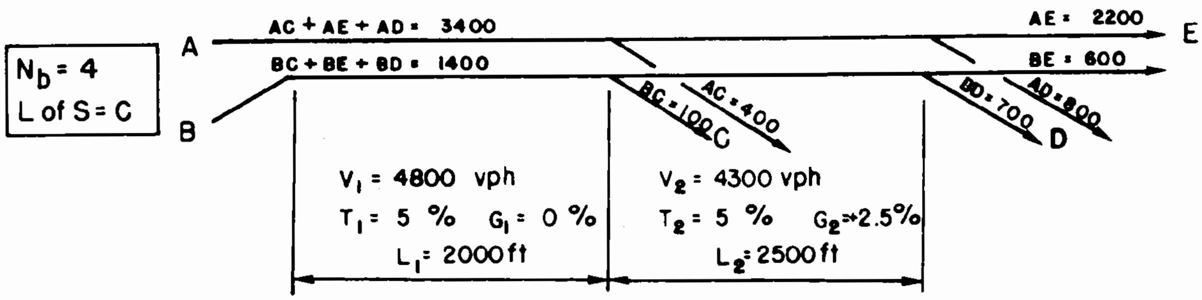
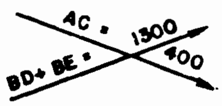


Figure 47.—Problem 35 illustrated.

PROJECT BELTWAY PROBLEM 35 BY VFS DATE 7/13/74
LOCATION DUKE STREET INTERCHANGE CHECKED CLK



E_{T1} = 2 F_{T1} = 0.95 E_{T2} = 6 F_{T2} = 0.80 E_{T12} = 5 F_{T12} = 0.83



W_1 = 1700 vph

$\overline{W_1} = \frac{W_1}{F_{T1}}$ = 1790 pcph

k_1 = 2.9

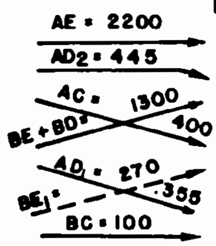
W_{12} = 1400 vph

$\overline{W_{12}} = \frac{W_{12}}{F_{T2}}$ = 1750 pcph

k_{12} = 1.4

$AD_1 = AD \frac{L_1}{L_1+L_2}$ = 355

$BE_1 = BE \frac{L_1}{L_1+L_2}$ = 270



$W_a + W_a'$ = 1700 vph

$\overline{W_a + W_a'} = \frac{W_a + W_a'}{F_{T1}}$ = 1790 pcph

$W_b' = AD_1$ = 355 vph

$\overline{W_b'} = \frac{W_b'}{F_{T1}}$ = 375 pcph

LENGTH REQUIREMENTS (MANUAL FIG. 7.4)

$\Sigma(\overline{W+W'}) = \overline{W_a+W_a'} + \overline{W_b'}$ = 2165 pcph

L_1 minimum = 1700 ft (curve III)

desirable = 3400 ft (curve II)

out-of-realm = 6700 ft (curve I)

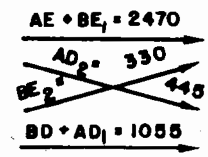
WIDTH REQUIREMENTS

$\overline{SV_1}$ = 1400 pcph $SV_1 = \overline{SV_1} \times F_{T1}$ = 1330 vph

$N = \frac{V_1 + (k_1-1)W_a' + (k_{12}-1)W_b'}{SV_1}$ = 4.3 lanes

$AD_2 = AD - AD_1$ = 445

$BE_2 = BE - BE_1$ = 330



$W_c + W_c'$ = 775 vph

$\overline{W_c + W_c'} = \frac{W_c + W_c'}{F_{T2}}$ = 970 pcph

LENGTH REQUIREMENTS (MAN. FIG. 7.4)

$\Sigma(\overline{W+W'}) = \overline{W_c+W_c'}$ = 970 pcph

L_2 minimum = 500 ft (curve III)

desirable = 1150 ft (curve II)

out-of-realm = 2100 ft (curve I)

WIDTH REQUIREMENTS

$\overline{SV_2}$ = 1400 pcph $SV_2 = \overline{SV_2} \times F_{T2}$ = 1120 vph

$N_2 = \frac{V_2 + (k_{12}-1)W_c'}{SV_2}$ = 3.9 lanes

NOTES: (1) W_a, W_b, W_c = LARGER OF TWO WEAVING MOVEMENTS IN EACH WEAVING PAIR
(2) W_a', W_b', W_c' = SMALLER OF TWO WEAVING MOVEMENTS IN EACH WEAVING PAIR
(3) BAR VALUES ($\overline{SV}$, $\overline{W}$ ETC.) ALL IN pcph
(4) GENERAL FORMULA FOR $N = \frac{V + \Sigma(k-1)W'}{SV}$

FIGURE 48.—Analysis worksheet for problem 35.

Chapter VI

DETERMINATION OF NUMBER AND ARRANGEMENT OF LANES ON FREEWAYS

The preceding chapters deal with the techniques of capacity analyses. Given the design hourly volumes and the number of traffic lanes together with related geometrics, the levels of service can be established; or, given the design hourly volumes and the levels of service desired, the number of lanes can be determined. The procedures presented provide the mechanics for performing capacity analyses on any component of a freeway facility. The determination of the number and arrangement of lanes along a freeway route is a most significant aspect of design. The available "tools" are fully applied here, but in consideration of the total facility, which functions as a system in itself, capacity analyses provide only a partial answer in the overall determination of the number and arrangement of lanes.

PEAK-HOUR TRAFFIC CONSIDERATIONS IN DESIGN

Although significant advances are continuing to be made in the methods of forecasting and assigning traffic volumes, the fact remains that such traffic volumes are predicated on developments assumed to take place some 20 or 30 years in the future. Accordingly, the estimation of future traffic volumes involves the human element and considers certain events over which the planner and designer have no control. The number of lanes on a freeway, therefore, cannot be properly determined by relating highway capacity to estimated traffic volume alone. It entails much more than that.

Design hourly volumes normally represent the repeated morning and evening peaks—peaks resulting largely from home-to-work and work-to-home movements. These are considered of primary importance in design. But there are several other situations which, on urban freeways, may cause different and sometimes unduly high traffic peaks:

a. unforeseen concentration of development
b. holiday or weekend travel
c. special events
d. stage construction or partial development of freeway networks
e. accidents
f. extensive maintenance operations.

Unforeseen Concentration of Development

Although, on an area-wide basis, travel may be forecast with some degree of accuracy, there is no assurance that in any one locale, the city traffic would materialize as predicted 20 to 30 years in the future. Unforeseen concentration of development, such as a large industrial plant, a commercial center, or a shift in residential development, could produce volumes or patterns of traffic on some sections of freeway or interchanges in considerable variance with the over-all forecast.

Holiday or Weekend Travel

In many instances holiday or weekend travel (particularly in the summer season) produces traffic volumes on some facilities substantially different from those accommodated during normal weekday periods. For example, at a given interchange during the evening or morning peak there may be a heavy turning movement in one quadrant, whereas on a holiday or a weekend the predominant flow may be in another quadrant or it may be a through movement. Thus, in such cases, the number or the arrangement of lanes may not be fully adequate. Furthermore, shopping trips in conjunction with some holidays and other special shopping periods may produce significant variations in traffic patterns.

Special Events

Unusual traffic peaks occasionally develop in cities due to special events. Some of these can be predicted, others cannot. Some are repetitive from time to time. Some entail 1 day, others may last for a week or more. The occasions may be sports events, political or religious gatherings, visits of dignitaries, special ceremonies, etc. Peaks caused by special events generally produce patterns and volumes of traffic quite different from those generated during normal weekdays or on weekends or holidays. Thus the number and arrangement of lanes normally provided may not be capable of handling special events traffic.

Stage Construction—Partial Development of Freeway Networks

Generally, it is not possible to provide, through one construction effort, a complete system of freeways in a city. Construction of the whole system must take time. Urban freeway developments frequently are programmed over a period of 20 years or more, and the traffic estimated and assigned to any one facility is that normally based on a full system of freeways. However, when one freeway or section of freeway is constructed at a time, different traffic patterns and loadings occur from those contemplated on the basis of the completed network. Although attempts are often made to account for this, the multitude of variables involved and the likely adjustments in construction schedules make prediction of interim traffic difficult, with the result that the number and arrangement of lanes may be inadequate for some period of time until the remainder of the system is built.

Accidents

Traffic congestion is sometimes the result of accidents. Some accidents occur directly on the freeway during which time it is expected that traffic would slow down and at times stop temporarily. Occupancy of a shoulder by a disabled vehicle, or the use of a shoulder by a stream of vehicles to bypass a disabled vehicle in a traffic lane, may allow for traffic to keep moving at reduced speed. (Provision of fully usable shoulders both on the right and left of each traveled way is a significant feature in this regard.) Congestion also can develop because of accidents which may occur on a parallel street or on some other part of the system. These accidents can reflect on a freeway in a significant manner by traffic altering its course in an attempt to avoid the direct effect of the accident. Such change in volume or pattern of traffic on a freeway may cause its number or arrangement of lanes to be ineffective at such times.

Extensive Maintenance Operations

As in the case of accidents, extensive maintenance operations can change the pattern and volume of traffic on a freeway. Such maintenance operations need not be directly on the freeway but may be on a parallel street or on some other part of the system which can cause traffic to be diverted to or extended over some portions of the freeway. Unless the freeway is capable of handling the adjusted traffic, congestion may have serious effects. Obviously, the problem of determining the proper number of lanes on a freeway is quite complex. Satisfying the requirements for the number of lanes on the basis of the regular morning and evening traffic peaks is only part of the answer. This has become evident through operational experience of freeways. Any one of the variables described above, which may cause traffic occasionally to deviate from the regular peak-hour norm, may produce a congested facility.

In a fairly large city the situations described are constant occurrrences. Frequently two or more combine to produce even more serious effect. At the design stage, it is impossible to predict the place and magnitude of resulting traffic peaks. It is not intended here that this problem be accounted for directly in design or by otherwise modifying the normal design-hour volumes. At this point it is suggested that these situations—which can cause significant changes in volume and pattern of traffic on certain sections of the freeway—be *recognized* in design when determining the number of lanes.

ANALYSIS STEPS IN LANE DETERMINATION

Basically, the design of a freeway should be predicated on the normal peak hours which are repeated daily during the morning (home-to-work) and the evening (work-to-home) periods. Secondly, the design should be checked and adjusted to accommodate any other known peak period, such as holiday or weekend concentrations, or peaks arising from existing or fully planned traffic generators such as shopping centers, sports stadia, etc. And thirdly, the design should be further adjusted in accordance with other events which may cause unduly high traffic loads but which cannot be quantitatively determined in advance. It is in conjunction with this latter step that a technique is developed to largely account for the various situations described—a technique which automatically provides a degree of capacity reserve and operational flexibility.

In the design of freeways the number of lanes is determined primarily through capacity analysis. This is a fundamental control and would be sufficient if the design hourly volume were more or less a fixed value and could be relied upon at all times. Since this is not the case, and various situations are apt to produce unforeseen traffic peaks which differ from the regular peaks, additional design controls in the analysis must be applied in determining width requirements on freeways. The following are the controlling features which, in combination, provide the necessary *analysis steps* for determining lane requirements on freeways:

a. Volume-capacity relations
b. Basic number of lanes
c. Lane balance
d. Special auxiliary lanes

Volume-Capacity Relations

The ratio of the design hourly volume to the design capacity generally indicates the number of lanes required on a freeway. Weekday morning and evening peaks, as well as holiday or weekend peaks, should be accounted for in the analysis. Other concentrations of traffic which may involve unusual shopping peaks or heavy truck operations also should be examined. The procedures for carrying out level of service or capacity analyses were presented in the previous chapters. These are fully utilized in the procedure outlined but within the context of the analysis steps a through d.

Basic Number of Lanes

Fundamental to establishing the number and arrangement of lanes on a freeway is the designation of the *basic number of lanes*. Any route of arterial character should maintain a certain consistency in the number of lanes provided along it. Thus, the basic number of lanes is defined as: a minimum number of lanes designated and maintained over a significant length of it, irrespective of changes in traffic volume and requirements for lane balance. Stating it another way, it is a constant number of lanes assigned to a route, exclusive of auxiliary lanes.

As illustrated in figure 49, the basic number of lanes on freeways is maintained over significant lengths of the routes, as *A* to *B* or *C* to *D*. The number of lanes is predicated on the general volume level of traffic over a substantial length of the facility. The volume considered here is the design hourly volume (normally representative of the a.m. or p.m. weekday peak). Localized variations are ignored,

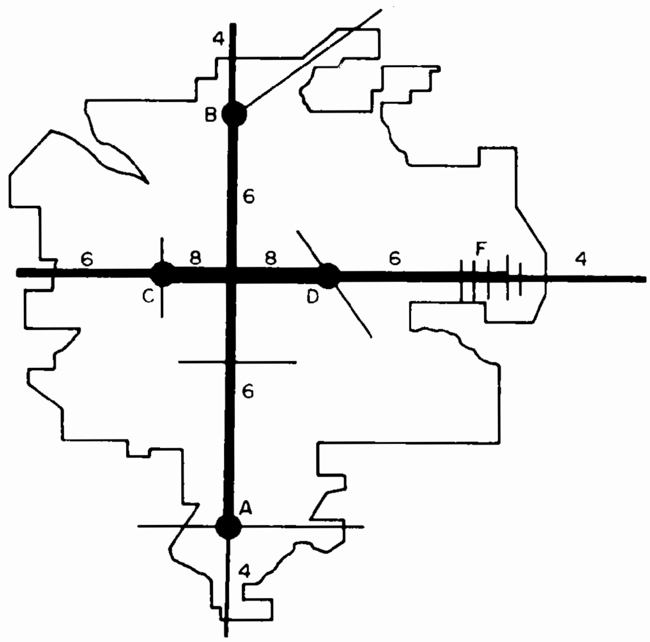


FIGURE 49.—Basic number of lanes—schematic.

so that volumes on short sections below the general level would theoretically have reserve capacity, while volumes on short sections somewhat above the general level would be compensated for by the addition of auxiliary lanes introduced within these sections.

Changes in the basic number of lanes, if required, normally should be effected as follows:

a. *Increase* in the basic number of lanes—where traffic builds up sufficiently to justify an extra lane and where such build-up raises the volume level over a substantial length of the following facility. The increase in width would take place at an interchange by adding a lane to the freeway as a continuation from an entrance ramp.

b. *Decrease* in the basic number of lanes—where traffic reduces sufficiently to drop a basic lane, by continuing the outer lane of the freeway onto the ramp, but only if the exit volume is large enough to cause a general lowering of the volume level on the freeway route *as a whole.*

Required changes in the number of basic lanes are generally accomplished at major junctions, as at freeway-to-freeway interchanges. In the case of an increase in basic lanes, the added lane is introduced via a 2-lane entering ramp at the freeway-to-freeway interchange. In the case of a decrease in basic lanes, the lane is not dropped at the ramp of the freeway-to-freeway interchange, discharging the heavy volume, but via an exit at the following interchange. Another case where the basic number of lanes may be reduced occurs when a series of exits, as in an outlying area of a city, causes the traffic load on the freeway to drop sufficiently to justify the smaller basic number of lanes, as demonstrated at point *F* in figure 49. The selection of the basic number of lanes should be a matter of planning and design policy consistent with the overall system of freeways in the urban area.

Lane Balance

The number of lanes as determined by the volume-capacity relations sometimes changes abruptly at points of entrance or exit. Whereas such changes may be logical in terms of capacity requirements, they are not always appropriate in achieving smooth operating characteristics. To ensure efficient operation and to realize the indicated capacity potential where merging, diverging, and weaving take place, a certain balance of lanes must be maintained. Lane balance should comply with the following relations:

a. At entrances: $N_c = N_f + N_e - 1$ or $N_c = N_f + N_e$ (see explanation of terms under point 2).

b. At exits: $N_c = N_f + N_e - 1$, where:

N_c = number of lanes for the combined flow beyond an entrance or in advance of an exit

N_f = number of lanes on the freeway upstream of an entrance or downstream from an exit

N_e = number of lanes on the entrance or on the exit. At exits this relation provides an "extra lane going away," that is, an *optional lane* on which the driver has a choice of either proceeding on the freeway or on the ramp..

c. At exits the freeway traveled way should not be reduced by more than 1 traffic lane at a time.

The principle of having an extra lane at the point of divergence is a form of "escape hatch," or a device which would tend to "flush" traffic away from the point of divergence because of greater exit capacity than approach capacity. This principle is part of the overall concept discussed (below) in the use of special auxiliary lanes which, when coupled with the other factors for determining the number of lanes, produces significant operational flexibility on freeways in handling traffic loads beyond design hourly volumes.

Lane balance and capacity obviously should be coordinated in determining lane requirements. The two often are

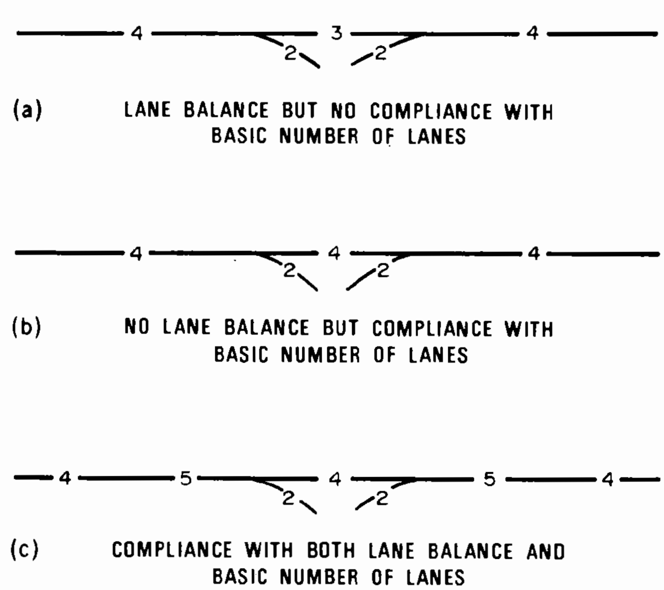


FIGURE 50.—Coordination of lane balance and basic number of lanes.

in harmony or can be brought into harmony by adjusting lane arrangements to satisfy both controls. Likewise, lane balance and basic number of lanes must be coordinated. The fact that at 2-lane exits a traffic lane must be removed from the freeway to achieve lane balance, although to comply with basic number of lanes it is necessary that the number of freeway lanes be maintained beyond 2-lane exits, seems to indicate that the two concepts are in conflict. Actually, this need not be so. The necessary requirements for maintaining both lane balance and basic number of lanes can be met by holding the basic number of lanes and then achieving lane balance by means of "building upon" the basic number of lanes; that is, by adding auxiliary lanes or removing auxiliary lanes from the basic width of the traveled way. Thus, in no case would there be less than the basic number of lanes on the freeway. To further illustrate the two situations discussed and how they can be coordinated to produce a third (desired) arrangement, reference is made to figure 50 where it is assumed that a 4-lane freeway in one direction of travel has a 2-lane exit followed by a 2-lane entrance. In figure 50–A, lane balance is provided but there is no compliance with the basic number of lanes. In figure 50–B, the basic number of lanes is maintained but there is no compliance with lane balance. In figure 50–C, however, there is compliance with both lane balance and basic number of lanes. In this example, the two concepts are combined by merely adding and removing auxiliary lanes to and from the basic number of lanes on the freeway. Here there are 4 lanes on the approach, 4 lanes between the exiting ramp and the entering ramp, and 4 lanes on the leaving end, thus maintaining the basic number of lanes. Lane balance, however, is introduced by adding a fifth (auxiliary) lane for some distance in advance of the exit. Likewise, lane balance is achieved in conjunction with the 2 lanes of merging traffic by the addition of a fifth (auxiliary) lane for some distance beyond the merging end.

Any application of auxiliary lanes, for the purpose described above, must take into consideration an *effective distance* in advance of the exit or beyond the entrance. Where interchanges are closely spaced and there is need for an auxiliary lane to be introduced at an entrance, the added lane should be carried to the exit of the following interchange. Or, an added lane required for an exit should be extended back to the entrance of the previous interchange. Such treatment is demonstrated in figure 51. Frequently an entrance followed by an exit forms a weaving section which requires the use of added width and certain minimum length (entrance to exit) in order to comply with capacity requirements for a weaving section. How-

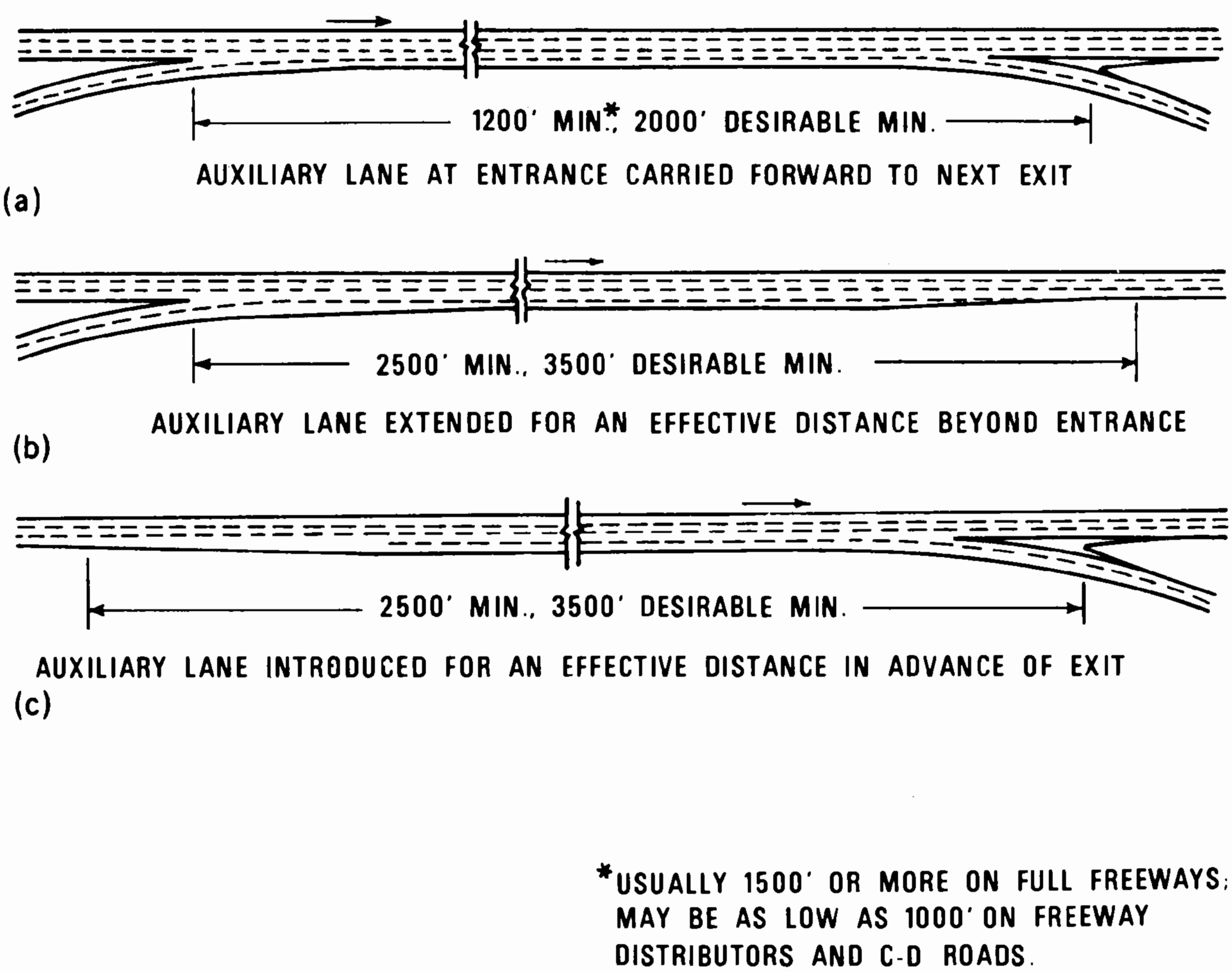


FIGURE 51.—Coordination of lane balance and basic number of lanes through application of auxiliary lanes.

ever, an effective length of auxiliary lane on a full freeway in conjunction with a 2-lane entrance or a 2-lane exit, for purposes of lane balance, preferably should be of the order of 2,000 feet or more and in no case less than 1,200 feet (600-foot maneuver areas back to back), as shown in figure 51–A. These controls govern where weaving capacity requirements alone may show lesser acceptable distances. On a facility serving as an adjunct to a freeway such as a C–D road or a freeway distributor, a normal minimum length of auxiliary lane between an entrance and an exit is taken to be 1,000 feet.

Where interchanges are widely spaced, it might not be feasible or necessary to extend the auxiliary lane from one interchange to the next. In such cases, the auxiliary lane picked up at a 2-lane entrance should be carried along the freeway for an effective distance beyond the merging point, as shown in figure 51–B, or an auxiliary lane introduced for a 2-lane exit should be carried along the freeway for an effective distance in advance of the exit and extended onto the ramp, as shown in figure 51–C. The effective length of the introduced auxiliary lane required under these circumstances is not known precisely, but experience indicates that minimum distances of about 2,500 feet, and desirably 3,500 feet, are needed to produce the necessary operational effect and to develop the full capacity of 2-lane entrances and exits on high-type facilities.

Auxiliary lanes are essential to provide balanced and efficient operation. The objective is to add and remove auxiliary lanes on the freeway as required to account for localized increases and decreases in traffic volumes and for frictional effects therewith, and thereby achieve more uniform level of service. With the basic number of lanes established and maintained, there will be need to add 1 auxiliary lane frequently, and 2 auxiliary lanes occasionally. Thus, considering 4 basic lanes in one direction as normal maximum on a single roadway, characteristic of a large urban area, a total of 5 lanes would occur frequently and a total of 6 lanes occasionally.

Special Auxiliary Lanes

The manner in which the lanes are arranged and finally adjusted on a freeway and on interchanges therewith, may make a vast difference in operational flexibility afforded. Maintaining the basic number of lanes, coupled with proper lane balance through the use of auxiliary lanes, as described above, is one of the primary features in lane arrangement which automatically provides a measure of flexibility to the operation of a freeway. Further ability of a facility to accommodate variations in pattern and volume of traffic can be achieved by more generous lane balance; that is, a final adjustment in the number and arrangement of lanes to furnish an occasional additional lane at strategic locations. This is accomplished as a final step in the analysis of lane determination, following capacity calculations, provision of the basic number of lanes, and establishment of the over-all lane balance. It is a step which independently, and from a broader view, places the facility into balance providing for a more uniform capacity reserve throughout the system, and avoids "weak links" or potential bottlenecks.

In the past, there has been a tendency to underestimate lane requirements, particularly in the use of auxiliary lanes. Too much reliance was placed on a more or less fixed peak-hour condition. This conservative approach sometimes proved to be a false economy approach which from time to time produced an imbalance in the operational characteristics of a freeway. Thus the flexibility and the useful life of a freeway facility can be significantly enhanced, at nominal cost, by an occasional strategically added lane as demonstrated in figure 52.

In figure 52–A(I), the single-lane exit on a 4-lane freeway section has been designed on the basis of capacity-volume relations, with adherence to the basic number of lanes and lane balance at the exit. In this typical case the design-hour volume approaches, but does not exceed, the design capacity of a single-lane exit and, for this reason, the layout in figure 52–A(I) was chosen. Any increase in turning volume during the peak hour beyond that predicted, or even a high rate of flow (surge of traffic) for short periods of time within the hour, particularly in combination with a high incidence of trucks at the same time, could produce a slow-down and serious loss of efficiency in operation. A much "safer" design is shown in figure 52–A(II), using a 2-lane exit and an added (fifth) lane for a distance in advance of the exit. The section of auxiliary lane on the freeway, plus the extra lane on the ramp, can absorb all kinds of variations in the turning traffic pattern. Moreover, the extra width allows traffic to shift lanes and sort itself for the diverging maneuver. It tends to compensate for frictional effects of internal weaving, and what otherwise could be a reduction in speed and an increase in density.

Figure 52–B may be assumed to be an extension of figure 52–A where the exiting traffic further divided into left-turn and right-turn movements through an interchange. The configuration in figure 52–B(I) would match the arrangement in figure 52–A(I), further illustrating the lack of operational flexibility. Figures 52–B(II) and 52–B(III) are alternative extensions of figure 52–A(II). The plan in figure 52–B(II), despite the 2-lane exit from the freeway, may also have its limitation. Should traffc approach capacity, even for short periods, on the 2-lane ramp leaving the freeway, congestion would occur unless there was nearly equal division of traffic at the second fork. Furthermore, should a lane on either of the single-lane ramps become blocked, the effect on operation of the freeway proper could be serious. The arrangement in figure 52–B(III), where an extra lane is provided on one of the diverging roadways, is much more flexible. The added lane can preclude the operational problems described above. The extra lane need not be installed initially in all cases as long as proper provision is made for adding it later.

Another example of increased operational flexibility by the use of an added lane over and above that called for by the fundamental design controls of volume-capacity relations, lane balance and basic number of lanes, is illustrated in figure 52–C. The arrangement in figure 52–C(I) is an example of the situation where the weaving action produced by the design hourly volumes on the successive ramps is just under the capacity requirement for an auxiliary lane. A "safer" design is shown in figure 52–C(II), where an extra (fourth) lane is used between the ramp terminals.

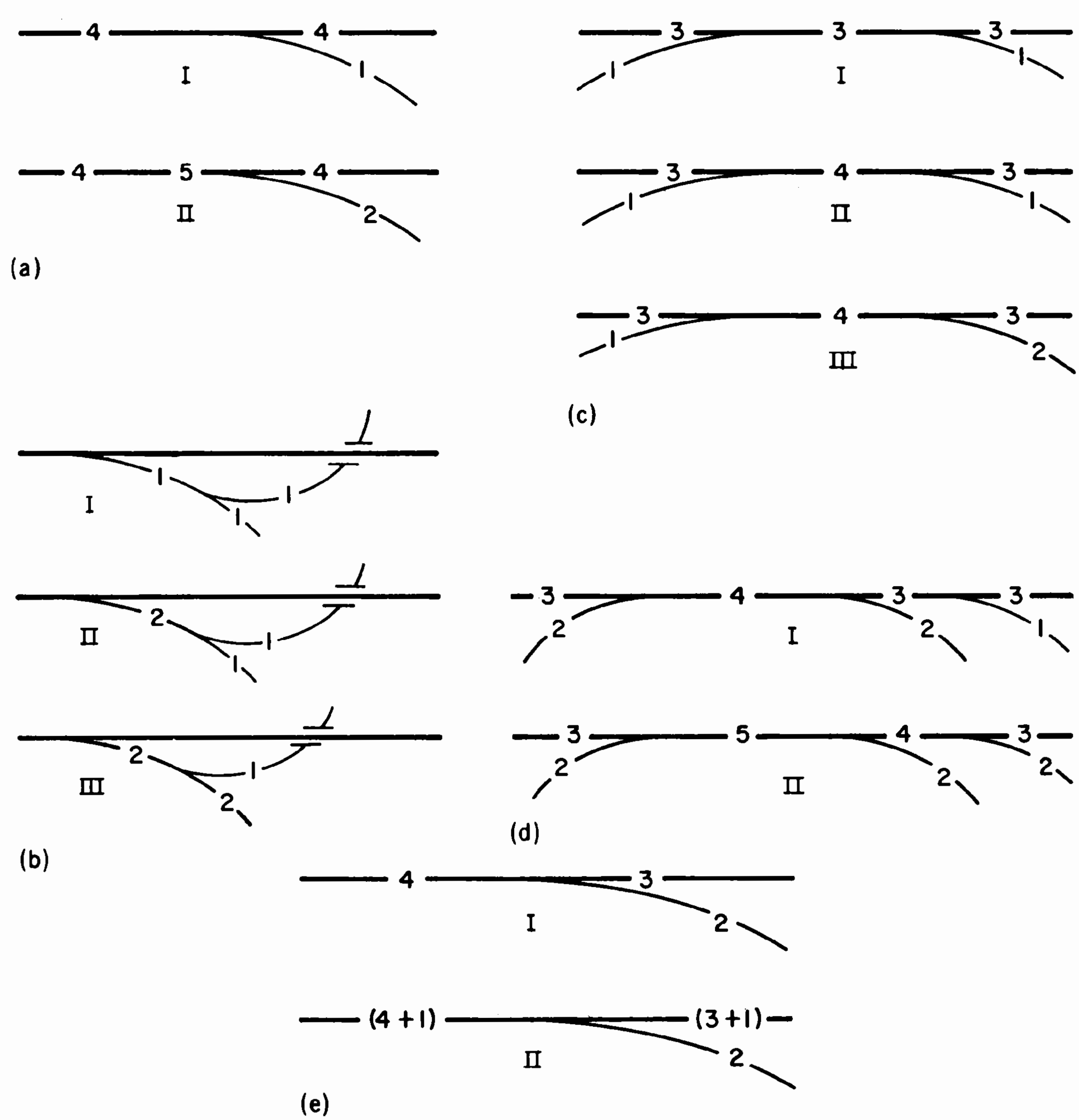


FIGURE 52.—Special use of auxiliary lanes.

The widening significantly raises the potential level of service along this section of the facility by increasing the capacity of the freeway and of the ramp entrance and exit. However, this does not comply with lane balance at the exit as set out previously.

This can be corrected and operational flexibility and capacity further enhanced by continuing the auxiliary lane onto the exit ramp, making it a 2-lane roadway, as shown in figure 52–C(III). A significant feature here, brought about by the full application of lane balance, is the arrangement of having the number of lanes "going-away" equal to 1 more than the number of lanes "approaching." As in the previous example, the extra lane (on the freeway and on the ramp) need not be installed initially as long as adequate provision is made for adding it in the future. Figure 52–D(I) also illustrates a ramp entrance followed by a ramp exit. In this case, an auxiliary (fourth) lane is provided on the freeway to achieve lane balance for the 2-lane entrance and exit. Capacity analysis may show that this is a workable arrangement on the basis of the design hourly volumes, but any changes in traffic pattern or volume through this section could cause the facility to break down. Obviously, making the weaving section 5 lanes wide (adding 2 auxiliary lanes), as shown in figure 52–D(II), provides for higher capacity and increased flexibility to handle possible traffic fluctuations. However, to fully realize these operational characteristics, 1 of the auxiliary lanes is carried on the freeway beyond the first exit so that the number of lanes going-away is 1 more than the number approaching (6 compared with 5). The fourth lane is then eliminated at the next exit, with 3 basic lanes retained beyond this point on the freeway.

Where the basic number of lanes changes, it is often good insurance to allow for the provision of extra lanes in the future, as shown in figure 52–E. The more flexible layout shown in figure 52–E(II) makes provision for an extra lane beyond the exit and for a length of auxiliary lane in advance of the exit. Thus, it may be possible to extend the basic number of lanes to the next interchange, as may be ultimately required by the outward extension of urban development.

Illustrative Example

The application of the analysis steps developed above is presented in the following example. The problem is typical of the situation encountered on most urban freeways. The solution clearly indicates the flexibility and capacity reserve provided, in the application of the techniques outlined, which permit the various traffic situations (producing unpredictable peaks) to be accounted for in design. The procedure emphasizes the system aspect of the freeway which, as a linear system, is brought into an operationally coordinated and balanced facility.

Problem 36

A 7-mile section of urban freeway is in design process, for which the arrangement of ramps and general gradients is indicated in figure 53. Also shown are design hourly volumes including wearing patterns. Truck traffic during peak hours is generally taken to be 5 percent throughout the system. Peaking conditions are assumed to be normal, for which an unrestrained overall PHF of 0.85 may be considered representative. As a full freeway, modern design standards are stipulated, including a design speed of 70 mph. Determine the number and arrangement of lanes for the ultimate and interim designs considering operational flexibility, compatability with stage construction, and consideration of an extended life of the facility.

Solution The overall approach to the solution, as covered above, is applied here utilizing the four analysis steps: volume-capacity relations; basic number of lanes; lane balance; and special auxiliary lanes. In the application of this technique, the detailed analysis normally takes the form of an 8-step procedure, as follows:

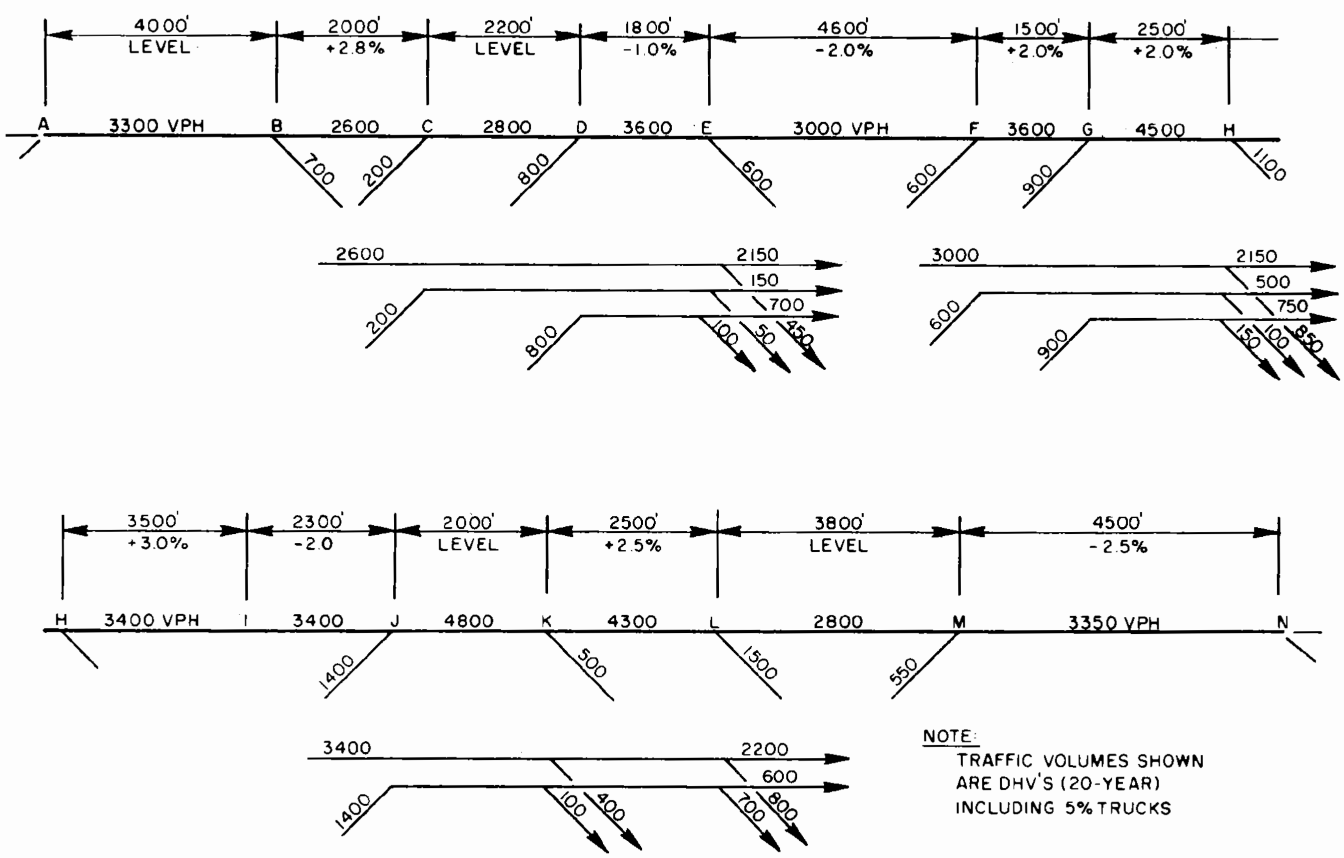


FIGURE 53.—Problem 36 illustrated.

a. Capacity analysis—general, each freeway segment
b. Determination of the basic number of lanes
c. Identification of ramp groupings
d. Capacity analysis—weaving sections
e. Initial lane balance
f. Capacity analysis—ramp exits and entrances
g. Provision of special auxiliary lanes and final lane balance
h. Determination of interim and ultimate lane arrangements of stage construction

a. Capacity Analysis—General, Freeway Segments

As a preliminary step to the determination of basic number of lanes, the procedure is to establish the number of lanes individually on each freeway segment, predicated purely on volume, regardless of weaving and ramp exits and entrances. The chart in figure 4 is used for this purpose. The same procedure as indicated in problem 2 is employed here with the following results:

Freeway Segment	*Profile Condition*	*Truck-passenger Car Equivalent, E_T*	*Traffic Volume, V*	*Number of Lanes on Segment, N*
A–B	level	2	3,300	2.6
B–C	2.8%, 0.4 mile	7	2,600	2.8
C–D	level	2	2,800	2.3
D–E	downgrade	2	3,600	2.9
E–F	downgrade	2	3,000	2.5
F–G	2.0%, 0.3 mile	4	3,600	3.1
G–H	2.0%, 0.5 mile	4	4,500	3.9
H–I	3.0%, 0.7 mile	8	3,400	3.3
I–J	downgrade	2	3,400	2.7
J–K	level	2	4,800	3.7
K–L	2.5%, 0.5 mile	6	4,300	3.9
L–M	level	2	2,800	2.3
M–N	downgrade	2	3,350	2.7

The E_T values are found in table 4. Segment *G–H* includes the effect of preceding length of grade, as does segment *H–I*, the E_T values for which are determined in problem 4 and figure 10.

b. Basic Number of Lanes

The numbers of fractional lanes found above serve as a direct guide in establishing the basic number of lanes on the freeway. Accordingly, the initial designation is as follows:

Freeway segments	*Basic lanes*
In advance of *G*	3 lanes
Between *G* and *L*	4 lanes
Beyond *L*	3 lanes

This is diagrammed in figure 54–A, recognizing that the basic number of lanes may be adjusted slightly in further consideration of lane balance and the relationship of the freeway to the overall arterial network.

c. Identification of Ramp Groupings

In order to give direction to capacity analyses of ramp junctions and determination of the effects of successive entrances and exits, the grouping of overlapping pairs of ramp junctions is essential, as indicated in figure 54–B.

d. Capacity Analysis—Weaving Sections

Weaving sections are analyzed prior to ramp entrances and exits which, together with basic number of lanes, establish lane requirements for a given level of service or capacity. This is a necessary step preceding the application of lane balance and analysis of ramp junctions. The ramp groupings, as shown in figure 54–B, identify three multiple weaving configurations. These are then tested for the realm of weaving, to determine whether they are multiple, simple or non-weaving sections.

Applying the test, as covered in chapter V with the use of figure 14 and table 15, the following forms of weaving sections are indicated:

Segment	*Length, ft.*	$\overline{W + W'}$, *pcph*	*k*	*Weaving Section*
C to *E*	4,000	640	1.0	out-of-realm
D–E	1,800	1,260	2.5	simple
F to *H*	4,000	1,930	1.9	multiple (2EN–EX)
J to *H*	4,500	1,750	1.3	multiple (EN–2EX)

The numbers of lanes required to satisfy weaving in segments *D–E*, *F–G*, *G–H*, *J–K*, and *K–L* were previously determined in chapter V, problems 33, 34, and 35. These are shown in figure 54–C, supplanting the previously calculated values. Also indicated are rounded numbers of lanes for each segment along the freeway.

e. Initial Lane Balance

Applying lane balance at this point in the analysis may seem out of place, since ramp capacities have not yet been determined and specific lane requirements on ramps are not available. However, the number and arrangement of lanes at points of entrance and exit along with lane balance are necessary to identify the "ramp case" applicable to the particular condition, in accordance with figures 14 and 15.

The procedure, therefore, requires assuming the number of lanes on the ramps in accordance with volumes carried as a preliminary step in order to achieve initial lane balance. Then, the principles of lane balance are applied as previously outlined. Following this, capacity analyses of ramps can be carried out and the design adjusted further as required. As a matter of policy, some of the highway agencies call for design of 2-lane ramps for volumes of the order of 800 to 1,000 vph. Such practice allows for considerable flexibility in operation and a capacity reserve, which can accommodate more effectively unusual peaks and future traffic growth. The policy recommended here and applied to this problem is: ramp volumes of approximately 800 vph or more are to be used to designate 2-lane ramps. Thus, the following numbers of lanes are applied to the ramps:

1-lane—Ramps *B*, *C*, *E*, *F*, *K*, and *M*
2-lane—Ramps *D*, *G*, *H*, *J*, and *L*

The numbers of lanes, based on the previous analyses of basic number of lanes, weaving, and initial assignment of ramp lanes, are indicated in figure 54–D. Applying lane balance principles a, b, and c presented earlier in this chapter, the arrangement shown in figure 54–E is achieved.

f. Capacity Analysis—Ramp Entrances and Exits

The ramp cases as shown in figure 54–E can now be identified in figures 14 and 15, and the effects of preceding

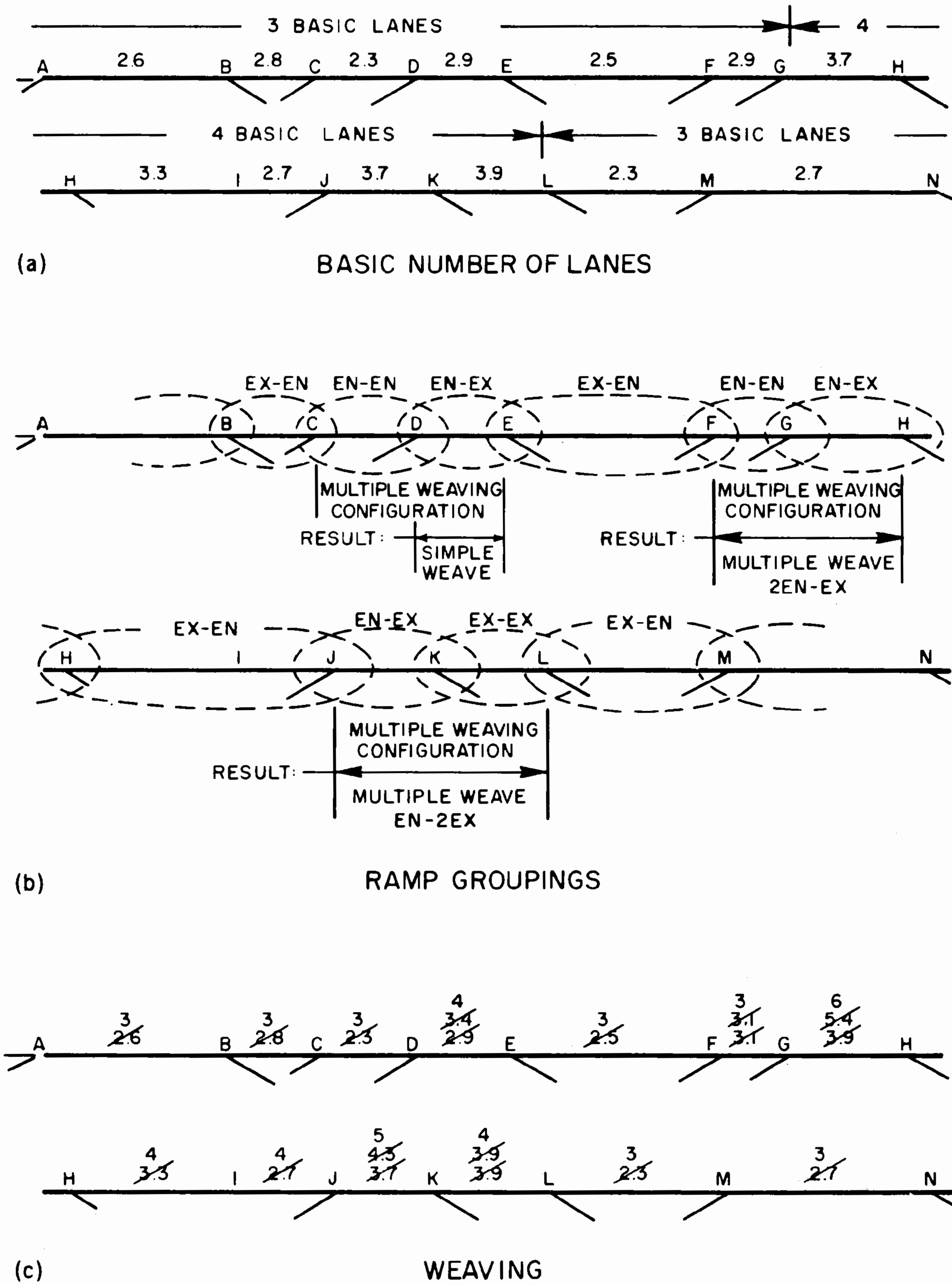
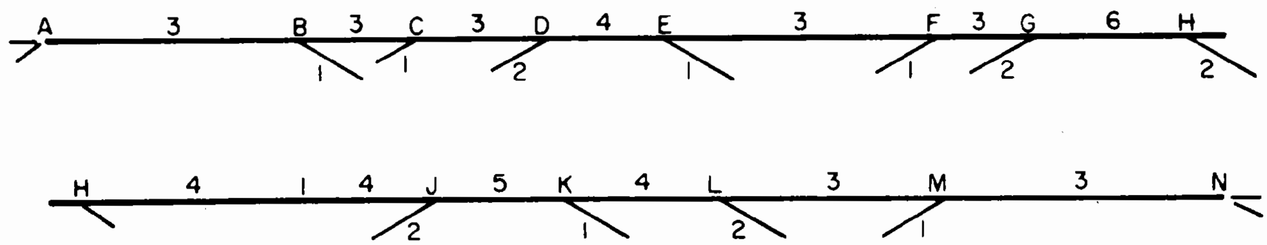
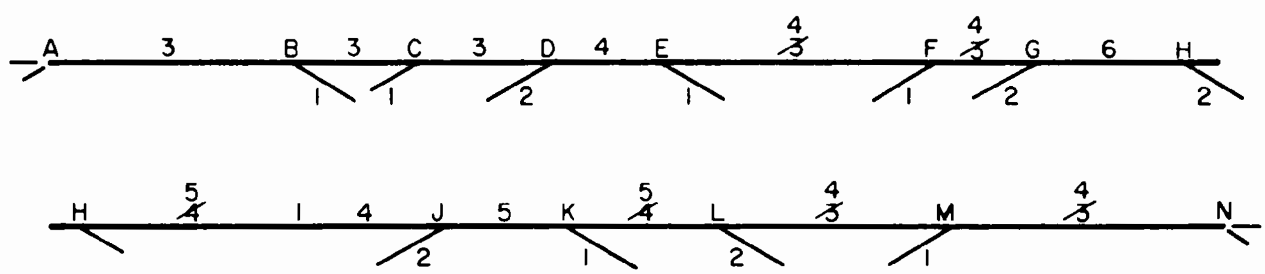


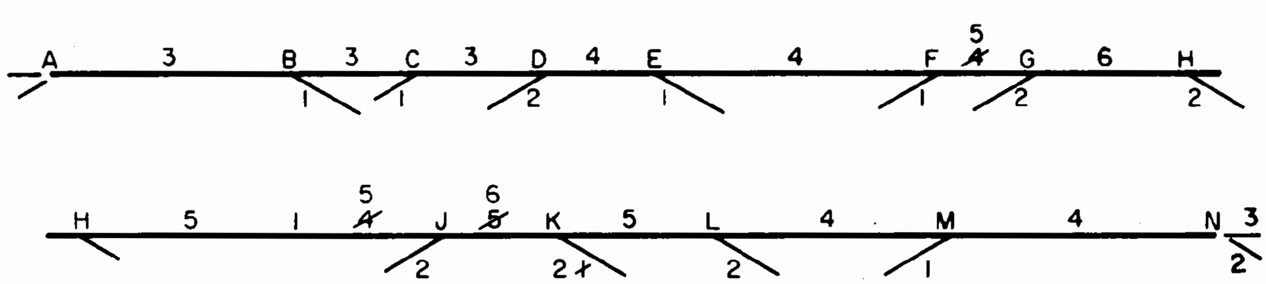
Figure 54.—Analysis steps, problem 36.



(d) INITIAL LANE BALANCE • TRIAL NUMBER OF LANES ON RAMPS



(e) INITIAL LANE BALANCE • RAMP ANALYSIS AND LANE BALANCE ADJUSTMENTS



(f) SPECIAL AUXILIARY LANES AND FINAL LANE BALANCE

FIGURE 54.—Continued.

or succeeding ramps can be determined in accordance with the grouping in figure 54–B and figure 16. The procedures for determining service volumes and capacities of ramp exits and entrances are demonstrated in chapter IV. The results of the analysis are summarized in the following tabulation:

Ramp	*Ramp Form*	*N*	*Volume Service*	*Demand Volume*	*Remarks*
B	Iso.[a] EX(1)	3	1,200	700	OK
C	Iso. EN(1)	3	1,000	200	OK
D	EN–EN EN(2)[b]	3	2,300	800	OK
E	Iso. EX(1)	4	1,100	600	OK
F	Iso. EN(1)	4	1,000	600	OK
G	Iso. EN(2 + a)[c]	4	2,700	900	OK
H	Iso. EX(2)	6	2,200	1,200	OK
J	Iso. EX(2)	4	2,200	1,400	OK
K	EX–EX EX(1)[d]	5	550	500	OK, close
L	Iso. EX(2)	5	1,900	1,500	OK
M	Iso. EN(1)	4	950	550	OK

[a] Iso. indicates isolated ramp effect.

[b] Second ramp of EN–EN grouping.

[c] The EN–EN grouping has no effect because of fully free 2-lane entrance.

[d] First ramp of EX–EX grouping: affected by SV of 1,100 in Lane *A* of Ramp *L*.

According to the above, the numbers of lanes assigned to the ramps meet the demand volumes indicated for the design hour. However, Ramp *K* shows a very close "fit," particularly in the light of the probable variation in volume and pattern of traffic and, therefore, does not provide the necessary capacity reserve as is evident in the other ramp entrances and exits. This aspect is accounted for in the next step.

g. Provision of Special Auxiliary Lanes and Final Lane Balance

At this point of analysis the arrangement is examined for weak links or possibilities where the system could break down should unusual traffic patterns develop due to such incidents as unforeseen development, delays or changes in stage construction of the overall freeway system, special events, maintenance, operations, accidents, etc. The application of special auxiliary lanes to place the facility into better balance becomes immediately evident in conjunction with Ramps *F* and *K*. The problem is overcome by making the entrance at *F* case (1 + a) and extending the introduced auxiliary lane forward to point *G*; and increasing the width of Ramp *K* to two lanes and extending the newly added lane therewith back to point *I*. As shown in figure 54–F, the additional length of 1,500 and 4,300 feet of extra lane along the freeway provides additional capacity reserve (a capability of handling 1,500 vph by the EN(1 + a) ramp at *F* and 2,100 vph by the EX(2) ramp at *K*) and improves other aspects of operation.

For example, the special auxiliary lane between *F* and *G* obviates two auxiliary lanes from joining the freeway simultaneously and provides traffic with improved and more efficient lane distribution. The special auxiliary lane between *I* and *K* establishes safer, more uniform operation by eliminating a lane drop (from 5 lanes to 4) at *I*; provides a greater capacity reserve in weaving section *J–K* and better balance with other elements of the freeway; and by continuing the auxiliary lane onto Ramp *K*, the 2-lane exit furnishes greater flexibility and is better adapted to operation of the primary weave within multiple weaving section *J–L*.

It may be noted in figure 54–F that the 4 basic lanes have been automatically extended from *D* to *H*. The latter point appropriately occurs at a minor interchange beyond the major exit. Another aspect in achieving a balanced facility is that whenever the initial weaving calculation shows a need for a total of 6 lanes in any one section, the freeway, as indicated in segment *G–H*, figure 54–C, plans should be tested for alternative arrangements to improve operations. In this particular case the consideration of C–D roads or the transposition of Ramps *G* and *H* with a grade separation (criss-cross) are variations which should be investigated. This aspect is emphasized although not included in this illustrative example.

h. Lane Arrangements as Part of Stage Development

The number of lanes finally determined in the scheme of figure 54–F is a complete solution, indicative of an ultimate or long-range plan. The arrangement is repeated in figure 55–A. In actual practice, it is frequently necessary to "trim down" the size of the facility in its initial stages either for budgetary purposes or for psychological reasons in facilitating decision making and approval of projects. The procedure of evolving the initial stage of development from the complete or ultimate development is illustrated in the three parts of figure 55. The simplest and most appropriate means of reducing the size of the facility is by deferring the construction of a traffic lane in each direction adjoining the median. This is shown in figure 55–B by the dotted line, together with the diminished number of lanes. A further reduction sometimes can be effected by removing certain auxiliary lanes from the complete design. Such an arrangement is indicated in figure 55–C where auxiliary lanes between *J* and *K* and between *G* and *H* are excluded including one lane on each of the joining ramps.

Either the plan in figure 55–B or in figure 55–C could represent the first construction stage. Preferably it would be the former, but if it should require the use of the latter, a full balance of lanes is maintained even though ingress and egress capacity is lowered. The arrangement in figure 55–C may be particularly advantageous as the initial planned staged to help "sell" the project because it is less formidable.

The important feature in this procedure is that the full plan can be realized because provision has been made for it in design. The build-up of stages in the reverse order—from the initial plan to the ultimate plan—now can take place in an orderly and logical fashion. The plausibility of achieving a design which provides operational flexibility and a degree of capacity reserve to accommodate unforeseen traffic peaks, is illustrated by this example. This is apparent by comparing the fractional lane requirements of figure 55–C and the final numbers of lanes shown for the

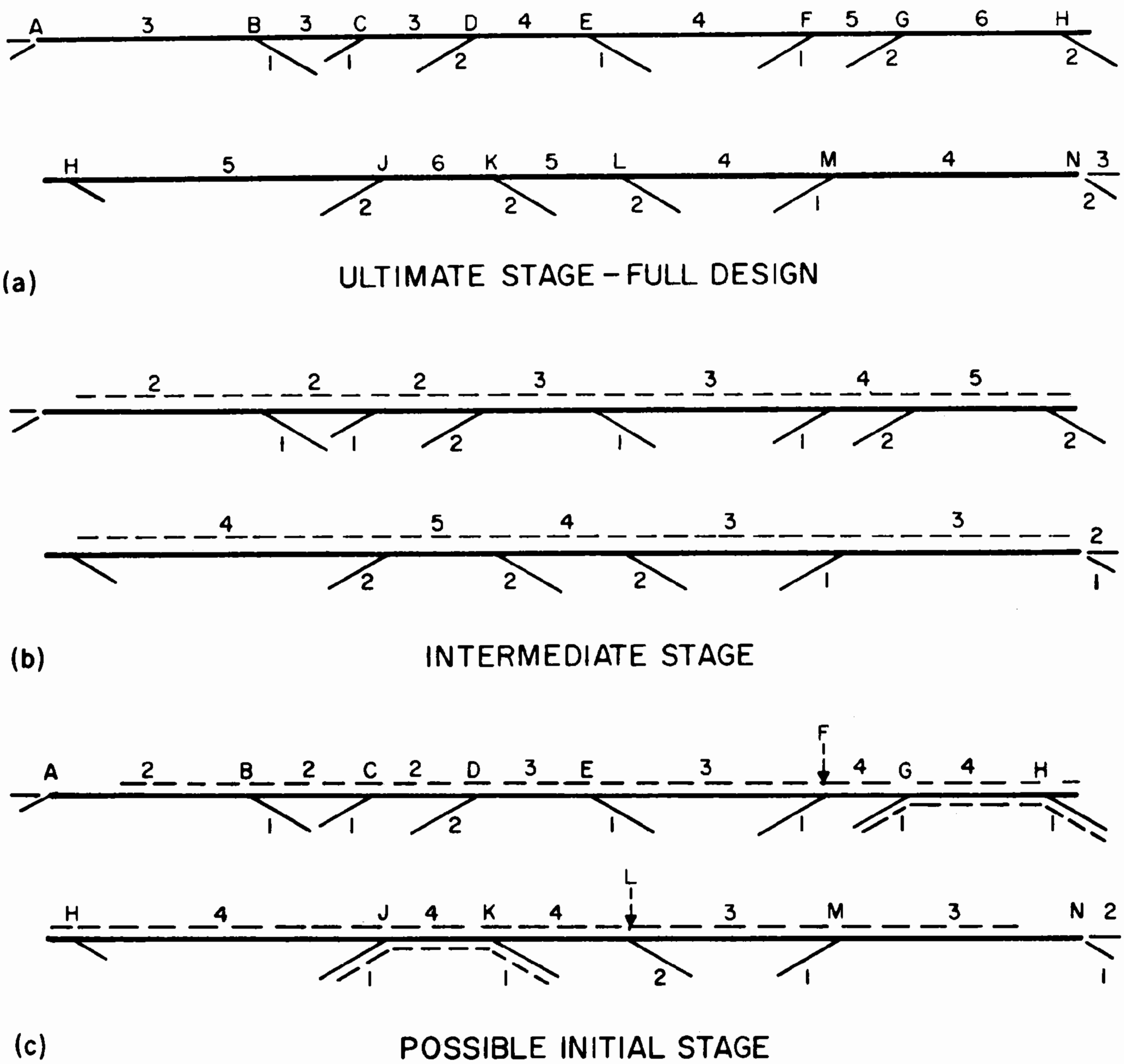


FIGURE 55.—Solution for problem 36—stage development.

full design in figure 55–A; and by comparing in the above tabulation the service volumes with the demand volumes on the ramps (including the adjusted service volumes determined for Ramps *F* and *K*). Thus, the technique presented, by the procedural application of volume-capacity relations, basic number of lanes, lane balance, and special auxiliary lanes, provides a sound basis for design of *balanced* freeway facilities to handle significant variations in pattern and volume of traffic, and at the same time extending the life of the facilities. This applies both to design of new freeways and to rehabilitation of existing (overloaded) freeways.

BIBLIOGRAPHY

"A POLICY ON GEOMETRIC DESIGN OF RURAL HIGHWAYS," AASHO, 1965.

BERRY, D. S., ROSS G. L. D., and PFEFER, R. C., "A Study of Left-hand Exit Ramps on Freeways," Highway Research Record No. 21, 1963.

BUREAU OF PUBLIC ROADS, Weaving study data, 1963 (unpublished).

DREW, D. R. and KEESE, C. J., "Freeway Level of Service as Influenced by Volume and Capacity Characteristics," Highway Research Record No. 99, 1965.

HESS, J. W., "Ramp-Freeway Terminal Operation as Related to Freeway Lane Volume Distribution and Adjacent Ramp Influence," Highway Research Record No. 99, 1965.

HIGHWAY CAPACITY MANUAL, Special Report 87, Highway Research Board, 1965.

MAY, A. D. and WAGNER, F. A., "Headway Characteristics and Interrelationships of Fundamental Characteristics of Traffic Flow," HRB Proceedings, Vol. 39, 1960.

MOSKOWITZ, K. and NEWMAN, L., "Notes on Freeway Capacity," Highway Research Record No. 27, 1963.

NORMANN, O. K., "Operations of Weaving Areas," HRB Bulletin 167, 1957.

POLYTECHNIC INSTITUTE OF NEW YORK, NCHRP Project No. 3–15, Weaving Area Operations Study, 1973.

WORRAL, R. D., DRAKE, J. S., BUHR, J. H., SOLTMAN, T. J., and BERRY, D. S., "Operational Characteristics of Left-hand Entrance and Exit Ramps on Urban Freeways," Highway Research Record No. 99, 1965.

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16. Abstract

In design of freeways the various geometric features, the configuration and spacing of interchanges, the number and arrangement of lanes, must be determined largely through level-of-service and capacity analyses. The 1965 Highway Capacity Manual basically furnishes this discipline, but it is not sufficiently oriented to design application, and lacks certain data and procedures. To meet these requirements the discussion herein develops a procedural framework for analysis. As part of the developed procedure, gaps in the Manual presentation dealing with ramps with high peaking characteristics are filled in by re-evaluation and more detailed rationalization of data and methods in the Manual, and by application of some data from other sources.

A special feature of the technique involves the grouping of successive ramps along a freeway into overlapping pairs. The influence of one ramp upon another is then isolated and each component of the freeway is dealt with by a separate analysis. Finally the results are combined to reach a composite solution for design of the complete freeway facility or for it as an operating system.

A graphical method, utilizing special nomographs, has been developed to facilitate the procedure. A series of illustrative problems and their solutions are presented, covering the full scope of problems to be encountered. Accordingly, the publication considers fully the aspects of practical application, and is presented to serve as a complete guide for level-of-service and capacity analyses in design and redesign of freeway facilities.

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