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PROPOSED CHAPTERS FOR THE 1985 HIGHWAY CAPACITY MANUAL

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Foreword

This Circular marks the first appearance of what is expected to be final text for portions of the Third Edition of the *HIGHWAY CAPACITY MANUAL*. It is appearing now in TRB Circular 281 for two reasons: To make new procedures available to practitioners as early as possible; and to provide a final review opportunity so that problems of clarity and the like will be minimized in the eventual complete Manual.

The Third Edition of the *HIGHWAY CAPACITY MANUAL,* expected in 1985, will be another milestone in a long history. Its first edition appeared in 1950. Research leading to a second edition began a few years later. Under the leadership of the Transportation Research Board (then Highway Research Board) Committee on Highway Capacity, these efforts led to the publication of Special Report 87, *Highway Capacity Manual—1965.* More research followed, and a formal project was initiated in 1977 under the National Cooperative Highway Research Program (NCHRP) to produce the third edition. A resulting step along the way was the 1980 publication of TRB Circular 212, *Interim Materials on Highway Capacity.*

This long publication history attests to the importance of the Highway Capacity Manual for highway engineering practitioners. Now in its tenth printing, the 1965 Manual has been the Board's most widely distributed publication with more than 30,000 copies made available. It has been translated into several languages, and it frequently serves as the primary reference for planning, design, and operational analyses of highway capacity all over the world—this despite the fact that the data upon which it is based come from North American experience. TRB Circular 212 has itself been through several printings, with more than 9,000 copies distributed in the past three years.

Yet, much has changed since the earlier editions of the manual, in the characteristics of travel and the information needs influencing highway capacity analyses. Research by many individuals, by private organizations, and by public agencies has led to new understandings and insights, and to procedural revisions and new techniques in capacity analysis. Because some public concerns have faded and new issues have taken their place, current requirements reflect new emphasis. All of these forces press for *the new publication.*

The chapters presented in this Circular are not necessarily interrelated, and they do not represent a complete section in the Third Edition. They are simply those where the work is complete and, thus, may be regarded as final and what will appear as chapters in the 1985 Manual. However, the conveyance to TRB of any discovery of errors or recommendations for improved clarity is invited and will be received with gratitude. Although one or more Circulars may follow this one before the entire Manual is assembled, it is anticipated that production of the complete manual will begin no later than *November 1984*. Comments, suggestions, or criticisms should be sent before then.

Three of the chapters presented here represent revisions and updates of material contained in TRB Circular 212, *Interim Materials on Highway Capacity*. Chapter 3, "Basic Freeway Segments," replaces the section with the same title. Chapter 5 replaces another section in the freeway capacity procedures entitled "Ramps and Ramp Junctions." Chapter 10 replaces the procedures in "Unsignalized Interesections." The fourth chapter, Chapter 7, represents new material on the procedures for multilane highways given in the *1965 Highway Capacity Manual*.

The fifteen chapters in the Third Edition come from several sources. In some cases they represent the results of funded research specifically commissioned for the development of new Manual material. In other cases, and at the other extreme, they represent the voluntary contributions of members of the TRB Committee on Highway Capacity and Quality of Service. Still others represent mixed sources of inputs that become nearly impossible to accredit. Nevertheless, all chapters have two features in common. Each has been prepared by the research team assembled under Dr. Roger P. Roess of the Polytechnic Institute of New York and Dr. Carroll J. Messer of Texas Transportation Institute. And each has been thoroughly reviewed by members of the NCHRP project panel monitoring the work, by members of the TRB Committee on Highway Capacity and Quality of Service and its subcommittees, and by many individuals not affiliated with either group who have volunteered their time and interest.

What follows is a listing of those groups and agencies whose contributions to the evolution of the new Manual merit recognition. Despite attempts to be inclusive, there may be omissions; there are simply too many people, to mention individually, who have supplied helpful comments without which the value of the new manual would be greatly reduced.

- The National Cooperative Highway Research Program, under the management and guidance of the NCHRP staff and project panel, has been responsible for much of the work leading to the development of these and the remaining chapters. Other research has been funded by the Federal Highway Administration, under the Office of Research, Development and Technology.
- The principal research agencies have been JHK & Associates, Texas A&M Research Foundation, Polytechnic Institute of New York, PRC Voorhees, and Jack E. Leisch & Associates. Others include the Traffic Institute at Northestern University, KLD Associates, Inc., and the Minnesota Department of Transportation.
- The final responsibility for what appears in this Circular belongs to the Transportation Research Board Committee on Highway Capacity and Quality of Service and supporting staff including the Editorial and Production Offices.

Contents

Chapter Organization of the New Highway Capacity Manual *

* Chapters 3, 5, 7, and 10, are published in this *Circular.* It should be pointed out that some of the technical graphics do not represent a final effort and will be redrawn in the complete edition of the 1985 HCM.

CHAPTER 3

BASIC FREEWAY SEGMENTS

CONTENTS

I. INTRODUCTION

A *freeway* may be defined as a divided highway facility having two or more lanes for the exclusive use of traffic in each direction and full control of access and egress.

The freeway is the only type of highway facility that provides completely "uninterrupted" flow. There are no external interruptions to traffic flow, such as signalized or STOP-controlled intersections. Access to and egress from the facility occur only at ramps, which are generally designed to permit high-speed merging and diverging maneuvers to take place, thus minimizing disruptions to mainline traffic.

Because of these characteristics, operating conditions primarily result from interactions among vehicles in the traffic stream, and between vehicles and the geometric characteristics of the freeway. Operations are also affected by environmental conditions, such as weather, pavement conditions, and/or the occurrence of traffic incidents.

The procedures contained in this chapter relate the probable operating conditions of a freeway to the geometric and traffic conditions which exist during a defined time interval on a specified segment of freeway. This chapter details procedures for the operational analysis, design, and planning of basic freeway segments. Weaving areas are treated in Chapter 4, and ramp junctions are considered in Chapter 5. This chapter is based primarily on material presented in Ref. *1*.

COMPONENTS OF A FREEWAY

In general, a freeway is composed of three different types of component subsections:

1. *Basic freeway segments*—Sections of the freeway that are unaffected either by merging or diverging movements at nearby ramps or by weaving movements.
2. *Weaving areas*—Sections of the freeway where two or more vehicle flows must cross each other's path along a length of the freeway. Weaving areas are usually formed when merge areas are closely followed by diverge areas. They are also formed when a freeway on-ramp is followed by an off-ramp and the two are connected by a continuous auxiliary lane.
3. *Ramp junctions*—Points at which on- and off-ramps join the freeway. The junction formed at this point is an area of turbulence due to concentrations of merging or diverging vehicles.

Basic freeway segments are located outside of the influence area of any ramp or weaving area. In general, the influence area of ramp junctions or weaving areas may be taken to be:

1. *On-ramps*—500 ft upstream and 2,500 ft downstream of the ramp junction.
2. *Off-ramps*—2,500 ft upstream and 500 ft downstream of the ramp junction.
3. *Weaving areas*—500 ft upstream of the merge point marking the beginning of the weaving area, and 500 ft downstream of the diverge point forming the end of the weaving area.

The foregoing guidelines refer to stable operations. During congested or breakdown conditions, merge, diverge, or weaving areas can produce queues of widely varying size, up to several miles in length.

Figure 3-1 shows the various types of freeway components.

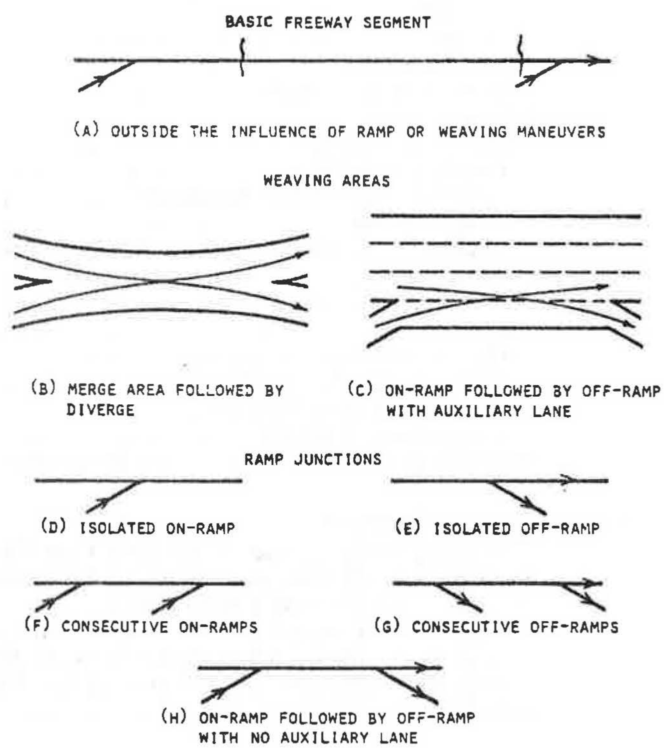


Figure 3-1. Freeway components.

The influence areas of these components are illustrated in Figure 3-2.

OVERALL CONSIDERATIONS

The procedures set forth in Chapters 3, 4, and 5 treat only the isolated characteristics of the segment under consideration. The procedures assume:

1. Good pavement conditions.
2. No traffic incidents.
3. Good weather conditions.

Should any of these conditions not exist, the user must use judgment to alter the results of the analysis, consider this when interpreting results, or both.

In practice, it is essential to analyze sections of freeway in an integrated manner to estimate overall capacity of the freeway system and to identify points of minimum capacity, which could become potential bottlenecks. The interactions between and among adjacent freeway subsegments are of extreme importance, particularly when a breakdown in one causes queues to extend into upstream segments. Procedures for overall freeway systems analysis are presented in Chapter 6.

Chapter 6 also treats a number of subjects which can impact overall operations, but which are not explicitly considered in the analysis of individual segments. These include:

1. Lane balance and configuration.
2. Traffic incidents.
3. Impacts of high-occupancy vehicle lanes.
4. Impacts of work zones and maintenance operations.
5. Weather and other environmental factors.
6. Impacts of freeway surveillance and control systems.

The user should refer to Chapter 6 for detailed discussions of these factors.

DEFINITIONS AND TERMINOLOGY

The following terms and definitions are of specific interest to material in this chapter. The basic traffic flow parameters used in this chapter are defined in Chapter 1. Other definitions are introduced as used in subsequent discussion.

1. *Freeway capacity* is the maximum sustained (15-min) rate of flow at which traffic can pass a point or uniform segment of freeway under prevailing roadway and traffic conditions. Capacity is defined for a single direction of flow, and is expressed in vehicles per hour (vph).

2. *Roadway characteristics* are the geometric characteristics of the freeway segment under study; these include the number and width of lanes, lateral clearances at the roadside and median, design speeds, grades, and lane configurations.

3. *Traffic conditions* refer to any characteristic of the traffic

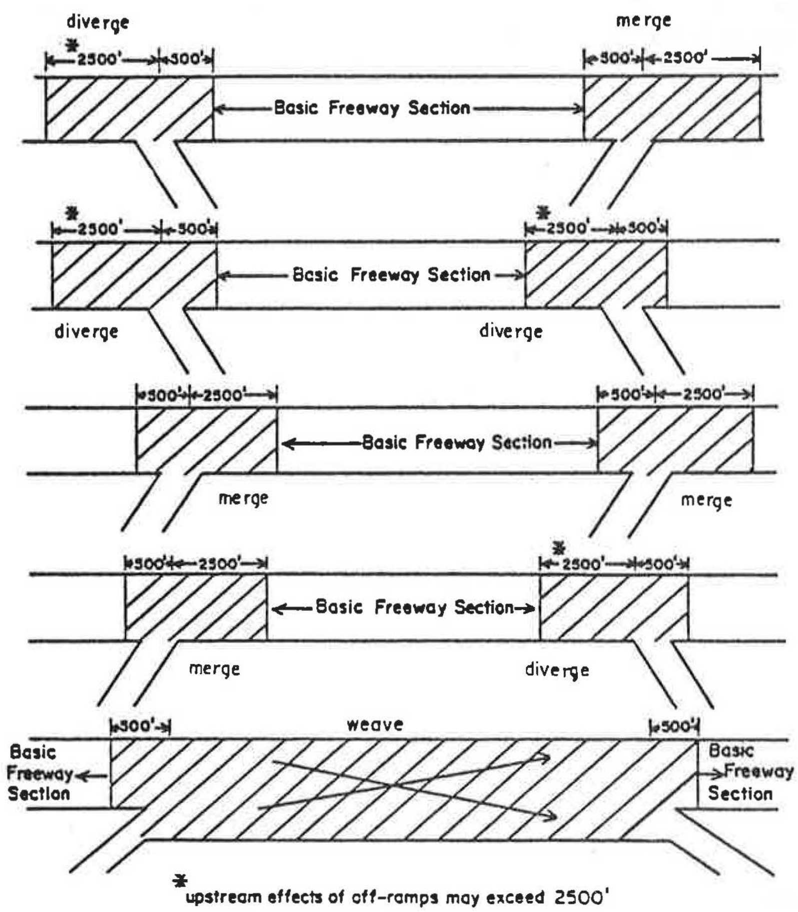


Figure 3-2. Influence areas of freeway components.

stream that affects capacity or operations. These include the percentage composition of the traffic stream by vehicle type, lane distribution characteristics, and driver characteristics (such as the differences between weekday commuters and recreational drivers).

It should be noted that capacity analysis is based on point locations or freeway segments of *uniform* roadway and traffic conditions. If either of these prevailing conditions changes significantly, the capacity of the segment and its likely operating conditions change as well.

Such segments also should have reasonably uniform design speeds. Accordingly, all straight and level segments of freeway are considered to have a design speed of 70 mph. It may be necessary to consider isolated elements with lower design speeds separately, such as a curve with a design speed significantly lower than 70 mph. On the other hand, a long segment of freeway dominated by many geometric elements with reduced design speed could be analyzed as a single unit, based on the reduced design speed.

CHARACTERISTICS OF FREEWAY FLOW

Freeway Flow Under Ideal Conditions

Chapter 1 of this manual includes a discussion of the general characteristics of uninterrupted traffic flow. The specific speed-flow-density relationship depends on the prevailing roadway and traffic conditions for the segment in question. The base characteristics used in this chapter have been estimated for a set of "ideal conditions," as follows:

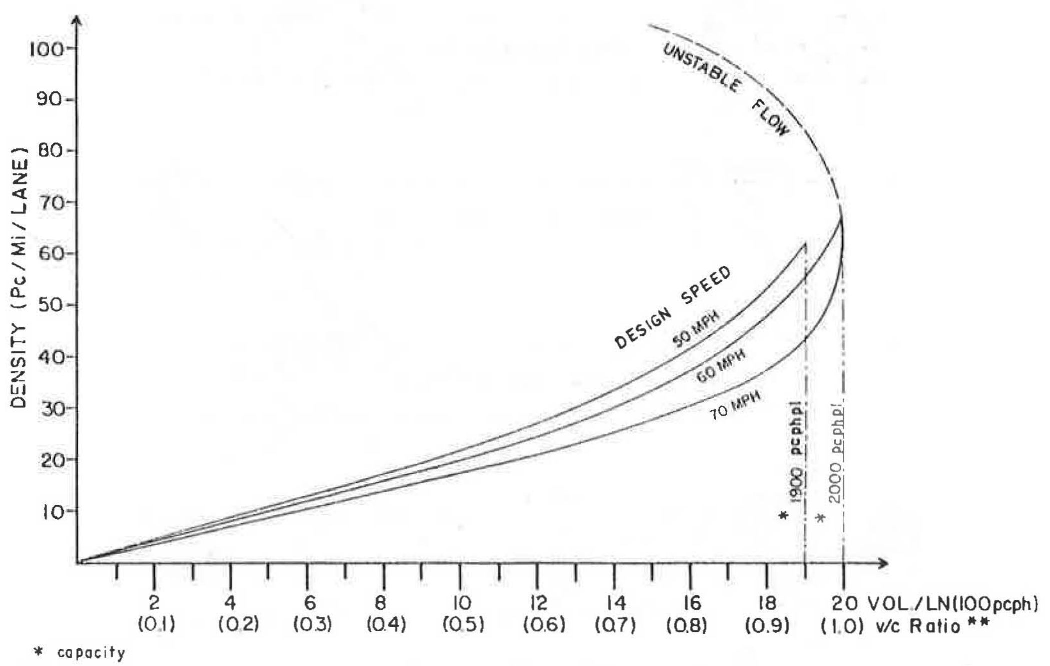
1. Twelve-foot minimum lane widths.
2. Six-foot minimum lateral clearance between the edge of the travel lanes and the nearest obstacle or object on the roadside or in the median (note that certain types of median barriers do not represent an "obstacle," even when closer than 6 ft to the pavement edge, as is discussed later).
3. All passenger cars in the traffic stream.
4. Driver characteristics typical of weekday commuter traffic streams in urban areas, or regular users in other areas.

It should be noted that these conditions are "ideal" only from the point of view of capacity, and do not relate to safety or other factors.

Typical flow characteristics for these conditions and various design speeds are illustrated in Figures 3-3 and 3-4. Figure 3-3 shows the typical relationship between density and rate of flow, while Figure 3-4 depicts the relationship between average travel speed and rate of flow. The relationships shown reflect the influence of a 55-mph speed limit.

The curves show a capacity of 2,000 pcphpl for 70-mph and 60-mph design speeds, and 1,900 pcphpl for 50-mph design speeds, all for ideal conditions. The speed-flow curves show minor differences between four-, six-, and eight-lane freeways for 70-mph design speed that are not shown on the density-flow curves. When plotted on a density-flow plane, the differences become so small as to be virtually impossible to depict.

The curves depict two important characteristics that greatly influence the use and interpretation of the procedures contained in this chapter.



* capacity
** v/c ratio based on 2000 pcphpl - valid only for-60 and 70-MPH design speeds

Figure 3-3. Density-flow relationships under ideal conditions.

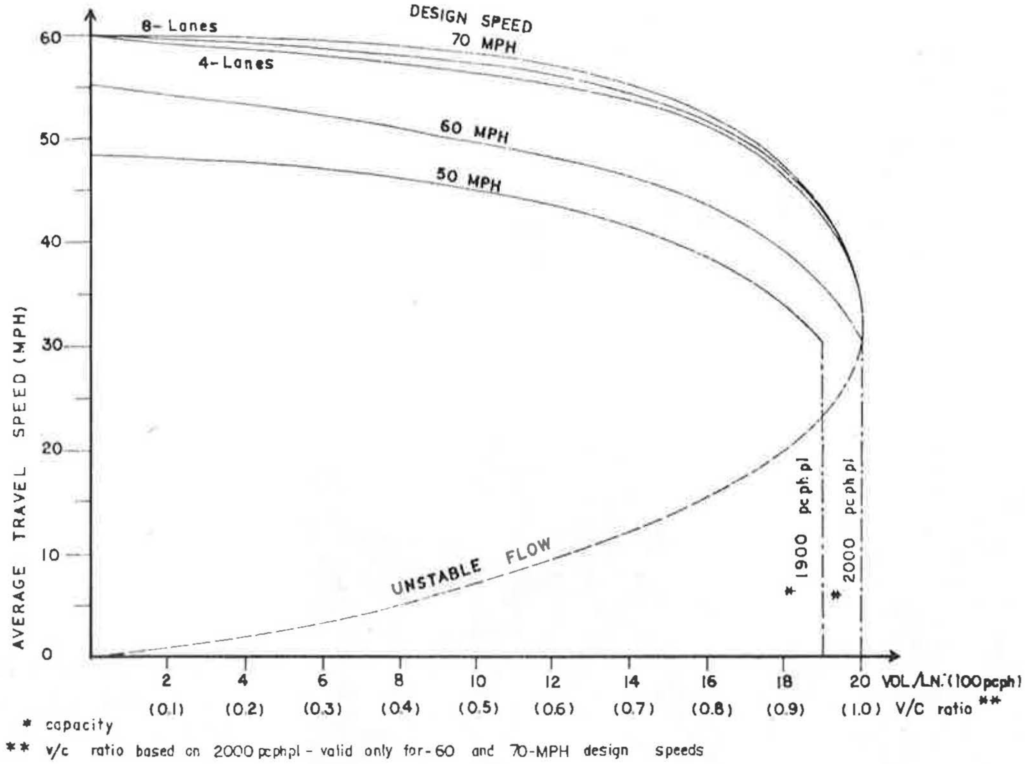


Figure 3-4. Speed-flow relationships under ideal conditions.

1. There is a substantial range of flow over which speed is relatively insensitive to flow; this range extends to fairly high flow rates.
2. As flow approaches capacity, speed drops off at an extremely sharp rate.

These characteristics are most pronounced for 70-mph design speed freeway elements. As capacity is approached, small changes in volume or rate of flow will produce extremely large changes in operating conditions, i.e., speed and density. Level-of-service criteria for freeways reflect this, with the poorer levels defined for reasonably large ranges in speed and density, while the corresponding range in flow rates is quite small.

Factors Affecting Flow Under Ideal Conditions

Any prevailing condition that differs from the ideal conditions defined above will cause changes in the typical speed-flow-density relationship.

1. *Lane width and lateral clearance*—When lane widths are less than 12 ft, drivers are forced to travel laterally closer to one another than they would normally desire. Drivers tend to compensate for this by observing longer spacings between vehicles in the same lane.

The effect of restricted lateral clearance is similar. When roadside or median objects are located too close to the pavement edge, drivers tend to "shy" away from them, positioning themselves further from the pavement edge than under normal or ideal conditions. This has the same effect as narrow lanes, usually forcing drivers closer together laterally. Again, drivers generally compensate by leaving more distance between vehicles in the same lane.

When drivers allow longer spacing for a given speed, the volume accommodated decreases. The same effect can be viewed in reverse—for a given spacing, drivers will slow down when lateral clearance and/or lane width restrictions exist—again resulting in reduced flow.

Illustrations 3-1 and 3-2 depict the impacts of lane width and lateral clearance on freeway flow.

2. *Reduced design speed*—As indicated in Figure 3-3, a reduction in the design speed of a freeway segment below 70 mph will have a substantial impact on freeway operations. Because restrictive geometrics require greater vigilance on the part of the driver, observed speeds for any given volume will generally be lower than on similar segments of 70-mph design.

3. *Trucks, buses, and recreational vehicles*—The presence of vehicles other than passenger cars in the traffic stream affects flow in two ways: (a) such vehicles are larger than passenger cars, and therefore occupy more roadway space than passenger cars, and (b) the operating capabilities of such vehicles (acceleration, deceleration, maintenance of speed, etc.) are generally inferior to those of passenger cars; when introduced into a mixed traffic stream, these different performance capabilities lead to the formation of gaps in the traffic stream that cannot be readily filled by passing maneuvers.

The second impact is particularly significant on long sustained upgrades, on which trucks may be forced to slow considerably, thereby creating extremely large gaps in the traffic stream.

Illustrations 3-3 and 3-4 depict the impact of trucks and other heavy vehicles on freeway traffic streams.

4. *Driver population*—The ideal conditions defined for the typical speed-flow-density relationships assume a driver population consisting primarily of weekday commuters or other regular users. A variety of studies across the nation show that other driver populations do not display the same characteristics.

Recreational traffic streams consisting primarily of weekend or occasional drivers have been observed to operate with considerably less efficiency than commuter traffic. Capacity reductions of as much as 20 to 25 percent have been observed for such traffic streams.

Illustration 3-1. Vehicles shy away from both roadside and median barriers, driving as close to the lane marking as possible. The existence of narrow lanes compounds the problem, making it difficult for two vehicles to travel alongside each other.

Illustration 3-2. In this case, vehicles shy away from the roadside barrier. This causes a shift towards the median in the placement of vehicles in each lane. This is also an indication that the median barrier illustrated here does not present an obstruction to drivers.

Illustration 3-3. Note formation of large gaps in front of slow-moving trucks climbing upgrade.

Illustration 3-4. Large gaps in front of trucks or other heavy vehicles are often unavoidable even on relatively level terrain.

II. METHODOLOGY

This section describes the general structure of the capacity analysis procedures for basic freeway segments. Detailed instructions for the application of these procedures in operational analysis, design, and planning are presented in a subsequent section.

LEVELS OF SERVICE

Measures of Effectiveness

Freeway operating characteristics include a wide range of rates of flow over which speed is relatively constant. This means that speed alone is not adequate as a performance measure by which to define levels of service.

Although speed is a primary concern of drivers with respect to service quality, freedom to maneuver and proximity to other vehicles are also important parameters. These other qualities are directly related to the *density* of the freeway traffic stream. Further, rate of flow increases with increasing density throughout the full range of stable flows (see Figure 3-3).

For these reasons, density is the parameter used to define levels of service for basic freeway segments. The densities used to define the various levels of service (LOS) are as follows:

Level of Service	*Density (pc/mi/ln)*
A	12
B	20
C	30
D	42
E	67

These values are boundary conditions representing the maximum allowable densities for the associated level of service. The LOS-E boundary of 67 pc/mi/ln has been generally found to be the *critical density* at which capacity most often occurs. This corresponds to an average travel speed of 30 mph and a capacity of 2,000 pcphpl for 60-mph and 70-mph design speeds. The exact speed and density, however, at which capacity occurs may vary somewhat from location to location.

Level-of-Service Criteria

Level-of-service criteria for basic freeway segments are given in Table 3-1 for 70-mph, 60-mph, and 50-mph design speed elements. To be within a given level of service, the *density* criterion must be met. The average travel speeds and maximum service flow rates indicated in the table are expected to exist under *ideal* conditions for the given densities. Actual average travel speeds for traffic streams under nonideal conditions may be somewhat lower than the values shown.

Design speed depends on the combination of horizontal and vertical alignment. Other influences on driver behavior, such as the development environment, local driving habits, and other factors, may cause the relationship among density, speed, and flow to differ from the typical values of Table 3-1. Where local speed-flow-density data are available, they may be used as a guide in determining which design speed best represents local conditions.

DESCRIPTION OF LEVELS OF SERVICE

Operational characteristics for the six levels of service are shown in Illustrations 3-5 to 3-10.

The levels of service have been defined to represent reasonable ranges in the three critical variables: average travel speed, density, and flow rate. The basic shape of the typical speed-density-flow curves requires that as level of service moves from A to F, the range of densities and speeds covered by each level becomes larger, while the corresponding range of service flow rates becomes smaller.

The values in Table 3-1 reflect the influence of the 55-mph speed limit. Even with this speed limit clearly signed and reasonably enforced, average travel speeds for the better levels of service are still expected to be slightly higher than the 55-mph limit. Where enforcement is particularly stringent, or where lower speed limits are posted, speeds may be somewhat lower than those given in Table 3-1.

TABLE 3-1. LEVELS OF SERVICE FOR BASIC FREEWAY SECTIONS

LOS	DENSITY (PC/MI/LN)	70 MPH DESIGN SPEED			60 MPH DESIGN SPEED			50 MPH DESIGN SPEED		
		SPEED[b] (MPH)	v/c	MSF[a] (PCPHPL)	SPEED[b] (MPH)	v/c	MSF[a] (PCPHPL)	SPEED[b] (MPH)	v/c	MSF[a] (PCPHPL)
A	≤ 12	≥ 60	0.35	700	—	—	—	—	—	—
B	≤ 20	≥ 57	0.54	1,100	≥ 50	0.49	1,000	—	—	—
C	≤ 30	≥ 54	0.77	1,550	≥ 47	0.69	1,400	≥ 43	0.67	1,300
D	≤ 42	≥ 46	0.93	1,850	≥ 42	0.84	1,700	≥ 40	0.83	1,600
E	≤ 67	≥ 30	1.00	2,000	≥ 30	1.00	2,000	≥ 28	1.00	1,900
F	> 67	< 30	c	c	< 30	c	c	< 28	c	c

[a] Maximum service flow rate per lane under ideal conditions.
[b] Average travel speed.
[c] Highly variable, unstable.
NOTE: All values of *MSF* Rounded to the nearest 50 vph.

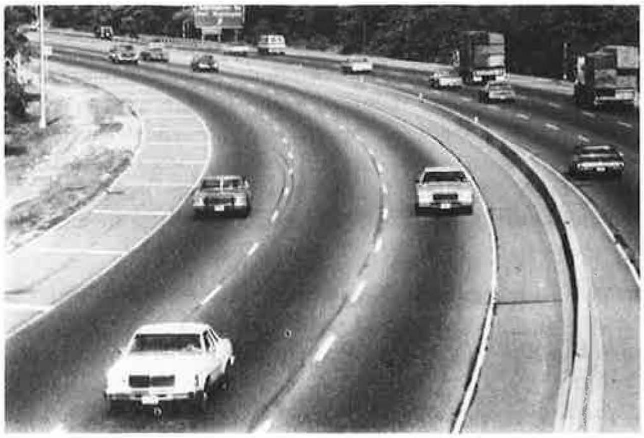


Illustration 3-5. Level-of-service A.

Illustration 3-6. Level-of-service B.

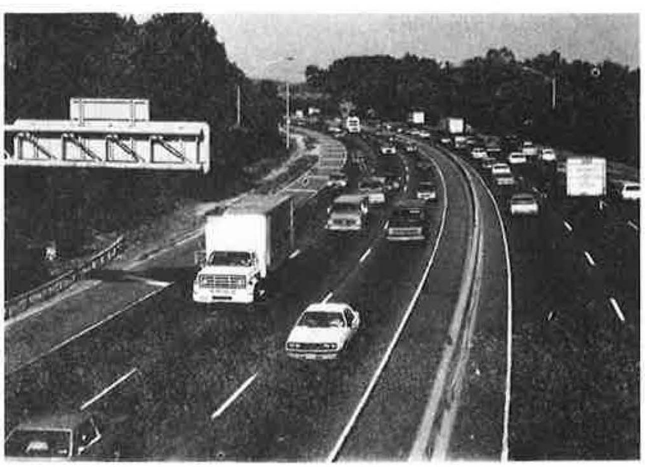


Illustration 3-7. Level-of-service C.

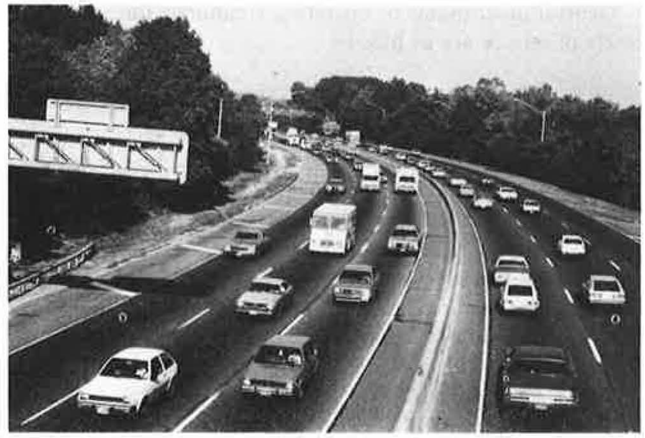


Illustration 3-8. Level-of-service D.

Illustration 3-9. Level-of-service E.

Illustration 3-10. Level-of-service F.

General descriptions of operating conditions for each of the levels of service are as follows:

1. *Level-of-service A*—Level A describes primarily free flow operations. Average travel speeds near 60 mph generally prevail on 70-mph freeway elements. Vehicles are almost completely unimpeded in their ability to maneuver within the traffic stream. The average spacing between vehicles is about 440 ft, or 22 car-lengths, with a maximum density of 12 pc/mi/ln. This affords the motorist a high level of physical and psychological comfort. The effects of minor incidents or breakdowns are easily absorbed at this level. Although they may cause a deterioration in LOS in the vicinity of the incident, standing queues will not form, and traffic quickly returns to LOS A on passing the disruption.

2. *Level-of-service B*—Level B also represents reasonably free-flow conditions, and speeds of over 57 mph are maintained on 70-mph freeway elements. The average spacing between vehicles is about 260 ft, or 13 car-lengths, with a maximum density of 20 pc/mi/ln. The ability to maneuver within the traffic stream is only slightly restricted, and the general level of physical and psychological comfort provided to drivers is still high. The effects of minor incidents and breakdowns are still easily absorbed, though local deterioration in service would be more severe than for LOS A.

3. *Level-of-service C*—Level C provides for stable operations, but flows approach the range in which small increases in flow will cause substantial deterioration in service. Average travel speeds are still over 54 mph. Freedom to maneuver within the traffic stream is noticeably restricted at LOS C, and lane changes require additional care and vigilance by the driver. Average spacings are in the range of 175 ft, or 9 car-lengths, with a maximum density of 30 pc/mi/ln. Minor incidents may still be absorbed, but the local deterioration in service will be substantial. Queues may be expected to form behind any significant blockage. The driver now experiences a noticeable increase in tension due to the additional vigilance required for safe operation.

4. *Level-of-service D*—Level D borders on unstable flow. In this range, small increases in flow cause substantial deterioration in service. Average travel speeds of 46 mph or more can still be maintained on 70-mph freeway elements. Freedom to maneuver within the traffic stream is severely limited, and the driver experiences drastically reduced physical and psychological comfort levels. Even minor incidents can be expected to create substantial queuing, because the traffic stream has little space to absorb disruptions. Average spacings are about 125 ft, or 6 car-lengths, with a maximum density of 42 pc/mi/ln.

5. *Level-of-service E*—The lower boundary of LOS E describes operation at capacity. Operations in this level are extremely unstable, because there are virtually no usable gaps in the traffic stream. Vehicles are spaced at approximately 80 ft, or 4 car-lengths, at relatively uniform headways. This, however, represents the minimum spacing at which stable flow can be accommodated. Any disruption to the traffic stream, such as a vehicle entering from a ramp, or a vehicle changing lanes, causes following vehicles to give way to admit the vehicle. This condition establishes a disruption wave which propagates through the upstream traffic flow. At capacity, the traffic stream has no ability to dissipate even the most minor disruptions. Any incident can be expected to produce a serious breakdown with extensive queuing. The range of flows encompassed by LOS E is relatively small compared to other levels, but reflects a substantial deterioration in service. Maneuverability within the traffic stream is extremely limited, and the level of physical and psychological comfort afforded to the driver is extremely poor. Average travel speeds at capacity are approximately 30 mph.

6. *Level-of-service F*—Level F describes forced or breakdown flow. Such conditions generally exist within queues forming behind breakdown points. Such breakdowns occur for a number of reasons:

a. Traffic incidents cause a temporary reduction in the capacity of a short segment, such that the number of vehicles arriving at the point is greater than the number of vehicles that can traverse it.

b. Recurring points of congestion exist, such as merge or weaving areas and lane drops, where the number of vehicles arriving is greater than the number of vehicles traversing the point.

c. In forecasting situations, any location presents a problem when the projected peak hour (or other) flow rate exceeds the estimated capacity of the location.

It is noted that in all cases, breakdown occurs when the ratio of actual arrival flow rate to actual capacity or the forecasted flow rate to estimated capacity exceeds 1.00. Operations at such a point will generally be at or near capacity, and downstream operations may be better as vehicles pass the bottleneck (assuming that there are no additional downstream problems). The LOS F operations observed within a queue are the result of a breakdown or bottleneck at a downstream point. The designation "LOS F" is used, therefore, to identify the point of the breakdown or bottleneck, as well as the operations within the queue which forms behind it.

The extent of queuing, and the delays caused by queuing, are of great interest in the analysis of congested freeway segments. Chapter 6 contains a methodology for estimating the queue length and delays behind a bottleneck with known arrival and discharge rates. The procedure allows a rough quantification of the extent of congestion created by a LOS F situation.

BASIC RELATIONSHIPS

Table 3-1 presents criteria for maximum service flow rate, *MSF*, under ideal conditions, for 70-mph, 60-mph, and 50-mph design speed elements. These values are computed from the volume-to-capacity ratios, v/c, as follows, then rounded to the nearest 50 pcphpl.

$$MSF_i = c_j \times (v/c)_i \tag{3-1}$$

where:

MSF_i = the maximum service flow rate per lane for LOS i under ideal conditions, in pcph;

$(v/c)_i$ = the maximum volume-to-capacity ratio associated with LOS i;

c_j = capacity under ideal conditions for freeway element of design speed j; 2,000 pcphpl for 60-mph and 70-mph freeway elements, 1,900 pcphpl for 50-mph freeway elements; the value of c_j is synonymous with the maximum service flow rate for LOS E in Table 3-1.

Note that all values of *MSF* given in Table 3-1 have been rounded to the nearest 50 pcph.

These values represent ideal conditions of 12-ft lanes, adequate lateral clearances, and all passenger cars in the traffic stream. Therefore, the maximum service flow rates of Table 3-1 must be adjusted to reflect any prevailing conditions that are other than ideal, and to reflect the total number of lanes in one direction on the freeway. This is accomplished by using several correction factors, as follows:

$$SF_i = MSF_i \times N \times f_w \times f_{HV} \times f_p \qquad (3\text{-}2)$$

where:

SF_i = the service flow rate for LOS i under prevailing roadway and traffic conditions for N lanes in one direction, in vph;
N = the number of lanes in one direction of the freeway;
f_w = factor to adjust for the effects of restricted lane widths and/or lateral clearances;
f_{HV} = factor to adjust for the effect of heavy vehicles (trucks, buses, and recreational vehicles) in the traffic stream; and
f_p = factor to adjust for the effect of driver population.

Even the adjusted service flow rate, however, assumes an absence of traffic incidents and the existence of good weather and pavement conditions. Any existing conditions differing from these could cause further reductions in the flow rates which are accommodated at any given level of service. A more detailed discussion of these issues is contained in Chapter 6.

Equations 3-1 and 3-2 can be combined as follows. The combined form is useful when a computation of SF is desired using v/c values directly, rather than MSF values.

$$SF_i = c_j \times (v/c)_i \times N \times f_w \times f_{HV} \times f_p \qquad (3\text{-}3)$$

These three basic relationships form the basis of all capacity analysis applications for basic freeway segments.

ADJUSTMENTS TO MAXIMUM SERVICE VOLUME

Adjustment for Restricted Lane Width and/or Lateral Clearance

The MSF for any freeway segment with lane widths narrower than 12 ft and/or objects closer to the edge of the travel lanes than 6 ft (at the roadside or in the median) is adjusted to reflect these prevailing conditions using the factor f_w.

Considerable judgment must be used in determining whether or not roadside and/or median objects and barriers present a true "obstruction." Such obstructions may be continuous, such as a retaining wall, or may be periodic objects, such as light supports or bridge abutments. In some cases, drivers may become accustomed to certain types of obstructions, in which case, their effect on traffic flow becomes negligible. Certain common types of traffic barrier, for example, have no impact on traffic, even when closer than 6 ft to the traveled way. These include the reinforced-concrete traffic barriers and the W-beam barriers often used on freeways.

Illustrations 3-1 and 3-2, shown earlier in this chapter, depict these conditions. In Illustration 3-1, vehicles are affected by both the roadside retaining wall and the low median barrier, as they "shy" away from both. This low median barrier type is rarely used in modern design, and has a significant impact on driver behavior. Illustration 3-2 shows the impact of the roadside obstructions, but the median barrier has little effect, with drivers actually driving closer to it than normal in response to the lateral shifts caused by the roadside obstructions. Illustrations 3-11 and 3-12, in contrast, depict designs in which there are no lane width or lateral clearance restrictions. Neither of the median treatments illustrated represents an effective obstruction in most cases. Some median barriers may restrict sight distance on horizontal curves, and may therefore influence behavior due to this factor.

The adjustment factor, f_w, is given in Table 3-2. The factor is based on the lane width, the distance to the nearest obstruction, the number of lanes on the freeway, and whether the obstruction exists on one or both sides of the freeway. An obstruction on *both* sides of the freeway means that obstructions exist at the roadside and in the median. The left side of the freeway travel lanes in any direction is the median. If the distances to obstructions at the roadside and in the median are different, the average distance is used, and a factor for obstructions on *both* sides of the freeway is selected. Thus, if a freeway had a lateral obstruction 3 ft from the travel lanes at the roadside, and other obstructions 5 ft from the travel lanes in the median, a factor would be selected for obstructions on both sides of the freeway at 4 ft. The factor for 12-ft lanes and obstructions $\geq$ 6 ft from travel lanes is 1.00, as this represents ideal conditions.

As an example, consider an older four-lane freeway which has the following characteristics:

1. Frequent abutments and other obstructions located in the shoulder area, 2 ft from the edge of the travel lanes.
2. A median barrier of the type shown in Illustration 3-1, immediately at the edge of the pavement edge.
3. Eleven-foot lanes.

Table 3-2 is entered with 11-ft lanes, obstructions on *both* sides of the roadway at an *average* of 1 ft from the pavement edge, for a four-lane freeway. The factor found is *0.85*, suggesting that 15 percent of the freeway's ideal capacity is lost due to the lane width and lateral clearance restrictions present.

Adjustment for the Presence of Heavy Vehicles in the Traffic Stream

Values of MSF must be adjusted to reflect the prevailing conditions of taffic streams containing trucks, buses, and/or recreational vehicles. This adjustment is made using the factor f_{HV}.

The factor f_{HV} is found in a two-step process, as follows:

1. Determine the passenger-car equivalent (pce) for each truck, bus, and/or recreational vehicle for the traffic and roadway conditions under study. These values (E_T, E_B, and E_R for trucks, buses, and recreational vehicles respectively) represent the number of passenger cars that would consume the same percentage of the freeway's capacity as one truck, bus, or recreational vehicle under prevailing roadway and traffic conditions.
2. Compute the heavy vehicle adjustment factor f_{HV} using the values of E_T, E_B, E_R, and the proportion of each type of vehicle in the traffic stream (P_T, P_B, and P_R).

Illustration 3-11. This cross section illustrates ideal conditions of lane width and lateral clearance. The concrete median barrier does not cause vehicles to shift their lane position, and therefore would not be considered an "obstruction."

Illustration 3-12. The freeway section shown here is also ideal with respect to lane width and lateral clearances. The W-beam median barrier is another type of barrier which generally does not cause vehicles to shift their lateral lane placement, and also would not be considered an "obstruction" in most cases.

The impact of heavy vehicles on traffic flow depends on the grade conditions as well as the traffic composition. Passenger-car equivalents can be selected for two conditions:

1. *Extended general freeway segments*—It is often possible to consider an extended length of freeway containing a number of upgrades, downgrades, and level segments, as a single uniform segment. This is possible where no one grade is long enough or steep enough to have a significant impact on the overall operation of the general segment. As a rule, extended general segment analysis may be used where *no one grade of 3 percent or greater is longer than ½ mi, or longer than 1 mi for grades less than 3 percent.*

2. *Specific grades*—Any grade less than 3 percent and longer than 1 mi, or any grade of 3 percent or more and longer than ½ mi, is usually analyzed as a separate segment. Such grades may have a significant impact on traffic flow, and must therefore be considered for this possibility.

The choice of which procedure to use is subject to some judgment on the part of the user. Extended general segment analysis is used where no one grade will cause operating conditions to deteriorate significantly below those generally prevailing in the section. Thus, individual steep grades within a generally mountainous terrain might not require separate analysis, whereas one such grade within a generally level terrain would.

TABLE 3-2. ADJUSTMENT FACTOR FOR RESTRICTED LANE WIDTH AND LATERAL CLEARANCE

DISTANCE FROM TRAVELED PAVEMENT[a] (FT)	ADJUSTMENT FACTOR, f_w							
	OBSTRUCTIONS ON ONE SIDE OF THE ROADWAY				OBSTRUCTIONS ON BOTH SIDES OF THE ROADWAY			
	LANE WIDTH (FT)							
	12	11	10	9	12	11	10	9
	4-LANE FREEWAY (2 LANES EACH DIRECTION)							
≥ 6	1.00	0.97	0.91	0.81	1.00	0.97	0.91	0.81
5	0.99	0.96	0.90	0.80	0.99	0.96	0.90	0.80
4	0.99	0.96	0.90	0.80	0.98	0.95	0.89	0.79
3	0.98	0.95	0.89	0.79	0.96	0.93	0.87	0.77
2	0.97	0.94	0.88	0.79	0.94	0.91	0.86	0.76
1	0.93	0.90	0.85	0.76	0.87	0.85	0.80	0.71
0	0.90	0.87	0.82	0.73	0.81	0.79	0.74	0.66
	6- or 8- LANE FREEWAY (3 or 4 LANES EACH DIRECTION)							
≥ 6	1.00	0.96	0.89	0.78	1.00	0.96	0.89	0.78
5	0.99	0.95	0.88	0.77	0.99	0.95	0.88	0.77
4	0.99	0.95	0.88	0.77	0.98	0.94	0.87	0.77
3	0.98	0.94	0.87	0.76	0.97	0.93	0.86	0.76
2	0.97	0.93	0.87	0.76	0.96	0.92	0.85	0.75
1	0.95	0.92	0.86	0.75	0.93	0.89	0.83	0.72
0	0.94	0.91	0.85	0.74	0.91	0.87	0.81	0.70

[a] Certain types of obstructions, high-type median barriers in particular, do not cause any deleterious effect on traffic flow. Judgment should be exercised in applying these factors.

The methodology for finding the appropriate value of f_{HV} is discussed in the following sections:

1. *Passenger car equivalents for extended general freeway segments*—Whenever extended general segment analysis is used, the terrain of the freeway must be classified in one of three categories:

a. *Level terrain*—Any combination of grades and horizontal or vertical alignment permitting heavy vehicles to maintain approximately the same speed as passenger cars; this generally includes short grades of no more than 1 to 2 percent.

b. *Rolling terrain*—Any combination of grades and horizontal or vertical alignment causing heavy vehicles to reduce their speeds substantially below those of passenger cars, but *not* causing heavy vehicles to operate at crawl speeds for any significant length of time.

c. *Mountainous terrain*—Any combination of grades and horizontal or vertical alignment causing heavy vehicles to operate at crawl speeds for significant distances or at frequent intervals.

"Crawl speed" is the maximum sustained speed which trucks can maintain on an extended upgrade of a given percent. If any grade is long enough, trucks will be forced to decelerate to the crawl speed which they will then be able to maintain for extended distances. Appendix I to this chapter contains truck performance curves which illustrate crawl speed and the length of grade over which trucks have usually decelerated to this speed.

The exact categorization of terrain depends on the terrain itself and the prevailing mix of heavy vehicles present. Grades causing large trucks to operate at crawl speed, for example, may not have the same effect on recreational vehicles or buses, or perhaps even smaller trucks.

Passenger-car equivalents for heavy vehicles on general freeway segments are given in Table 3-3.

TABLE 3-3. PASSENGER-CAR EQUIVALENTS ON EXTENDED GENERAL FREEWAY SEGMENTS

FACTOR	TYPE OF TERRAIN		
	LEVEL	ROLLING	MOUNTAINOUS
E_T for Trucks	1.7	4.0	8.0
E_B for Buses	1.5	3.0	5.0
E_R for RV's	1.6	3.0	4.0

2. *Passenger-car equivalents for specific grades*—Any freeway grade of more than 1 mi for grades less than 3 percent, or ½ mi for grades of 3 percent or more is usually considered as a separate segment. For such segments, analysis procedures must consider the upgrade conditions and the downgrade conditions separately, and whether or not the grade is a single, isolated grade of constant percent, or part of a series of grades forming a composite segment.

The performance of heavy vehicles on significant grades varies considerably among the classes of vehicles and among the individual vehicles of a particular category. This is particularly true of trucks and recreational vehicles, both of which cover a wide cross section of vehicles. Intercity buses tend to be more uniform in their characteristics, though there is some variability in this class as well.

Several studies have indicated that freeway truck populations have an average weight-to-horsepower ratio of between 125 and 150 lb/hp. In capacity analysis, however, heavier trucks have a greater impact on traffic flow than lighter trucks. Thus, for capacity analysis purposes, the "typical" truck population is assumed to have a characteristic ratio of 200 lb/hp. Procedures provide options for use where the truck population is either more or less powerful than usual. Tabulations are provided for a more powerful truck population with a ratio of 100 lb/hp and a less powerful population with a ratio of 300 lb/hp.

Recreational vehicles (RV's) vary considerably in both type and characteristics. These vehicles range from cars with trailers of various types to self-contained mobile campers. In addition, drivers of recreational vehicles are not professionals, and their degree of skill in handling such vehicles covers a broad range. "Typical" weight-to-horsepower ratios of recreational vehicles range from 30 to 60 lb/hp. Passenger-car equivalents for RV's vary from one-third to one-half of comparable values for a typical truck.

There has been comparatively little research on the performance characteristics of buses over the past decade, and current information on passenger-car equivalents is limited to that available in the early 1960's.

a. *Upgrades*—Tables 3-4 through 3-8 give values of passenger-car equivalents for use in capacity analysis. These represent the upgrade condition only, and are as follows:

Table	*Value*	*Tabulated Vehicle Type*
3.4	E_T	Typical Trucks (200 lb/hp)
3.5	E_T	Light Trucks (100 lb/hp)
3.6	E_T	Heavy Trucks (300 lb/hp)
3.7	E_R	Recreational Vehicles
3.8	E_B	Buses

Passenger-car equivalent values depend on number of variables, including the type of vehicle, the percentage and length of grade, and the percentage of heavy vehicles in the traffic stream. As heavy vehicles travel up a grade, their impact becomes progressively severe as their speeds decrease. Thus, for most analyses, passenger-car equivalents are selected for a point at the end of the grade. There are occasions, however, when an intermediate grade point will be of interest. If a ramp junction occurred on an extended upgrade, for example, the length and percent of grade to the junction would be of interest for analyzing the merge or diverge movements. If a composite grade started with a 5 percent upgrade followed by a 2 percent upgrade, heavy vehicles would be traveling the slowest at the end of the 5 percent portion of the grade. That point would then be of primary interest.

The length of grade is generally taken from a profile of the highway in question, and generally includes the straight portion of the grade plus some portion of the vertical curves at the beginning and end of this grade. It is suggested that one-quarter of the length of the vertical curves at the beginning and end of the grade be included in the total grade length. Where two consecutive upgrades are joined by a vertical curve, one-half of the length of curve is included with each portion of the grade.

b. *Downgrades*—Very little specific data exist on the impact of heavy vehicles on traffic flow on downgrades. In general, if a downgrade is not so severe as to cause heavy vehicles to shift into a low gear, it may be treated as if it were a level terrain segment, and passenger-car equivalents are selected accordingly from Table 3-3. Grades less than 4 percent or shorter than 3,000 ft would generally fall into this category. Where more severe downgrades occur, the passenger-car equivalent is best estimated by taking field measurements of speed and using the equivalent for a comparable upgrade condition. The "equivalent" upgrade

TABLE 3-4. PASSENGER-CAR EQUIVALENTS FOR TYPICAL TRUCKS (200 LB/HP)

GRADE (%)	LENGTH (MI)	PASSENGER-CAR EQUIVALENT, E_T 4-LANE FREEWAYS								6–8 LANE FREEWAYS							
PERCENT TRUCKS		2	4	5	6	8	10	15	20	2	4	5	6	8	10	15	20
< 1	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
1	0–1/2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	1/2–1	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	≥ 1	4	3	3	3	3	3	3	3	4	3	3	3	3	3	3	3
2	0–1/4	4	4	4	3	3	3	3	3	4	4	4	3	3	3	3	3
	1/4–1/2	5	4	4	3	3	3	3	3	5	4	4	3	3	3	3	3
	1/2–3/4	6	5	5	4	4	4	4	4	6	5	5	4	4	4	4	4
	3/4–1½	7	6	6	5	4	4	4	4	7	5	5	5	4	4	4	4
	≥ 1½	8	6	6	6	5	5	4	4	8	6	6	5	4	4	4	4
3	0–1/4	6	5	5	5	4	4	4	3	6	5	5	5	4	4	4	3
	1/4–1/2	8	6	6	6	5	5	5	4	7	6	6	6	5	5	5	4
	1/2–1	9	7	7	6	5	5	5	5	9	7	7	6	5	5	5	5
	1–1½	9	7	7	7	6	6	5	5	9	7	7	6	5	5	5	5
	≥ 1½	10	7	7	7	6	6	5	5	10	7	7	6	5	5	5	5
4	0–1/4	7	6	6	5	4	4	4	4	7	6	6	5	4	4	4	4
	1/4–1/2	10	7	7	6	5	5	5	5	9	7	7	6	5	5	5	5
	1/2–1	12	8	8	7	6	6	6	6	10	8	7	6	5	5	5	5
	≥ 1	13	9	9	9	8	8	7	7	11	9	9	8	7	6	6	6
5	0–1/4	8	6	6	6	5	5	5	5	8	6	6	6	5	5	5	5
	1/4–1/2	10	8	8	7	6	6	6	6	8	7	7	6	5	5	5	5
	1/2–1	12	11	11	10	8	8	8	8	12	10	9	8	7	7	7	7
	≥ 1	14	11	11	10	8	8	8	8	12	10	9	8	7	7	7	7
6	0–1/4	9	7	7	7	6	6	6	6	9	7	7	6	5	5	5	5
	1/4–1/2	13	9	9	8	7	7	7	7	11	8	8	7	6	6	6	6
	1/2–3/4	13	9	9	8	7	7	7	7	11	9	9	8	7	6	6	6
	≥ 3/4	17	12	12	11	9	9	9	9	13	10	10	9	8	8	8	8

NOTE: If a length of grade falls on a boundary condition, the equivalent for the longer grade category is used. For any grade steeper than the percentage shown, use the next higher grade category.

TABLE 3-5. PASSENGER-CAR EQUIVALENTS FOR LIGHT TRUCKS (100 LB/HP)

GRADE (%)	LENGTH (MI)	PASSENGER-CAR EQUIVALENT, E_T — 4-LANE FREEWAYS								6–8 LANE FREEWAYS							
PERCENT TRUCKS		2	4	5	6	8	10	15	20	2	4	5	6	8	10	15	20
≤ 2	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
3	0–1/4	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	1/4–1/2	4	4	4	3	3	3	3	3	4	4	4	3	3	3	3	3
	1/2–3/4	4	4	4	4	3	3	3	3	4	4	4	3	3	3	3	3
	3/4–1	5	4	4	4	3	3	3	3	5	4	4	4	3	3	3	3
	> 1	6	5	5	5	4	4	4	3	6	5	5	4	4	4	3	3
4	0–1/4	4	4	4	3	3	3	3	3	5	4	4	4	3	3	3	3
	1/4–1/2	5	5	5	4	4	4	4	4	5	4	4	4	4	4	4	4
	1/2–1	6	5	5	5	4	4	4	4	6	5	5	4	4	4	4	4
	> 1	7	6	6	5	4	4	4	4	7	5	5	5	4	4	4	4
5	0–1/4	6	5	5	5	4	4	4	3	6	5	5	5	4	4	4	3
	1/4–1	8	7	7	6	5	5	5	5	8	7	7	6	5	5	5	5
	> 1	9	7	7	6	5	5	5	5	8	7	7	6	5	5	5	5
6	0–1/4	7	5	5	5	4	4	4	4	7	5	5	5	4	4	3	3
	1/4–1	9	7	7	6	5	5	5	5	8	7	7	6	5	5	5	5
	> 1	9	7	7	7	6	6	5	5	9	7	7	6	5	5	5	5

NOTE: If a length of grade falls on a boundary condition, the equivalent from the longer grade category is used. For any grade steeper than the percentage shown, use the next higher grade category.

TABLE 3-6. PASSENGER-CAR EQUIVALENTS FOR HEAVY TRUCKS (300 LB/HP)

GRADE (%)	LENGTH (MI)	PASSENGER-CAR EQUIVALENT, E_T — 4-LANE FREEWAYS								6–8 LANE FREEWAYS							
PERCENT TRUCKS		2	4	5	6	8	10	15	20	2	4	5	6	8	10	15	20
< 1	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
1	0–1/4	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	1/4–1/2	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	1/2–3/4	4	4	4	4	3	3	3	3	4	4	4	3	3	3	3	3
	3/4–1	5	4	4	4	3	3	3	3	5	4	4	4	3	3	3	3
	1–1½	6	5	5	5	4	4	4	3	6	5	5	4	4	4	3	3
	> 1½	7	5	5	5	4	4	4	3	7	5	5	5	4	4	3	3
2	0–1/4	4	4	4	3	3	3	3	3	4	4	4	3	3	3	3	3
	1/4–1/2	7	6	6	5	4	4	4	4	7	5	5	5	4	4	4	4
	1/2–3/4	8	6	6	5	5	4	4	4	8	6	6	6	5	5	4	4
	3/4–1	8	6	6	6	5	5	5	5	8	6	6	6	5	5	5	5
	1–1½	9	7	7	7	6	6	5	5	9	7	7	6	5	5	5	5
	> 1½	10	7	7	7	6	6	5	5	10	7	7	6	5	5	5	5
3	0–1/4	6	5	5	5	4	4	4	3	6	5	5	5	4	4	4	3
	1/4–1/2	9	7	7	6	5	5	5	5	8	7	7	6	5	5	5	5
	1/2–3/4	12	8	8	7	6	6	6	6	10	8	7	6	5	5	5	5
	3/4–1	13	9	9	8	7	7	7	7	11	8	8	7	6	6	6	6
	> 1	14	10	10	9	8	8	7	7	12	9	9	8	7	7	7	7
4	0–1/4	7	5	5	5	4	4	4	4	7	5	5	5	4	4	3	3
	1/4–1/2	12	8	8	7	6	6	6	6	10	8	7	6	5	5	5	5
	1/2–3/4	13	9	9	8	7	7	7	7	11	9	9	8	7	6	6	6
	3/4–1	15	10	10	9	8	8	8	8	12	10	10	9	8	7	7	7
	> 1	17	12	12	10	9	9	9	9	13	10	10	9	8	8	8	8
5	0–1/4	8	6	6	6	5	5	5	5	8	6	6	6	5	5	5	5
	1/4–1/2	13	9	9	8	7	7	7	7	11	8	8	7	6	6	6	6
	1/2–3/4	20	15	15	14	11	11	11	11	14	11	11	10	9	9	9	9
	> 3/4	22	17	17	16	13	13	13	13	17	14	14	13	12	11	11	11
6	0–1/4	9	7	7	7	6	6	6	6	9	7	7	6	5	5	5	5
	1/4–1/2	17	12	12	11	9	9	9	9	13	10	10	9	8	8	8	8
	> 1/2	28	22	22	21	18	18	18	18	20	17	17	16	15	14	14	14

NOTE: If a length of grade falls on a boundary condition, the equivalent from the longer grade category is used. For any grade steeper than the percent shown, use the next higher grade category.

TABLE 3-7. PASSENGER-CAR EQUIVALENTS FOR RECREATIONAL VEHICLES

GRADE (%)	LENGTH (MI)	PASSENGER-CAR EQUIVALENT, E_R															
		4-LANE FREEWAYS								6–8 LANE FREEWAYS							
PERCENT RV's		2	4	5	6	8	10	15	20	2	4	5	6	8	10	15	20
<2	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
3	0–1/2	3	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	≥ 1/2	4	3	3	3	3	3	3	3	4	3	3	3	3	3	3	3
4	0–1/4	3	2	2	2	2	2	2	2	3	2	2	2	2	2	2	2
	1/4–3/4	4	3	3	3	3	3	3	3	4	3	3	3	3	3	3	3
	≥ 3/4	5	4	4	4	3	3	3	3	4	4	4	4	3	3	3	3
5	0–1/4	4	3	3	3	3	3	3	3	4	3	3	3	2	2	2	2
	1/4–3/4	5	4	4	4	4	4	4	4	5	4	4	4	4	4	4	4
	≥ 3/4	6	5	4	4	4	4	4	4	5	5	4	4	4	4	4	4
6	0–1/4	5	4	4	4	3	3	3	3	5	4	4	3	3	3	3	3
	1/4–3/4	6	5	5	4	4	4	4	4	6	4	4	4	4	4	4	4
	≥ 3/4	7	6	6	6	5	5	5	5	6	5	5	5	4	4	4	4

NOTE: If a length of grade falls on a boundary condition, the equivalent from the longer grade category is used. For any grade steeper than the percent shown, use the next higher grade category.

TABLE 3-8. PASSENGER-CAR EQUIVALENTS FOR BUSES

GRADE (%)	PASSENGER-CAR EQUIVALENT, E_B
0–3	1.6
4[a]	1.6
5[a]	3.0
6[a]	5.5

[a] Use generally restricted to grades more than 1/4-mi long.

is a length of upgrade of percent equal to the existing downgrade which results in the same final speed of trucks as measured on the actual downgrade. The truck performance curves of Appendix I are used for this purpose. Where such field measurements are not practical, the downgrade equivalent may be estimated very roughly as one-half the corresponding upgrade equivalent.

c. *Composite grades*—The vertical alignment of most freeways results in a continuous series of grades. It is often necessary to find the impact of a series of significant grades in succession. Consider the following example. A 3 percent grade of ½ mi is followed immediately by a 4 percent grade of 1 mi. The analysis problem of interest is the maximum impact of heavy vehicles, which would occur at the end of the 4 percent segment. The most straightforward technique is to compute the *average grade* to the point in question. The average grade is defined as the total rise (in feet) from the beginning of the composite grade divided by the length of the grade (in feet). For the example cited:

$$\text{Total Rise} = 2{,}640 \times 0.03 + 5{,}280 \times 0.04 = 290.4 \text{ ft}$$
$$\text{Average Grade} = 290.4/7{,}920 = 0.037 \text{ or } 3.7 \text{ percent}$$
$$\text{Note: } 2{,}640 \text{ ft} = \tfrac{1}{2} \text{ mi}$$

Passenger-car equivalents for this composite grade would be found for a 4 percent grade (values are usually rounded to the nearest percent), 1½ mi in length.

The average grade technique is an acceptable approach for grades less than 4 percent or shorter than 3,000 ft in total length. For more severe composite grades, a detailed technique is presented in Appendix I to this chapter. That more exact technique uses vehicle performance curves and equivalent speeds to determine the effective simple grade for analysis.

3. *Computing the adjustment factor for heavy vehicles*—Once the values of E_T, E_B, and E_R are found, the determination of the adjustment factor, f_{HV}, is a straightforward algebraic exercise:

$$f_{HV} = 1/[1 + P_T(E_T - 1) + P_R(E_R - 1) + P_B(E_B - 1)] \quad (3\text{-}4)$$

where:

f_{HV} = the adjustment factor for the combined effect of trucks, recreational vehicles, and buses on the traffic stream;

E_T, E_R, E_B = the passenger-car equivalents for trucks, recreational vehicles, and buses respectively; and

P_T, P_R, P_B = the proportion of trucks, recreational vehicles, and buses, respectively, in the traffic stream.

In many cases, only one heavy vehicle type will be present in the traffic stream to a significant degree. Where the percentage of RV's and buses is small in comparison to the percentage of trucks, it is sometimes convenient to consider all vehicles to be trucks. Thus, a traffic stream consisting of 15 percent trucks, 2 percent RV's, and 1 percent buses might be analyzed as having 18 percent trucks. It is generally acceptable to do this where the percentage of trucks in the traffic stream is at least 5 times the total percentage of RV's plus buses present. In such cases, the adjustment factor, f_{HV}, may be obtained from Table 3-9, instead of computing it using Eq. 3-4. This table may also be used if all heavy vehicles are RV's or buses.

If the problem noted previously were for a freeway with

TABLE 3-9. ADJUSTMENT FACTOR FOR THE EFFECT OF TRUCKS, BUSES, OR RECREATIONAL VEHICLES IN THE TRAFFIC STREAM

PCE[a] E_T E_R or E_B	ADJUSTMENT FACTOR, f_{HV} — PROPORTION OF TRUCKS, P_T; RV'S, P_R; OR BUSES, P_B														
	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09	0.10	0.12	0.14	0.16	0.18	0.20
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
16	0.87	0.77	0.69	0.63	0.57	0.53	0.49	0.45	0.43	0.40	0.36	0.32	0.29	0.27	0.25
17	0.86	0.76	0.68	0.61	0.56	0.51	0.47	0.44	0.41	0.38	0.34	0.31	0.28	0.26	0.24
18	0.85	0.75	0.66	0.60	0.54	0.49	0.46	0.42	0.40	0.37	0.33	0.30	0.27	0.25	0.23
19	0.85	0.74	0.65	0.58	0.53	0.48	0.44	0.41	0.38	0.36	0.32	0.28	0.26	0.24	0.22
20	0.84	0.72	0.64	0.57	0.51	0.47	0.42	0.40	0.37	0.34	0.30	0.27	0.25	0.23	0.21
21	0.83	0.71	0.63	0.56	0.50	0.45	0.41	0.38	0.36	0.33	0.29	0.26	0.24	0.22	0.20
22	0.83	0.70	0.61	0.54	0.49	0.44	0.40	0.37	0.35	0.32	0.28	0.25	0.23	0.21	0.19
23	0.82	0.69	0.60	0.53	0.48	0.43	0.39	0.36	0.34	0.31	0.27	0.25	0.22	0.20	0.19
24	0.81	0.68	0.59	0.52	0.47	0.42	0.38	0.35	0.33	0.30	0.27	0.24	0.21	0.19	0.18
25	0.80	0.67	0.58	0.51	0.46	0.41	0.37	0.34	0.32	0.29	0.26	0.23	0.20	0.18	0.17

[a] Passenger-car equivalent, obtained from Table 3-3, 3-4, 3-5, or 3-6.
NOTE: This table should not be used when the combined percentage of buses and RV's in the traffic stream is more than one-fifth the percentage of trucks.

generally rolling terrain, Table 3-9 would be used as follows. Enter the table with 18 percent trucks and a value of E_T of 4 (from Table 3-3). The value of f_{HV} is read directly as 0.65.

Adjustment for Driver Population

The traffic stream characteristics on which the criteria presented in this section are based are representative of regular weekday drivers in a commuter traffic stream or other regular users of a facility. It is generally accepted that traffic streams with different characteristics (weekend, recreational, perhaps even mid-day) use freeways less efficiently. Although data are sparse, and reported results vary substantially, capacities in the range of 1,500 to 1,600 pcphpl have been reported on weekends, particularly in recreational areas. It may generally be assumed that this reduction in capacity extends to service flow rates for other levels of service as well.

The adjustment factor f_p is used to reflect the influence of driver population. Table 3-10 provides values that can be used with caution. The use of this factor calls for judgment in determining its exact value, and the analyst should apply general knowledge of the subject facility and its environs in selecting a value. Where great accuracy is needed, comparative field studies of weekday and weekend traffic flows and speeds are recommended.

In some cases, it may be useful to conduct sensitivity analyses using a range of values for f_p, including the minimum value of 0.75, to determine whether the selection of a precise value seriously affects the results of the analysis. Practical application of this methodology in operational analysis, design, and planning of freeways is detailed in the next section.

TABLE 3-10. ADJUSTMENT FACTOR FOR THE CHARACTER OF THE TRAFFIC STREAM

TRAFFIC STREAM TYPE	FACTORS, f_p
Weekday or Commuter	1.0
Other	0.75–0.90[a]

[a] Engineering judgment must be exercised in selecting an exact value.

III. PROCEDURES FOR APPLICATION

The methodology presented in the previous section is most often used in one of three applications:

1. *Operational analysis*—Operational analysis involves the consideration of a known present or projected future freeway. Given known or projected geometric roadway conditions and known or projected traffic conditions, the analysis yields an estimate of the level of service and of the speed and density of the traffic stream. This is the most detailed of the three applications, and requires precise input information for roadway and traffic conditions. Operational analysis also provides the most versatile use of the methodology. It is extremely useful in evaluating the likely impacts of proposed spot or segment improvements, and can be used to evaluate alternative design proposals.
2. *Design*—In design, a forecast demand volume is used in conjunction with known design standards for geometric features and a desired level of service to compute the number of lanes required for the freeway section in question. The design application is straightforward for each usage, but trial-and-error operational analyses may be required to evaluate alternative designs. Design requires a detailed traffic forecast, including volumes, peaking characteristics, traffic composition, and specifics of vertical and horizontal alignment for the sections under study.
3. *Planning*—The objective of a planning application is the same as for design: determination of the number of lanes required for a segment of freeway. The planning application, however, focuses on an early and approximate determination before the details of a complete traffic forecast and the vertical and horizontal alignment of the facility are known. Given a general forecast average annual daily traffic, *AADT*, the approximate percentage of trucks, the general terrain classification (level, rolling, mountainous), and the desired level of service, a preliminary estimate of the number of lanes needed can be made.

The user is cautioned that these procedures are intended to be used as a guide, and do not replace the responsibility for decision-making or selection among viable alternatives. Procedures outlined herein will give the analyst additional information on either likely operating conditions and/or the number of lanes needed to provide for specified desired operating conditions. This information is an important input to decision-making on freeway projects. There are other criteria, however, including cost-effectiveness and environmental impacts. No result from these procedures should be construed as mandating a particular solution to a specific problem. The procedures do not make decisions, rather, they provide meaningful information to the engineers and planners who must.

OPERATIONAL ANALYSIS

Objectives of Operational Analysis

An operational analysis is an analytic evaluation of operations on an existing freeway segment. The same type of analysis may be applied to evaluate probable operating conditions on a future facility. In either case, all traffic and roadway conditions must be specified, as well as traffic volumes. The output of operational analysis is an estimate of the level of service for the segment in question and of the approximate speed and density at which the traffic stream operates.

Data Requirements

Operational analysis requires detailed information concerning the freeway segment(s) in question. These data must be available from field studies of an existing site, or must be forecast for future evaluations. The following information is required:

1. *Traffic volumes* for the peak hour (or any other hour of interest).
2. *Traffic characteristics,* including composition (percentage of trucks, RV's, and buses), the peak hour factor (PHF), and the driver population (weekday, commuter, recreational, etc.).
3. *Roadway characteristics,* including lane widths, lateral clearances, design speeds, grades, etc.

Segmenting the Freeway for Analysis

An analysis must consider freeway segments with uniform characteristics. Thus, in each segment analyzed, each of the data elements noted previously, i.e., traffic volumes, traffic characteristics, and roadway characteristics, must be constant. A change in any of the data indicates the need to separate the freeway into an additional segment for analysis.

In considering a long section of freeway, there are critical locations which generally serve as boundaries for analysis segments. Ramp junctions are often boundary points because the demand volume changes at these points. Weaving areas should be isolated for separate analysis (see Chapter 4), and freeway segments on either side of a weaving section are most often considered separately. Isolated grades having a significant impact on operations are also segmented for separate analysis. Any other points bounding a marked change in terrain similarly would be candidates for identifying separate freeway segments for analysis.

The designation of uniform segments for analysis requires some judgment, and the guidelines discussed herein should be viewed as general suggestions, not absolute criteria.

Procedural Steps

The general procedure for performing an operational analysis is to use the basic Eq. 3-2 or Eq. 3-3 to compute the effective maximum service flow rate, *MSF,* or the effective v/c ratio, for the segment in question. Either of these values can be used in conjunction with Table 3-1 to determine the level of service, and with Figures 3-3 and 3-4 to determine the approximate density and speed conditions of the traffic stream. The following step-by-step procedure can be used in performing these computations:

1. Convert all volumes to peak 15-min flow rates. Note that as a computational device, the service flow rate, *SF*, is set equal to the actual peak flow rate, as follows:

$$SF = V/\text{PHF}$$

where:

SF = the service flow rate for the segment in question, in vph;
V = the actual hourly demand volume for the segment in question, in vph; and
PHF = the peak hour factor for the segment in question.

2. Adjustment factors and passenger-car equivalents for prevailing conditions are obtained from the appropriate tables:

f_w (Table 3-2)
E_T (Table 3-3, 3-4, 3-5, or 3-6)
E_R (Table 3-3 or 3-7)
E_B (Table 3-3 or 3-8)
f_{HV} (Table 3-9, or compute from Eq. 3-4)
f_p (Table 3-10)

3. Determine the effective *MSF* or *v/c* ratio using Eq. 3-2 or Eq. 3-3, as follows:

$$MSF = SF/[N \times f_w \times f_{HV} \times f_p]$$

or

$$v/c = SF/[c_j \times N \times f_w \times f_{HV} \times f_p]$$

Either equation may be used, because both *MSF* and *v/c* ratio are tabulated for the various levels of service, and the two values are related on a one-to-one basis.

4. Compare the effective *MSF* or the effective *v/c* ratio to the criteria of Table 3-1 to determine level of service. *MSF* or *v/c* must be less than the tabulated criteria to fall within a given level of service.

5. Using the effective *MSF* or *v/c* ratio, Figure 3-3 is used to find the approximate density of the traffic stream, and Figure 3-4 is used to find the approximate average travel speed of the traffic stream.

Figure 3-5 illustrates a worksheet that may be used to summarize operational analysis computations.

For example, if a 70-mph freeway were found to have an *MSF* of 1,685 pcphpl, Table 3-1 would be used to find the level of service. Because 1,685 pcphpl is less than 1,850 pcphpl (the

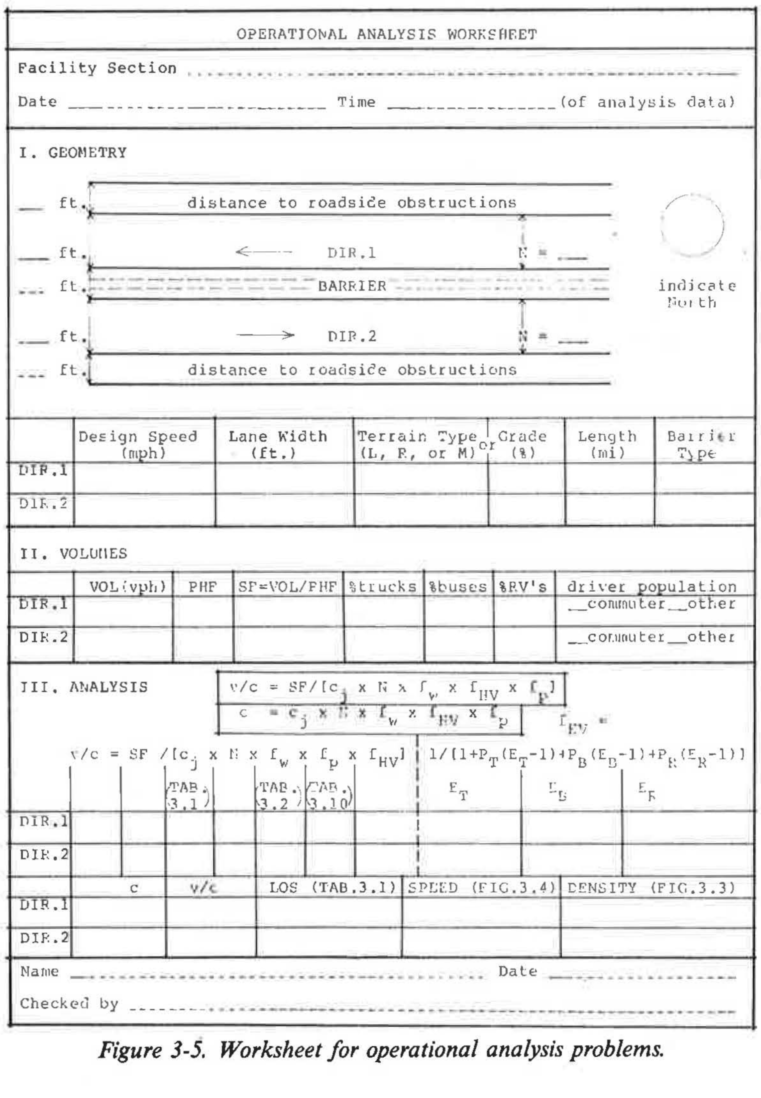


Figure 3-5. Worksheet for operational analysis problems.

maximum value for LOS D), but more than 1,550 pcphpl (the maximum value for LOS C), the segment is operating at level-of-service D.

Further, Figures 3-3 and 3-4 would be entered with 1,685 pcphpl to find the approximate speed and density as shown in Figure 3-6. The results are a speed of 53 mph and a density of 32 pc/mi/ln, as illustrated in Figure 3-6.

Interpretation of Results

The results of an operational analysis yield a description of the probable operating conditions for a given traffic stream on a given segment of freeway. These estimates are based on the typical speed-flow-density conditions illustrated herein. There will, however, be some variation from these estimates because of regional driver habits or other unique local characteristics.

Densities greater than 42 pc/mi/ln are generally unstable, and small increases in flow or minor incidents will cause rapid breakdown of the traffic stream. This is the same flow range in which speed deteriorates rapidly with small increases in flow.

Operational analysis of freeway segments can be used to evaluate current operations or likely future operations. It is also used to find and evaluate "trouble spots" of congestion and potential remedies to such situations.

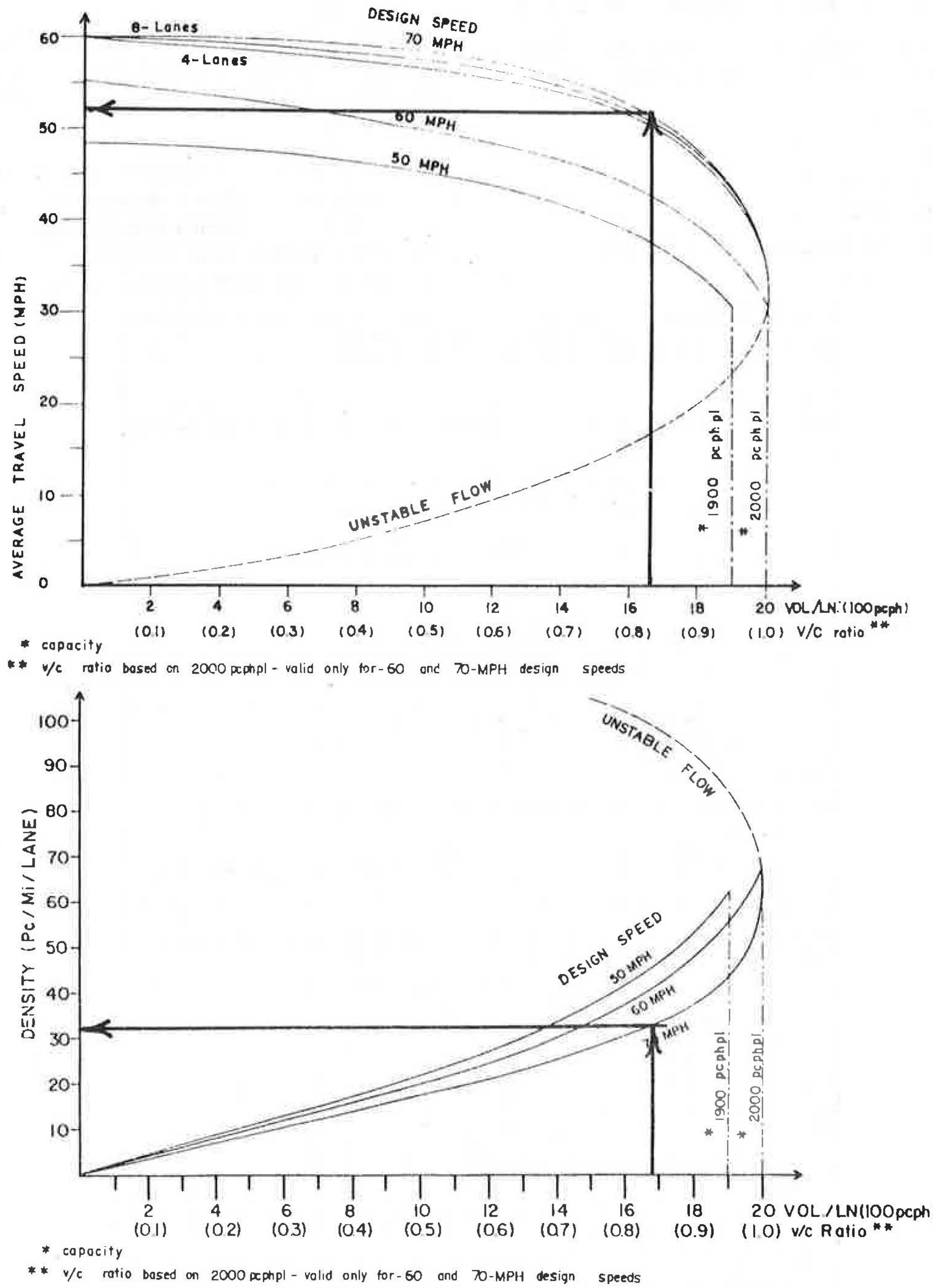


Figure 3-6. Example solutions for approximate density and speed of a freeway traffic stream.

DESIGN

Objectives of Design

A design analysis is made to determine the number of lanes required on the freeway to provide the desired level of service for the forecasted traffic volume and traffic characteristics.

Data Requirements

Design analysis requires information concerning the projected directional design hour volume, *DDHV,* and the traffic characteristics that describe it. Design standards, such as design speed, lane widths, and lateral clearances, must also be specified. The horizontal and vertical alignment of the facility would generally be established before the consideration of capacity, so that details of grades and horizontal curvature would also be available.

The following information is required:

1. *Geometric design standards* must be selected for lane width, lateral clearance, and design speed. The design speed will be influenced by the horizontal and vertical alignments of the facility.
2. The *directional design hour volume, DDHV,* must be forecast for the design year.
3. *Traffic characteristics* must be specified: composition (percentage of trucks, RV's, and buses), the peak hour factor, PHF, and the driver population (weekday, commuter, recreational, etc.).

Segmenting the Freeway for Design

The freeway must be divided into segments yielding uniform characteristics. The horizontal and vertical alignments must be examined to identify points at which the terrain changes, and to isolate specific grades requiring separate analysis. It is often necessary to segment the freeway at ramps and major junctions because the volume generally will change at these points.

Design Criteria

Design analysis also requires the selection of a design level of service, which determines the design value of v/c. The characteristics of modern freeway flow make it difficult to use Table 3-1 directly for this purpose. At LOS C, D, and E, the range of flows is quite small, while at LOS A and B it is quite large. This is a result of speed and density characteristics, both of which deteriorate rapidly with small changes in flow as capacity is approached. This, however, gives the designer a rather small range of practical options.

In design, Table 3-11 is used to select a design v/c ratio. Values of v/c, in increments of 0.10 from 0.30 to 0.80, are given, as are the equivalent values of *MSF,* together with the LOS, speed, and density which would occur at such values. Using these design values, a design may be attempted at points throughout the LOS range, not just at the boundaries between levels.

Relationship of Design Criteria to AASHTO Standards

Current AASHTO design standards refer to level-of-service criteria that are not the same as those in this and other chapters of this manual.

AASHTO standards recommend that urban freeways should not operate with volumes higher than 1,500 to 1,700 pcphpl, and rural freeways no higher than 1,000 to 1,200 pcphpl. With respect to design levels of service, current AASHTO recommendations are *approximately* comparable to the following v/c ratios:

Rural Freeways	0.60
Urban and Suburban Freeways	0.80

It is important to note, therefore, that AASHTO policies based on previous documents may not be applied directly to this procedure because LOS designations and criteria are not the same.

Procedural Steps

The basic analytic procedure for design purposes is to solve for the number of lanes needed (in each direction) on each freeway segment by using Eq. 3-3 or Eq. 3-4. The following steps are used:

1. Convert the directional design hour volume, *DDHV,* to an equivalent peak flow rate, which is set equal to the service flow rate, *SF:*

$$SF = DDHV/\mathrm{PHF}$$

All terms are as previously defined.

2. Find all adjustment factors and passenger-car equivalents, based on forecast traffic characteristics and selected design standards:

f_w (Table 3-2)
E_T (Table 3-3, 3-4, 3-5, or 3-6)
E_R (Table 3-3 or 3-7)
E_B (Table 3-3 or 3-8)
f_{HV} (Table 3-9, or compute from Eq. 3-4)
f_p (Table 3-10)

3. Select a design v/c ratio, or corresponding *MSF,* from Table 3-11.
4. Solve for *N,* the number of lanes needed in each direction as follows:

$$N = SF/[c_j \times (v/c) \times f_w \times f_{HV} \times f_p]$$

or

$$N = SF/[MSF \times f_w \times f_{HV} \times f_p]$$

where c_j = 2,000 pcphpl for 60- and 70-mph freeway elements, and 1,900 pcphpl for 50-mph freeway elements.

TABLE 3-11. VALUES OF VOLUME-TO-CAPACITY RATIO FOR USE IN DESIGN

v/c RATIO	MSF[a] (PCPHPL)	RESULTING PERFORMANCE CHARACTERISTICS		
		LOS[b]	DENSITY (PC/MI/LN)	SPEED (MPH)
		70-MPH ELEMENTS		
0.30	600	A	10.5	60
0.35[c]	700	A	12.0	60
0.40	800	B	14.0	59
0.50	1,000	B	17.5	58
0.54[c]	1,100	B	20.0	57
0.60	1,200	C	21.0	56
0.70	1,400	C	25.0	55
0.77[c]	1,550	C	30.0	54
0.80	1,600	D	30.5	52
		60-MPH ELEMENTS		
0.30	600	B	12.0	52
0.40	800	B	15.5	52
0.49[c]	1,000	B	20.0	50
0.60	1,200	C	25.0	48
0.69[c]	1,400	C	30.0	47
0.80	1,600	D	37.5	43
		50-MPH ELEMENTS		
0.30	550	C	13.0	47
0.40	750	C	17.0	47
0.50	950	C	22.0	45
0.60	1,150	C	27.0	44
0.67[c]	1,300	C	30.0	43
0.70	1,350	D	34.0	41
0.80	1,500	D	42.0	40

[a] Values rounded to the nearest 50 pcphpl.
[b] Design may be within LOS bounds, not necessarily at maximum condition for LOS.
[c] Maximum permissible value for the LOS shown.

Interpretation of Results

The design procedure results in a direct computation of N for a given freeway segment. Care should be exercised in such design computations because N may be different for successive segments (geometric and/or traffic conditions change) or even for two directions of the same segment (particularly on significant grades).

A special procedure for the consideration of truck climbing lanes is given later in this chapter, and should be consulted wherever the initial analysis indicates an additional lane or lanes are required on the upgrade.

Also note that the solution for N will most often yield a fractional result. A decision must then be made to go either to the next full integer, or to raise the design v/c value to allow the next smaller integer value. This is often a complex decision that may include economic and other considerations. The operational result of either option should be investigated by subjecting the alternative designs to operational analysis as described in the previous section.

It should also be noted that a decision on the number of lanes to be used on a specific segment of freeway cannot be made without a review of the lane requirements throughout the freeway system in question. Lane additions or subtractions for specific segments must consider the availability of appropriate locations for such changes. Lane continuity related to major traffic flows must also be considered. Consult Chapter 6 for a more detailed discussion of freeway system requirements and analysis.

Figure 3-7 presents a worksheet which may be used in conjunction with design computations.

PLANNING

Objectives in Freeway Planning

The objectives of a freeway capacity analysis at the planning level are principally the same as those of a design analysis: determine the number of freeway lanes needed to achieve a

DESIGN WORKSHEET

Facility Section ______________________________

Date ________________ Time ____________ (of analysis data)

I. DESIGN STANDARDS

	LOS	v/c (TAB. 3.1)	Des. Speed (mph)	Lane width (ft.)	Lateral clearance (ft.) roadside	Lateral clearance (ft.) median	Terrain or (L, P, or M)	Grade (%)	length (mi)
DIR. 1									
DIR. 2									

II. TRAFFIC FORECASTS

	DDHV (vph)	PHF	SF = (DDHV/PHF)	%trucks	%buses	%RV's	driver population
DIR. 1							__commuter __other
DIR. 2							__commuter __other

III. DESIGN ANALYSIS

$$\boxed{N = SF/[c_j \times v/c \times f_w \times f_{HV} \times f_p]}$$

$$f_{HV} = 1/[1 + P_T(E_T - 1) + P_B(E_B - 1) + P_R(E_R - 1)]$$

	N	SF	c_j (TAB. 3.1)	(v/c)	f_w (TAB. 3.2)	f_p (TAB. 3.10)	f_{HV} (TAB. 3.9)	E_T	E_B	E_R
DIR. 1										
DIR. 2										

IV. SKETCH DESIGN

Name ____________________ Date ____________

Checked by ____________________________

Figure 3-7. Worksheet for design analysis problems.

desired level of service for the projected traffic flows and characteristics. The primary difference between design and planning analyses is the amount and detail of information available as inputs into the analysis.

In the planning stage, details of specific grades and other geometric features do not exist. Further, traffic forecasts are not precise. Thus, at the planning level, capacity analysis is approximate, and serves to give a general idea of the freeway geometrics required. This determination, however, must be subjected to a full segment-by-segment design analysis when these details become available.

Data Requirements for Planning

To conduct a planning analysis, only the following information is needed:

1. A forecast of *AADT* in the anticipated design year.
2. A forecast of the likely truck percentage.
3. A general classification of terrain type.

The *AADT* is a necessary input for any highway planning, and will generally be available for capacity analysis. Vertical alignment and truck presence may be only estimates on the part of the analyst, based on the general terrain conditions of the area through which the freeway will pass and on the anticipated character of traffic which is intended to be served.

Procedural Steps in Planning

The following steps are involved in conducting a planning analysis:

1. Convert *AADT* to *DDHV* using Eq. 3-6:

$$DDHV = AADT \times K \times D \qquad (3\text{-}6)$$

where:

$AADT$ = forecast average annual daily traffic, in vpd;
$DDHV$ = directional design hour volume, in vph;
K = percent of *AADT* occurring in peak hour; and
D = percent of peak hour traffic in the heaviest direction.

Values of K and D should be based on local or regional characteristics. If such information is unavailable, the following approximations may be used:

For K:	Urban Freeways	0.07 – 0.10
	Suburban Freeways	0.10 – 0.15
	Rural Freeways	0.15 – 0.20

In general, as the density of land use increases, the K-factor decreases, because traffic demand is distributed more smoothly throughout the day.

For D:	Urban Circumferential Freeways	0.50
	Urban Radial Freeways	0.55
	Rural Freeways	0.65

2. Select an appropriate value of *SFL*, the service flow rate per lane, from Table 3-12 for the prevailing truck percentage and terrain, and for the desired LOS. Table 3-12 values are based on a number of assumptions concerning likely conditions. These include an assumption that all heavy vehicles are 200-lb/hp trucks, that lane widths and lateral clearances are ideal, and that the alignment has a 70-mph design speed.

3. Compute the number of lanes that would be required in each direction of the freeway using Eq. 3-7:

$$N = DDHV/[SFL \times \text{PHF}] \qquad (3\text{-}7)$$

The inclusion of the PHF in the equation automatically considers the peak 15-min flow rate in the determination of N.

Interpretation of Results

The results of a planning analysis are straightforward. It should be remembered, however, that it is based on general planning information which may change as the freeway project moves from planning to design. The results of a planning analysis should not be used directly for design purposes. Design analysis on a segment-by-segment basis is always necessary in the design stage, irrespective of the results of planning analysis.

SPECIAL APPLICATION—CLIMBING LANES, DESIGN AND/OR OPERATIONAL ANALYSIS

On many long and/or steep upgrades, it is necessary to consider adding a climbing lane for trucks and other heavy vehicles. This is not the same as adding another general purpose lane to the freeway, since it will normally contain 100 percent trucks and/or other heavy vehicles. Although the climbing lane will have a traffic composition of virtually 100 percent heavy vehicles, not all heavy vehicles will use the lane and some will remain in the other normal traffic lanes as part of a mixed traffic stream.

There are no precise capacity analysis procedures for the treatment of climbing lanes. The following approximate technique, however, can be used to obtain a general idea of how such a lane would operate and what its impact on operations in adjacent normal freeway lanes would be.

First, it is necessary to estimate the capacity of the climbing lane and the number of heavy vehicles that are likely to use it. Because this procedure is approximate, computations may be simplified by assuming that all heavy vehicles are trucks. The appropriate value of E_T for the grade and length of grade in question is selected from Table 3-4, 3-5, or 3-6. Because the lane will contain 100 percent trucks, the value selected will be the *minimum value for the grade and length of grade shown in the table.* This is reasonable, because the value of E_T decreases as the percentage of trucks increases. The capacity of the climbing lane may then be computed as:

TABLE 3-12. SERVICE FLOW RATES PER LANE (SFL) FOR USE IN PLANNING ANALYSIS

TYPE OF TERRAIN	LEVEL OF SERVICE	PERCENT TRUCKS				
		0	5	10	15	20
Level	A	700	650	650	600	600
	B	1,100	1,050	1,000	950	950
	C	1,550	1,500	1,450	1,350	1,300
	D	1,850	1,800	1,700	1,600	1,550
	E	2,000	1,900	1,850	1,750	1,700
Rolling	A	700	600	550	500	450
	B	1,100	950	850	750	700
	C	1,550	1,350	1,200	1,050	1,000
	D	1,850	1,600	1,400	1,300	1,150
	E	2,000	1,750	1,550	1,400	1,250
Mountainous	A	700	500	400	350	250
	B	1,100	800	650	550	400
	C	1,550	1,150	900	750	550
	D	1,850	1,350	1,100	900	650
	E	2,000	1,500	1,200	1,000	700

Base assumptions for Table 3-12:

70-mph design speed
All heavy vehicles are trucks
Lane widths are 12 ft
Lateral clearances > 6 ft

NOTE: All values rounded to the nearest 40 vphpl.

$C_T = 2{,}000/E_T$ for 60- and 70-mph design speeds) (3-8a)

$C_T = 1{,}900/E_T$ for 50-mph design speed) (3-8b)

If it is intended that the climbing lane will operate at approximately the same v/c ratio as the remaining normal freeway lanes, the service flow rate using the climbing lane can be estimated as:

$$SF_T = c_T \times (v/c)_i \quad (3\text{-}9)$$

where:

SF_T = service flow rate in the climbing lane, in vph;
c_T = capacity of the climbing lane, in vph; and
$(v/c)_i$ = v/c ratio for LOS i, from Table 3-1 for operational analysis, or from Table 3-11 for design.

The assumption that the v/c for the climbing lane will be approximately the same as for mixed traffic lanes presumes that vehicles will make use of the total available lanes in a manner that achieves similar service for all vehicles. The analyst may choose to make other assumptions on the occupancy of the climbing lane if local data or judgment so indicates.

Remaining trucks and heavy vehicles are assumed to share mixed traffic lanes with passenger cars. The mixed lanes are evaluated using standard techniques for operational or design analysis as described in previous sections.

In operational analysis, this will require a trial-and-error (iterative) procedure, because a LOS must be assumed for the climbing lane, and then computed for the remaining lanes. Trials are complete when both values are the same.

In design, the LOS is known and the solution is direct. It should be noted that this procedure should be employed in any situation where standard design analysis indicates the need for more lanes in the upgrade direction than in the downgrade direction.

Capacity is not the only criterion used in the consideration of climbing lanes. Truck speed reductions, delay, and other factors may also be considered in accordance with State and/or local practice.

IV. SAMPLE CALCULATIONS

The following problems serve to illustrate the use of the procedures and methodologies discussed in this chapter. Each problem is presented in step-by-step detail, with full discussion of results. In practice, the presentation of solutions would be shorter and less detailed.

CALCULATION 1—OPERATIONAL ANALYSIS OF A BASIC CASE

1. *Description*—An older four-lane urban freeway with a 60-mph design speed serves a directional peak hour volume of 2,100 vph with 6 percent trucks and a PHF of 0.95. The freeway has 11-ft lanes, obstructions immediately at the pavement edge at both the roadside and median, and generally rolling terrain. Evaluate the level of service on the facility. Determine how much additional traffic could be accommodated before reaching capacity. Field studies of average travel speed indicate that during the peak 15 min of flow, speed is 35 mph.

2. *Solution*—To find the level of service, the effective v/c ratio for the facility described would be computed as:

$$v/c = SF/[c_j \times N \times f_w \times f_{HV} \times f_p]$$

where:

c_j = 2,000 pcphpl (Table 3-1);
N = 2 (Given);
E_T = 4 (Table 3-3, rolling terrain);
f_{HV} = 0.85 (Table 3-9, 0.06 trucks, $E_T = 4$);
f_w = 0.79 (Table 3-2, 11-ft lanes, obs. both sides at 0 ft); and
f_p = 1.00 (Table 3-10, weekday).

The service flow rate is taken to be the existing volume, which must be adjusted to reflect a peak flow rate:

$$SF = 2{,}100/0.95 = 2{,}211 \text{ vph}$$

Then:

$$v/c = 2{,}211/[2{,}000 \times 2 \times 0.79 \times 0.85 \times 1.00] = 0.82$$

Comparing this result with the criteria of Table 3-1 indicates that the resulting LOS is D, which is expected to occur for v/c values in the range of 0.69 to 0.84.

Figures 3-3 and 3-4 can be entered with the effective v/c ratio of 0.82 to find the approximate speed and density of the traffic stream. The speed would be *43.0 mph* and the density would be *40 pc/mi/ln*. Comparing the density of 40 pc/mi/ln with the LOS criteria or Table 3-1 shows that the result is consistent with the earlier determination of LOS D. These solutions and the worksheet for this problem are illustrated in Figure 3-8.

Because actual field data on speed were collected in this instance, the LOS could be found directly. During the peak 15 min of flow, the flow rate is 2,211 vph and the observed average travel speed is given as 35 mph. Therefore, the density of the traffic stream is:

$$2{,}211/35 = 63.2 \text{ vpm or } 63.2/2 = 31.6 \text{ v/mi/ln}$$

The density criteria of Table 3-1, however, are expressed in pc/mi/ln. Thus, to determine the LOS from field values, the above density must be converted to units of pc/mi/ln. Note that 6 percent of the traffic stream consists of trucks, with each truck being the equivalent of 4 passenger cars. Thus:

OPERATIONAL ANALYSIS WORKSHEET

Facility Section INTERBOROUGH PKWY, MP 25

Date 12/8/83 Time ______ (of analysis data)

I. GEOMETRY

0 ft. distance to roadside obstructions

22 ft. ← DIR.1 N = 2

0 ft. BARRIER

22 ft. → DIR.2 N = 2

0 ft. distance to roadside obstructions

indicate North

	Design Speed (mph)	Lane Width (ft.)	Terrain Type (L, R, or M)	or Grade (%)	Length (mi)	Barrier Type
DIR.1	60	11	R	—	—	CONCRETE
DIR.2	60	11	R	—	—	CONCRETE

II. VOLUMES

	VOL(vph)	PHF	SF=VOL/PHF	%trucks	%buses	%RV's	driver population
DIR.1	2,100	0.95	2,211	6	—	—	X commuter __other
DIR.2	2,100	0.95	2,211	6	—	—	X commuter __other

III. ANALYSIS

$v/c = SF/[c_j \times N \times f_w \times f_{HV} \times f_p]$

$c = c_j \times N \times f_w \times f_{HV} \times f_p$

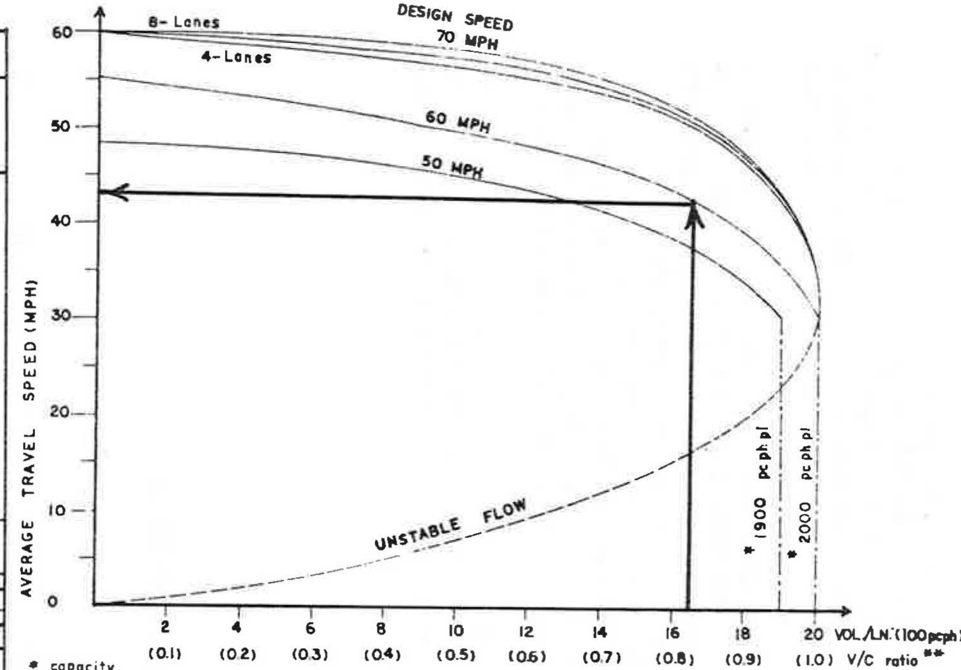
$f_{HV} =$

	v/c =	SF /	[c_j (TAB. 3.1) x	N x	f_w (TAB. 3.2) x	f_p (TAB. 3.10) x	f_{HV}]	$1/[1+P_T(E_T-1)+P_B(E_B-1)+P_R(E_R-1)]$ E_T	E_B	E_R
DIR.1	0.82	2211	2000	2	0.79	1.00	0.85	4	—	—
DIR.2	0.82	2211	2000	2	0.79	1.00	0.85	4	—	—

	c	v/c	LOS (TAB.3.1)	SPEED (FIG.3.4)	DENSITY (FIG.3.3)
DIR.1	2,686	0.82	D	43 MPH	40 PC/MI/LN
DIR.2	2,686	0.82	D	43 MPH	40 PC/MI/LN

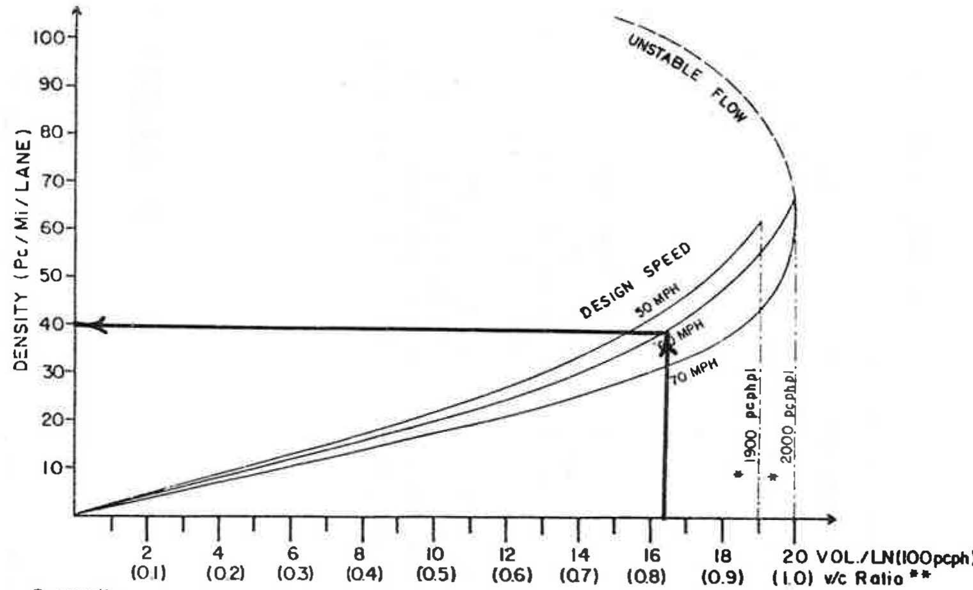
Name ______ Date ______

Checked by ______



* capacity

** v/c ratio based on 2000 pcphpl - valid only for 60 and 70-MPH design speeds



* capacity

** v/c ratio based on 2000 pcphpl - valid only for 60 and 70-MPH design speeds

Figure 3-8. Illustration of solution to Calculation 1.

Density (pc/mi/lm)

$$= (31.6 \times 0.06 \times 4) + (31.6 \times 0.94) = 37.3$$

When compared to the criteria of Table 3-1, this density also yields a level-of-service of D. It should be noted that the field value of density is very close to the value predicted by the methodology (40 pc/mi/ln). The measured speed of 35 mph, however, is lower than the predicted value of 43 mph. This is a reflection of the impact of nonideal conditions of speed. The predicted values from Figure 3-4 assume ideal conditions. The existing conditions in this situation include trucks, rolling terrain, and severe lane width and lateral clearance restrictions, all of which impact speed negatively.

The second part of the problem asks for an evaluation of the maximum additional traffic demand which could be accommodated by the freeway. The v/c ratio during the peak 15 min is 0.82, compared to capacity, at which v/c is 1.00. The capacity of the facility is computed as:

$$c = SF_E = c_j \times N \times (v/c) \times f_w \times f_{HV} \times f_p$$

where v/c is equal to 1.00. Then:

$$c = 2{,}000 \times 2 \times 1.00 \times 0.79 \times 0.85 \times 1.00 = 2{,}686 \text{ vph}$$

Thus:

Capacity	= 2,686 vph
Actual flow rate	= 2,211 vph
	475 vph

An additional flow of 475 vph can be accommodated during the peak 15 min. This can be converted to an equivalent full peak hour value by multiplying by the PHF. Thus, an additional 475 × 0.95 or 451 vph can be accommodated in the peak hour without exceeding the capacity of the section.

CALCULATION 2—OPERATIONAL ANALYSIS OF A COMPOSITE GRADE

1. *Description*—A six-lane freeway with a 70-mph design speed carries a peak hour volume of 3,500 vph, with 5 percent trucks and a PHF of 0.85. The freeway has 12-ft lanes, a 20-ft clear median, and rock cliffs 2 ft from the pavement edge.

The freeway segment in question is the composite grade illustrated in Figure 3-9. Determine the level of service at which the freeway operates during peak periods—upgrade and downgrade.

2. *Solution*—The key to the upgrade solution is to find an equivalent grade of 2 mi in length which results in the same final speed of trucks as the sequence of grades illustrated in Figure 3-9. This is done using the procedure of Appendix I with the performance curves for a 200-lb/hp standard truck. The solution is shown in Figure 3-10.

The performance curves are entered by constructing vertical line 1 at 2,640 ft, finding the intersection with the 2 percent deceleration curve. A horizontal line drawn through this point to the vertical axis indicates a speed of trucks of 49 mph.

Vertical line 2 is constructed from the intersection of the 49-mph horizontal line and the 3 percent deceleration curve, indicating that trucks enter the 3 percent grade as if they had been on it for 1,000 ft. Vertical line 3 is drawn at the 1,000 + 2,640 or 3,640-ft mark, and carried to the intersection with the 3 percent deceleration curve. A horizontal line through

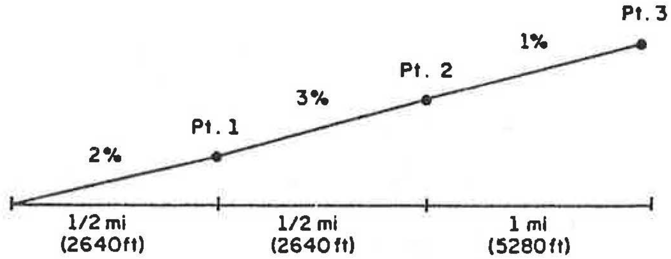


Figure 3-9. Composite grade for Calculation 2.

this point to the vertical axis indicates a speed of 40 mph at the end of the 3 percent grade.

The 40-mph horizontal line, however, does *not* intersect with the 1 percent deceleration curve. This is because trucks entering a 1 percent curve from a 3 percent curve would be expected to accelerate. Thus, vertical line 4 is drawn from the intersection of the 40-mph horizontal line with the 1 percent *acceleration* curve, indicating that trucks enter the grade as if they had traveled on it for 2,100 ft.

Vertical line 5 is constructed from the 2,100 + 5,280 or the 7,380-ft mark. The intersection of this line with the 1 percent acceleration curve yields the final speed of trucks of 50 mph.

The solution for an equivalent grade is now an unknown percent grade of 2 mi that results in a final truck speed of 50 mph. This, however, would be misleading. The minimum truck speed of 40 mph is reached at the end of the 3 percent grade segment, and it is at this point that trucks would have the maximum impact on operations. Therefore, the solution point sought should be an unknown percent grade of *1 mi* that results in a final speed of trucks of *40 mph*.

This is given by the intersection of vertical line 6 (constructed at 5,280 ft) and the 40-mph horizontal, and yields an equivalent grade of 2.8 percent, which will be taken as 3 percent, 1 mi long, for the analysis. Then:

$$v/c = SF/[c_j \times N \times f_w \times f_{HV} \times f_p]$$

where:

SF	=	3,500/0.85 = 4,118 vph (Given);
c_j	=	2,000 pcphpl (Table 3-1);
N	=	3 (Given);
f_w	=	0.97 (Table 3-2, 12-ft lanes, obs. one side at 2 ft);
f_p	=	1.00 (Table 3-10, weekday);
E_T (Upgrade)	=	7 (Table 3-4, 3 percent grade, 1 mi length);
E_T (Downgrade)	=	1.7 (Table 3-3, level terrain);
f_{HV} (Upgrade)	=	0.77 (Table 3-9, $E_T = 7$, 0.05 trucks); and
f_{HV} (Downgrade)	=	$1/[1 + 0.05(1.7 - 1)] = 0.97$.

Then:

v/c (Upgrade)	=	$4{,}118/[2{,}000 \times 3 \times 0.97 \times 0.77 \times 1.00] = 0.92$
v/c (Downgrade)	=	$4{,}118/[2{,}000 \times 3 \times 0.97 \times 0.97 \times 1.00] = 0.73$

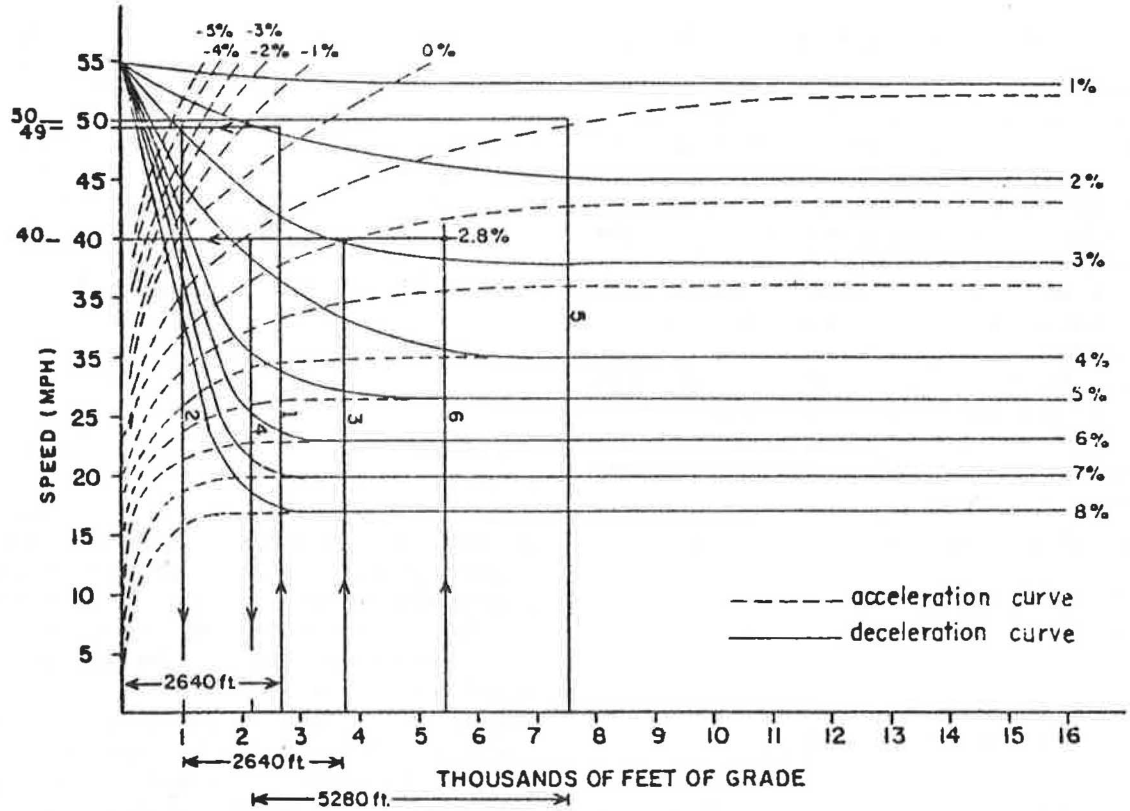


Figure 3-10. Solution of composite grade for Problem 2.

From Table 3-1, the respective levels of service are D for the upgrade and C for the downgrade.

Figures 3-3 and 3-4 may be entered with the above v/c values to obtain approximate speeds and densities for the upgrade and downgrade conditions described. For the upgrade, speed is *46 mph* and density is *40 pc/mi/ln;* for the downgrade, speed is *54 mph* and density is *28 pc/mi/ln.* These solutions and the worksheet for Calculation 2 are shown in Figure 3-11.

The relatively high value of v/c for the upgrade might suggest consideration of a truck climbing lane for this location.

CALCULATION 3—DESIGN OF A BASIC CASE

1. *Description*—An extended section of freeway in level terrain in an urban area is to be designed to operate at level-of-service C. The section is expected to carry a directional design hour volume of 4,500 vph, with 12 percent trucks, no buses or RV's, and a PHF of 0.90. The driver population consists primarily of commuters. Determine the number of lanes which must be provided through the section.

2. *Solution*—The solution involves the computation of the minimum number of lanes required to provide an acceptable LOS C design for a peak flow rate of 4,500/0.90 = 5,000 vph.

Table 3-11 shows the maximum v/c for LOS C to be 0.77 for a 70-mph design. Table 3-11 also indicates several potential design values of v/c less than 0.77 that are also within LOS C. Because AASHTO policies suggest the use of 0.80 for urban freeways, the 0.77 value seems reasonable, and will be used.

The following geometric parameters are assumed as design standards: 70-mph design speed, 12-ft lanes, and no lateral obstructions. Then:

$$N = SF/[c_j \times (v/c) \times f_w \times f_{HV} \times f_p]$$

where:

SF = 5,000 vph (Given);
c_j = 2,000 (Table 3-1);
v/c = 0.77 (Table 3-11);
f_w = 1.00 (Table 3-2);
f_p = 1.00 (Table 3-10);
E_T = 1.7 (Table 3-3, level terrain);
f_{HV} = $1/[1 + 0.12(1.7 - 1)] = 0.92$; and
N = $5{,}000/[2{,}000 \times 0.77 \times 1.00 \times 0.92 \times 1.00] = 3.5$ lanes.

Because a v/c of 0.77 is the maximum acceptable value for LOS C, and since 0.5 lanes cannot be provided, the minimum LOS C design would be *four* lanes in each direction, or an *eight*-lane freeway. The worksheet for this problem is illustrated in Figure 3-12.

The design problem itself ends here. Because the design provides for some excess lanes, the designer may wish to determine the resulting level of service.

To analyze this situation, an operational analysis is performed, setting the known demand equal to SF to compute the effective v/c ratio:

$$v/c = SF/[c_j \times N \times f_w \times f_{HV} \times f_p]$$
$$v/c = 5{,}000/[2{,}000 \times 4 \times 1.00 \times 0.92 \times 1.00] = 0.68$$

OPERATIONAL ANALYSIS WORKSHEET

Facility Section CROSS CREEK EXPWY - MP 110

Date 8/16/84 Time 2:00 P.M. (of analysis data)

I. GEOMETRY

2 ft. distance to roadside obstructions

36 ft. ← DIR.1 N = 3

20 ft. BARRIER

36 ft. → DIR.2 N = 3

2 ft. distance to roadside obstructions

indicate North

	Design Speed (mph)	Lane Width (ft.)	Terrain Type (L, R, or M)	or Grade (%)	Length (mi)	Barrier Type
DIR.1	70	12		+3	1	CLEAR
DIR.2	70	12		-3	1	CLEAR

II. VOLUMES

	VOL(vph)	PHF	SF=VOL/PHF	%trucks	%buses	%RV's	driver population
DIR.1	3,500	0.85	4,118	5	—	—	X commuter __ other
DIR.2	3,500	0.85	4,118	5	—	—	X commuter __ other

III. ANALYSIS

$v/c = SF/[c_j \times N \times f_w \times f_{HV} \times f_p]$

$c = c_j \times N \times f_w \times f_{HV} \times f_p$

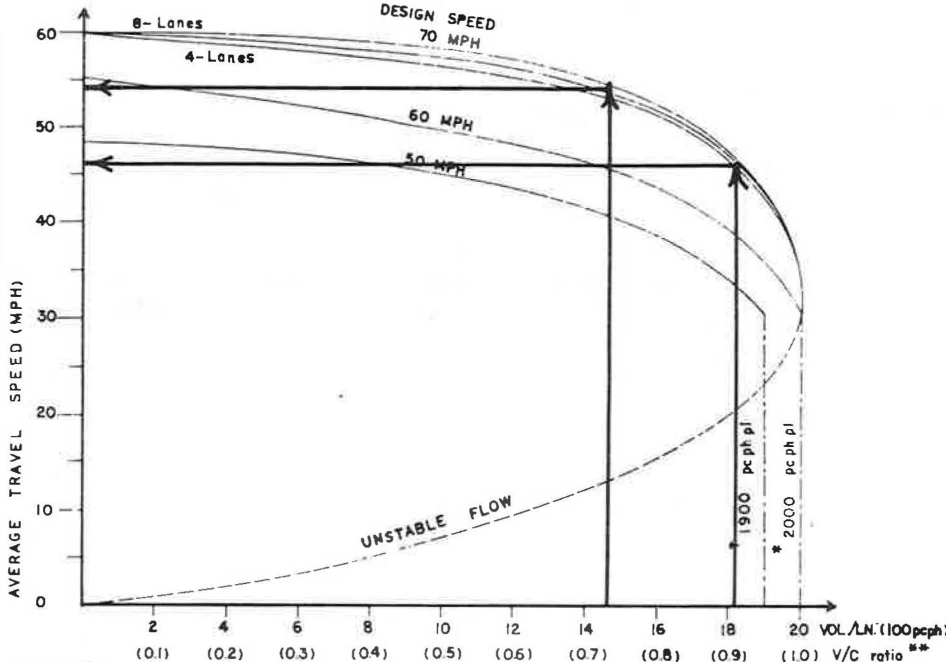
$f_{HV} = 1/[1+P_T(E_T-1)+P_B(E_B-1)+P_R(E_R-1)]$

	v/c = SF	/ [c_j (TAB. 3.1)	x N	x f_w (TAB. 3.2)	x f_p (TAB. 3.10)	x f_{HV}]	E_T	E_B	E_R
DIR.1	0.92	4118	2000	3	0.97	1.00	0.77	7	—	—
DIR.2	0.73	4118	2000	3	0.97	1.00	0.97	1.7	—	—

	c	v/c	LOS (TAB.3.1)	SPEED (FIG.3.4)	DENSITY (FIG.3.3)
DIR.1	4,481	0.92	D	46 MPH	40 PC/MI/LN
DIR.2	5,645	0.73	C	54 MPH	28 PC/MI/LN

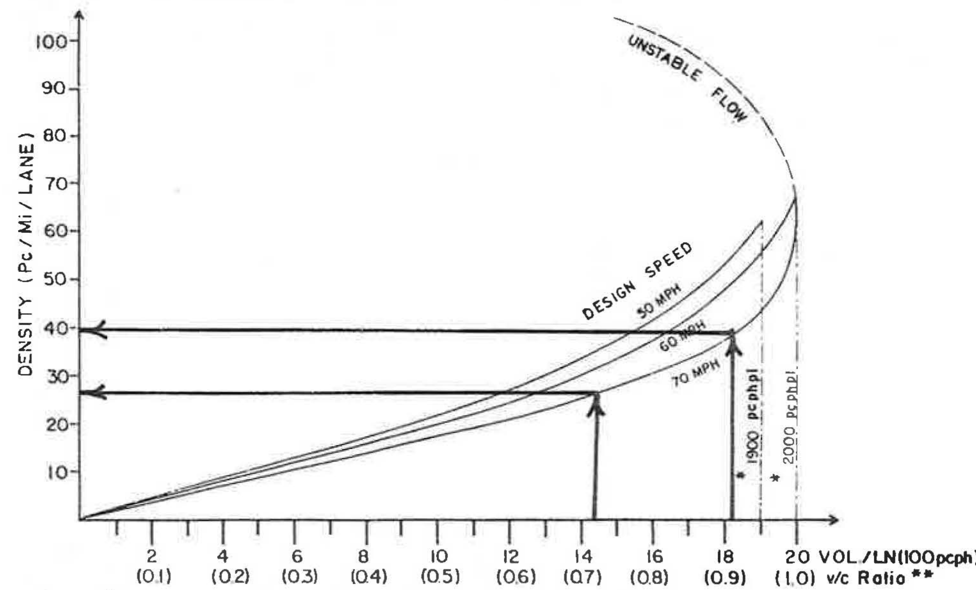
Name THOMAS JONES Date

Checked by



* capacity

** v/c ratio based on 2000 pcphpl - valid only for-60 and 70-MPH design speeds



* capacity

** v/c ratio based on 2000 pcphpl - valid only for-60 and 70-MPH design speeds

Figure 3-11. Illustration of solution to Calculation 2.

DESIGN WORKSHEET

Facility Section ROCK ISLAND FWY - MP. 15

Date 1/8/84 Time 11:00 AM (of analysis data)

I. DESIGN STANDARDS

	LOS	v/c (TAB. 3.1)	Des. Speed (mph)	Lane width (ft.)	Lateral clearance (ft.) roadside	Lateral clearance (ft.) median	Terrain or (L,R,or M)	Grade (%)	length (mi)
DIR.1	C	0.77	70	12	≥6	≥6	L	–	–
DIR.2	C	0.77	70	12	≥6	≥6	L	–	–

II. TRAFFIC FORECASTS

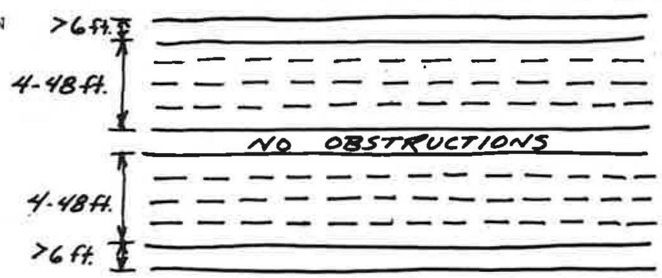
	DDHV(vph)	PHF	SF=(DDHV/PHF)	%trucks	%buses	%RV's	driver population
DIR.1	4500	0.90	5,000	12	–	–	X commuter __other
DIR.2	4500	0.90	5,000	12	–	–	X commuter __other

III. DESIGN ANALYSIS

$$\boxed{N = SF/[c_j \times v/c \times f_w \times f_{HV} \times f_p]}$$

N =	SF	/ [c_j (TAB. 3.1)	x (v/c)	x f_w (TAB. 3.2)	x f_p (TAB. 3.10)	x f_{HV}] (TAB. 3.9)	f_{HV} = 1/[1+P_T(E_T−1)+P_B(E_B−1)+P_R(E_R−1)] E_T	E_B	E_R
DIR.1 3.5	5000	2000	0.77	1.00	1.00	0.92	1.7	–	–
DIR.2 3.5	5000	2000	0.77	1.00	1.00	0.92	1.7	–	–

IV. SKETCH DESIGN



Name ______________ Date ______________

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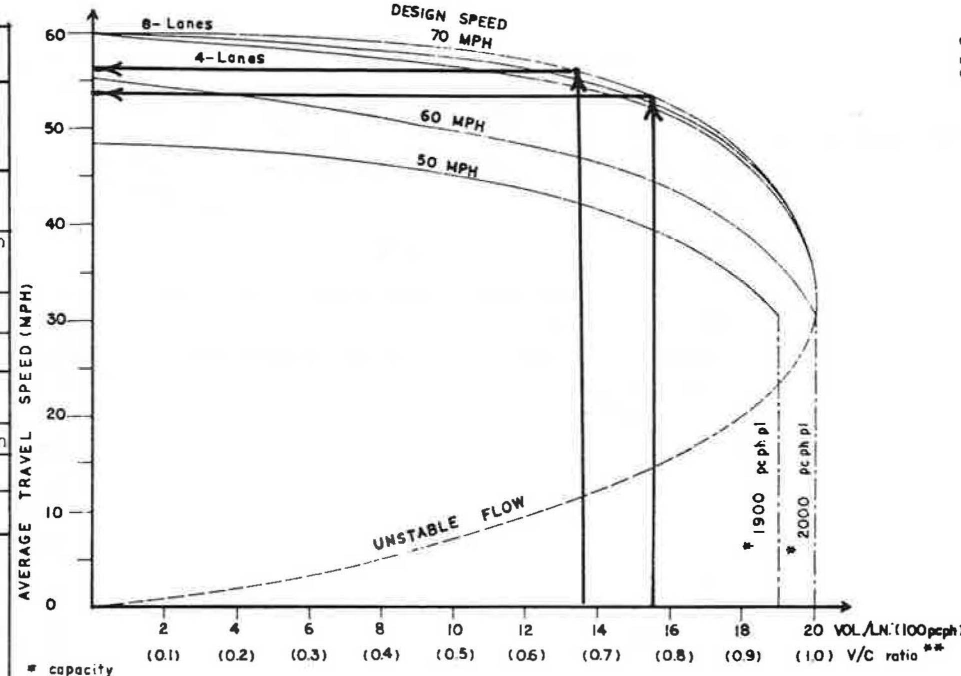
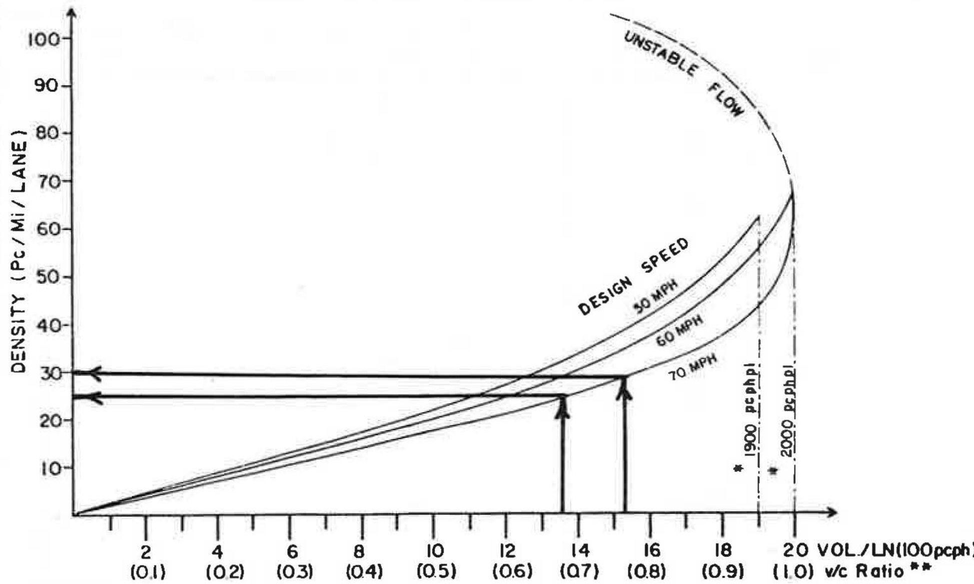


* capacity

** v/c ratio based on 2000 pcphpl - valid only for-60 and 70-MPH design speeds



* capacity

** v/c ratio based on 2000 pcphpl - valid only for-60 and 70-MPH design speeds

Figure 3-12. Illustration of solution to Calculation 3.

From Table 3-1 or 3-11, the LOS provided is still within LOS C. The v/c ratio has, however, been improved. This improvement can be quantified by entering Figures 3-3 and 3-4 with v/c ratios of 0.77 and 0.68 respectively.

An operation at $v/c = 0.77$ would result in an approximate density of 29 pc/mi/ln and a speed of 54 mph. The actual operation at a v/c ratio of 0.68 yields an expected density of 23 pc/mi/ln and a speed of 56 mph. Thus, the additional 0.5 lanes added to the minimum design provides better service than anticipated in the original solution. Figure 3-12 also illustrates this part of the analysis.

CALCULATION 4—DESIGN OF A TRUCK CLIMBING LANE

1. *Description*—A long segment of rural freeway is to be designed for level-of-service B. The *DDHV* is 2,200 vph (weekday), including 20 percent trucks and a PHF of 0.95. A 5-mi segment of level terrain is followed by a 3 percent sustained grade of 1 mi. How many lanes will be required on both the level terrain and sustained grade segments?

2. *Solution*—The following design standards are assumed to be adopted for this solution: 70-mph design speed, 12-ft lanes, and no lateral obstructions.

From Table 3-11, a design value of 0.54 will be used for v/c, the maximum permissible value for LOS B. The required number of lanes is found as:

$$N = SF/[c_j \times (v/c) \times f_w \times f_{HV} \times f_p]$$

where:

SF = 2,200/0.95 = 2,316 vph (Given);
c_j = 2,000 pcphpl (Table 3-1);
v/c = 0.54 (Table 3-11);
f_w = 1.00 (Table 3-2);
f_p = 1.00 (Table 3-10, weekday);
E_T (Downgrade) = 1.7 (Table 3-3, level terrain);
E_T (Upgrade) = 5 (Table 3-4, 3 percent grade, 1 mi long, 6-lanes assumed);
f_{HV} (Downgrade) = $1/[1 + 0.20\ (1.7 - 1)] = 0.88$ (level terrain); and
f_{HV} (Upgrade) = 0.56 (Table 3-9, $E_T = 5$, 20 percent trucks).

Then:

N (Level Terrain and Downgrade) = $2{,}316/[2{,}000 \times 0.54 \times 1.00 \times 0.88 \times 1.00] = 2.4$ lanes

N (Upgrade) = $2{,}316/[2{,}000 \times 0.54 \times 1.00 \times 0.56 \times 1.00] = 3.8$ lanes

These results suggest that the design should consist of a six-lane freeway, with a potential climbing lane on the upgrade. This should be checked using the special procedure for climbing lanes, as follows.

The capacity of the truck climbing lane may be estimated as:

$$c_T = 2{,}000/E_T = 2{,}000/5 = 400 \text{ trucks/hour}$$

Using the design v/c value, it would be expected that the following volume of trucks actually use the lane:

$$SF_T = c_T \times (v/c) = 400 \times 0.54 = 216 \text{ trucks/hour}$$

Thus, the remaining freeway lanes would serve 2,200 − 216 = 1,984 vph, of which (2,200 × 0.20) − 216 = 224 vph are trucks (11.3 percent). A design for the remaining freeway lanes must therefore be conducted for a *DDHV* of 1,984 vph and 11 percent (rounded to the nearest percent) trucks.

$$N = SF/[c_j \times (v/c) \times f_w \times f_{HV} \times f_p]$$

where:

SF = 1,984/0.95 = 2,088 vph;
E_T = 5 (Table 3-4, 3 percent grade, 1 mi long); and
f_{HV} = 0.70 (Table 3-9, $E_T = 5$, 11 percent trucks).

Then:

$$N = 2{,}088/[2{,}000 \times 0.54 \times 1.00 \times 0.70 \times 1.00] = 2.8 \text{ lanes}$$

As the requirement for remaining vehicles in mixed traffic lanes is less than three lanes, the design of a six-lane freeway with a truck climbing lane is appropriate.

CALCULATION 5—DESIGN OF A FREEWAY WITH HEAVY RECREATIONAL TRAFFIC

1. *Descirption*—A sustained upgrade of 5 percent, 1½ mi in length, is to be redesigned on a freeway serving a national park. The redesigned road is expected to carry a *DDHV* of 1,000 vph, 20 percent of which are recreational vehicles, and 5 percent of which are buses. The PHF is 0.95. A design for a v/c ratio of 0.60 (an intermediate point within LOS C) is deisred. Determine the number of lanes which will be required.

2. *Solution*—For the purposes of this solution, it will be assumed that 12-ft lanes and adequate lateral clearances are to be provided. The design speed will be 70 mph. Then:

$$N = SF/[c_j \times (v/c) \times f_w \times f_{HV} \times f_p]$$

where:

SF = 1,000/0.95 = 1,053 (Given);
c_j = 2,000 pcphpl (Table 3-1);
v/c = 0.60 (Given);
f_w = 1.00 (Table 3-2);
f_p = 0.75–0.90—Select 0.85 (Table 3-10, recreational);
E_R = 4 (Table 3-7, 5 percent, 1½ mi long, 20 percent RV's);
E_B = 3 (Table 3-8, 5 percent buses);
f_{HV} = $1/[1 + 0.20\ (4 - 1) + 0.05(3 - 1)] = 0.59$; and
N = $1{,}053/[2{,}000 \times 0.60 \times 1.00 \times 0.59 \times 0.85] = 1.7$ lanes.

The selection of a value of f_p would be based on knowledge of local driving characteristics. For this solution, the value of 0.85 was arbitrarily selected as an illustration.

It is clear from the foregoing results that a two-lane upgrade section is sufficient. No separate analysis of the downgrade would be needed because two lanes is the minimum number of lanes in each direction which may be constructed on a freeway. Thus, a simple four-lane freeway, with no climbing lanes, would be the recommended design. The worksheet for this problem is shown in Figure 3-13.

DESIGN WORKSHEET

Facility Section PRAIRIE FREEWAY

Date 11/11/83 Time 1:00 PM (of analysis data)

I. DESIGN STANDARDS

	LOS	v/c (TAB. 3.11)	Des.Speed (mph)	Lane width (ft.)	Lateral clearance (ft.) roadside	median	Terrain or (L,R,or M)	Grade (%)	length (mi)
DIR.1	C	0.60	70	12	≥6	≥6		+5	1 1/2
DIR.2									

II. TRAFFIC FORECASTS

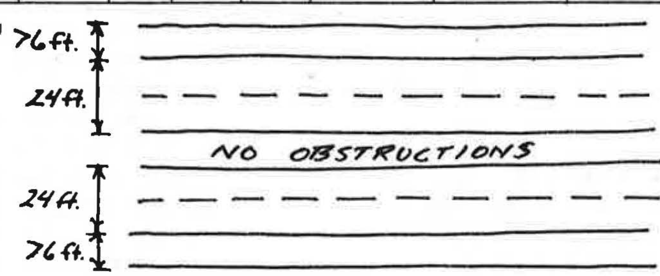
	DDHV(vph)	PHF	SF=(DDHV/PHF)	%trucks	%buses	%RV's	driver population
DIR.1	1,000	0.95	1,052	–	5	20	__commuter X other
DIR.2							__commuter __other

III. DESIGN ANALYSIS

$$\boxed{N = SF/[c_j \times v/c \times f_w \times f_{HV} \times f_p]}$$

	N =	SF /	[c_j (TAB. 3.1)	x (v/c)	x f_w (TAB. 3.2)	x f_p (TAB. 3.10)	x f_{HV}] (TAB. 3.9)	f_{HV} = 1/[1+P_T(E_T-1)+P_B(E_B-1)+P_R(E_R-1)] E_T	E_B	E_R
DIR.1	1.7	1,053	2000	0.60	1.00	0.85	0.59	–	3	4
DIR.2										

IV. SKETCH DESIGN



Name ROGER McCORMACK Date

Checked by

Figure 3-13. Worksheet for Calculation 5.

CALCULATION 6—DESIGN OF A RURAL FREEWAY WITH FARM TRUCKS

1. *Description*—A rural freeway segment of 3/4 mi on a 3 percent grade is to be designed for a v/c ratio of 0.60, the value recommended by AASHTO for rural freeways. It will have a *DDHV* of 1,900 vph, with 15 percent trucks, and a PHF of 0.95. Trucks are expected to be primarily of the farm-to-market variety, with high weight-to-horsepower ratios. Heavily loaded farm trucks are traveling in the direction of the upgrade. Determine the number of lanes required on the grade.

2. *Solution*—It will be assumed that 70-mph design speed, 12-ft lanes, and adequate lateral clearances are provided. For these conditions, a v/c ratio of 0.60 provides for LOS C (Table 3-11).

As trucks are expected to be heavier than normal, Table 3-6 will be used to select E_T values. Then:

$$N = SF/[c_j \times (v/c) \times f_w \times f_{HV} \times f_p]$$

where:

SF = 1,900/0.95 = 2,000 vph (Given);

c_j = 2,000 (Table 3-1);

v/c = 0.60 (Given);

f_w = 1.00 (Table 3-2);

f_p = 1.00 (Table 3-10);

E_T (Upgrade) = 7—This assumes a 4-lane freeway (Table 3-6, 3 percent grade, 3/4 mi long, 15 percent trucks);

E_T (Downgrade) = 1.7 (Table 3-3);

f_{HV} (Upgrade) = 0.53 (Table 3-9, E_T = 7, 15 percent trucks);

f_{HV} (Downgrade) = $1/[1 + 0.15(1.7 - 1)] = 0.90$;

N (Upgrade) = $2{,}000/[2{,}000 \times 0.60 \times 1.00 \times 0.53 \times 1.00)$ = 3.1 lanes; and

N (Downgrade) = $2{,}000/[2{,}000 \times 0.60 \times 1.00 \times 0.90 \times 1.00)$ = 1.9 lanes.

From these results, it appears that a truck climbing lane should be considered for the upgrade, added to a basic four-lane freeway. Although the upgrade technically requires more than three lanes, it is generally not practical to add two truck climbing lanes to the upgrade, or to expand the entire freeway to six lanes with an upgrade truck climbing lane for the sake of 0.1 lanes. The situation of a four-lane freeway with a single truck climbing lane, however, should be carefully examined.

The capacity of the truck climbing lane would be:

$$c_T = 2{,}000/7 = 286 \text{ trucks/hour}$$

and the expected service flow rate:

$$SF_T = 286 \times 0.60 = 172 \text{ trucks/hour}$$

The remaining freeway lanes would then carry 1,900 − 172, or 1,728 vph, of which 1,900 (0.15) − 172, or 113 are trucks (7 percent). The required normal freeway lanes may then be computed as:

$$N = SF/[c_j \times (v/c) \times f_w \times f_{HV} \times f_p]$$

where

c_j, v/c, f_w, f_p = as before;

SF = 1,728/0.95 = 1,819 vph;

E_T = 8 (Table 3-6, 3 percent grade, 3/4 mi long, 7 percent trucks);

f_{HV} = 0.67 (Table 3-9, E_T = 8, 7 percent trucks); and

$$N = 1{,}819/[2{,}000 \times 0.60 \times 1.00 \times 0.67 \times 1.00] = 2.3 \text{ lanes.}$$

This result suggests that two normal freeway lanes plus a climbing lane is *not* sufficient to provide for v/c of 0.60. The actual v/c provided would be:

$$v/c = SF/[c_j \times N \times f_w \times f_{HV} \times f_p]$$
$$v/c = 1{,}819/[2{,}000 \times 2 \times 1.00 \times 0.67 \times 1.00] = 0.68$$

Although further trial-and-error solutions could be attempted, it is obvious that traffic in the climbing lane and in mixed lanes would balance out at a v/c ratio in the range of 0.60 to 0.68. As this range is well within LOS C boundaries (Table 3-1), it is most probable that the four-lane design with a single upgrade climbing lane would be adopted.

CALCULATION 7—PLANNING

1. *Description*—A freeway is being planned to service a radial route in an urban area. It is expected to have an *AADT* of 80,000 vpd, with approximately 10 percent trucks. A PHF of 0.90 is anticipated. The region through which it will travel has generally rolling terrain. Determine the number of freeway lanes that will likely be required to provide for LOS C?

2. *Solution*—It is first necessary to convert the *AADT* to a *DDHV*, using the equation:

$$DDHV = AADT \times K \times D$$

From the general recommendations given in this chapter, K will be selected as 0.09 for urban areas, and D will assumed to be 0.55 for radial routes. Then:

$$DDHV = 80{,}000 \times 0.09 \times 0.55 = 3{,}960 \text{ vph}$$

From Table 3-12, for rolling terrain and 10 percent trucks, the per lane service volume for LOS C is 1,200 vphpl, and:

$$N = DDHV/[SFL \times \text{PHF}]$$
$$N = 3{,}960/[1{,}200 \times 0.90] = 3.7 \textit{ or } \text{Say 4 lanes}$$

It is clear that an eight-lane freeway should be anticipated, subject to final design at a later date. Note that this determination assumes ideal geometrics for the design.

Note also that the planning solution is a very approximate one, based on early data available in the planning process. It gives a general idea as to the type and geometrics of the facility being contemplated, but requires detailed design and operational analysis to consider design details such as horizontal and vertical alignments, ramp junctions and weaving areas, lane configurations, and other factors.

V. REFERENCES

This chapter is based primarily on research reported in Ref. *1*. Passenger-car equivalents were based on a variety of sources reported in Refs. *2* through *11*. The level-of-service concept for freeways is discussed in Ref. *12*. The composite grade analysis technique was adapted from Ref. *13*. For design standards, users should consult the current AASHTO policies.

1. Roess, R., McShane, W., Linzer, E., and Pignataro, L., *Freeway Capacity Analysis Procedures*. Final Report, Project No. DOT-FH-11-9336, Polytechnic Institute of New York, Brooklyn N.Y. (May 1978).
2. Cunagin, W., and Messer, C., *Passenger Car Equivalents for Rural Highways*. Final Report, Project No. DTFH61-80-C-00128, Herbert G. Whyte Associates and the Texas Transportation Institute, College Station, Texas (May 1982).
3. Seguin, E., Crowley, K., and Zweig, W., *Passenger Car Equivalents on Urban Freeways*. Interim Report, Project No. DTFH61-80-C-00106, Institute for Research, State College, Penn. (Aug. 1982).
4. Craus, J., "A Revised Method for the Determination of Passenger Car Equivalents." *Transportation Research*, Vol. 14a, Pergamon Press, London, England (1980).
5. St. John, A., *Freeway Design and Control Strategies as Affected by Trucks and Traffic Regulations*. Report No.

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6. Linzer, E., McShane, W., and Roess, R., "Effects of Trucks, Buses, and Recreational Vehicles on Freeway Capacity and Service Volume." *Transportation Research Record 699*, Transportation Research Board, Washington, D.C. (1979).
7. Messer, C., "Two-Lane Two-Way Rural Highway Capacity." Final Report, NCHRP Project 3-28A, Texas Transportation Institute, College Station, Texas (Feb. 1983).
8. "Review of Truck Weight/Horsepower Ratios as Related to Passing Lane Design." NCHRP Project 20-7, Penn State University, College Station, Penn. (1978).
9. St. John, A., and Koppett, J., "Grade Effects on Traffic Flow, Stability, and Capacity." *NCHRP Report 185*, Transportation Research Board, Washington, D.C. (1978).
10. Ching, P., and Rooney, F., *Truck Speeds on Grades in California*. California Department of Transportation, Sacramento, Calif. (June 1979).
11. St. John, A., "Truck Population on High-Type Rural Highways." 59th Annual Meeting of the Transportation Research Board, Washington D.C. (Jan. 1980).
12. Roess, R., McShane, W., and Pignataro, L. "Freeway Level of Service: A Revised Approach." *Transportation Research Record 699*, Transportation Research Board, Washington D.C. (1980).
13. Leisch, J., *Capacity Analysis Techniques for Design and Operation of Freeway Facilities*. Federal Highway Administration, Washington D.C. (1974).

APPENDIX I

A PRECISE PROCEDURE FOR DETERMINING PASSENGER-CAR EQUIVALENTS OF TRUCKS ON COMPOSITE UPGRADES

In capacity analysis, an overall average grade can be substituted for a series of grades if no single grade exceeds 4 percent or 3,000 ft in length. For grades outside these limits, the following technique is recommended. It estimates the continuous grade that would result in the same final speed of trucks as the actual series of grades. The solution for this equivalent grade uses performance curves for trucks on grades that are included in this appendix.

The technique is best illustrated by example. Consider a composite grade consisting of 5,000 ft of 2 percent grade followed by 5,000 ft of 6 percent grade. If the average grade technique were used:

$$\text{Total Rise} = 5{,}000 \times 0.02 + 5{,}000 \times 0.06 = 400 \text{ ft}$$
$$\text{Average Grade} = 400/10{,}000 = 0.04 \text{ or } 4 \text{ percent}$$

The more precise technique would find a percent grade of 10,000 ft which would result in the same final speed of trucks as 5,000 ft of 2 percent grade followed by 5,000 ft of 6 percent grade. The solution for this point is illustrated in Figure I.3-1, which depicts the acceleration and deceleration performance curves for a standard truck with weight-to-horsepower ratio of 200 lb/hp.

The curve is entered on the horizontal axis at 5,000 ft to find the speed of trucks at the end of the 2 percent grade. A vertical line is drawn at 5,000 ft to the intersection with the 2 percent grade deceleration line. This is indicated as point 1 on the figure.

The speed of trucks is found by drawing a horizontal line from this point to the vertical axis, where the speed is read at point 2 as 47 mph.

The speed of trucks at the end of the 2 percent grade is now determined to be 47 mph. This is also the speed at which trucks *enter* the 6 percent grade.

The intersection of the horizontal line between points 1 and 2 with the 6 percent deceleration curve is found (point 3). A vertical line is constructed from this point to the horizontal axis at point 4. This point indicates that at 47 mph, trucks enter the 6 percent grade *as if they had already been on it for 750 ft, starting from level terrain.*

As trucks will now travel an additional 5,000 ft on the 6 percent grade, this is added to the 750 ft determined above to find point 5, at 5,750 ft. A vertical is constructed at this point to the intersection with the 6 percent deceleration curve to find the final speed of trucks at the end of the 6 percent grade, at point 6. A horizontal line from point 6 to point 7 on the vertical axis determines this speed to be 23 mph.

It is now desired to find a percent of grade of 10,000-ft length that would result in a final speed of trucks of 23 mph. This is found by the intersection of the horizontal line at 23 mph and a vertical line constructed at 10,000 ft (point 8). The equivalent grade is found to be *6 percent,* not 4 percent as indicated by the average grade.

The value of E_T would now be selected for a 6 percent grade, 10,000 ft long.

In general, the following steps describe the solution for equivalent grade:

1. Enter the appropriate truck acceleration-deceleration performance curves with the initial grade and length of grade. Find the speed of trucks at the end of the first grade, which is the speed at which they enter the second grade.

2. Find the length along the second grade which results in the same speed as found in step 1. This is used as the starting point along the second grade.

3. Starting with the length found in step 2, add the length of the second grade, and find the speed at the end of the second grade.

4. If there are additional grades, repeat steps 1 through 3 for each subsequent grade until the final speed is found.

5. Enter the truck performance curves with the final speed of trucks and the total length of composite grade to find the equivalent uniform grade percent, which may be used in finding E_T.

Note that this analysis can be applied to any number of successive grades. A given series of grades may even include some downgrade portions, or segments of level terrain. *Such segments should not be used as points of demarkation between analysis sections unless the speed of trucks can be shown to have returned to 55 mph under free-flow conditions.*

Figures I.3-2, I.3-3, and I.3-4 give performance curves for standard, light, and heavy truck populations, respectively. This precise analysis is generally not undertaken for RV's or buses due to the approximate nature of equivalents for these vehicle types.

Note also that the procedure uses discrete grade segments, and ignores the vertical curves that join them. This simplifies computations, and results in sufficient accuracy for capacity analysis purposes.

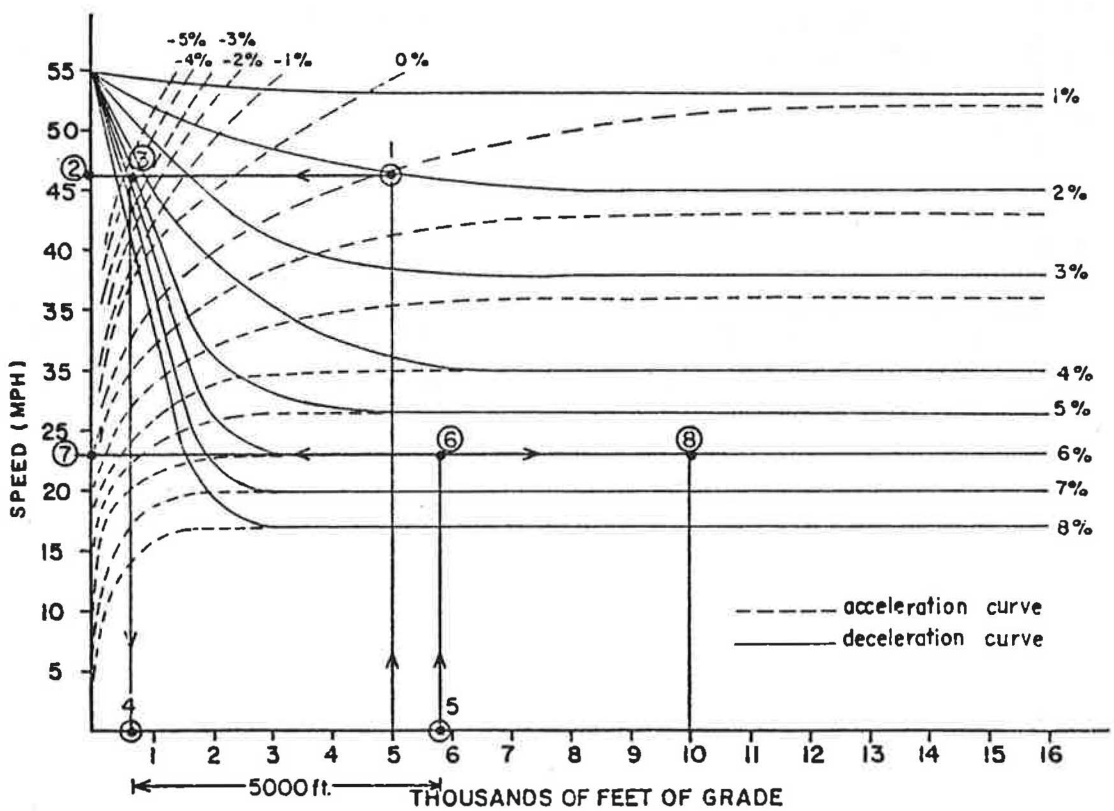


Figure I.3-1. Sample solution for equivalent grade using 200-lb/hp performance curves.

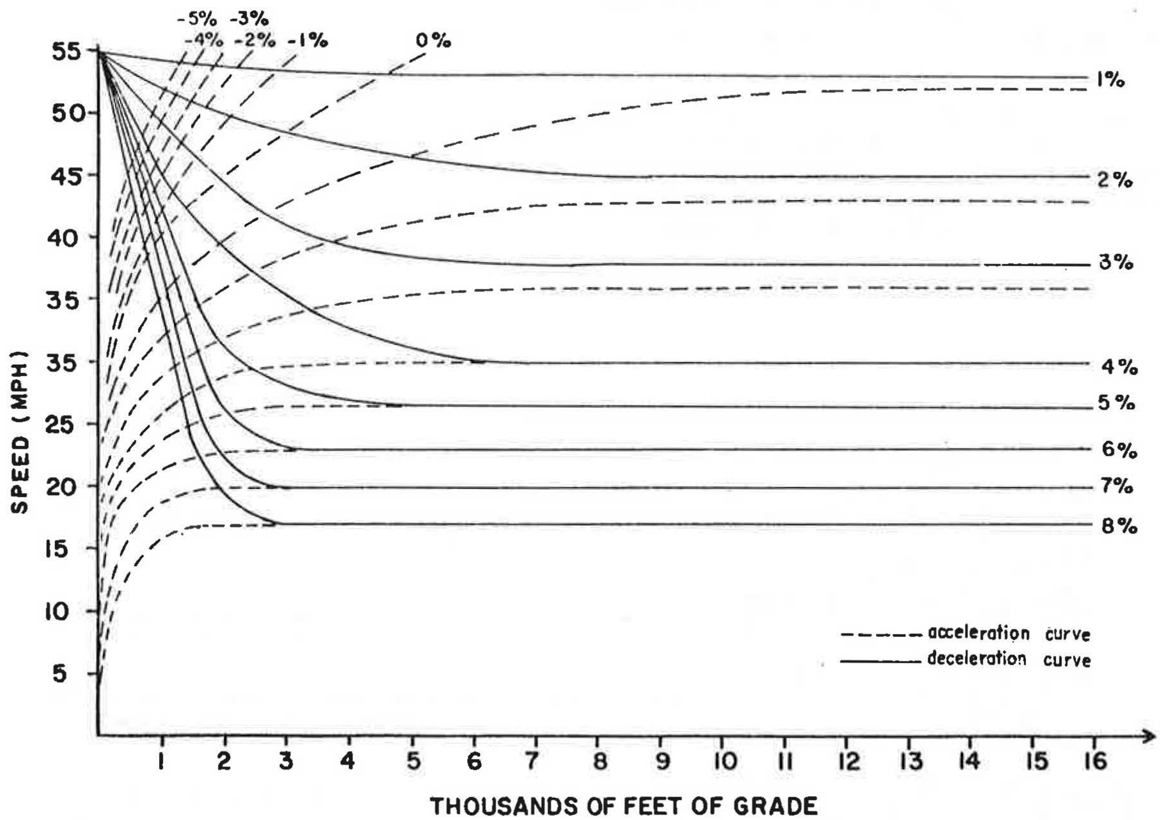


Figure I.3-2. Performance curves for a standard truck (200 lb/hp).

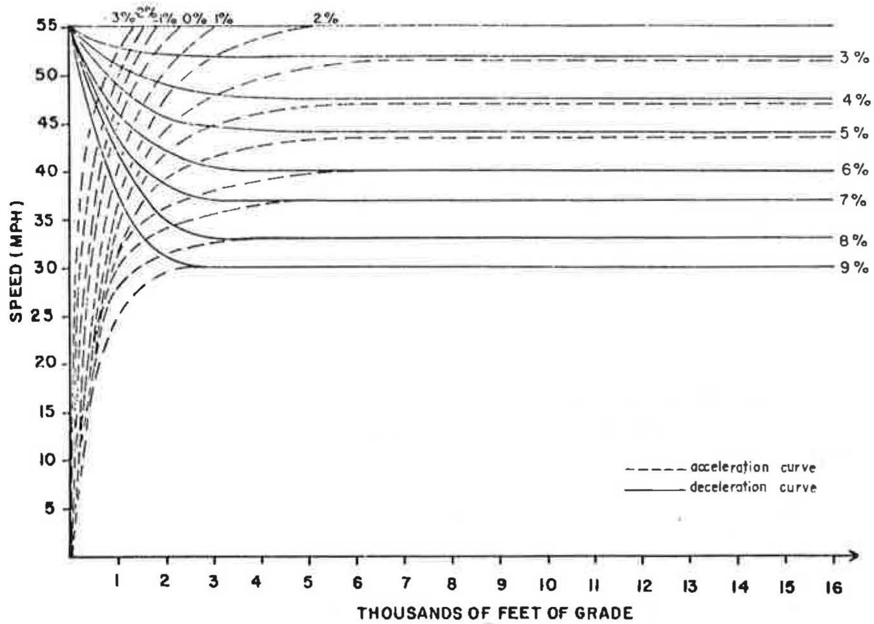


Figure I.3-3. Performance curves for light trucks (100 lb/hp).

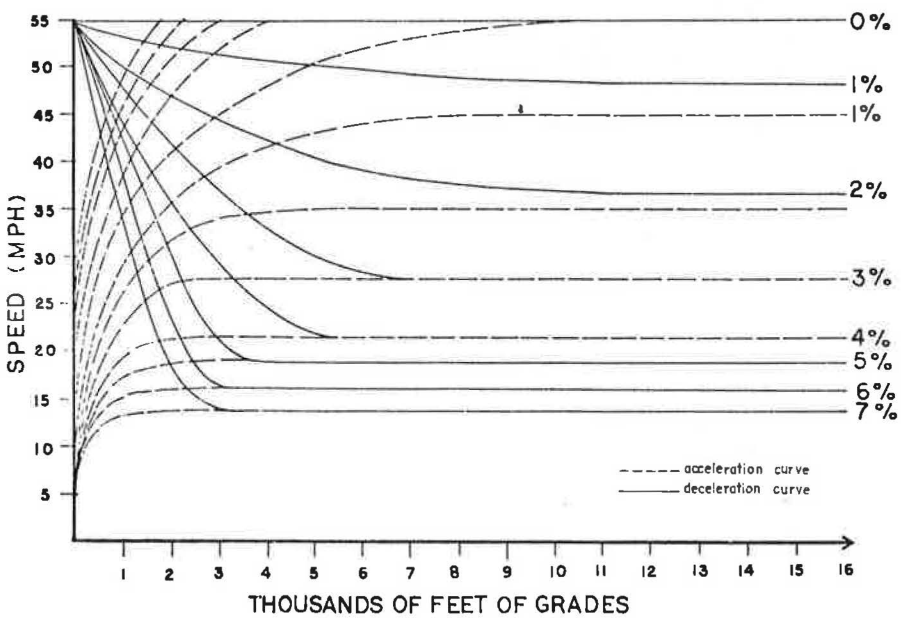


Figure I.3-4. Performance curves for heavy trucks (300 lb/hp).

APPENDIX II

TABLES, FIGURES, AND WORKSHEETS FOR USE IN THE CAPACITY ANALYSIS OF BASIC FREEWAY SECTIONS

TABLE 3-1. LEVELS OF SERVICE FOR BASIC FREEWAY SECTIONS

		70 MPH DESIGN SPEED			60 MPH DESIGN SPEED			50 MPH DESIGN SPEED		
LOS	DENSITY (PC/MI/LN)	SPEED[b] (MPH)	v/c	MSF[a] (PCPHPL)	SPEED[b] (MPH)	v/c	MSF[a] (PCPHPL)	SPEED[b] (MPH)	v/c	MSF[a] (PCPHPL)
A	$\leq$ 12	$\geq$ 60	0.35	700	—	—	—	—	—	—
B	$\leq$ 20	$\geq$ 57	0.54	1,100	$\geq$ 50	0.49	1,000	—	—	—
C	$\leq$ 30	$\geq$ 54	0.77	1,550	$\geq$ 47	0.69	1,400	$\geq$ 43	0.67	1,300
D	$\leq$ 42	$\geq$ 46	0.93	1,850	$\geq$ 42	0.84	1,700	$\geq$ 40	0.83	1,600
E	$\leq$ 67	$\geq$ 30	1.00	2,000	$\geq$ 30	1.00	2,000	$\geq$ 28	1.00	1,900
F	$>$ 67	$<$ 30	[c]	[c]	$<$ 30	[c]	[c]	$<$ 28	[c]	[c]

[a] Maximum service flow rate per lane under ideal conditions.
[b] Average travel speed.
[c] Highly variable, unstable.
NOTE: All values of *MSF* Rounded to the nearest 50 vph.

TABLE 3-2. ADJUSTMENT FACTOR FOR RESTRICTED LANE WIDTH AND LATERAL CLEARANCE

DISTANCE FROM TRAVELED PAVEMENT[a] (FT)	ADJUSTMENT FACTOR, f_w							
	OBSTRUCTIONS ON ONE SIDE OF THE ROADWAY				OBSTRUCTIONS ON BOTH SIDES OF THE ROADWAY			
	LANE WIDTH (FT)							
	12	11	10	9	12	11	10	9
	4-LANE FREEWAY (2 LANES EACH DIRECTION)							
$\geq$ 6	1.00	0.97	0.91	0.81	1.00	0.97	0.91	0.81
5	0.99	0.96	0.90	0.80	0.99	0.96	0.90	0.80
4	0.99	0.96	0.90	0.80	0.98	0.95	0.89	0.79
3	0.98	0.95	0.89	0.79	0.96	0.93	0.87	0.77
2	0.97	0.94	0.88	0.79	0.94	0.91	0.86	0.76
1	0.93	0.90	0.85	0.76	0.87	0.85	0.80	0.71
0	0.90	0.87	0.82	0.73	0.81	0.79	0.74	0.66
	6- or 8- LANE FREEWAY (3 or 4 LANES EACH DIRECTION)							
$\geq$ 6	1.00	0.96	0.89	0.78	1.00	0.96	0.89	0.78
5	0.99	0.95	0.88	0.77	0.99	0.95	0.88	0.77
4	0.99	0.95	0.88	0.77	0.98	0.94	0.87	0.77
3	0.98	0.94	0.87	0.76	0.97	0.93	0.86	0.76
2	0.97	0.93	0.87	0.76	0.96	0.92	0.85	0.75
1	0.95	0.92	0.86	0.75	0.93	0.89	0.83	0.72
0	0.94	0.91	0.85	0.74	0.91	0.87	0.81	0.70

[a] Certain types of obstructions, high-type median barriers in particular, do not cause any deleterious effect on traffic flow. Judgment should be exercised in applying these factors.

TABLE 3-3. PASSENGER-CAR EQUIVALENTS ON EXTENDED GENERAL FREEWAY SEGMENTS

FACTOR	TYPE OF TERRAIN LEVEL	ROLLING	MOUNTAINOUS
E_T for Trucks	1.7	4.0	8.0
E_B for Buses	1.5	3.0	5.0
E_R for RV's	1.6	3.0	4.0

TABLE 3-4. PASSENGER-CAR EQUIVALENTS FOR TYPICAL TRUCKS (200 LB/HP)

GRADE (%)	LENGTH (MI)	PASSENGER-CAR EQUIVALENT, E_T 4-LANE FREEWAYS								6–8 LANE FREEWAYS							
PERCENT TRUCKS		2	4	5	6	8	10	15	20	2	4	5	6	8	10	15	20
<1	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
1	0–1/2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	1/2–1	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	≥1	4	3	3	3	3	3	3	3	4	3	3	3	3	3	3	3
2	0–1/4	4	4	4	3	3	3	3	3	4	4	4	3	3	3	3	3
	1/4–1/2	5	4	4	3	3	3	3	3	5	4	4	3	3	3	3	3
	1/2–3/4	6	5	5	4	4	4	4	4	6	5	5	4	4	4	4	4
	3/4–1½	7	6	6	5	4	4	4	4	7	5	5	5	4	4	4	4
	≥ 1½	8	6	6	6	5	5	4	4	8	6	6	5	4	4	4	4
3	0–1/4	6	5	5	5	4	4	4	3	6	5	5	5	4	4	4	3
	1/4–1/2	8	6	6	6	5	5	5	4	7	6	6	6	5	5	5	4
	1/2–1	9	7	7	6	5	5	5	5	9	7	7	6	5	5	5	5
	1–1½	9	7	7	7	6	6	5	5	9	7	7	6	5	5	5	5
	≥ 1½	10	7	7	7	6	6	5	5	10	7	7	6	5	5	5	5
4	0–1/4	7	6	6	5	4	4	4	4	7	6	6	5	4	4	4	4
	1/4–1/2	10	7	7	6	5	5	5	5	9	7	7	6	5	5	5	5
	1/2–1	12	8	8	7	6	6	6	6	10	8	7	6	5	5	5	5
	≥ 1	13	9	9	9	8	8	7	7	11	9	9	8	7	6	6	6
5	0–1/4	8	6	6	6	5	5	5	5	8	6	6	6	5	5	5	5
	1/4–1/2	10	8	8	7	6	6	6	6	8	7	7	6	5	5	5	5
	1/2–1	12	11	11	10	8	8	8	8	12	10	9	8	7	7	7	7
	≥ 1	14	11	11	10	8	8	8	8	12	10	9	8	7	7	7	7
6	0–1/4	9	7	7	7	6	6	6	6	9	7	7	6	5	5	5	5
	1/4–1/2	13	9	9	8	7	7	7	7	11	8	8	7	6	6	6	6
	1/2–3/4	13	9	9	8	7	7	7	7	11	9	9	8	7	6	6	6
	≥ 3/4	17	12	12	11	9	9	9	9	13	10	10	9	8	8	8	8

NOTE: If a length of grade falls on a boundary condition, the equivalent for the longer grade category is used. For any grade steeper than the percentage shown, use the next higher grade category.

Table 3-5. Passenger-Car Equivalents for Light Trucks (100 lb/hp)

Grade (%)	Length (mi)	Passenger-Car Equivalent, E_T — 4-Lane Freeways								6–8 Lane Freeways							
Percent Trucks		2	4	5	6	8	10	15	20	2	4	5	6	8	10	15	20
≤ 2	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
3	0–1/4	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	1/4–1/2	4	4	4	3	3	3	3	3	4	4	4	3	3	3	3	3
	1/2–3/4	4	4	4	4	3	3	3	3	4	4	4	3	3	3	3	3
	3/4–1	5	4	4	4	3	3	3	3	5	4	4	4	3	3	3	3
	> 1	6	5	5	5	4	4	4	3	6	5	5	4	4	4	3	3
4	0–1/4	4	4	4	3	3	3	3	3	5	4	4	4	3	3	3	3
	1/4–1/2	5	5	5	4	4	4	4	4	5	4	4	4	4	4	4	4
	1/2–1	6	5	5	5	4	4	4	4	6	5	5	4	4	4	4	4
	> 1	7	6	6	5	4	4	4	4	7	5	5	5	4	4	4	4
5	0–1/4	6	5	5	5	4	4	4	3	6	5	5	5	4	4	4	3
	1/4–1	8	7	7	6	5	5	5	5	8	7	7	6	5	5	5	5
	> 1	9	7	7	6	5	5	5	5	8	7	7	6	5	5	5	5
6	0–1/4	7	5	5	5	4	4	4	4	7	5	5	5	4	4	3	3
	1/4–1	9	7	7	6	5	5	5	5	8	7	7	6	5	5	5	5
	> 1	9	7	7	7	6	6	5	5	9	7	7	6	5	5	5	5

NOTE: If a length of grade falls on a boundary condition, the equivalent from the longer grade category is used. For any grade steeper than the percentage shown, use the next higher grade category.

Table 3-6. Passenger-Car Equivalents for Heavy Trucks (300 lb/hp)

Grade (%)	Length (mi)	Passenger-Car Equivalent, E_T															
		4-Lane Freeways								6–8 Lane Freeways							
Percent Trucks		2	4	5	6	8	10	15	20	2	4	5	6	8	10	15	20
< 1	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
1	0–1/4	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	1/4–1/2	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	1/2–3/4	4	4	4	4	3	3	3	3	4	4	4	3	3	3	3	3
	3/4–1	5	4	4	4	3	3	3	3	5	4	4	4	3	3	3	3
	1–1½	6	5	5	5	4	4	4	3	6	5	5	4	4	4	3	3
	> 1½	7	5	5	5	4	4	4	3	7	5	5	5	4	4	3	3
2	0–1/4	4	4	4	3	3	3	3	3	4	4	4	3	3	3	3	3
	1/4–1/2	7	6	6	5	4	4	4	4	7	5	5	5	4	4	4	4
	1/2–3/4	8	6	6	5	5	4	4	4	8	6	6	6	5	5	4	4
	3/4–1	8	6	6	6	5	5	5	5	8	6	6	6	5	5	5	5
	1–1½	9	7	7	7	6	6	5	5	9	7	7	6	5	5	5	5
	> 1½	10	7	7	7	6	6	5	5	10	7	7	6	5	5	5	5
3	0–1/4	6	5	5	5	4	4	4	3	6	5	5	5	4	4	4	3
	1/4–1/2	9	7	7	6	5	5	5	5	8	7	7	6	5	5	5	5
	1/2–3/4	12	8	8	7	6	6	6	6	10	8	7	6	5	5	5	5
	3/4–1	13	9	9	8	7	7	7	7	11	8	8	7	6	6	6	6
	> 1	14	10	10	9	8	8	7	7	12	9	9	8	7	7	7	7
4	0–1/4	7	5	5	5	4	4	4	4	7	5	5	5	4	4	3	3
	1/4–1/2	12	8	8	7	6	6	6	6	10	8	7	6	5	5	5	5
	1/2–3/4	13	9	9	8	7	7	7	7	11	9	9	8	7	6	6	6
	3/4–1	15	10	10	9	8	8	8	8	12	10	10	9	8	7	7	7
	> 1	17	12	12	10	9	9	9	9	13	10	10	9	8	8	8	8
5	0–1/4	8	6	6	6	5	5	5	5	8	6	6	6	5	5	5	5
	1/4–1/2	13	9	9	8	7	7	7	7	11	8	8	7	6	6	6	6
	1/2–3/4	20	15	15	14	11	11	11	11	14	11	11	10	9	9	9	9
	> 3/4	22	17	17	16	13	13	13	13	17	14	14	13	12	11	11	11
6	0–1/4	9	7	7	7	6	6	6	6	9	7	7	6	5	5	5	5
	1/4–1/2	17	12	12	11	9	9	9	9	13	10	10	9	8	8	8	8
	> 1/2	28	22	22	21	18	18	18	18	20	17	17	16	15	14	14	14

NOTE: If a length of grade falls on a boundary condition, the equivalent from the longer grade category is used. For any grade steeper than the percent shown, use the next higher grade category.

TABLE 3-7. PASSENGER-CAR EQUIVALENTS FOR RECREATIONAL VEHICLES

GRADE (%)	LENGTH (MI)	PASSENGER-CAR EQUIVALENT, E_R 4-LANE FREEWAYS								6–8 LANE FREEWAYS							
PERCENT RV'S		2	4	5	6	8	10	15	20	2	4	5	6	8	10	15	20
<2	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
3	0–1/2	3	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	≥ 1/2	4	3	3	3	3	3	3	3	4	3	3	3	3	3	3	3
4	0–1/4	3	2	2	2	2	2	2	2	3	2	2	2	2	2	2	2
	1/4–3/4	4	3	3	3	3	3	3	3	4	3	3	3	3	3	3	3
	≥ 3/4	5	4	4	4	3	3	3	3	4	4	4	4	3	3	3	3
5	0–1/4	4	3	3	3	3	3	3	3	4	3	3	3	2	2	2	2
	1/4–3/4	5	4	4	4	4	4	4	4	5	4	4	4	4	4	4	4
	≥ 3/4	6	5	4	4	4	4	4	4	5	5	4	4	4	4	4	4
6	0–1/4	5	4	4	4	3	3	3	3	5	4	4	3	3	3	3	3
	1/4–3/4	6	5	5	4	4	4	4	4	6	4	4	4	4	4	4	4
	≥ 3/4	7	6	6	6	5	5	5	5	6	5	5	5	4	4	4	4

NOTE: If a length of grade falls on a boundary condition, the equivalent from the longer grade category is used. For any grade steeper than the percent shown, use the next higher grade category.

TABLE 3-8. PASSENGER-CAR EQUIVALENTS FOR BUSES

GRADE (%)	PASSENGER-CAR EQUIVALENT, E_B
0–3	1.6
4[a]	1.6
5[a]	3.0
6[a]	5.5

[a] Use generally restricted to grades more than 1/4 mi long.

TABLE 3-9. ADJUSTMENT FACTOR FOR THE EFFECT OF TRUCKS, BUSES, OR RECREATIONAL VEHICLES IN THE TRAFFIC STREAM

PCE[a] E_T E_R or E_B	ADJUSTMENT FACTOR, f_{HV} — PROPORTION OF TRUCKS, P_T; RV's, P_R; or BUSES, P_B														
	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09	0.10	0.12	0.14	0.16	0.18	0.20
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
16	0.87	0.77	0.69	0.63	0.57	0.53	0.49	0.45	0.43	0.40	0.36	0.32	0.29	0.27	0.25
17	0.86	0.76	0.68	0.61	0.56	0.51	0.47	0.44	0.41	0.38	0.34	0.31	0.28	0.26	0.24
18	0.85	0.75	0.66	0.60	0.54	0.49	0.46	0.42	0.40	0.37	0.33	0.30	0.27	0.25	0.23
19	0.85	0.74	0.65	0.58	0.53	0.48	0.44	0.41	0.38	0.36	0.32	0.28	0.26	0.24	0.22
20	0.84	0.72	0.64	0.57	0.51	0.47	0.42	0.40	0.37	0.34	0.30	0.27	0.25	0.23	0.21
21	0.83	0.71	0.63	0.56	0.50	0.45	0.41	0.38	0.36	0.33	0.29	0.26	0.24	0.22	0.20
22	0.83	0.70	0.61	0.54	0.49	0.44	0.40	0.37	0.35	0.32	0.28	0.25	0.23	0.21	0.19
23	0.82	0.69	0.60	0.53	0.48	0.43	0.39	0.36	0.34	0.31	0.27	0.25	0.22	0.20	0.19
24	0.81	0.68	0.59	0.52	0.47	0.42	0.38	0.35	0.33	0.30	0.27	0.24	0.21	0.19	0.18
25	0.80	0.67	0.58	0.51	0.46	0.41	0.37	0.34	0.32	0.29	0.26	0.23	0.20	0.18	0.17

[a] Passenger-car equivalent, obtained from Table 3-3, 3-4, 3-5, or 3-6.

NOTE: This table should not be used when the combined percentage of buses and RV's in the traffic stream is more than one-fifth the percentage of trucks.

TABLE 3-10. ADJUSTMENT FACTOR FOR THE CHARACTER OF THE TRAFFIC STREAM

TRAFFIC STREAM TYPE	FACTORS, f_p
Weekday or Commuter	1.0
Other	0.75–0.90[a]

[a] Engineering judgment must be exercised in selecting an exact value.

TABLE 3-11. VALUES OF VOLUME-TO-CAPACITY RATIO FOR USE IN DESIGN

v/c RATIO	MSF[a] (PCPHPL)	RESULTING PERFORMANCE CHARACTERISTICS		
		LOS[b]	DENSITY (PC/MI/LN)	SPEED (MPH)
		70-MPH ELEMENTS		
0.30	600	A	10.5	60
0.35[c]	700	A	12.0	60
0.40	800	B	14.0	59
0.50	1,000	B	17.5	58
0.54[c]	1,100	B	20.0	57
0.60	1,200	C	21.0	56
0.70	1,400	C	25.0	55
0.77[c]	1,550	C	30.0	54
0.80	1,600	D	30.5	52
		60-MPH ELEMENTS		
0.30	600	B	12.0	52
0.40	800	B	15.5	52
0.49[c]	1,000	B	20.0	50
0.60	1,200	C	25.0	48
0.69[c]	1,400	C	30.0	47
0.80	1,600	D	37.5	43
		50-MPH ELEMENTS		
0.30	550	C	13.0	47
0.40	750	C	17.0	47
0.50	950	C	22.0	45
0.60	1,150	C	27.0	44
0.67[c]	1,300	C	30.0	43
0.70	1,350	D	34.0	41
0.80	1,500	D	42.0	40

[a] Values rounded to the nearest 100 pcphpl.
[b] Design may be within LOS bounds, not necessarily at maximum condition for LOS.
[c] Maximum permissible value for the LOS shown.

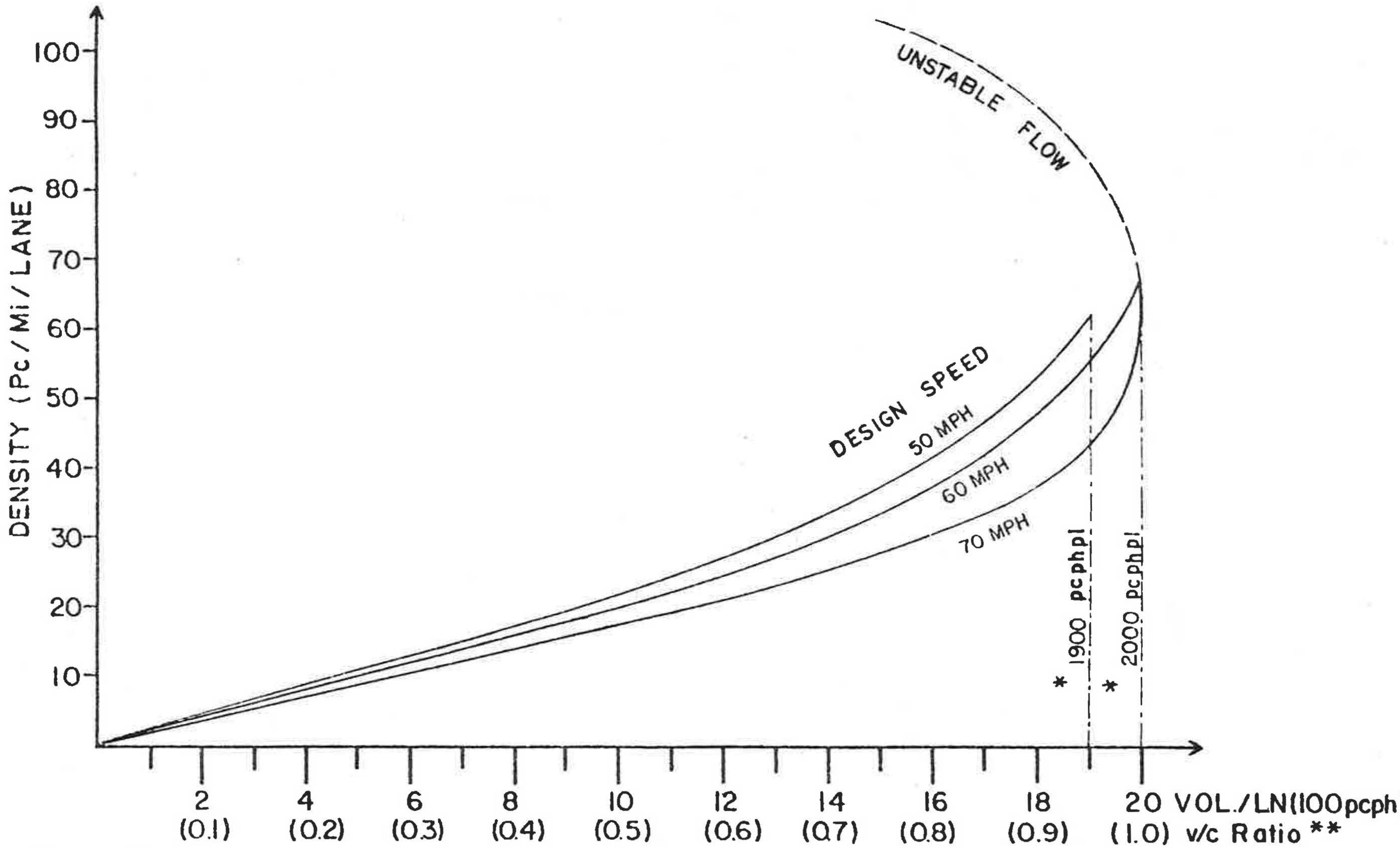
TABLE 3-12. SERVICE FLOW RATES PER LANE (SFL) FOR USE IN PLANNING ANALYSIS

TYPE OF TERRAIN	LEVEL OF SERVICE	PERCENT TRUCKS 0	5	10	15	20
Level	A	700	650	650	600	600
	B	1,100	1,050	1,000	950	950
	C	1,550	1,500	1,450	1,350	1,300
	D	1,850	1,800	1,700	1,600	1,550
	E	2,000	1,900	1,850	1,750	1,700
Rolling	A	700	600	550	500	450
	B	1,100	950	850	750	700
	C	1,550	1,350	1,200	1,050	1,000
	D	1,850	1,600	1,400	1,300	1,150
	E	2,000	1,750	1,550	1,400	1,250
Mountainous	A	700	500	400	350	250
	B	1,100	800	650	550	400
	C	1,550	1,150	900	750	550
	D	1,850	1,350	1,100	900	650
	E	2,000	1,500	1,200	1,000	700

Base assumptions for Table 3-12:

70-mph design speed
All heavy vehicles are trucks
Lane widths are 12 ft
Lateral clearances > 6 ft

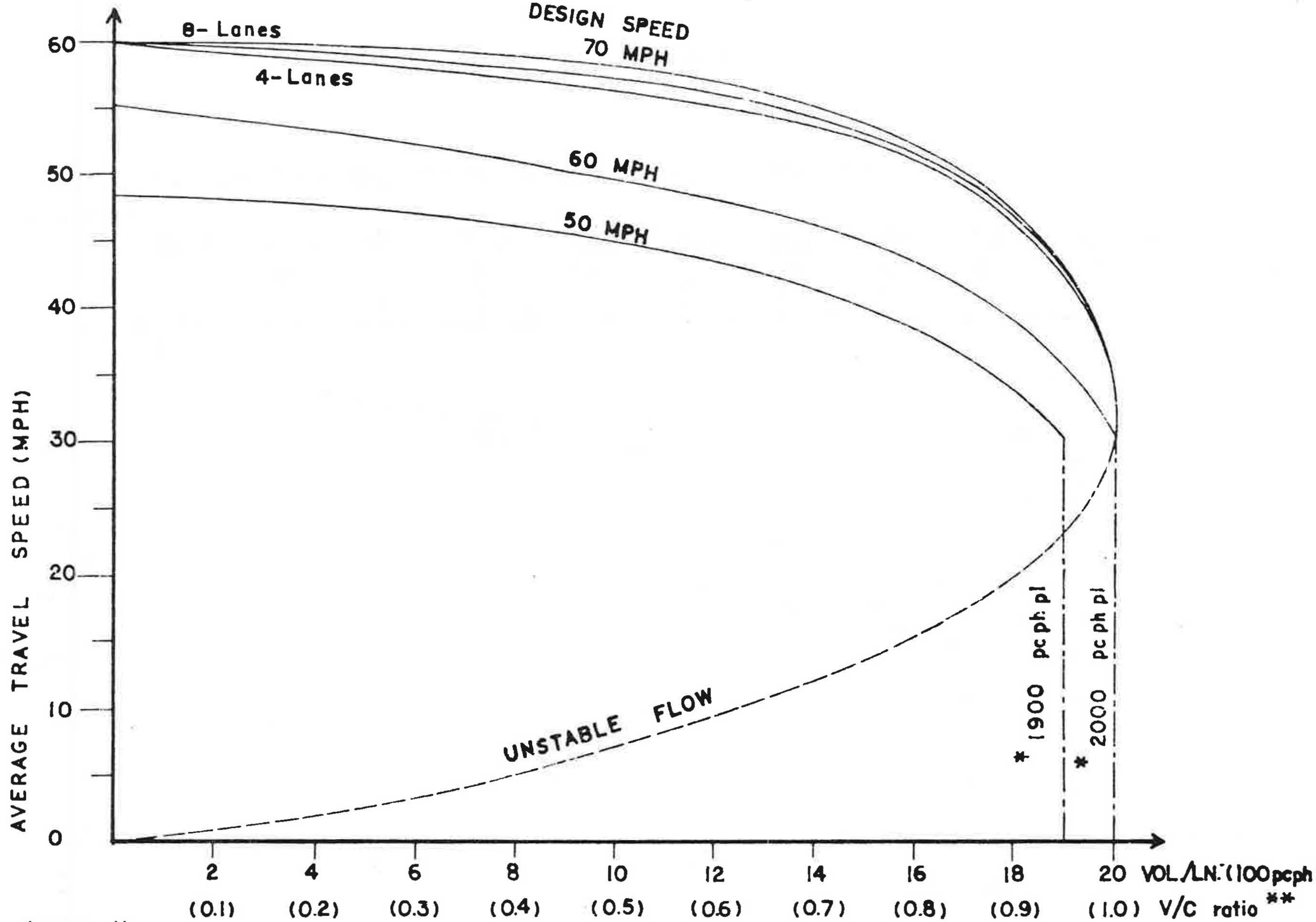
NOTE: All values rounded to the nearest 40 vphpl.



* capacity

** v/c ratio based on 2000 pcphpl - valid only for-60 and 70-MPH design speeds

Figure 3-3. Density-flow relationships under ideal conditions.



* capacity

** v/c ratio based on 2000 pcphpl - valid only for - 60 and 70-MPH design speeds

Figure 3-4. Speed-flow relationships under ideal conditions.

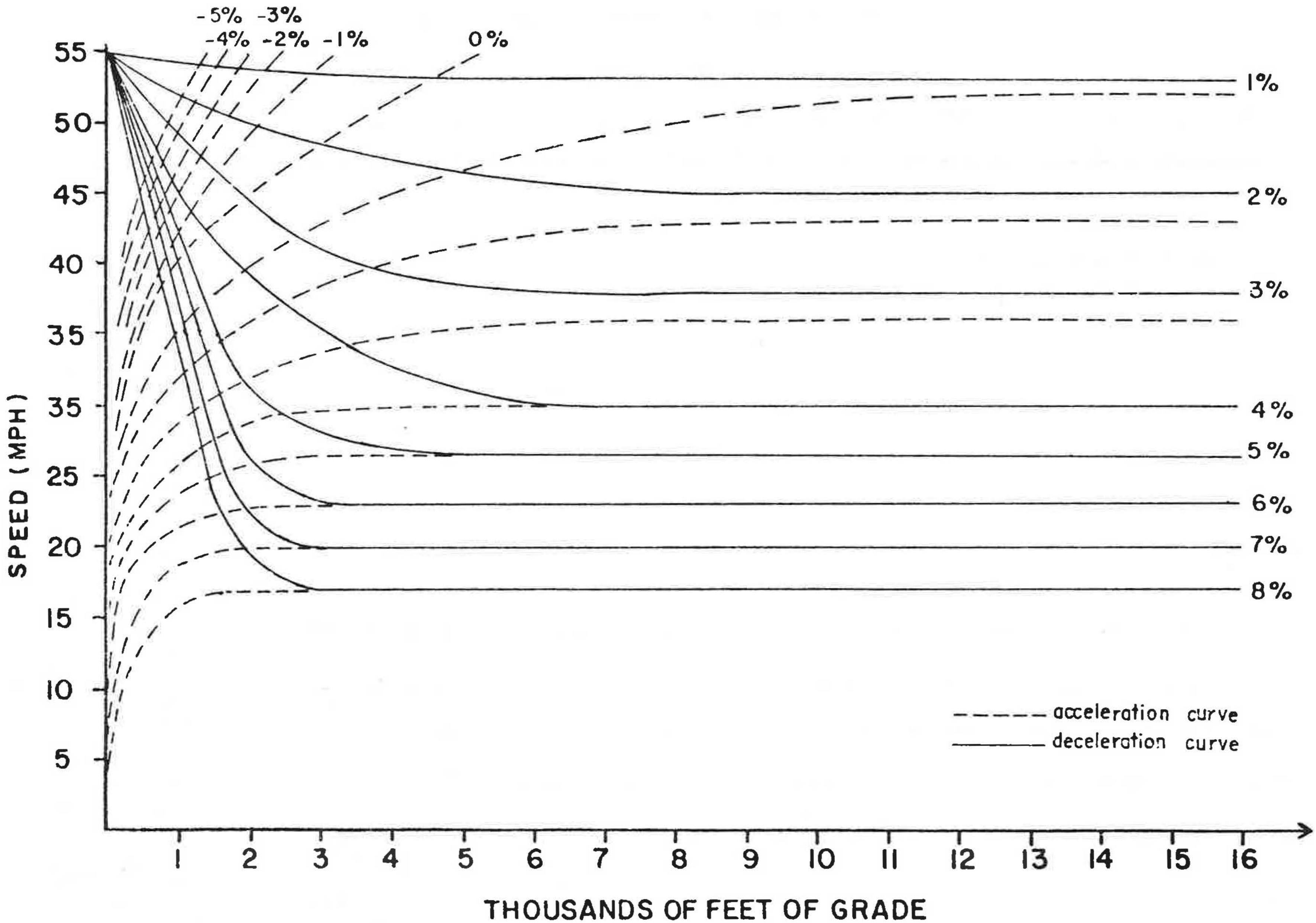


Figure I.3-2. Performance curves for a standard truck (200 lb/hp).

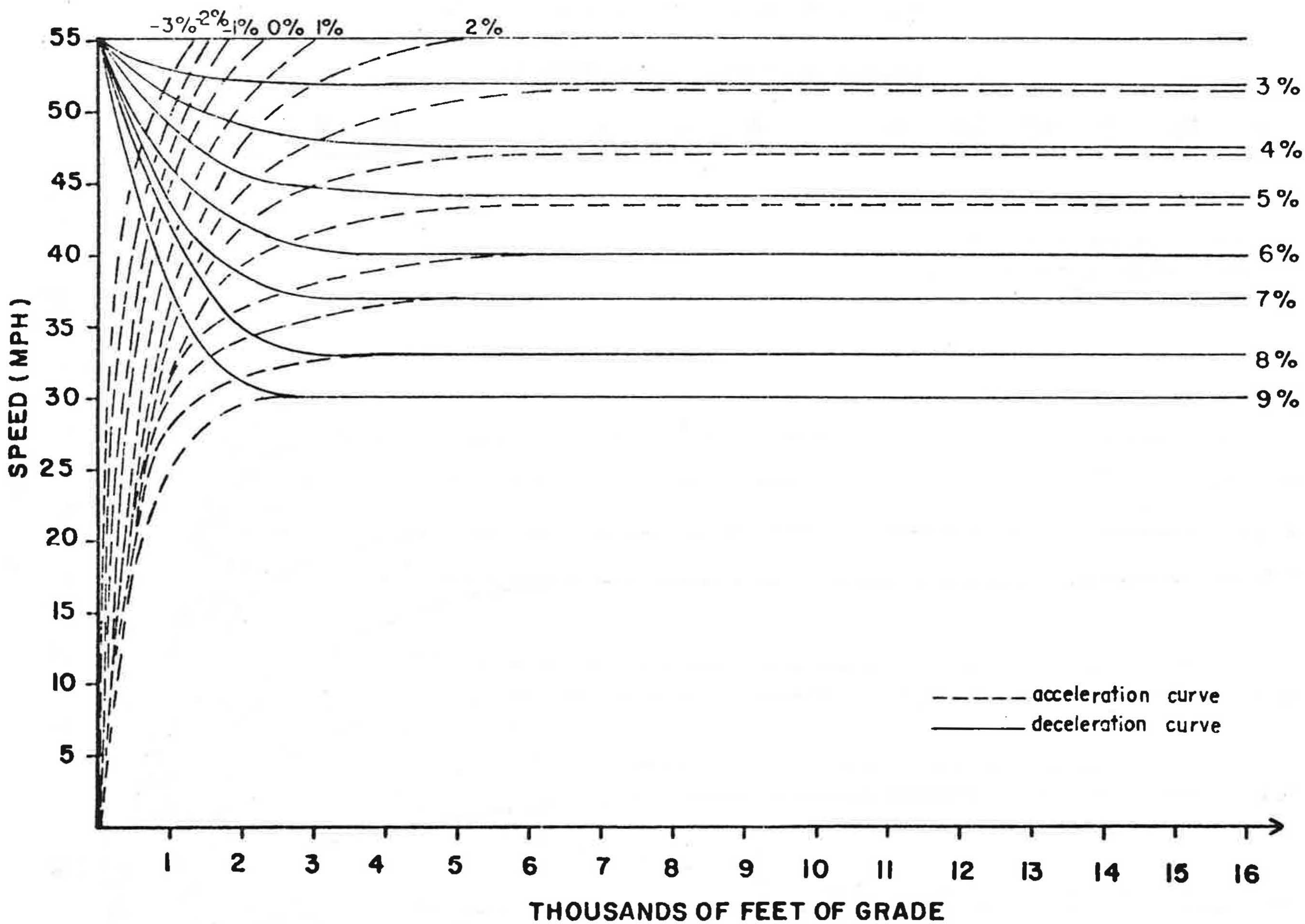


Figure I.3-3. Performance curves for light trucks (100 lb/hp).

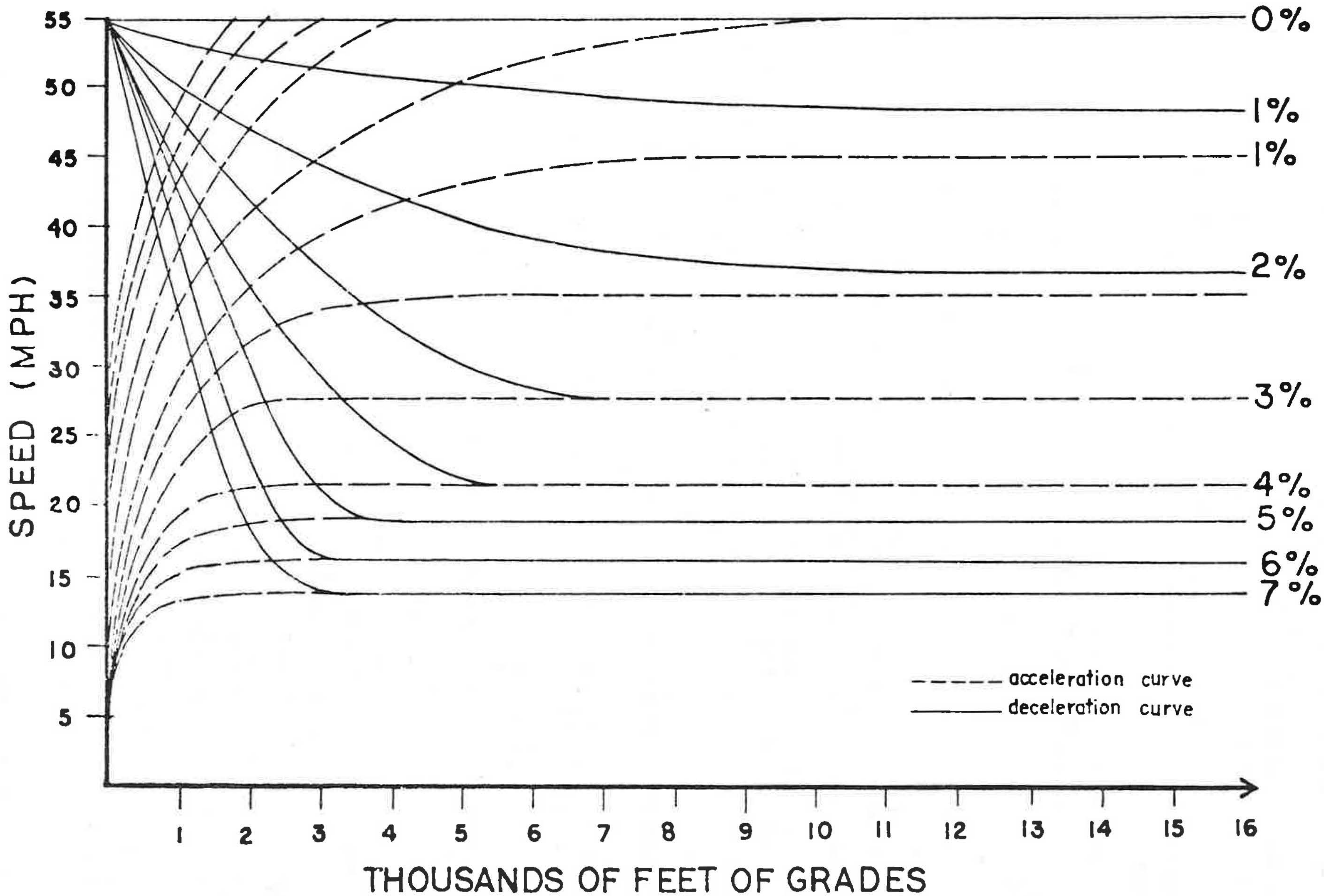


Figure I.3-4. Performance curves for heavy trucks (300 lb/hp).

OPERATIONAL ANALYSIS WORKSHEET

Facility Section ____

Date ________________________ Time ________________(of analysis data)

I. GEOMETRY

___ ft. distance to roadside obstructions

___ ft. ← DIR.1 N = ___

___ ft. BARRIER

___ ft. → DIR.2 N = ___

___ ft. distance to roadside obstructions

indicate North

	Design Speed (mph)	Lane Width (ft.)	Terrain Type (L, R, or M) or Grade (%)		Length (mi)	Barrier Type
DIR.1						
DIR.2						

II. VOLUMES

	VOL(vph)	PHF	SF=VOL/PHF	%trucks	%buses	%RV's	driver population
DIR.1							__commuter__other
DIR.2							__commuter__other

III. ANALYSIS

$v/c = SF/[c_j \times N \times f_w \times f_{HV} \times f_p]$

$c = c_j \times N \times f_w \times f_{HV} \times f_p$

$f_{HV} =$

	$v/c = SF$	$/[c_j$ (TAB. 3.1)	$\times N$	$\times f_w$ (TAB. 3.2)	$\times f_p$ (TAB. 3.10)	$\times f_{HV}]$	$1/[1+P_T(E_T-1)+P_B(E_B-1)+P_R(E_R-1)]$ E_T	E_B	E_R
DIR.1									
DIR.2									

	c	v/c	LOS (TAB.3.1)	SPEED (FIG.3.4)	DENSITY (FIG.3.3)
DIR.1					
DIR.2					

Name ______________________________________ Date __________________

Checked by ____

DESIGN WORKSHEET

Facility Section ______________________________

Date ______________________ Time ______________ (of analysis data)

I. DESIGN STANDARDS

	LOS	v/c (TAB. 3.1)	Des. Speed (mph)	Lane width (ft.)	Lateral clearance (ft.) roadside	Lateral clearance (ft.) median	Terrain or (L,R,or M)	Grade (%)	length (mi)
DIR.1									
DIR.2									

II. TRAFFIC FORECASTS

	DDHV(vph)	PHF	SF=(DDHV/PHF)	%trucks	%buses	%RV's	driver population
DIR.1							__commuter__other
DIR.2							__commuter__other

III. DESIGN ANALYSIS

$$\boxed{N = SF/[c_j \times v/c \times f_w \times f_{HV} \times f_p]}$$

$$f_{HV} = 1/[1+P_T(E_T-1)+P_B(E_B-1)+P_R(E_R-1)]$$

	N =	SF	/ [c_j (TAB. 3.1)	x (v/c)	x f_w (TAB. 3.2)	x f_p (TAB. 3.10)	x f_{HV}] (TAB. 3.9)	E_T	E_B	E_R
DIR.1										
DIR.2										

IV. SKETCH DESIGN

Name ______________________________ Date ____________________

Checked by ______________________________

APPENDIX III

GLOSSARY AND SYMBOLS

GLOSSARY

adjustment factor—A multiplicative factor that adjusts a capacity or service volume from ideal or base conditions to prevailing conditions, for a given characteristic.

average annual daily traffic—The total volume passing a point or segment of a highway facility, in both directions, for one year, divided by the number of days in the year.

basic freeway segment—A section of a freeway facility on which operations are unaffected by weaving, diverging, or merging maneuvers.

composite grade—A series of adjacent grades along a freeway having a comulative effect on operations, which is more severe than if each grade were considered separately.

crawl speed—The maximum speed which trucks or other heavy vehicles can maintain on a continuous sustained upgrade of a given percent; value depends on the type of vehicle and the percent of grade.

density—The number of vehicles occupying a given length of highway or highway lane, averaged over time; usually expressed as vehicles per mile, averaged over a one-hour or 15-min flow period.

directional design hour volume—the traffic volume for the design hour in the peak direction of flow; usually a forecast of the relevant peak hour volume.

freeway—A multilane divided highway facility that has a minimum of two lanes for the exclusive use of traffic in each direction and full control of access and egress.

freeway capacity—The maximum rate of flow at which vehicles can pass a point or segment of a freeway in one direction under prevailing traffic and roadway conditions.

heavy vehicle—Any vehicle with more than two axles and/or more than four tires touching the pavement, generally falling into one of three categories: trucks, recreational vehicles, and buses.

ideal conditions—A set of traffic and roadway conditions considered to be the best possible; includes uninterrupted flow, a minimum of two lanes for the exclusive use of traffic in each direction, 12-ft lanes, no lateral obstructions closer than 6 ft to the traveled way, and 70-mph design speed; the exact specification of ideal conditions varies with the type of facility.

level of service—A letter designation (from A to F) which generally characterizes the quality of traffic service experienced by motorists in any given situation; intended to reflect such characteristics as travel time, freedom to maneuver, comfort and convenience, safety, and others.

level terrain—Any combination of grades, length of grades, horizontal or vertical alignment, which permits heavy vehicles to maintain speeds that are approximately equal to the speeds of passenger cars.

maximum service flow rate—The highest 15-min rate of flow that can be accommodated on a highway facility under ideal conditions, while maintaining operating characteristics for a stated level of service; a value of maximum service flow rate is specific to a given level of service.

mountainous terrain—Any combination of grades, length of grades, horizontal or vertical alignment, which causes trucks and/or other heavy vehicles to reduce their speed to crawl speed for considerable distances or at frequent intervals.

passenger-car equivalent—The number of passenger cars that are displaced by a single heavy vehicle of a particular type under prevailing roadway and traffic conditions.

ramp junction—A short segment of highway along which vehicles transfer from an on-ramp to the main roadway, or from the main roadway to an off-ramp.

roadway conditions—A set of geometric characteristics which define a particular roadway: number and width of lanes, shoulders and lateral clearances, design speed, horizontal and vertical alignment, etc.

rolling terrain—Any combination of grades, length of grades, horizontal or vertical alignment, that causes trucks and/or other heavy vehicles to reduce their speed substantially below that of passenger cars, but does not involve operation at crawl speeds for substantial distances or at frequent intervals.

service flow rate—The maximum 15-min rate of flow that can be accommodated past a point or short segment of highway in one direction, under prevailing traffic and roadway conditions, while maintaining operating characteristics for a stated level of service; value is specific to a given level of service.

traffic conditions—A set of characteristics that describes the traffic stream, including percent composition by vehicle type, weekday vs. weekend and recreational traffic, lane distribution, and other factors.

weaving area—A length of highway over which traffic streams cross each other's path without the aid of traffic signals over a length of highway, doing so through the execution of lane-changing maneuvers; formed between merge and diverge points, as well as between on-ramps and off-ramps on limited access facilities.

SYMBOLS

$AADT$	average annual daily traffic, in vehicles per day.
c_j	capacity per lane for a freeway of design speed j under ideal conditions, in passenger cars per hour per lane.
c_T	capacity of a climbing lane under prevailing conditions, in vehicles per hour.
$DDHV$	directional design hour traffic, in vehicles per hour.
E_B	passenger-car equivalent for buses.
E_R	passenger-car equivalent for recreational vehicles.
E_T	passenger-car equivalent for trucks.
f_{HV}	adjustment factor for the presence of heavy vehicles in the traffic stream.

f_p adjustment factor for driver population type.
f_w adjustment factor for restricted lane widths and/or lateral clearances.
LOS level of service.
PHF peak hour factor.
P_B proportion of buses in the traffic stream, expressed as a decimal.
P_R proportion of recreational vehicles in the traffic stream, expressed as a decimal.
P_T proportion of trucks in the traffic stream, expressed as a decimal.
SFL service flow rate per lane, in vehicles per hour per lane.
MSF maximum service flow rate per lane, in passenger cars per hour per lane.
SF service flow rate, in vehicles per hour.

CHAPTER 5

RAMPS AND RAMP JUNCTIONS

CONTENTS

I. INTRODUCTION

A ramp may be described as a length of roadway providing an exclusive connection between two highway facilities. This chapter contains procedures for the analysis of ramp roadways and ramp-freeway junctions. The latter may be approximately applied to analyze ramp junctions with facilities other than freeways, such as expressways and multilane and two-lane highways, provided that the junctions involve merge or diverge movements that are not controlled by traffic signals, STOP signs, or YIELD signs. For ramp-street junctions controlled by such devices, the procedures of Chapter 9, "Signalized Intersections," and Chapter 10, "Unsignalized Intersections," should be applied.

RAMP COMPONENTS

A ramp may consist of up to three geometric elements of interest:

1. The ramp-freeway junction.
2. The ramp roadway.
3. The ramp-street junction.

A ramp-freeway junction is generally designed to permit high-speed merging or diverging movements to take place with a minimum of disruption to the adjacent freeway traffic stream. The geometric characteristics of ramp-freeway junctions vary. Elements such as the provision and length of acceleration/deceleration lanes, angle of convergence or divergence, relative grades on the freeway and ramp, and other aspects may impact ramp operations. Although the procedures of this chapter are primarily applicable to high-type designs, many of the relationships used were calibrated using data from a variety of geometric cases, including some which could be termed "substandard." Thus, these relationships can be applied to cases with less than ideal geometrics, as noted in the procedures. Geometric design standards for ramps and ramp junctions are given in the AASHTO policies *(1, 2)*.

The ramp roadway itself may also vary widely from location to location. Ramps vary in the number of lanes (usually one or two), length, design speed, grades, and horizontal curvature. The ramp roadway itself is rarely a source of operational difficulties, unless a traffic incident causes a disruption along its length.

The ramp-street junction can be of a type permitting uncontrolled merging of diverging movements to take place, or it can take the form of an at-grade intersection.

This chapter provides procedures for the capacity analysis of ramp-freeway junctions and ramp roadways. At-grade intersections may be analyzed using the procedures of Chapter 9, "Signalized Intersections," or Chapter 10, "Unsignalized Intersections." This chapter also contains a brief discussion of ramp control and its potential impacts on traffic and operations.

The last subject is treated qualitatively, with general quantitative guidelines. It is a topic which will have increasing importance in facility rehabilitation and management. However, no work to date has shown that actual ramp capacity increases due to ramp control. The enhancements fall into the categories of operational safety improvements at certain sites, and of management of the facility's overall capacity.

OPERATIONAL CHARACTERISTICS

A ramp-freeway junction is an area of competing traffic demands for space. Upstream freeway demand competes with on-ramp demand in merge areas. On-ramp demand is usually generated locally, although collector and arterial streets may bring vehicles to the ramp from more distant origins. The freeway flow upstream of an on-ramp is the composite of upstream demands from a variety of sources.

In the merge area, on-ramp vehicles try to find openings, or "gaps," in the adjacent freeway lane traffic stream. As most ramps are on the right side of the facility, the freeway lane most directly impacted is the shoulder lane, designated lane 1 herein. In this manual, lanes are numbered from 1 to N, from the shoulder to the median.

As the on-ramp flow increases, the entering vehicles impact the distribution of traffic among the freeway lanes as traffic shifts to avoid the turbulence and conflicts in the merging area. The situation is a dynamic one in which the flows interact, with the on-ramp flow generally having a significant influence on overall operations. In the relationships used in this chapter, the on-ramp volume is specified independently, and the lane 1 volume is thought of as being dependent on it as well as on other variables.

Under breakdown conditions, drivers often allow an "alternate merge" between on-ramp and lane 1 traffic. The actual merge pattern may vary, however, and it will have a significant impact on the length of main-line and ramp queues.

At off-ramps, the basic maneuver is a diverge. Exiting vehicles must occupy the lane adjacent to the ramp (or dedicated to the ramp exit), so that there is a net effect of other drivers redistributing themselves amongst the other lanes. Where two-lane off-ramps are present, the influence of diverging movements may spread over several lanes of the freeway.

Procedures in this chapter treat the freeway and ramp volumes as inputs to a ramp capacity analysis, with the level of service as the output or result of the analysis. Thus, the methodology presented is applied in the "Operational Analysis" mode. This is logical, because the ramp is a point location on an overall facility for which the volumes are either known or specified.

A ramp will operate efficiently only if all of its elements, the junctions with freeways and/or streets and the ramp roadway, have been properly designed. It is critical to note that a breakdown on any one of these elements will adversely affect the operation of the entire ramp. It should be further noted that a breakdown on a ramp may also extend to the facilities it connects.

II. METHODOLOGY

The focus of this chapter is the operation of ramp-freeway terminals. This element is often the determinant of overall ramp operation, and has a significant impact on the operation of the freeway itself. Merging and diverging maneuvers which occur at these junctions should take place at the speed of the freeway traffic stream and without disruption to that stream.

Because merging and diverging maneuvers occur in the freeway lane adjacent to the ramp, the amount and character of traffic in this lane is a principal concern in analysis. For the most common case of a right-hand ramp, lane 1, the shoulder lane, is adjacent to the ramp. Most of the computational procedures presented in this chapter concentrate on estimating the volume in lane 1 immediately upstream of an on- or off-ramp. In general, lane 1 volume has been shown to be dependent on:

1. The ramp volume, V_r.
2. The total freeway volume *upstream* of the ramp, V_f.
3. The distance to the adjacent upstream and/or downstream ramps, D_u, D_d.
4. The volumes on the adjacent upstream and/or downstream ramps, V_u, V_d.
5. The type of ramp (on- or off-ramp, number of lanes at the junction, etc.).

The location of, and volume on, adjacent ramps is a critical factor in determining lane 1 volume, because these characteristics greatly influence the lane distribution of freeway vehicles. For example, a heavy volume entering a freeway 500 ft upstream of a subject ramp would cause a large volume to remain in lane 1, because few of these vehicles would have had the opportunity to leave lane 1 within 500 ft.

RAMP CONFIGURATIONS

As the characteristics of adjacent upstream and downstream ramps influence the operations at any given location, ramp analysis must consider ramp sequences rather than each ramp in an isolated fashion. To avoid treating an unreasonable number of different configurations, ramps are generally examined in *pairs.* Thus, where a ramp has both adjacent upstream and downstream ramps close enough to impact its operation, *it will generally be considered twice—one in conjunction with the upstream ramp, and then in conjunction with the downstream ramp.* This is discussed in the "Procedures for Application" section and illustrated in the sample problems.

This chapter specifically addresses the following ramp configurations:

1. *Isolated on-ramp*—An on-ramp with no adjacent ramps close enough to influence its operations. The term, "close enough," varies, depending on volumes and other factors; however, ramp spacings greater than 6,000 ft are always considered beyond the range of influence.
2. *Isolated off-ramp*—An off-ramp with no adjacent ramps close enough to influence its operations.
3. *Adjacent on-ramps*—Two consecutive on-ramps close enough to mutually influence their behavior.
4. *Adjacent off-ramps*—Two consecutive off-ramps close enough to mutually influence their behavior.
5. *On-ramp followed by off-ramp*—An on-ramp, off-ramp sequence spaced closely enough to mutually influence each other's behavior. If the ramps are joined by a continuous auxiliary lane, the section is treated as a ramp-weave area and analyzed using the procedures of Chapter 4; if no auxiliary lane is present, the procedures in this chapter are used.
6. *Off-ramp followed by on-ramp*—An off-ramp, on-ramp sequence spaced closely enough to mutually influence each other's behavior. Such a ramp sequence often operates as if the ramps were isolated.
7. *Lane additions*—A one-lane on-ramp that results in the addition of a continuous freeway lane at the ramp-freeway junction.
8. *Lane drops*—A one-lane off-ramp that results in the deletion of one freeway lane at the ramp-freeway junction.
9. *Major diverge point*—The separation of a freeway segment into two multilane freeway or collector/distributor roadways. Refers only to those configurations for which the total number of lanes departing the diverge point is equal to the number of lanes approaching it plus one.
10. *Major merge point*—The joining of two multilane freeway or collector/distributor roadways into a single freeway segment. Refers only to configurations in which two approach lanes (one from each approach) are merged into a single lane.
11. *Two lane ramps*—Two-lane on-ramps or off-ramps where there are no lane additions or drops at the ramp-freeway junction.

These configurations are shown schematically in Figure 5-1. Illustration 5-1 contains photographs of typical freeway ramp configurations.

CRITICAL ELEMENTS FOR ANALYSIS

Once the lane 1 volume is known, it is possible to consider critical components of the traffic stream. For ramp configurations, these components are:

1. *Merge volume,* V_m—This term applies to on-ramps and is the total volume in the traffic streams which will join. For the case of a one-lane, right-side on-ramp, the merge volume is the sum of the lane 1 volume plus the ramp volume.
2. *Diverge volume,* V_d—This term applies to off-ramps. It is the total volume in the traffic stream which will separate. For the case of a one-lane, right-side off-ramp, the diverge volume is equal to the lane 1 volume immediately upstream of the subject ramp.
3. *Freeway volume,* V_f—At any merge or diverge location, the total freeway volume must also be considered. The freeway volume is generally considered at the point where it is at the maximum level, i.e., upstream of an off-ramp and downstream of an on-ramp.

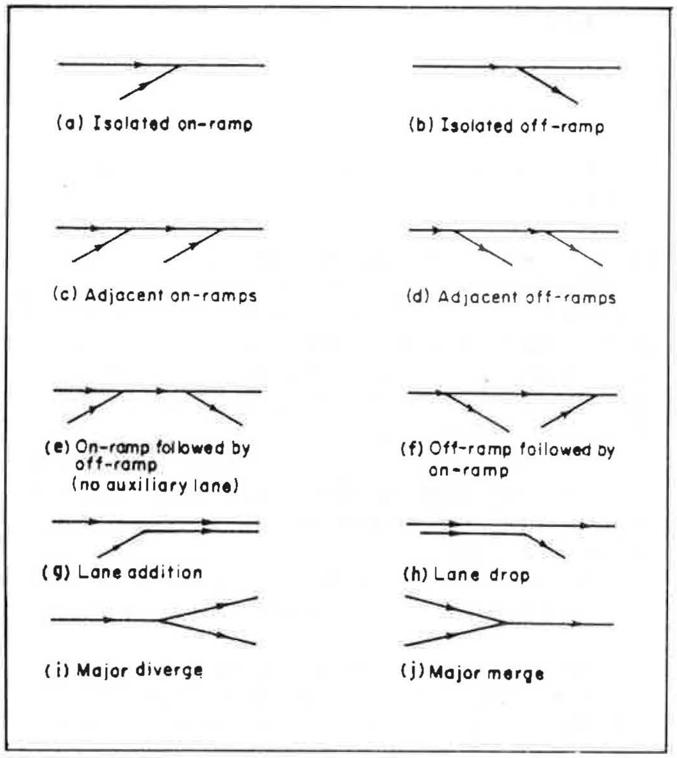


Figure 5-1. Ramp configurations covered by procedures.

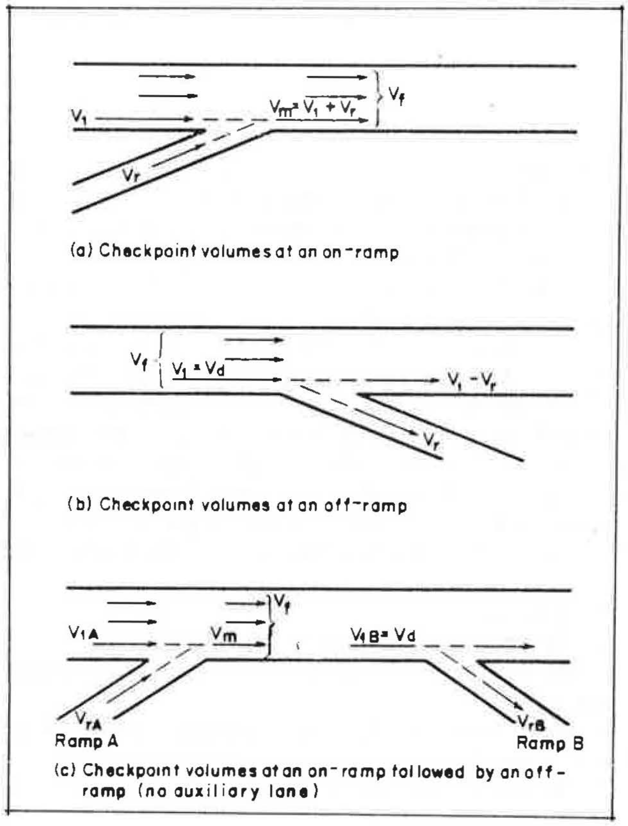


Figure 5-2. Checkpoint volumes for ramp-freeway terminals.

Figure 5-2 shows the relationships among these critical volumes and other volume elements. The merge, diverge, and freeway volumes are often referred to as "checkpoint" volumes, as it is these values to which level-of-service criteria are applied.

LEVEL-OF-SERVICE CRITERIA

Level-of-service criteria for merge, v_m, diverge, v_d, and freeway, v_f, flow rate checkpoints are given in Table 5-1. The criteria for freeway flow rates are the same as those given in Chapter 3, but are repeated here for the convenience of the user.

Note that criteria are stated in terms of *flow rates.* As in Chapters 3 and 4, computational procedures include the conversion of peak-hour volumes to equivalent hourly flow rates representing flow during the peak 15-min interval.

The criteria of Table 5-1 are not specifically correlated to measures of operational quality. They are intended, however, to reflect flow rates which may be accommodated while permitting the freeway as a whole to operate at the designated level of service in the vicinity of the ramp. Thus, the quality of operations is expected to be as described in Chapter 3, with some local turbulence in lane 1.

Level-of-service A represents unrestricted operation. Merging and diverging vehicles have little effect on other freeway flows. Merging is smoothly accomplished with only minor speed adjustments required to fill gaps; diverge movements encounter no significant turbulence.

At *level-of-service B,* merging vehicles have to adjust their speed slighlty to fill lane 1 gaps; diverging vehicles still do not experience any significant turbulence. Freeway vehicles not involved in merging or diverging movements are not seriously affected, and flow may be described generally as smooth and stable.

Level-of-service C, though still stable, approaches the range in which small changes in flow result in large changes in operating quality. Both lane 1 and on-ramp vehicles must adjust their speed to accomplish smooth merging, and under heavy on-ramp flows, minor ramp queuing may occur. Some slowing may also occur in diverge areas. Turbulence from on- and off-ramp maneuvers is more widespread, and the effects of this turbulence may extend into freeway lanes adjacent to lane 1. Overall speed and density of freeway vehicles are not expected to be seriously deteriorated.

At *level-of-service D,* smooth merging becomes difficult to achieve. Both entering and lane 1 vehicles must frequently adjust their speed to avoid conflicts in the merge area. Slowing in the vicinity of diverge areas is also significant. Turbulence from merge and diverge movements will affect several freeway lanes. At heavily used on-ramps, ramp queues may become a disruptive factor.

Level-of-service E represents capacity operation. Merge movements create significant turbulence, but continue without noticeable freeway queuing. On-ramp queues, however, may be significant. Diverge movements are significantly slowed, and some queuing may occur in the diverge area. All vehicles are affected by turbulence, and vehicles not involved in ramp movements attempt to avoid this turbulence by moving towards the median lanes.

At *level-of-service F* all merging is on a stop-and-go basis, and ramp queues and lane 1 breakdowns are extensive. Much turbulence is created as vehicles attempt to change lanes to avoid

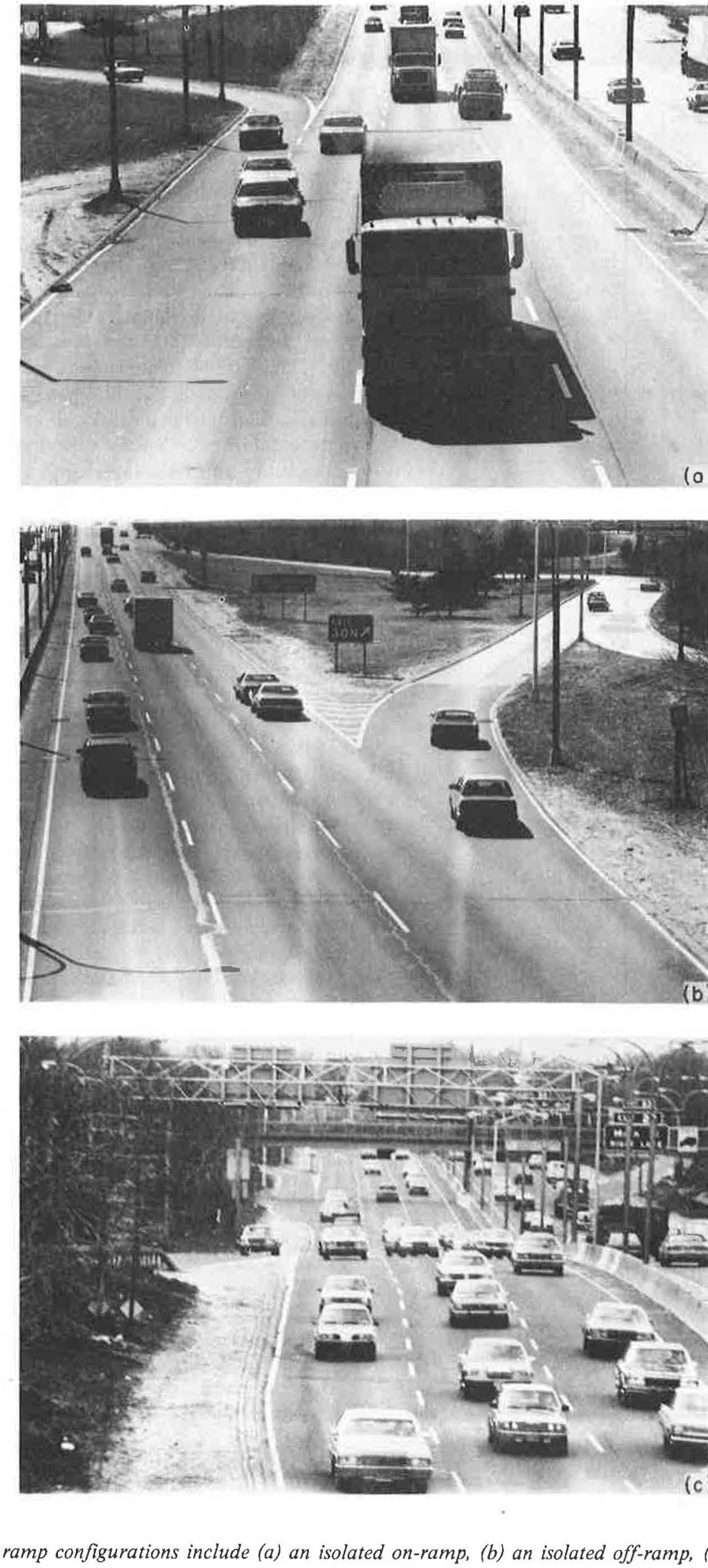


Illustration 5-1. Typical ramp configurations include (a) an isolated on-ramp, (b) an isolated off-ramp, (c) an on-ramp off-ramp sequence.

TABLE 5-1. LEVEL-OF-SERVICE CRITERIA FOR CHECKPOINT FLOW RATES AT RAMP-FREEWAY TERMINALS

LEVEL OF SERVICE	MERGE FLOW RATE (PCPH)[a] v_m	DIVERGE FLOW RATE (PCPH)[b] v_d	FREEWAY FLOW RATES (PCPH)[c], v_f								
			70-MPH DESIGN SPEED			60-MPH DESIGN SPEED			50-MPH DESIGN SPEED		
			4-LANE	6-LANE	8-LANE	4-LANE	6-LANE	8-LANE	4-LANE	6-LANE	8-LANE
A	≤ 600	≤ 650	≤ 1,400	≤ 2,100	≤ 2,800	d	d	d	d	d	d
B	≤ 1,000	≤ 1,050	≤ 2,200	≤ 3,300	≤ 4,400	≤ 2,000	≤ 3,000	≤ 4,000	d	d	d
C	≤ 1,450	≤ 1,500	≤ 3,100	≤ 4,650	≤ 6,200	≤ 2,800	≤ 4,200	≤ 5,600	≤ 2,600	≤ 3,900	≤ 5,200
D	≤ 1,750	≤ 1,800	≤ 3,700	≤ 5,550	≤ 7,400	≤ 3,400	≤ 5,100	≤ 6,800	≤ 3,200	≤ 4,800	≤ 6,400
E	≤ 2,000	≤ 2,000	≤ 4,000	≤ 6,000	≤ 8,000	≤ 4,000	≤ 6,000	≤ 8,000	≤ 3,800	≤ 5,700	≤ 7,600
F	WIDELY VARIABLE										

[a] Lane-1 flow rate plus ramp flow rate for one-lane, right-side on-ramps.
[b] Lane-1 flow rate immediately upstream of off-ramp for one-lane, right-side ramps.
[c] Total freeway flow rate in one direction upstream of off-ramp and/or downstream of on-ramp.
[d] Level of service not attainable due to design speed restrictions.

merge and diverge areas. Considerable delay is encountered in the vicinity of the ramp terminal (and perhaps for some distance upstream on the freeway), and conditions may vary widely, from minute to minute, as unstable conditions create "waves" of alternatively good and forced flow.

COMPUTING LANE 1 VOLUME

The computation of lane 1 volume, V_1, is the critical step in any ramp analysis. As noted previously, the lane distribution of freeway volume is affected by a number of variables including freeway and ramp volumes, the type of ramp under consideration, the location and characteristics of adjacent ramps, and the volumes on adjacent ramps. Lane 1 volume is computed for a point just upstream of the subject merge or diverge area.

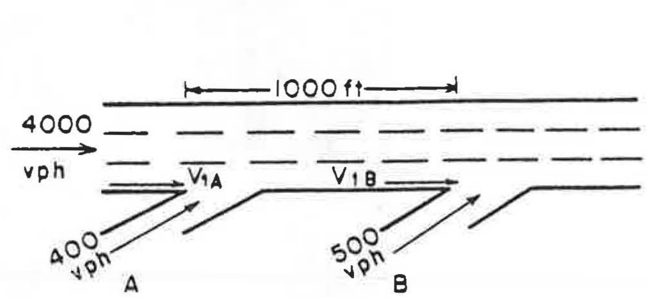
Table 5-2 contains an index to various equations and associated nomographs that are used in the computation of lane 1 volumes. Appendix I to this chapter contains these nomographs (with equations), which cover the various ramp configurations enumerated earlier. Because of the numerous ramp configurations which can occur, the nomographs do not cover all possible situations. For those cases in which none of the nomographs apply, an approximation procedure is used.

Nomograph Procedure

Each of the nomographs included in Appendix I contains a complete set of instructions for use, and details the conditions under which its use is acceptable. These should be carefully noted. Instructions are included for the use of default values extending the use of the nomographs to configurations that closely, but not exactly, resemble the same configurations as those treated. The equation for each nomograph is also prominently displayed. Where greater precision is desired, the direct use of the equations is recommended, although in most cases, the precision provided by the nomographs is adequate.

It should also be noted that all nomographs (and accompanying equations) have been calibrated in terms of mixed vehicles per hour (vph) for a full hour. Thus, the lane 1 volume computation occurs *before* volumes are converted to equivalent flow rates in passenger cars per hour (pcph).

The nomograph procedure for computation of lane 1 volume is best illustrated by example. Consider the following two on-ramps. Consideration of these ramps must begin by finding the lane 1 volume immediately upstream of ramps A and B, as shown.



NOTE: no upstream or downstream ramps within influence area of Ramps A and B.

TABLE 5-2. INDEX TO THE USE OF NOMOGRAPHS AND APPROXIMATION PROCEDURE FOR THE COMPUTATION OF LANE 1 VOLUME

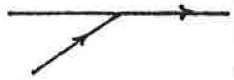
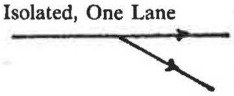
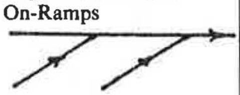
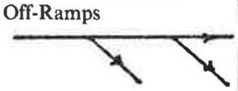
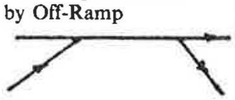
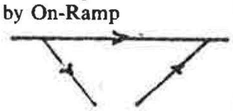
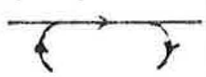
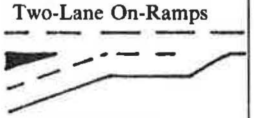
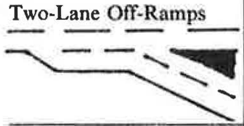
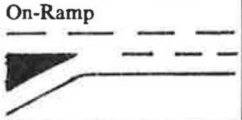
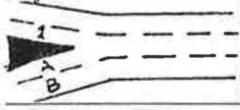
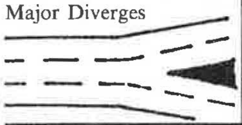
CONFIGURATION	4-LANE FREEWAY (2 LANES EACH DIRECTION)		6-LANE FREEWAY (3 LANES EACH DIRECTION)		8-LANE FREEWAY (4 LANES EACH DIRECTION)	
	1st RAMP	2nd RAMP	1st RAMP	2nd RAMP	1st RAMP	2nd RAMP
Isolated, One Lane 	Fig. I.5-1	—	Fig. I.5-6	—	Fig. I.5-9	—
Isolated, One Lane 	Fig. I.5-2	—	Fig. I.5-7	—	Approximate Using Table 5-3 and Fig. 5-5	—
Adjacent One-Lane On-Ramps 	Fig. I.5-1	Fig. I.5-5	Fig. I.5-6	Fig. I.5-8	Approximate Using Table 5-3 and Fig. 5-5	Approximate Using Table 5-3 and Fig. 5-5
Adjacent One-Lane Off-Ramps 	See Note 1	Fig. I.5-2	See Note 2	Fig. I.5-7	Approximate Using Table 5-3 and Fig. 5-5	Approximate Using Table 5-3 and Fig. 5-5
On-Ramp Followed by Off-Ramp 	Fig. I.5-1	Fig. I.5-3	Fig. I.5-6	Fig. I.5-7	Fig. I-5-10	Approximate Using Table 5-3 and Fig. 5-5
Off-Ramp Followed by On-Ramp 	Treat as Isolated Ramps			Fig. I.5-6	Treat as Isolated Ramps	
Loop Ramps 	Fig. I.5-4	Fig. I.5-3	Fig. I.5-6	Fig. I.5-7	Fig. I.5-10	Approximate Using Table 5-3 and Fig. 5-5
Two-Lane On-Ramps 	See Note 3	—	Fig. I.5-11	—	See Note 3	—
Two-Lane Off-Ramps 	See Note 4	—	Fig. I.5-12	—	See Note 4	—
Addition of Lane at On-Ramp 	Merge criteria in Table 5-1 may be applied directly to the on-ramp flow rate as a checkpoint.					
Dropping a Lane to the Off- at Off-Ramp 	Diverge criteria in Table 5-1 may be applied directly to the off-ramp flow rate as a checkpoint.					

TABLE 5-2. INDEX TO THE USE OF NOMOGRAPHS AND APPROXIMATION PROCEDURE FOR THE COMPUTATION OF LANE 1 VOLUME (CONTINUED)

CONFIGURATION	4-LANE FREEWAY (2 LANES EACH DIRECTION) 1st RAMP	4-LANE FREEWAY (2 LANES EACH DIRECTION) 2nd RAMP	6-LANE FREEWAY (3 LANES EACH DIRECTION) 1st RAMP	6-LANE FREEWAY (3 LANES EACH DIRECTION) 2nd RAMP	8-LANE FREEWAY (4 LANES EACH DIRECTION) 1st RAMP	8-LANE FREEWAY (4 LANES EACH DIRECTION) 2nd RAMP
Major Junctions 	Assume that lane B carries an amount of traffic equal to the merge checkpoint volume in Table 5-1 for the assumed level of service. Ramp lane A then carries the remaining ramp traffic. Compute lane 1 volume using Figure I.5-1 (4-lane freeway), Figure I.5-5 (6-lane freeway), or Figure I.5-9 (8-lane freeway), entering with ramp volume = lane A volume. Find checkpoint levels of service. Continue computations until assumed LOS agrees with results.					
Major Diverges 	Not Available	—	Fig. I.5-13	—	Not Available	—

NOTES:

1. Use Figure I.5-2 to find V_1 in advance of the first ramp, but enter with a V_r which is equal to the total volume on *both* ramps. This technique is valid where the distance between ramps is less than 800 ft. Where the distance betwen ramps is between 800 and 4,000 ft, use Table 5-3 and Figure 5-5 to approximate the situation. If the distance between ramps is greater than 4,000 ft, consider ramps to be isolated and consider separately.
2. Use Figure I.5-7 to find V_1 in advance of the first ramp, but enter with a V_r which is equal to the total volume on *both* off-ramps. This technique is valid where the distance between ramps is less than 800 ft. For other distance, see note 1.
3. Treat as two successive on-ramps separated by 400 ft; divide ramp volume equally between two ramp lanes.
4. Treat as two successive off-ramps separated by 400 ft; divide off-ramp volume equally between two ramp lanes.

Table 5-2 indicates that Figure I.5-6 should be used to compute the lane 1 volume immediately upstream of ramp A, V_{1A}, while Figure I.5-8 should be used for ramp B, V_{1B}.

Note that Figure I.5-6 is for an on-ramp on a six-lane freeway with both adjacent upstream and downstream off-ramps. Its use in the subject problem is, therefore, an approximation, and requires the use of default values as described under "Conditions for Use" on the nomograph. Instruction 2 of these conditions requires that the volume on the upstream adjacent off-ramp be set at 50 vph, because no such ramp exists for the subject problem. Instruction 3 indicates that the value of 640 (V_d/D_d) be set at 5, because no downstream off-ramp exists (the downstream ramp is an on-ramp in this case). With these default values, the equation or nomograph may be used:

$$V_1 = -121 + 0.244\,V_f - 0.085\,V_u + 640\,(V_d/D_d)$$

where:

$$V_f = 4{,}000 \text{ vph;}$$
$$V_u = 50 \text{ vph (default value);}$$
$$640(V_d/D_d) = 5 \text{ (default value); and}$$
$$V_{1A} = -121 + 0.244(4{,}000) - 0.085(50) + 5$$
$$V_{1A} = 856 \text{ vph}$$

Figure 5-3 illustrates the same solution using the nomograph, and results in $V_{1A} = 860$ vph.

Figure I.5-8 may be applied directly for the determination of V_{1B}. Note that when ramp B is considered, the freeway volume, V_f, is equal to 4,000 vph *plus* the 400 vph entering at ramp A, or 4,400 vph. Using the equation:

$$V_1 = 574 + 0.228\,V_f - 0.194\,V_r - 0.714\,D_u + 0.274\,V_u$$

where:

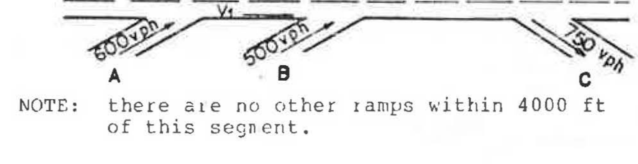
$$V_f = 4{,}400 \text{ vph;}$$
$$V_r = 500 \text{ vph;}$$
$$D_u = 1{,}000 \text{ ft;}$$
$$V_u = 400 \text{ vph; and}$$
$$V_{1B} = 574 + 0.228(4{,}400) - 0.194(500) - 0.714(1{,}000) + 0.274(400)$$
$$V_{1B} = 876 \text{ vph.}$$

Figure 5-4 illustrates the same solution using the nomograph. V_{1B} is found to be 870 vph. The difference between nomograph and equation solutions is due to the scale precision of the nomographs.

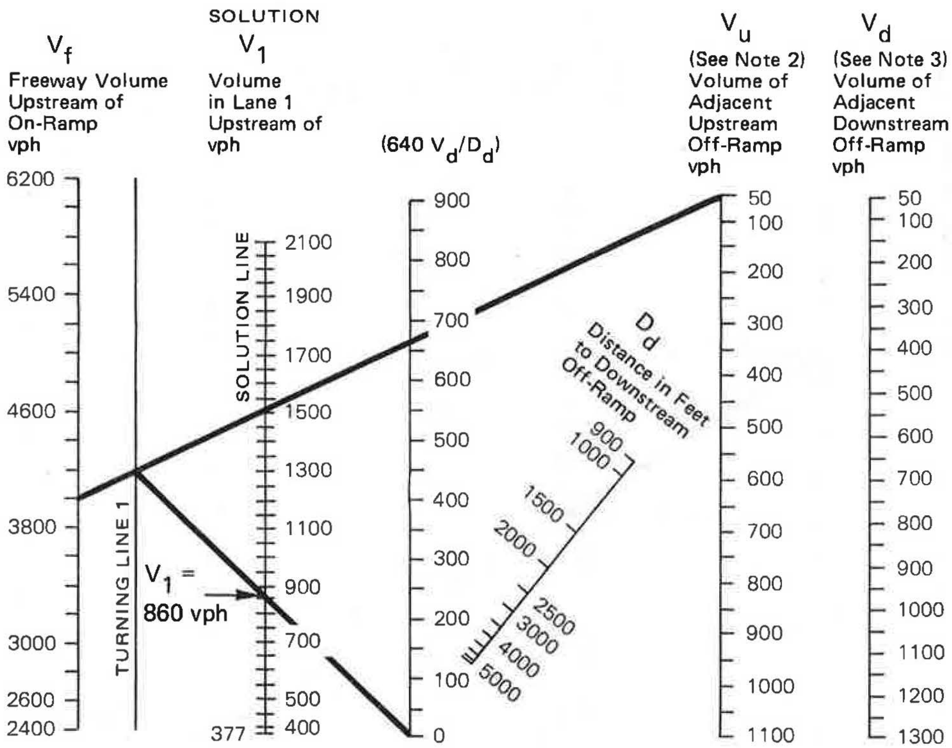
Approximation Procedure

Those cases for which no nomograph applies are analyzed using an approximate procedure. This most often occurs for ramps on eight-lane freeways, and for specific geometries that fall outside the range of variables for which a particular nomograph applies. Table 5-3 and Figure 5-5 are used to develop approximate estimates of lane 1 volume at ramps. It is emphasized that this procedure is used *only* where nomographs are not applicable to the particular configuration being studied.

Table 5-3 gives the percentage of "through" vehicles remaining in lane 1 in the vicinity of a subject ramp, where a through vehicle is defined as one not involved in *any* ramp movement within 4,000 ft of the subject ramp. Figure 5-5 shows the percentage of on- and off-ramp vehicles in lane 1 at various distances from the ramps on which they enter or leave the freeway. To find the total volume in lane 1, the through volume and each ramp volume within 4,000 ft of the subject ramp must be considered separately. Consider the following example:

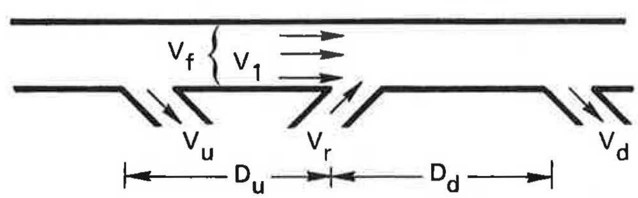


NOTE: there are no other ramps within 4000 ft of this segment.



Equation: $V_1 = -121 + 0.244V_f - 0.085V_u + 640\ V_d/D_d$

Diagram:



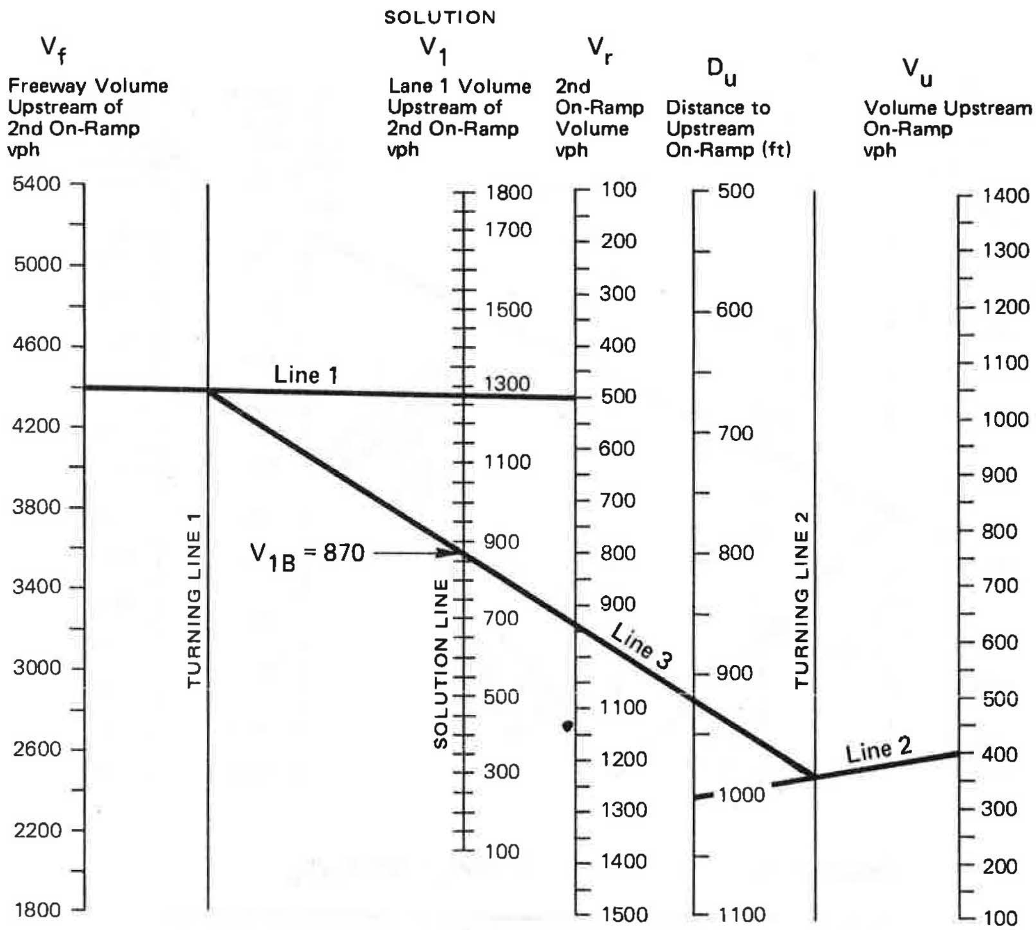
Conditions for Use:

1. Single lane on-ramps on 6-lane freeways with or without upstream and/or downstream off-ramps, with or without acceleration lane.
2. If there is no upstream off-ramp within 2600 ft, use V_u = 50.
3. If there is no downstream off-ramp within 5,700 ft, and V_f < 5000 vph, use 640 V_d/D_d = 5, and skip step 2 below.
4. Normal range of use: V_f = 2400 to 6200 vph; V_u = 50 to 1100 vph; V_d = 50-1300 vph
   V_r = 100 to 1700 vph; D_d = 900 to 5700 ft; D_u = 900-2600 ft

Steps in Solution:

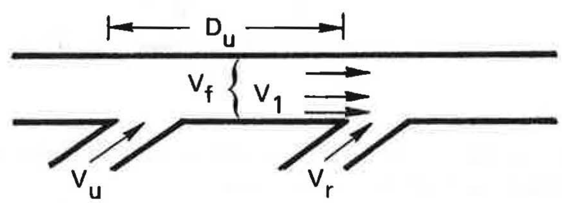
1. Draw a line from V_f value to V_u value, intersecting turning line 1.
2. Draw a line from V_d value to D_d value, intersecting 640 V_d/D_d line.
3. Draw a line from the step 1 intersection with turning line 1 to the 640 V_d/D_d value of step 2; read solution at intersection with V_1 line.

Figure 5-3. Nomograph solution for V_{1A} using Figure I.5-6 in Appendix I.



Equation: $V_1 = 574 + 0.228V_f - 0.194V_r - 0.714D_u + 0.274V_u$

Diagram:



Conditions for Use:

1. Single lane on-ramps on 6-lane freeways with adjacent upstream on-ramps, with or without acceleration lanes.
2. Normal range of use: V_f = 1800 to 5400 vph; V_r = 100 to 1500 vph
   V_u = 100 to 1400 vph; D_u = 500 to 1000 ft

Steps in Solution:

1. Draw a line from V_f value to V_r value, intersecting turning line 1.
2. Draw a line from V_u value to D_u value, intersecting turning line 2.
3. Draw a line from intersection of step 1 to that of step 2; read solution on V_1 line.

Figure 5-4 Nomograph solution for V_{1B} using Figure I.5-8 in Appendix I.

In this problem, the lane 1 volume immediately upstream of ramp B is sought. Before the solution can proceed, it is necessary to determine the "through" volume on the freeway. For such determinations, it is assumed that *no* vehicles entering the freeway in the subject segment also leave within it, unless planning or field information indicates otherwise. Thus, in the above illustration, the 750-vph exiting at ramp C are assumed to originate among the 5,000 vph on the freeway. The through volume for this problem is, therefore, 5,000 − 750 = 4,250 vph.

From Table 5-3 for an eight-lane freeway with 4,250-vph through volume, 8 percent of the through volume is expected to be in lane 1, and

$$V_{1B} \text{ (Through)} = 0.08 \times 4{,}250 = 340 \text{ vph}$$

Ramp B is 1,000 ft downstream of ramp A, on which 600 vph enter the freeway. Figure 5-5(II) indicates that 60 percent of on-ramp vehicles are expected to remain in lane 1, 1,000 ft downstream of the merge point. Therefore:

$$V_{1B} \text{ (Ramp A)} = 0.60 \times 600 = 360 \text{ vph}$$

Ramp B is also 1,500 ft upstream of ramp C, on which 750 vph exit the freeway. Figure 5-5(I) indicates that 79 percent of off-ramp vehicles are in lane 1 at a point 1,500 ft upstream of the diverge point. Thus:

$$V_{1B} \text{ (Ramp C)} = 0.79 \times 750 = 593 \text{ vph}$$

The total lane 1 volume immediately upstream of ramp B is the sum of these three components, or:

$$V_{1B} = 340 + 360 + 593 = 1{,}293 \text{ vph}$$

The approximation procedure traces the contribution of each ramp movement and the through volume to the lane 1 volume at any given point. When used, the procedure gives useful results, although they are generally not as accurate as the results of nomograph computations. This approximate procedure was developed and calibrated in California in the early 1960's, and is most properly applicable to volumes in the vicinity of level-of-service D, and is less accurate when applied at other levels.

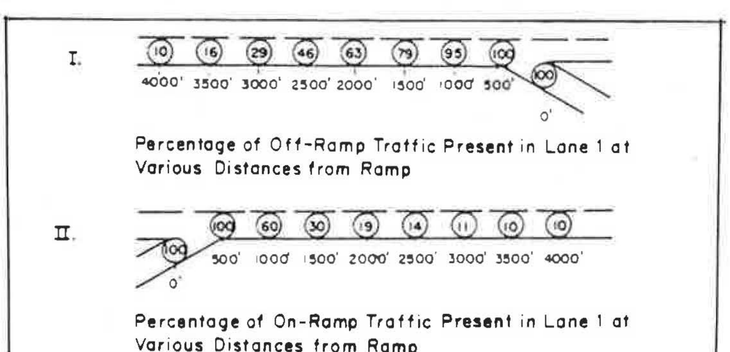
TRUCK PRESENCE IN LANE 1

Once the volume in lane 1 of the freeway is established immediately in advance of subject ramps, it is necessary to examine the likely percentage of trucks in that volume. Just as total volume does not distribute equally among all freeway lanes, neither do trucks. Trucks and other heavy vehicles tend to concentrate in the shoulder lane, with truck presence decreasing in lanes closer to the median. In some areas, trucks and other heavy vehicles are prohibited from using the median lane on six, or more, lane freeways. Thus, the volume in lane 1 generally has a disproportionately high percentage of trucks compared to other lanes.

TABLE 5-3. APPROXIMATE PERCENTAGE OF THROUGH TRAFFIC[a] REMAINING IN LANE 1 IN THE VICINITY OF RAMP TERMINALS

TOTAL THROUGH VOLUME, ONE DIRECTION (VPH)	THROUGH VOLUME REMAINING IN LANE 1 (%)		
	8-LANE FREEWAY	6-LANE FREEWAY	4-LANE FREEWAY
≥ 6500	10	—	—
6000 – 6499	10	—	—
5500 – 5999	10	—	—
5000 – 5499	9	—	—
4500 – 4999	9	18	—
4000 – 4499	8	14	—
3500 – 3999	8	10	—
3000 – 3499	8	6	40
2500 – 2999	8	6	35
2000 – 2499	8	6	30
1500 – 1999	8	6	25
≤ 1499	8	6	20

[a] Through traffic not involved in *any* ramp within 4,000 ft of the subject location.



NOTE: If the percentage found in this figure is less than the percent of through volume in lane 1 from Table 5-3, use the percentage given for through volume in Table 5-3.

Figure 5-5. Percentage of ramp vehicles in lane 1.

For the purposes of ramp analysis, in which performance criteria for levels of service are only generally defined, *all heavy vehicles are considered as trucks* to simplify computations.

Figure 5-6 describes the *percentage of total trucks located in lane 1.* This *is not* the proportion of trucks in the lane 1 volume, which must be computed from the results of Figure 5-6. Consider the following problem concerning an isolated on-ramp on a six-lane freeway:

V_f = 4,000 vph (Before Merge), 8 percent Trucks
V_r = 400 vph, 10 percent Trucks
V_1 = 856 vph (Found from Figure I.5-6)

The problem is to determine the proportion of trucks in the lane 1 volume, and the proportion of trucks in the total freeway volume after the merge.

Figure 5-6 is entered on the horizontal axis with a freeway volume of 4,000 vph (read on the scale as 40), rising vertically to the "6-lane freeway" curve, and projecting horizontally to the vertical axis. Here it is found that 52 percent of all trucks on the freeway are expected to be in lane 1. Then:

1. Number of trucks on freeway = 4,000 × 0.08 = 320 Trucks.
2. Number of trucks in lane 1 = 320 × 0.52 = 166 Trucks.
3. Proportion of trucks in lane 1 volume = 166/856 = 0.194 = 19.4 percent, say 19 Percent.
4. Number of trucks on freeway after merge = 320 + (0.10 × 400) = 360 Trucks.
5. Total freeway volume after merge = 4,000 + 400 = 4,400 vph.
6. Proportion of trucks in freeway volume after merge = 360/4,400 = 0.082 = 8.2 percent, say 8 Percent.

Note that, for computational purposes, truck presence is generally rounded to the nearest percent. This avoids the need to interpolate in passenger-car equivalent tables (of Chapter 3), and provides adequate precision.

Once the proportion of trucks in lane 1 and on the freeway (after the merge) is computed, all volumes may be converted to passenger cars per hour (pcph) by dividing by the heavy vehicle adjustment factor, f_{HV}, extracted from the appropriate tables of Chapter 3. Assuming that both the ramp and freeway illustrated here are in level terrain, volumes are converted as follows:

Item	Volume (vph)	Proportion of Trucks	E_T[a]	f_{HV}[b]	Equivalent Volume (pcph)
V_f (Before Merge)	4,000	0.08	1.7	0.95	4,000/0.95 = 4,211
V_f (After Merge)	4,400	0.08	1.7	0.95	4,400/0.95 = 4,632
V_r	400	0.10	1.7	0.93	400/0.93 = 430
V_1	856	0.19	1.7	0.88	856/0.88 = 973

[a] From Table 3-3
[b] Computed as $1/[1 + P_T(E_T - 1)]$

In problems where the ramp and freeway are on specific grades, the passenger-car equivalent values would be selected from Table 3-4. In these cases, the grade for the ramp and freeway would generally be different, and equivalents would be selected accordingly.

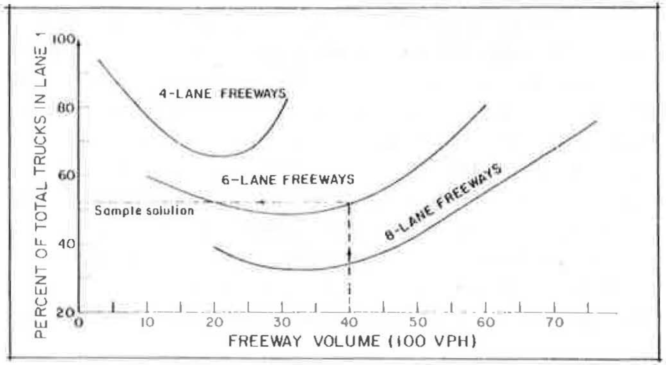


Figure 5-6. Truck presence in lane 1.

Figure 5-6 is based on expected national norms for the lane distribution of trucks. Local regulations restricting truck occupancy to certain lanes will affect this distribution, and local data should be checked wherever possible.

CHECKPOINT VOLUMES AND LEVEL-OF-SERVICE DETERMINATIONS

Once lane 1 volumes have been computed, and all volumes have been converted to equivalent passenger cars per hour, the remainder of the methodology is straightforward. Checkpoint volumes, i.e., all relevant merge, diverge, and freeway volumes, are computed and converted to peak flow rates by dividing by the peak hour factor (PHF).

As noted previously, the nomographs for computation of lane 1 volume are calibrated in terms of mixed vehicles per hour and full-hour volumes. Thus, the conversions to pcph and flow rates must be done *after* lane 1 volume computations are complete.

Level-of-service determinations are made by comparing checkpoint volumes to the criteria of Table 5-1.

III. PROCEDURES FOR APPLICATION

When the design of a ramp is being considered, the ramp location and general freeway design are already established (at least for a particular computational trial). Thus, ramp and freeway demand volumes are also established either from existing data or future forecasts, and are available as inputs to computations. In analysis, existing geometrics and volumes are known.

The computational procedures for ramp-freeway terminals are intended to find the level of service for a known existing or future forecast situation. Design is established by trial-and-error analyses. This design approach is not difficult because the number of options in any given case is generally limited. As other major elements of the freeway are most often already considered, the location of ramps is constrained by the location of intersecting facilities, and the geometry is constrained by terrain and fixed design features of the freeway itself.

A step-by-step computational procedure for the analysis of ramp terminals is given as follows.

STEP 1—ESTABLISH RAMP GEOMETRY AND VOLUMES

In analysis, these two factors are known. In design trials, a geometric configuration is assumed, and forecast volumes are assigned to the freeway and ramp(s).

The establishment of a configuration includes the type, location of, and volumes on adjacent ramps. Configuration is also the basis for selection of a nomograph (or equation) or approximation procedure for computation of lane 1 volume. Because nomographs deal primarily with ramp pairs, an individual ramp with both upstream and downstream adjacent ramps will often be considered *twice,* as part of a pair with each. For initial consideration, any adjacent ramp within 6,000 ft of the subject ramp should be treated as influencing ramp junction behavior. Individual nomographs include more detailed criteria for when an "adjacent" ramp may be considered to be isolated, and when it must be considered as part of a combination with adjacent ramps.

STEP 2—COMPUTE LANE 1 VOLUME

Lane 1 volume is computed using either one of 13 nomographs included in Appendix I or the approximation procedure described by Table 5-3 and Figure 5-5. Table 5-2 gives an index to these procedures. The choice of a specific nomograph or approximation procedure depends on (1) the ramp configuration in conjunction with adjacent ramps, (2) the number of lanes on the freeway, and (3) whether the ramp in question is the first or second of a paired configuration.

Each of the nomographs (Figures I.5-1 through I.5-13) in Appendix I contains a complete set of instructions for use, and details the conditions under which use is acceptable. These instructions and conditions should be carefully noted, particularly where an approximation is involved. Special instructions for such cases are provided. The equation for each nomograph is also prominently displayed. Where greater precision is desired, the direct use of the equation is recommended, although for many cases the precision provided by nomographs is adequate.

Table 5-3 and Figure 5-5 are used *only* where nomographs are not available for the particular configuration being considered. These exhibits were calibrated in California using data for periods of heavy volume (LOS D) and, when used, yield approximate results.

STEP 3—CONVERT ALL VOLUMES TO PASSENGER CARS PER HOUR

All lane 1 volumes, ramp volumes, and freeway volumes must be converted to equivalent volumes in passenger cars per hour (pcph). Volumes in mixed vehicles per hour may be converted to pcph by dividing by the appropriate heavy vehicle factor, f_{HV}, selected from Table 3-9 or computed using procedures described in Chapter 3.

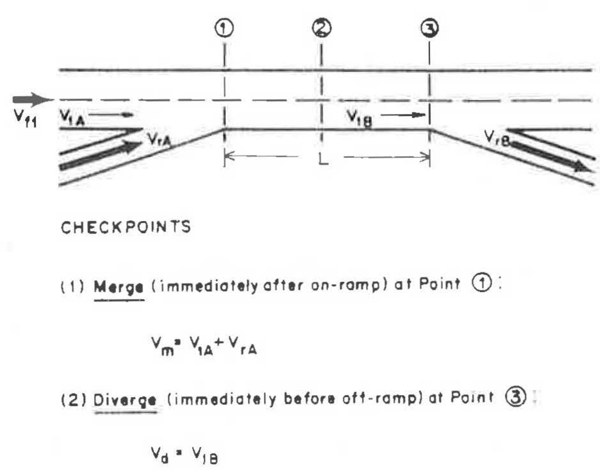
Before converting lane 1 volume to pcph, it is necessary to determine truck presence in this lane. Figure 5-6 or local data are used to estimate the percentage of total freeway trucks in lane 1, from which the proportion of trucks in the lane 1 volume may be computed.

STEP 4—COMPUTE CHECKPOINT VOLUMES

For each ramp analysis, there are up to three checkpoint volumes for each ramp or pair of ramps:

1. *Merge volume,* V_m — In any merge situation, two lanes will join to form a single lane. The merge volume is the sum of the volumes in the two lanes which join. In the most common case of a one-lane, right-side on-ramp, the merge volume equals the sum of the ramp volume plus the lane 1 volume immediately in advance of the ramp: $V_m = V_r + V_1$.
2. *Diverge volume,* V_d — The diverge volume is the total volume in a freeway lane immediately upstream of a point where the lane divides into two separate lanes. For the most common case of a one-lane, right-side, off-ramp, the diverge volume equals the lane 1 volume immediately in advance of the ramp: $V_d = V_1$.
3. *Total freeway volume,* V_f — The total volume on the freeway is checked at critical points. It is generally checked immediately upstream of an off-ramp and/or immediately downstream of an on-ramp.

Figure 5-7 illustrates the computation of checkpoint volumes for the case of an on-ramp followed by an off-ramp. Note that only one freeway volume checkpoint is needed, and that it is taken at a point between the two ramps where the freeway volume is at a maximum. This is consistent with the procedure outlined above, because the point selected is both upstream of the off-ramp and downstream of the on-ramp.



CHECKPOINTS

(1) Merge (immediately after on-ramp) at Point 1:

$V_m = V_{1A} + V_{rA}$

(2) Diverge (immediately before off-ramp) at Point 3:

$V_d = V_{1B}$

(3) Freeway Checkpoint Volume (upstream of off-ramp, downstream of on-ramp, between the ramps) at Point 2:

$V_f = V_{f1} + V_{rA}$

Figure 5-7. Computation of checkpoint volumes for an on-ramp followed by an off-ramp.

STEP 5—CONVERT CHECKPOINT VOLUMES TO PEAK FLOW RATES

Before comparing checkpoint volumes with the level-of-service criteria of Table 5-1, they must be adjusted to reflect peak flow rates rather than full-hour volumes. This is accomplished by dividing each checkpoint volume by the peak hour factor (PHF). Off-peak periods may be checked similarly.

STEP 6—FIND RELEVANT LEVELS OF SERVICE

The level of service for a given analysis is found by comparing the checkpoint flow rates for merging, diverging, and total freeway volume with the criteria given in Table 5-1.

In many cases, the various operational elements (merges, diverges, freeway flows) will not be in balance, i.e., have the same level of service. In such cases, the worst resultant LOS is assumed to govern the overall operation of the section in question. The analysis, however, will clearly identify those operational elements controlling the situation. These elements would then be candidates for improvement if the resulting LOS is considered unacceptable. Thus, if a merge is a congesting element in a segment of freeway, efforts at improvement would be targeted to the design and operation of the troublesome merge point.

It is desirable to have point locations such as ramp junctions operating in balance with the freeway as a whole. The most desirable operation would have the LOS of merge and diverge points equal to or better than the LOS for total freeway volume. Where merge and/or diverge points are the controlling element on a freeway segment, point congestion disrupts overall operation and prohibits the freeway from achieving a better level of service. Improvements at such locations should, therefore, be directed at removing point impediments and allowing the total freeway flow to determine operating conditions.

SPECIAL APPLICATIONS

The analysis steps outlined above apply to ramp-freeway junctions under a broad range of commonly occurring situations. There are, however, a number of less prevalent cases which also arise, and which may be treated using the general methodology with minor modifications. A number of these "special applications" are discussed in the following.

Ramp Junctions on Five-Lane Freeway Segments

Freeway segments with five lanes in a single direction are not common, but do occur in some major urban areas. These segments involve ramp junctions that need to be designed or analyzed. While no specific relationships exist for computing lane 1 volumes on five-lane segments, Ref. *4* contains an approximate procedure which can be applied.

Table 5-4 gives the approximate criteria for considering five-lane segments as equivalent four-lane segments (eight-lane freeway) by computing an equivalent freeway volume which can be used in conjunction with procedures for eight-lane freeways to determine lane 1 volume. The table in effect estimates the volume in the 5th lane, and subtracts it from the total freeway volume, allowing the remaining lanes to be treated as an eight-lane freeway.

TABLE 5-4. CONVERSION FACTORS FOR CONSIDERATION OF RAMPS ON FIVE-LANE SEGMENTS

RAMP TYPE	5-LANE FREEWAY VOLUME (VPH)	CONVERSION FACTOR
On-Ramp	All Volumes	0.78
Off-Ramp	≤ 4,000	1.00
	4,001 – 5,500	0.90
	5,501 – 7,000	0.85
	≥ 7,001	0.80

For example, if an off-ramp on a five-lane segment with a total freeway volume of 6,400 vph were being considered, procedures for an eight-lane freeway would be used, but with a freeway volume of 6,400 × 0.85 = 5,440 vph, where 0.85 is the conversion factor drawn from Table 5-4.

The lane 1 volume computed in this way is an approximation of the actual lane 1 volume for the five-lane segment.

When considering such cases, other special considerations include the following:

1. *Trucks in lane 1*—Truck presence in lane 1 may be computed using the eight-lane freeway curve of Figure 5-6. This is a "worst case" assumption, as little field data exist on truck distributions on five-lane segments.
2. *Freeway checkpoint*—The freeway flow rate checkpoint cannot be made directly using Table 5-1. The per lane freeway flow should be computed by dividing the total flow rate by 5, and the per lane freeway flow rate may then be compared to freeway LOS criteria in Table 3-1 of Chapter 3.

Left-Side Ramps

Although not normally recommended, left-side ramps do exist on some freeways, and thus often occur on collector-distributor roadways. Reference *4* again contains an approximate procedure for treating such ramps, involving two modifications to normal procedures:

1. *Lane i volumes*—The freeway lane of interest for a left-side ramp is not lane 1, but the median, or left-most lane of the freeway, designated herein as lane i. To compute lane i volumes, which are higher than corresponding lane 1 volumes, the lane 1 volume is computed as if a right-side ramp existed. Then:

Lane i volume = 1.25 × Lane 1 volume (On-Ramps)
Lane i volume = 1.10 × Lane 1 volume (Off-Ramps)

Note that the computation of "lane 1 volume" presumes that a right-hand ramp is present. The multipliers used here correct the result to reflect (1) the presence of a left-side ramp, and (2) a left-lane volume.

2. *Truck presence in lane i*—The proportion of trucks in lane i is approximated as follows:

a. For four-lane freeways, the proportion of *through* trucks in lane i is taken to be 25 percent of the total through trucks on the freeway. In the case of on-ramps, no additional trucks would be in lane i (immediately in advance of the merge point); in the case of off-ramps, *all* exiting trucks would be in lane i (immediately in advance of the diverge point).

b. For six- or more lane freeways, *no* through trucks are assumed to be in lane i. No on-ramp trucks would be in lane i, but *all* off-ramp trucks would be in lane i immediately in advance of the ramp.

Effects of Ramp Geometry

The methodology presented herein is calibrated for a wide variety of ramp configurations and geometries, not all of which are ideal. While no specific data exist, such specific geometric features as angle of approach or divergence, differential between freeway and ramp grade, and the existence and length of acceleration and deceleration lanes can have a dramatic impact on the operation of merge and diverge areas.

Drew (*7*) demonstrated, using gap acceptance models, that the gap acceptance capacity of an on-ramp would be reduced by as much as 90 percent when a 2-deg angle of convergence and a 1,200 ft acceleration lane were reduced to 10 deg and 400 ft respectively. The user is cautioned that "gap acceptance capacity" is not synonymous with "capacity" as defined in this chapter, and that the procedures herein do not assume ideal convergence angles or acceleration lanes, nor do they even define such criteria.

The designer or analyst should be aware, however, that such features do affect operations. Where extremely poor conditions exist, it is recommended that field studies be made to compare actual volumes with those predicted by the procedures herein.

Designers should be careful to provide for adequate ramp geometry, as defined in AASHTO policies (*1,2*), and analysts should be aware that poorly designed ramps may not operate as well as predicted by these procedures. Some extremely high merge volumes, however, have been observed at ramps with poor geometrics, particularly where drivers are familiar with the site. The effect of poor geometry may have a greater impact on operating quality and service flow rates than on capacity.

Ramp Roadways

There is very little information concerning operational characteristics on ramp roadways. Because most operational problems occur at ramp terminals, most quantitative studies have been concerned with terminal operations, not the ramp roadway itself.

Some basic design standards exist in AASHTO policies (*1,2*), but these are not related to specific operational characteristics. Leisch (*4*) has adapted this material to provide a broader set of criteria, but again, they are not related to specific operational characteristics.

Ramps differ considerably from the freeway mainline in that:

1. They are roadways of limited length and width (often one lane).
2. The design speed of the ramp is frequently lower than that of the roadways it connects.
3. On single-lane ramps, where passing is not possible, the adverse effect of trucks and other slow-moving vehicles is more pronounced than on a multilane roadway.
4. Acceleration and deceleration often take place on the ramp itself.
5. At ramp-street system interfaces, queuing may develop on the ramp.

Because of these distinct characteristics, it is difficult to adjust basic freeway criteria to approximate criteria for ramps. Reference *4* gives instructions for estimating the capacity of ramp roadways. Service flow rates for other levels of service are not as easily found, nor are there clear definitions of what type of operation is associated with each level. Table 5-5 gives approximate service flow rates for ramp roadways. Capacity estimates were generated from Ref. *4*, and other flow rates were approximately taken at similar v/c ratios as for the various levels of service on freeways. Extant data do not permit each level to be precisely described in terms of operating characteristics.

These values may be adjusted for heavy vehicle presence and lane width restrictions using the factors of Chapter 3. Their use in this context is, however, approximate.

TABLE 5-5. APPROXIMATE SERVICE FLOW RATES FOR SINGLE-LANE RAMPS[a] (pcph)

LOS	RAMP DESIGN SPEED (MPH)				
	≤ 20	21–30	31–40	41–50	≥ 51
A	[b]	[b]	[b]	[b]	600
B	[b]	[b]	[b]	900	900
C	[b]	[b]	1,100	1,250	1,300
D	[b]	1,200	1,350	1,550	1,600
E	1,250	1,450	1,600	1,650	1,700
F	WIDELY VARIABLE				

[a] For two-lane ramps, multiply the values in the table by: 1.7 for ≤ 20 mph
1.8 for 21–30 mph
1.9 for 31–40 mph
2.0 for ≥ 41 mph

[b] Level of service not attainable due to restricted design speed.

It should be noted that Table 5-5 refers only to the ramp roadway itself. Even though up to 1,700 pcph may be accommodated in a single-lane ramp, this does not guarantee that they can be accommodated in a single-lane ramp terminal, or at the ramp-street junction. As a general rule-of-thumb, where volumes exceed 1,500 pcph, a two-lane ramp-freeway terminal will be needed, and a two-lane ramp should be provided.

Further, even where a one-lane ramp and ramp terminal are sufficient from the capacity point of view, a two-lane ramp is generally provided if:

1. The ramp is longer than 1,000 ft, to provide opportunities to pass stalled or slow-moving vehicles.
2. Queues are expected to form on the ramp from a controlled ramp-street junction, to provide additional storage.
3. The ramp is located on a steep grade or has minimal geometrics.

If a two-lane ramp is provided for any of the above reasons, it is generally tapered to a single lane at the ramp-freeway junction.

It is difficult to maintain two-lane flow on loop ramps because of their severe horizontal alignment. In cases where two-lane loop ramps are deemed necessary, lane widths must be larger than 12 ft. Many states require lane-widening on loop ramps based on the off-tracking characteristics of trucks on such ramps.

The guidelines included herein are most useful in design where alternative ramp configurations may be developed for detailed analysis using ramp-freeway terminal procedures. In analysis, the total ramp flow may be quickly checked to ensure that adequate capacity is provided. Rarely, however, will the ramp roadway itself be a controlling factor in either design or analysis.

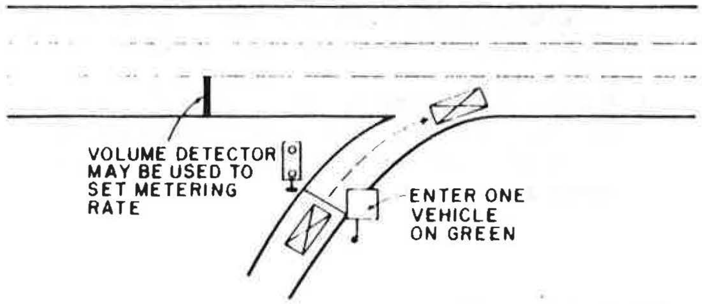


Figure 5-8. A typical ramp metering installation.

Ramp-Street Interface

This chapter does not address the subject of ramp-street system interfaces. Chapter 9 contains detailed procedures for the analysis of signalized junctions. A procedure for the analysis of unsignalized intersections is included in Chapter 10.

Where the ramp-street interface is itself a merge or diverge ramp junction of high-type design, the procedures in this chapter may be approximately applied.

Ramp Metering

Ramp metering has been used as an effective method of improving freeway operations at a number of on-ramp locations, and is now a generally accepted practice. Signals are placed on the ramp, at a point in advance of the acceleration lane, to control the entry of vehicles. One vehicle at a time is permitted to enter the freeway with each "green" flash of the signal. Figure 5-8 shows a typical installation of ramp control.

Signals may be set to allow a single vehicle to enter at regular intervals (typically 5 to 10 sec), or they may be operated by freeway detectors which sense approaching flow or occupancy in lane 1, allowing vehicles to enter when gaps are available. Reference *8* is a comprehensive treatment of ramp metering and system use of ramp controls. Chapter 6 contains a more complete discussion of ramp control in conjunction with overall freeway surveillance and control.

While the impact of ramp control on capacity is not thought to be great, the impact of control on operations is beneficial in two principal ways:

1. Ramp meters can be set to avoid breakdowns at ramp junctions; this allows the full capacity of dowstream sections to be effectively utilized by avoiding upstream bottlenecks which would prevent demand from reaching capacity levels.
2. Ramp meters can be set to allow a desired level of service to be attained and maintained on the facility.

Ramp control can also be used to ease operations at particular problem sites. It has been used to enhance the safety characteristics of ramps with poor sight distances or extremely short lengths. It has also been used to disperse platooned freeway entries from signalized street junctions.

The basic purpose of ramp metering is to assure that stable flow is maintained in freeway lanes without breakdown into congested flow with its attendant shock waves, stop-and-go operation, and resultant loss in service flow rates. It should be remembered, however, that vehicles diverted from ramps by the use of controls will either queue or find alternative routes, perhaps increasing congestion in nearby areas.

The procedures of this chapter are designed for uncontrolled ramps. Computations assume that the ramp volume, V_r, is a given value. Where ramp control is being considered, it is most useful to consider V_r to be a dependent variable, solving for an appropriate value to ensure that a given LOS is not violated at the merge point or on the freeway. This is a trial-and-error process, as computations for V_1 depend on a value of V_r. To compute the maximum value of V_r allowable for a given LOS, the following procedure may be followed:

1. Find the merge service flow rate, SF_m, from Table 5-1 for the LOS of interest, and convert this to an equivalent merge volume: $V_m = SF_m \times \text{PHF}$.
2. Assume a value of V_r.
3. Compute V_1 using the procedures described in this chapter.
4. Compute $V_r = V_m - V_1$.
5. Continue computations until the V_r assumed in (2) matches the value computed in (4).

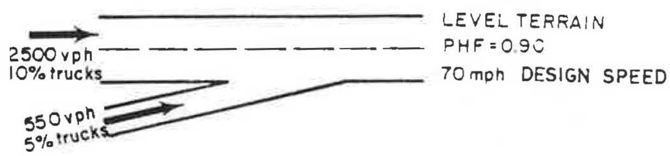
Of course, all values must be converted to passenger cars per hour and peak flow rates, as described elsewhere in this chapter. Sample Calculation 8 illustrates this process for determining an appropriate ramp metering rate.

There are, of course, many other considerations which bear on ramp-metering, including downstream freeway flows and levels of service, availability of and impact on alternate routes, and other factors.

IV. SAMPLE CALCULATIONS

CALCULATION 1—ISOLATED ON-RAMP

1. *Problem Description*—Consider the following on-ramp, which has no adjacent ramps within 6,000 ft, and may be considered to operate in a isolated manner:



What level of service would be expected to prevail?

2. *Solution*—Using the index provided in Table 5-2, it is seen that Figure I.5-1 of Appendix I is chosen as the appropriate nomograph for this case. Thus, the lane 1 volume immediately upstream of the on-ramp is computed as:

$$V_1 = 136 + 0.345\ V_f - 0.115\ V_r$$

where:

$$V_f = 2{,}500 \text{ vph};$$
$$V_r = 550 \text{ vph};$$
$$V_1 = 136 + 0.345(2{,}500) - 0.115(550) = \mathit{935\ vph}.$$

This value may be found from the nomograph as approximately 930 vph.

From Figure 5-6, about 67 percent of all trucks on the freeway will be in lane 1 immediately upstream of the ramp. Therefore:

Total trucks on freeway = 2,500 × 0.10 = 250 Trucks
Trucks in lane 1 = 250 × 0.67 = 168 Trucks
Proportion of trucks in lane 1 volume = 168/935 = 0.18 or 18 percent

At this point, the lane 1 ramp and freeway volumes must be converted to passenger cars per hour. Values of E_T are selected from Table 3-3 and values of f_{HV} are computed as $1/[1+P_T(E_T - 1)]$.

Item	Volume (vph)	E_T	Proportion of Trucks	f_{HV}	Vol. (pcph) = Vol. (vph)/f_{HV}
V_1	935	1.7	0.18	0.89	1,051
V_r	550	1.7	0.05	0.97	567
V_f	2,500	1.7	0.10	0.93	2,688

Checkpoint volumes may now be computed:

$$V_m = V_r + V_1 = 567 + 1{,}051 = 1{,}618 \text{ pcph}$$
$$V_f \text{ (After Merge)} = V_f \text{ (Before Merge)} + V_r$$
$$= 2{,}688 + 567 = 3{,}255 \text{ pcph}$$

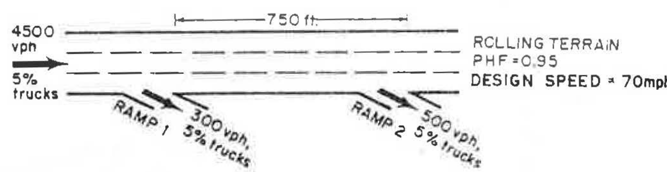
These values are now expanded to peak flow rates by dividing by the peak hour factor. The level of service is then found by comparing the merge and freeway checkpoint flow rates to the criteria of Table 5-1:

$$V_m = 1{,}618/0.90 = 1{,}798 \text{ pcph (LOS E, Table 5-1)}$$
$$V_f = 3{,}255/0.90 = 3{,}617 \text{ pcph (LOS D, Table 5-1)}$$

In this case, the merge area is the controlling feature (an undesirable condition), and the prevailing LOS is E.

CALCULATION 2—CONSECUTIVE OFF-RAMPS

1. *Problem Description*—Consider the following ramp configuration. There are no other ramps within the influence area of the ramps shown:



At what level of service would the two off-ramps be expected to operate?

2. *Solution*—As indicated in Table 5-2, note 2 must be consulted when analyzing the first ramp. Note 2 specifies the use of Figure I.5-7 for this ramp, but instructs that V_r be taken as equal to the *total* off-ramp volume on both ramps. Figure I.5-7 is also used for the second ramp.

• Ramp 1. Because there is no upstream on-ramp involved, the value "215 V_u/D_u" will be set at 2, as directed by item 2 under "Conditions for Use" on Figure I.5-7. As noted above, V_r will be taken as 300 + 500 = 800 vph for consideration of the first ramp. Then:

$$V_1 = 94 + 0.231\ V_f + 0.473\ V_r + 215\ V_u/D_u$$

$$V_1 = 94 + 0.231(4{,}500) + 0.473(800) + 2$$

$$V_1 = 1{,}514 \text{ vph}$$

• Ramp 2. For ramp 2, V_f equals 4,500 − 300 or 4,200 vph. Further, "215 V_u/D_u" will still be set equal to 2:

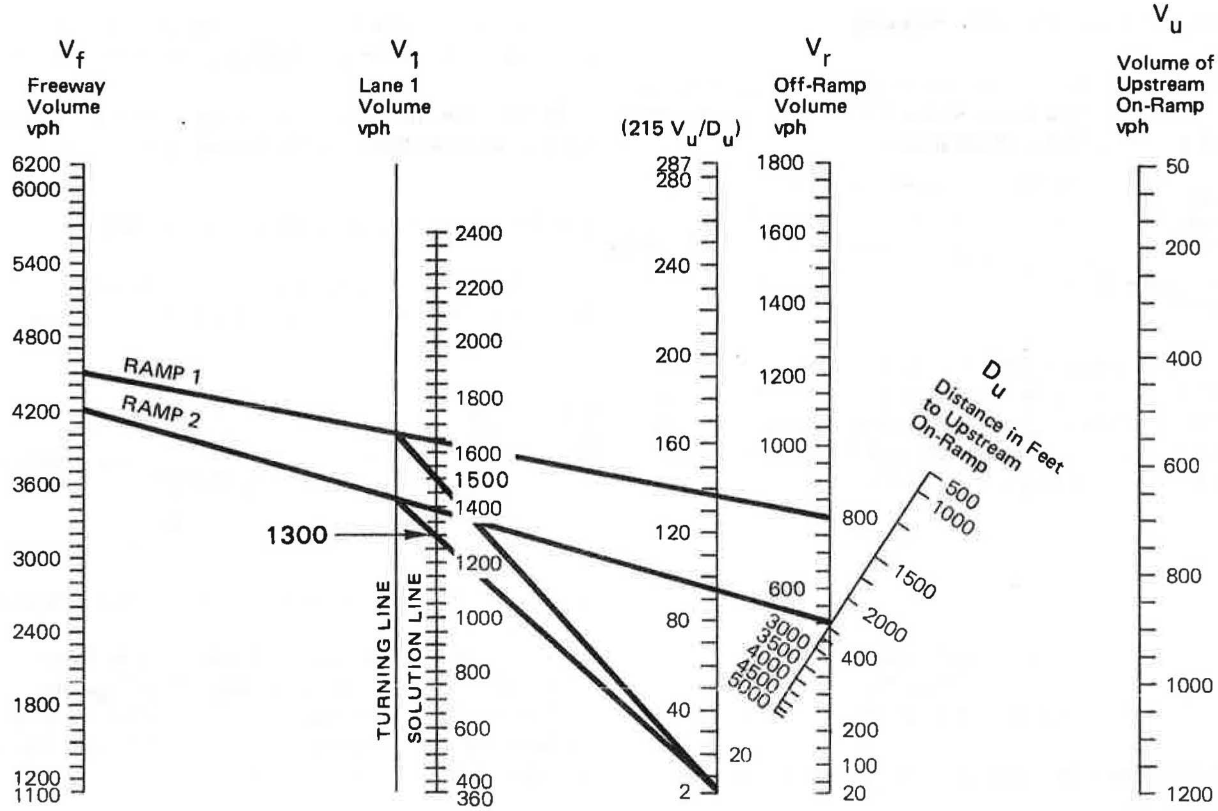
$$V_1 = 94 + 0.231(4{,}200) + 0.473(500) + 2$$

$$V_1 = 1{,}303 \text{ vph}$$

Figure 5-9 illustrates the nomograph solutions for both of these values. V_1 = 1,500 vph for ramp 1 and 1,303 for ramp 2.

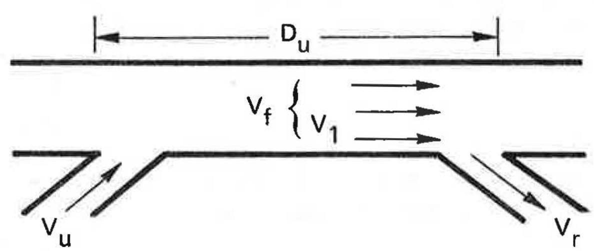
The proportion of trucks in the respective lane 1 volumes is now computed:

• Ramp 1
Percent total trucks in lane 1 = 56 percent (Figure 5-6)
Total trucks on freeway = 4,500 × 0.05 = 225 Trucks
Trucks in lane 1 = 225 × 0.56 = 126 Trucks
Proportion of trucks in lane 1 volume = 126/1,514
= 0.083, say 8 Percent



Equation: $V_1 = 94 + 0.231V_f + 0.473V_r + 215\ V_u/D_u$

Diagram:



Conditions for Use:

1. Single-lane off-ramp on a 6-lane freeway with or without upstream on-ramp, with or without deceleration lane.
2. If there is no upstream on-ramp within 5700 ft, skip step 2 below, and set $215\ V_u/D_u$ to 2.
3. Normal range of use: V_f = 1100 to 6200 vph; V_r = 20 to 1800 vph
   V_u = 50 to 1200 vph; D_u = 900 to 5700 ft

Steps in Solution:

1. Draw line from V_f value to V_r value, intersecting turning line.
2. Draw line from V_u value to D_u value, intersecting $215\ V_u/D_u$ line.
3. Draw line from intersection point of step 1 to that of step 2; read solution on V_1 line.

Figure 5-9. Nomograph solutions for Calculation 2 (Figure I.5-7 in Appendix I is the base nomograph).

• Ramp 2

Percent trucks in lane 1 = 53 Percent (Figure 5-6)
Total trucks in lane 1 = 4,200 × 0.05 = 210 Trucks
Trucks in lane 1 = 210 × 0.53 = 111 Trucks
Proportion of trucks in lane 1 volume = 111/1,303
= 0.085, say 9 Percent

Then:

Item	Volume (vph)	E_T[a]	Proportion of Trucks	f_{HV}[b]	Vol. (pcph) = Vol. (vph)/f_{HV}
V_f	4,500	4	0.05	0.87	5,172
V_r (1)	300	4	0.05	0.87	345
V_r (2)	500	4	0.05	0.87	575
V_1 (1)	1,514	4	0.08	0.81	1,869
V_1 (2)	1,303	4	0.09	0.79	1,649

[a] Table 3-4
[b] Table 3-9

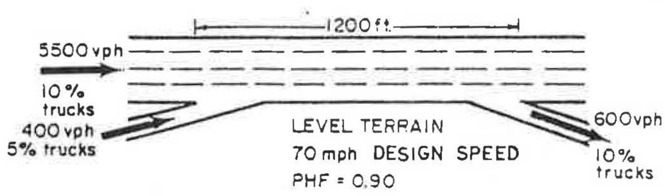
Three checkpoint volumes are of interest: (1) the freeway volume at the maximum point, *before* the two off-ramps, and (2) the diverge volumes before each of the off-ramps. Each checkpoint volume must be converted to a peak flow rate and compared with the criteria of Table 5-1.

V_f = 5,172/0.95 = 5,444 pcph (LOS D, Table 5-1)
V_d (1) = V_1 (1) = 1,869/0.95 = 1,967 pcph (LOS E, Table 5-1)
V_d (2) = V_1 (2) = 1,649/0.95 = 1,736 pcph (LOS D, Table 5-1)

In this situation, the diverge at ramp 1 is clearly the critical restrictive element on operations, and causes the overall LOS to be E. The high lane 1 volume at this point, however, is greatly influenced by the presence of a second, more heavily used, off-ramp within 750 ft. The diverge volume at ramp 1 is not really the problem per se, but the total lane 1 volume at that point is. This would not be an easy situation to remedy, although consideration to modifying the location of the ramps might be given, particularly if greater separation could be provided. The impacts of moving ramps on demand must be considered, however. The addition of a freeway lane in the vicinity of these ramps might be considered to separate off-ramp vehicles from the through volume in lane 1. This lane could be dropped at the first or second off-ramp.

CALCULATION 3—ON-RAMP FOLLOWED BY AN OFF-RAMP

1. *Problem Description*—Consider the following configuration. No other ramps influence the behavior of those shown:



At what level of service would the section operate?

2. *Solution*—Table 5-2 indicates that the on-ramp be analyzed using Figure I.5-10. The off-ramp situation must be approximated using Table 5-3 and Figure 5-5.

• *On-Ramp.* Note that the distance of 1,200 ft between ramps falls outside of the calibrated range of 1,500 to 3,000 ft for Figure I.5-10. Thus, the analyst must choose between extending this range and using the nomograph for stated case, or using Table 5-3 and Figure 5-5 as an approximation. Both methods are illustrated as follows.

Using Figure I.5-10:

$V_1 = -353 + 0.199\ V_f - 0.057\ V_r + 0.486\ V_d$
$V_1 = -353 + 0.199(5,500) - 0.057(400) + 0.486(600)$
V_1 = 1,010 vph

Using Table 5-3 and Figure 5-5:

Through volume = 5,500 − 600 = 4,900 vph
Percent through volume in lane 1 = 9 Percent (Table 5-3)
Percent off-ramp volume in lane 1, 1,200 ft upstream = 89 Percent (Figure 5-5)
V_1 (Through) = 4,900 × 0.09 = 441 vph
V_1 (Off) = 600 × 0.89 = 534 vph
V_1 = 975 vph

Because the lane 1 volume is higher when the nomograph is used, the value of 1,010 vph will be used as a worst case analysis.

From Figure 5-6, the percentage of total trucks in lane 1 is 49 percent. Therefore:

Total trucks on freeway = 5,500 × 0.10 = 550 Trucks
Trucks in lane 1 = 550 × 0.49 = 270 Trucks
Proportion of trucks in lane 1 volume = 270/1,010 = 0.267, say 27 Percent

• Off-Ramp. The freeway volume in advance of the off-ramp is 5,500 + 400 = 5,900 vph. The "through" volume is 5,900 − 600 − 400 = 4,900 vph. The lane 1 volume immediately in advance of the off-ramp consists of:

9 Percent of the through volume (Table 5-3)
100 Percent of the off-ramp volume (Figure 5-5I)
48 Percent of the on-ramp volume (Figure 5-5II, interpolate between 1,000 ft and 1,500 ft)

Thus:

$$V_1 = 0.09(4,900) + 1.00(600) + 0.48(400)$$

$$V_1 = 1,233 \text{ vph}$$

From Figure 5-6, this lane 1 volume contains 54 percent of the total trucks on the freeway:

Total trucks on freeway = (5,500 × 0.10) + (400 × 0.05) = 570 Trucks
Trucks in lane 1 = 570 × 0.54 = 308 Trucks
Proportion of trucks in lane 1 volume = 308/1,233 = 0.249, say 25 Percent

Now, each volume must be converted to passenger cars per hour and expanded to a peak flow rate by dividing by the PHF. Both steps are done in the table which follows for convenience. Note that the freeway volume is checked between the two ramps, where it is at a maximum. The proportion of trucks in the freeway volume at this point is 570/5,900 = 0.097, say 10 percent.

Item	Volume (vph)	Proportion of Trucks	E_T[a]	f_{HV}[b]	Flow Rate (pcph) = Vol. (vph)/f_{HV} × PHF
V_1 (On)	1,010	0.27	1.7	0.84	1,336
V_1 (Off)	1,233	0.25	1.7	0.85	1,612
V_f	5,900	0.10	1.7	0.93	7,049
V_r (On)	400	0.05	1.7	0.97	458
V_r (Off)	600	0.10	1.7	0.93	717

[a] Table 3-3
[b] Computed as $f_{HV} = 1/[1 + P_T(E_T - 1)]$

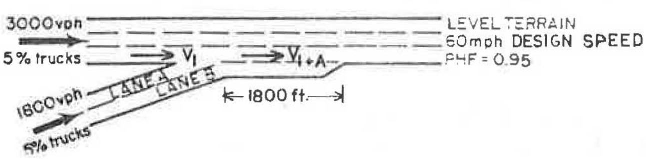
Critical checkpoint volumes may now be computed and compared with the criteria in Table 5-1.

$V_m = V_1$ (On) $+ V_r$ (On) $= 1{,}336 + 458 = 1{,}794$ pcph (LOS D, Table 5-1)
$V_d = V_1$ (Off) $= 1{,}612$ pcph (LOS D, Table 5-1)
$V_f = 7{,}049$ pcph (LOS D, Table 5-1)

In this case, level-of-service D will prevail, and all operational elements are in balance.

CALCULATION 4—TWO-LANE ON-RAMP

1. *Problem Description*—Consider the following two-lane on-ramp. There are no other ramps within 6,000 ft of the ramp shown:



What level of service would be expected at this location.

2. *Solution*—Table 5-2 indicates that Figure I.5-11 should be used for this problem. Note that the solution to this problem involves two merges—the first when lane 1 merges with lane A, and the second when lane B merges with the total volume from the first merge. The second merge is the most critical for the analysis. The nomograph is used to solve for V_1 and V_{1+A} in this problem, as shown in Figure 5-10.

From Figure 5-10: $V_{1+A} = 1{,}700$ vph
$V_1 = 352$ vph
$V_A = 1{,}700 - 352 = 1{,}348$ vph
$V_B = 1{,}800 - 1{,}348 = 452$ vph

V_f (After Merge) $= 4{,}800$ vph

Each of these must be converted to passenger cars per hour and peak flow rates. To accomplish this, it is necessary to assume that there are 5 percent trucks in both ramp lanes A and B. Procedures do not give specific guidance on this point, and lacking field data, a uniform distribution would be assumed. From Figure 5-6, 49 percent of the total trucks on the freeway are in lane 1 immediately in advance of the on-ramp.

Thus:

Total trucks on freeway = 3,000 × 0.05 = 150 Trucks
Trucks in lane 1 = 150 × 0.49 = 74 Trucks
Proportion of trucks in lane 1 volume = 74/352 = 0.21 or 21 Percent

Then:

Item	Volume (vph)	Proportion of Trucks	E_T[a]	f_{HV}[b]	Flow Rate (pcph) = Vol. (vph)/f_{HV} × PHF
V_1	352	0.21	1.7	0.87	426
V_{1+A}	1,700	0.08	1.7	0.95	1,884
V_A	1,348	0.05	1.7	0.97	1,463
V_B	452	0.05	1.7	0.97	491
V_f	4,800	0.05	1.7	0.97	5,209

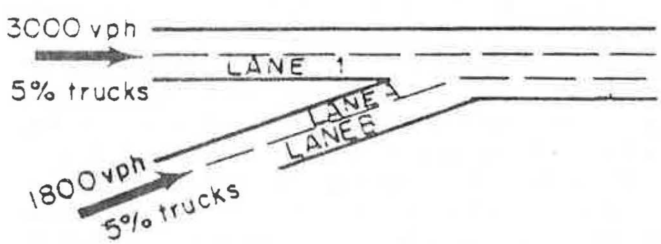
[a] Table 3-3
[b] Computed as $f_{HV} = 1/[1 + P_T(E_T - 1)]$

Checkpoint volumes may now be computed and compared with the criteria of Table 5-1:

$V_{m1} = V_1 + V_A = 426 + 1{,}463 = 1{,}889$ pcph (LOS E)
$V_{m2} = V_{1+A} + V_B = 1{,}884 + 491 = 2{,}375$ vph (LOS F)
$V_f = 5{,}209$ pcph (LOS E)

Obviously, the second merge volume of 2,375 pcph would not actually occur. However, it is clear that during peak periods of flow, great congestion will exist in the vicinity of this merge area. Level-of-service F is highly likely.

The addition of a lane, at this point, which would be carried for a significant distance might be considered. If this is not possible, the deletion of a lane from the main freeway approaching the merge might be considered, creating a major junction with the geometry shown below:

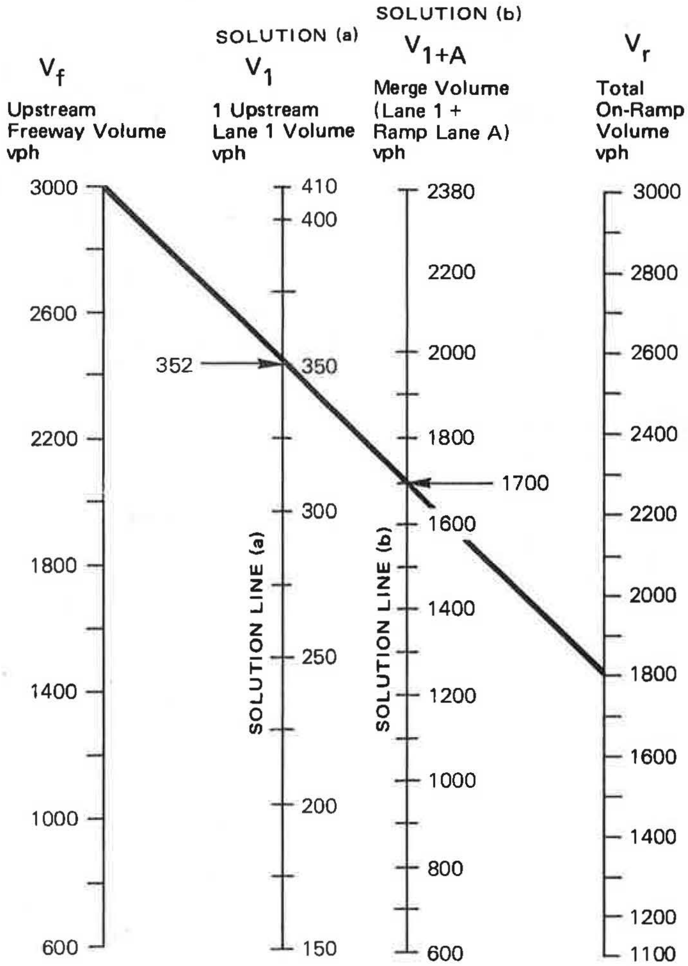


From Table 5-2, this alternative may be analyzed using a multistep trial-and-error process.

If LOS D is assumed, the lane B flow rate is assumed to be 1,750 pcph or a volume of 1,750 × 0.95 = 1,662 vph. Thus, lane A would carry only 1,800 − 1,662 = 138 vph. At LOS C, lane B would carry a flow rate of 1,450 pcph or a volume of 1,450 × 0.95 = 1,378 vph. Lane A would carry 1,800 − 1,378 = 422 vph. At LOS B, lane B carries a volume of 1,000 × 0.95 = 950 vph, and lane A would carry 1,800 − 950 = 850 vph. These values are drawn from Table 5-1. The 0.95 value is the peak hour factor used to convert flow rates to volumes. Because these values are selected for initial trials, the details of trucks presence are ignored in these assumed values, but will be included in subsequent computations.

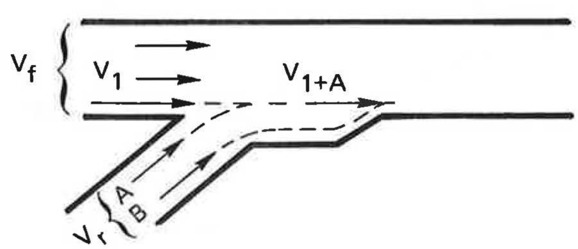
Table 5-2 indicates the use of Figure I.5-1 to compute V_1, but directs the use of only the lane A volume for V_r:

$$V_1 = 136 + 0.345\,V_f - 0.115\,V_r$$



Equation: (a) $V_1 = 54 + 0.070V_f + 0.049V_r$
(b) $V_{1+A} = -205 + 0.287V_f + 0.575V_r$

Diagram:



Conditions for Use:

1. Two-lane on-ramps on 6-lane freeways with acceleration lane of at least 800 ft in length.
2. Normal range of use: V_f = 600 to 3000 vph
   V_r = 1100 to 3000 vph

Steps in Solution:

1. Draw line from V_f value to V_r value. Read V_1 on V_1 line, V_{1+A} on V_{1+A} line.
2. Compute $V_A = V_{1+A} - 1$; $V_B = V_r - V_A$.
3. Check L. of S. for two merge points: $V_{m1} = V_1 + V_A$; $V_{m2} = V_{1+A} + V_B$.

Figure 5-10. Solution for V_{1+A} in Calculation 4 (Figure 1.5-11 in Appendix I is the base nomograph).

As the assumption of LOS B resulted in the most reasonable distribution of ramp traffic (at first glance), this case will be used to start computations. Thus:

$$V_1 = 136 + 0.345(3{,}000) - 0.115(850) = 1{,}073 \text{ vph}$$

From Figure 5-6, lane 1 will contain 80 percent of all trucks on the freeway, or:

Trucks in lane 1 = (3,000)(0.05)(0.80) = 120 Trucks

Proportion of trucks in lane 1 volume
= 120/1,073 = 0.112, say 11 Percent

The checkpoint of interest here is the merge volume consisting of the lane 1 volume plus the lane A volume. Converting these to passenger cars per hour and dividing by the PHF:

Item	Volume (vph)	Proportion of Trucks	E_T[a]	f_{HV}[b]	Flow Rate (pcph) = Vol. (vph)/f_{HV} × PHF
V_1	1,073	0.11	1.7	0.93	1,214
V_A	850	0.05	1.7	0.97	922

[a] Table 3-3
[b] $f_{HV} = 1/[1 + P_T(E_T - 1)]$

Then:

$$V_m = 1{,}214 + 922 = 2{,}136 \text{ vph (LOS F, Table 5-1)}$$

As LOS B was assumed, and LOS F resulted from computations, a second trial assuming an intermediate LOS is reasonable. Assuming LOS D, V_A would be taken as 138 vph, and:

$$V_1 = 136 + 0.345(3{,}000) - 0.115(138) = 1{,}155 \text{ vph}$$

As previously, lane 1 will contain 120 trucks, or 120/1,155 = 0.104, say 10 percent. Converting V_1 and V_A to passenger cars per hour and dividing by PHF:

Item	Volume (vph)	Proportion of Trucks	E_T[a]	f_{HV}[b]	Flow Rate (pcph) = Vol. (vph)/f_{HV} × PHF
V_1	1,155	0.10	1.7	0.93	1,307
V_A	138	0.05	1.7	0.97	150

[a] Table 3-3
[b] $f_{HV} = 1/[1 + P_T(E_T - 1)]$

Then:

$$V_m = 1{,}307 + 150 = 1{,}457 \text{ pcph (LOS D, Table 5-1)}$$

As this agrees with the assumed LOS, the proposed configuration would operate at LOS D, and is an improvement over the existing configuration which experienced LOS F.

The proposed geometry provides for a more orderly merge, and improves the overall operation significantly. The initial design forced vehicles into lane A, whereas the second makes more use of lane B. Further, by "adding" a lane, lane B vehicles do not merge. The removal of an upstream freeway lane is not critical, because the initial LOS for the approach was out of balance with the merge and downstream conditions. Two lanes are sufficient for balanced operation. A lane drop would have to be designed before approaching the vicinity of the merge in question.

Another alternative would be to merge the two ramp lanes into a single lane and, then, to add this single lane to the freeway. This would not be appropriate here because 1,800 vph is beyond the capacity of a single-lane ramp, as indicated in Table 5-5.

CALCULATION 5—RAMP ROADWAY

1. *Problem Description*—A loop ramp with a design speed of 25 mph is expected to carry 800 vph, 10 percent of which are trucks. If the PHF = 0.90 and the ramp is on a 1,400-ft, 4 percent upgrade, what design should be adopted, and what level of service can be expected?

2. *Solution*—Before proceeding with analysis, the demand volume is adjusted to reflect passenger cars per hour and a peak flow rate. Note that from Table 3-4 (Chapter 3), E_T is 5 for a 1,400-ft (¼ mile), 4 percent grade with 10 percent trucks. From Table 3-9, f_{HV} is 0.77. Thus, the adjusted demand flow rate is:

$$800/(0.71 \times 0.90) = 1{,}252 \text{ pcph}$$

From Table 5-5, a one-lane ramp would provide for level-of-service E if the design speed is 25 mph. Since the ramp is longer than 1,000 ft, paved shoulders wide enough to allow passing of stalled or slow-moving vehicles should be provided.

Provision of a better level of service requires an improvement in the design speed used. A 41- to 50-mph design speed ramp would result in LOS C operations, a more acceptable result.

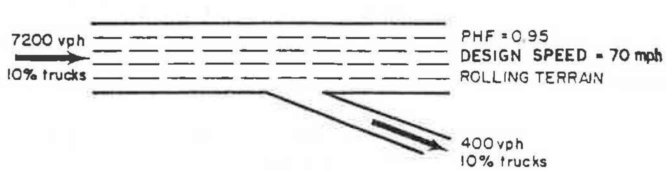
A 41- to 50-mph loop ramp, however, will create an extremely long loop, consuming a great deal of land in its wake. The designer is faced with several options:

1. Accept a lower LOS, using a loop ramp with design speed 25 mph.
2. Use a 41- to 50-mph loop ramp, and accept the inefficiency of the design.
3. Design a direct interchange not involving a loop ramp—an option involving costly structures.

A final decision would be based on extensive analysis of economic, land use, and environmental factors, as well as on capacity impacts.

CALCULATION 6—ISOLATED OFF-RAMP ON A FIVE-LANE FREEWAY SEGMENT

1. *Problem Description*—The following off-ramp occurs on a five-lane urban freeway segment. It is not within the operational influence of any adjacent ramps:



What level of service would be expected to prevail?

2. *Solution*—From Table 5-4, the segment may treated as though it were a four-lane segment (eight-lane freeway) with a volume of:

$$V_f = 7{,}200 \times 0.80 = 5{,}760 \text{ vph}$$

From Table 5-2, for an eight-lane freeway, the lane 1 volume must be approximated using Table 5-3 and Figure 5-5 (with a freeway volume of 5,760 vph). From Table 5-3, 10 percent of the through volume will remain in lane 1 at the off-ramp. From Figure 5-5, all off-ramp traffic must be in lane 1 immediately before the diverge. The "through" volume is 5,760 − 400 = 5,360 vph. Thus:

$$V_1 = (5{,}360 \times 0.10) + (1{,}00 \times 400) = 936 \text{ vph}$$

From Figure 5-6, for an eight-lane freeway with a volume of 5,760 vph, the percentage of total trucks in lane 1 is 52 percent. Then:

Total trucks on freeway = 5,760 × 0.10 = 576 vph
Total trucks in lane 1 = 576 × 0.52 = 300 vph

Proportion of trucks in lane 1 volume = 300/936 = 0.32 or 32 Percent

Then:

Item	Volume (vph)	Proportion of Trucks	E_T[a]	f_{HV}[b]	Flow Rate (pcph) = Vol. (vph)/f_{HV} × PHF
V_1	936	0.32	4	0.51	1,932
V_r	400	0.10	4	0.77	547
V_f	7,200	0.10	4	0.77	9,842

[a] Table 3-3
[b] Table 3-9

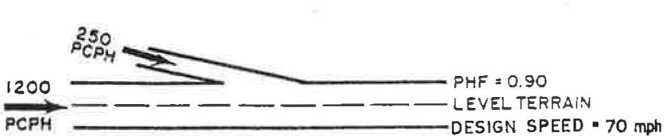
Computing the checkpoint volumes:

$V_d = V_1 = 1{,}932$ pcph (LOS E, Table 5-1)
$V_f = 9{,}842/5 = 1{,}968$ pcphpl (LOS E, Table 3-1)

The segment operates at level-of-service E. All operational elements are in balance.

CALCULATION 7—LEFT-SIDE ON-RAMP

1. *Problem Description*—Consider the left-side on-ramp shown below, which is far enough away from other ramps to be considered as isolated:



At what level of service would the section be expected to operate?

2. *Solution*—In this problem, the volume in the left-most lane must be computed immediately upstream of the on-ramp. Special procedures indicate that this volume, V_l can be approximated as 1.25 × V_1, where V_1 is computed as if the ramp were a right-side ramp.

From Table 5-2, V_1 is found using Figure I.5-1. Use of the nomograph results in:

$$V_1 = 520 \text{ vph}$$

and:

$$V_l = 520 \times 1.25 = 650 \text{ vph}$$

Note that this computation *does not* indicate that the lane 1 volume actually is 520 vph, in which case the left-lane volume would be 1,200 − 520 = 680 vph. That result assumes that a right-side ramp exists at this location. The method simply adjusts a right-side ramp computation to approximate V_l.

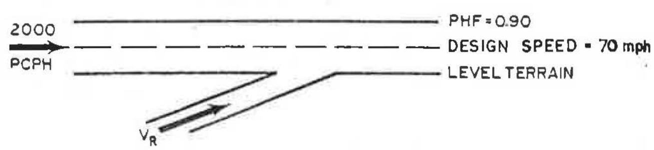
Computing checkpoint volumes and dividing by the PHF:

$V_m = (650) + 250)/0.90 = 1{,}000$ pcph (LOS B, Table 5-1)
$V_f = (1{,}200 + 250)/0.90 = 1{,}611$ pcph (LOS B, Table 5-1)

The facility will operate at level-of-service B, with all operational elements in balance.

CALCULATION 8—RAMP METERING

1. *Problem Description*—It is desired to control the on-ramp volume at an isolated ramp such that the prevailing level of service does not become worse than C. If a fixed-time meter is used, at what rate should ramp vehicles be permitted to enter the traffic stream to accomplish this?



2. *Solution*—The question asks for a solution of a maximum value of V_r such that the merge or freeway flow rates do not become more than the service flow rates for LOS C. It will be assumed that the merge checkpoint is the controlling factor to begin. As the computation of V_1 depends upon V_r, a trial-and-error process will be used.

From Table 5-1, the service flow rate for merging at level-of-service C is 1,450 pcph. For a peak hour factor of 0.90, this is equivalent to a full-hour volume of 1,450 × 0.90 = 1,305 vph. Considering the situation described in the problem, a tabular computation may be constructed as follows:

Assumed V_r	V_1 (Fig. I.5-1)	Computed V_r	Comparison
200	810	495	NG
400	775	530	NG
500	770	535	NG
550	765	540	OK

A metering rate of 550 pcph, or one vehicle every 3,600/550 = 6.55 sec, would be set.

These computations are naturally more complex where volumes contain mixed vehicles per hour, but the procedure and basic approach are as illustrated herein.

A more precise solution may be found by using the equation for Figure I.5-1 directly:

$$V_1 = 136 + 0.345\,V_f - 0.115\,V_r$$

and considering that:

$$V_r = 1{,}305 - V_1$$

Substituting for V_1:

$$V_r = 1{,}305 - (136 + 0.345\,V_f - 0.115\,V_r)$$

where $V_f = 2{,}000$ vph.

Solving for V_r:

$$V_r = (1{,}169 - 0.345)(2{,}000)/0.885 = 541\text{vph}$$

The freeway checkpoint should now be checked to ensure that it is not being violated. The total freeway volume after the merge is 2,000 + 541 = 2,541 pcph, or a flow rate of 2,541/0.90 = 2,823 pcph. Checking with Table 5-1, this is less than the service flow rate for LOS D on a four-lane freeway.

V. REFERENCES

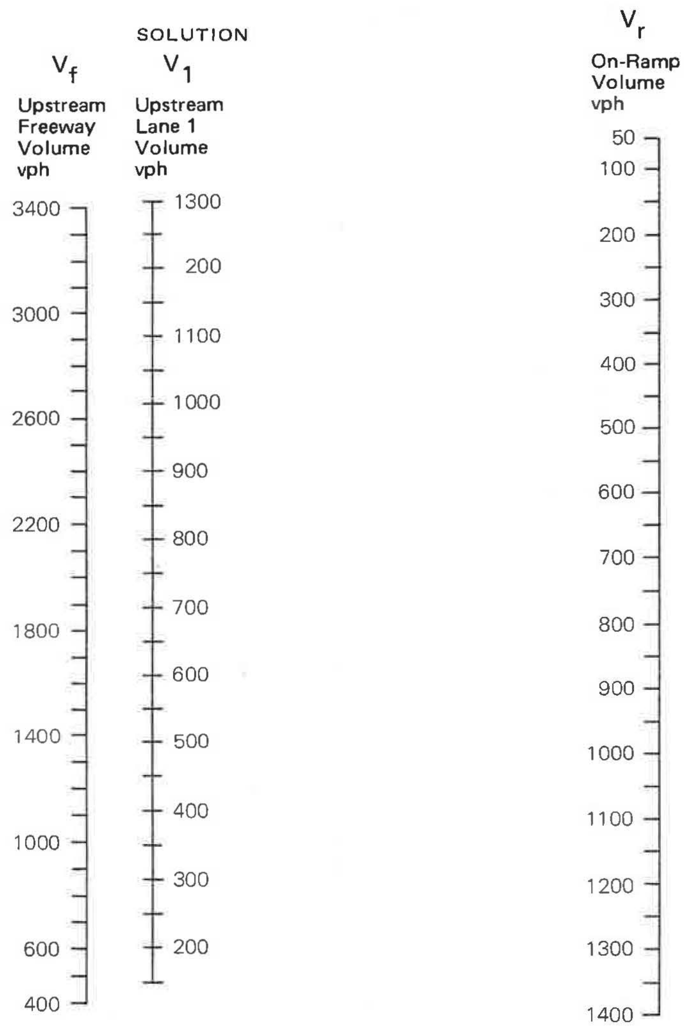
This chapter is based on the results of a study conducted by the then Bureau of Public Roads in the early 1960's. The statistical results of that study were verified and its application was modified as part of a study of weaving area operations conducted at the Polytechnic Institute of New York and sponsored by the National Cooperative Highway Research Program *(3)*. Some of these modifications appeared in a text by Pignataro *(5)*. Special applications for four-lane freeway segments and left-side ramps were adapted from a study by Leisch *(4)*, as were capacities for ramp roadways. References *6* and *7* provide a background in gap acceptance theory and its application to merge area analysis. AASHTO standards for geometric design of ramps are given in Refs. *1* and *2*.

1. *A Policy on Geometric Design of Rural Highways.* American Association of State Highway and Transportation Officials, Washington, D.C. (1965).
2. *A Policy on Design of Urban Highways and Arterial Streets.* American Association of State Highway and Transportation Officials, Washington, D.C. (1965).
3. PIGNATARO, L. J., MCSHANE, W. R., ROESS, R. P., LEE, B., and CROWLEY, K. W., "Weaving Areas—Design and Analysis." *NCHRP Report 159* (1975) 119 pp.
4. LEISCH, J., *Capacity Analysis Techniques for Design and Operation of Freeway Facilities.* Report No. FHWA-RD-74-24. Federal Highway Administration, Washington, D.C. (1974).
5. PIGNATARO, L. J., *Traffic Engineering: Theory and Practice.* Prentice-Hall Inc., Englewood Cliffs, N.J. (1974).
6. WATTLEWORTH, J., ET AL., "Operational Effects of Some Entrance Ramp Geometrics on Freeway Merging." *Texas Transportation Institute Report 430-4.* Texas A & M University, College Station, Texas (1967).
7. DREW, D., *Traffic Flow Theory and Control.* McGraw-Hill Inc., New York, N.Y. (1968).
8. EVERALL, P., *Urban Freeway Surveillance and Control: State of the Art.* Federal Highway Administration, USGPO Stock No. 50001-00058, Washington, D.C. (June 1973).

APPENDIX I
NOMOGRAPHS FOR THE SOLUTION OF LANE 1 VOLUMES

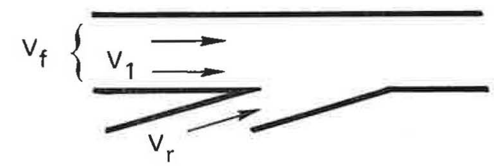
In using the nomographs of this appendix, note the following:

- *CONDITIONS FOR USE* specify the configurations for which the nomograph and accompanying equation apply. Where use is indicated for ramps both "with or without acceleration/deceleration lanes," the data base used in calibrating the relationship included both, and no statistically significant differences were observed between the two conditions. "Normal range of use" indicates the range of data used to calibrate the nomograph. Use outside this range should be limited to cases close to the range, and should be done with caution.
- *CONDITIONS FOR USE* also contain instructions for using nomographs to approximate configurations not covered elsewhere.
- *STEPS IN SOLUTION* are a step-by-step set of instructions for using each nomograph.
- *EQUATION* shows the mathematical relationship expressed by the nomograph, which may be used directly for greater precision in computations.



Equation: $V_1 = 136 + 0.345V_f - 0.115V_r$

Diagram:



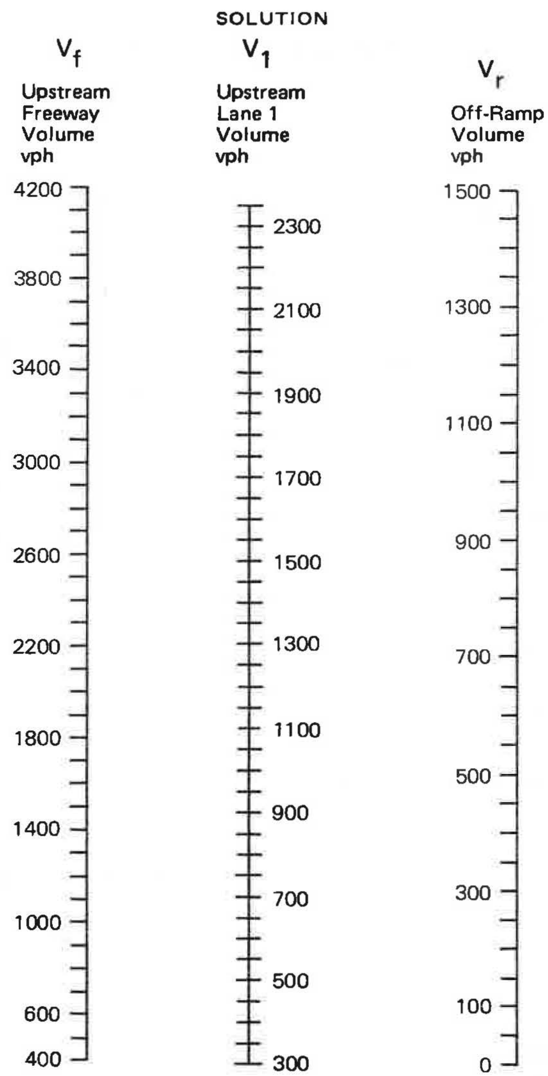
Conditions for Use:

1. Single-lane on-ramp (not a loop) on 4-lane freeway, with or without acceleration lane.
2. For use only when no adjacent upstream on-ramp exists within 2000 ft.
3. Normal range of use: V_f = 400 to 3400 vph
   V_r = 50 to 1400 vph

Steps in Solution:

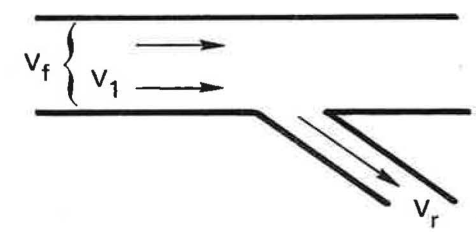
1. Draw line from V_f value to V_r value; read solution on V_1 line.

Figure I.5-1. Determination of lane 1 volume upstream of one-lane on-ramps on four-lane freeways (two lanes in each direction).



Equation: $V_1 = 165 + 0.345V_f + 0.520V_r$

Diagram:



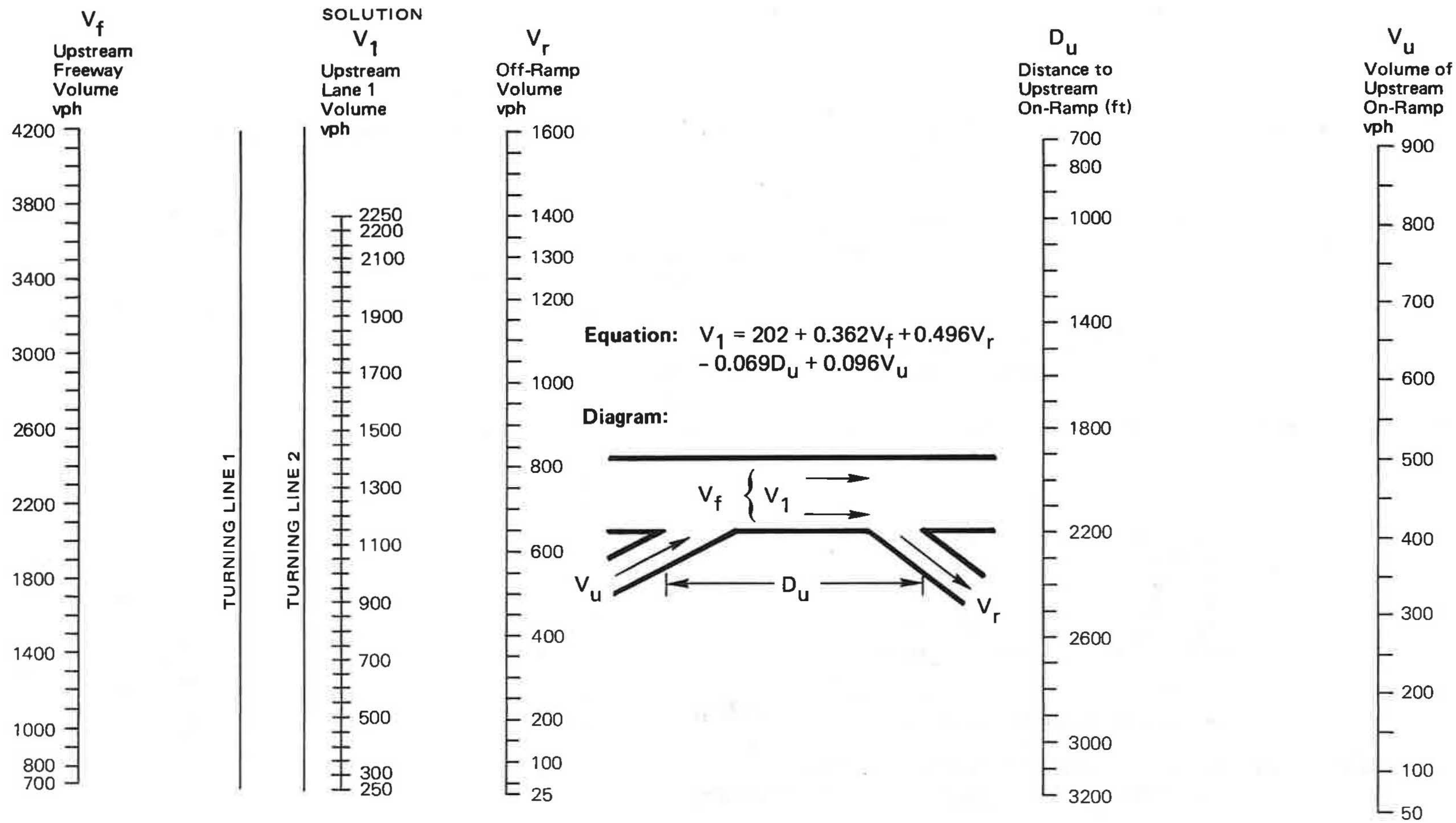
Conditions for Use:

1. Single-lane off-ramp on a 4-lane freeway, with or without a deceleration lane.
2. For use only when there is no adjacent upstream on-ramp within 3200 ft.
3. Normal range of use: V_f = 400 to 4200 vph
   V_r = 50 to 1500 vph

Steps in Solution:

1. Draw line from V_f value to V_r value; read solution on V_1 line.

Figure I.5-2. Determination of lane 1 volume upstream of one-lane off-ramps on four-lane freeways (two lanes in each direction).



Equation: $V_1 = 202 + 0.362V_f + 0.496V_r - 0.069D_u + 0.096V_u$

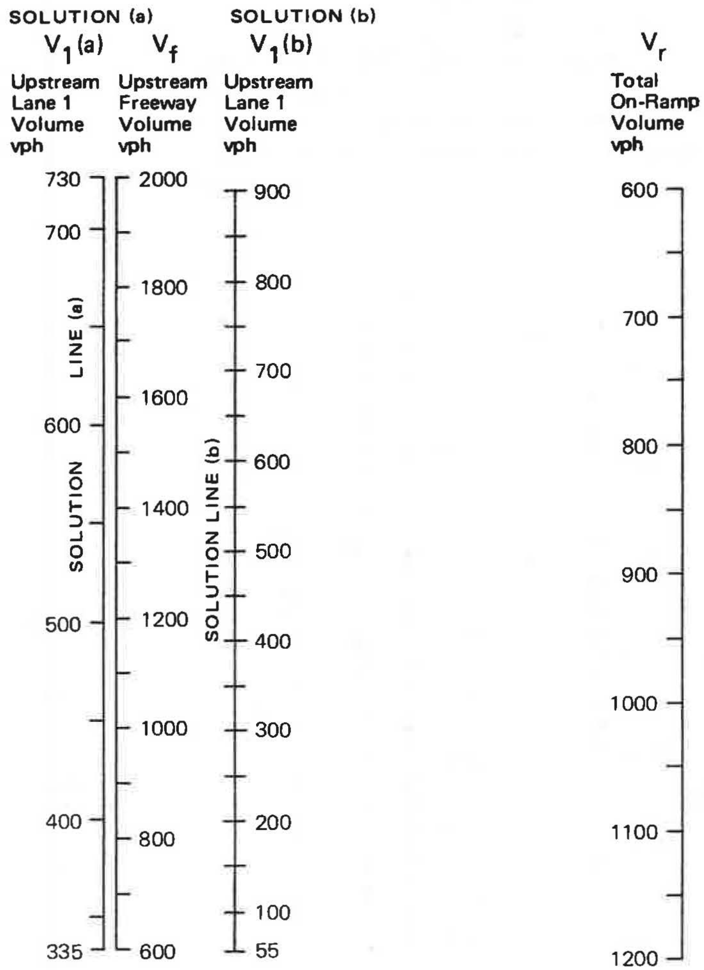
Conditions for Use:

1. Single-lane off-ramp on a 4-lane freeway, with or without a deceleration lane, with an adjacent upstream on-ramp within 3200 ft.
2. Normal range of use: V_f = 70 to 4200 vph
   V_r = 50 to 1600 vph
   V_u = 50 to 900 vph
   D_u = 700 to 3200 ft

Steps in Solution:

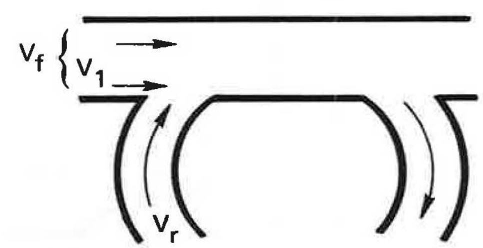
1. Draw line from V_f value to V_r value, intersecting turning line 1.
2. Draw a line from the point defined in step 1 to the D_u value, intersecting turning line 2.
3. Draw a line from the point defined in step 2 to the V_u value. Read the result from the V_1 line.

Figure I.5-3. Determination of lane 1 volume upstream of one-lane off-ramps on four-lane freeways (two lanes in each direction) with adjacent upstream on-ramps.



Equation: $aV_1 = 166 + 0.280V_f$ (for $V_r < 600$ vph)
$bV_1 = 128 + 0.482V_f - 0.301V_r$ (for V_r between 600-1200 vph)

Diagram:



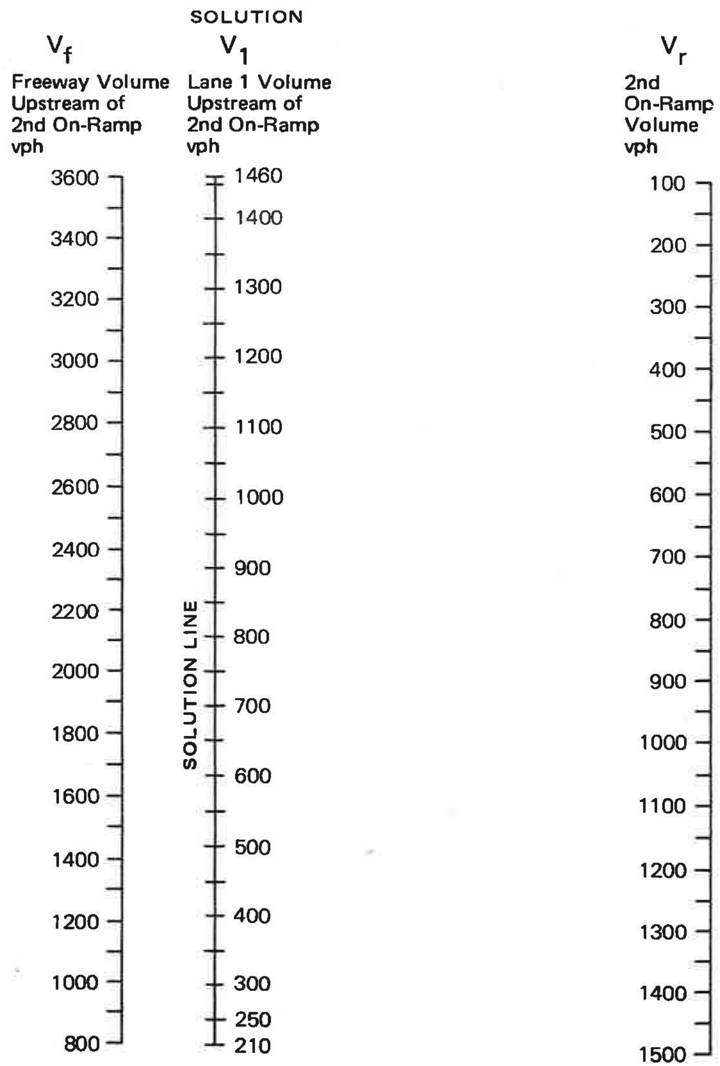
Conditions for Use:

1. Single-lane, loop-type on-ramp on a 4-lane freeway with or without an acceleration lane.
2. Normal range of use: V_f = 600 to 2000 vph
   V_r = 600 to 1200 vph (b)
   0 to 600 vph (a)

Steps in Solution:

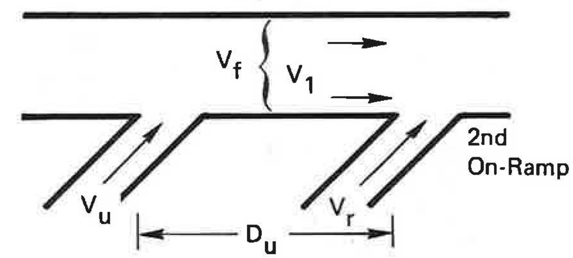
1. If $V_r < 600$ vph, read solution on solution line (a), horizontally to the left of V_f value.
2. If V_r is between 600 and 1200 vph, draw a line from V_f value to V_r value; read result on solution line (b).

Figure I.5-4. Determination of lane 1 volume upstream of one-lane, loop-type on-ramps on four-lane freeways (two lanes in each direction).



Equation: $V_1 = 123 + 0.376V_f - 0.142V_r$

Diagram:



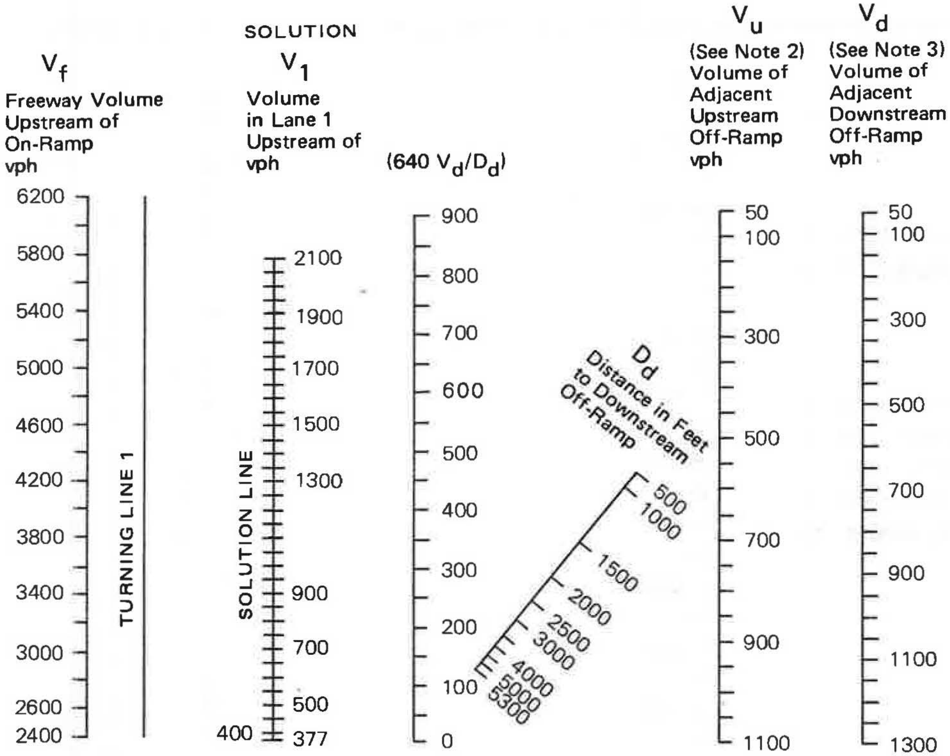
Conditions for Use:

1. Single-lane on-ramp on 4-lane freeway with adjacent upstream on-ramp within 400 to 2000 ft (with or without acceleration lane).
2. Not accurate where $D_u \leqslant 400$ or $V_u \geqslant 1000$ vph.
3. Normal range of use: V_f = 800 to 3600 vph
   V_r = 100 to 1500 vph
   V_u = 100 to 1000 vph
   D_u = 400 to 2000 ft

Steps in Solution:

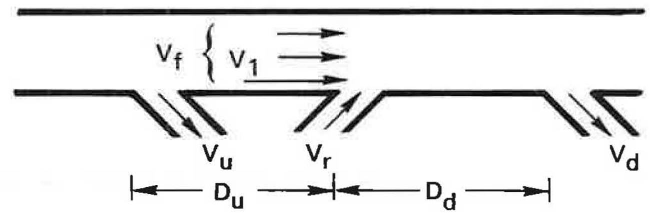
1. Draw a line from V_f value to V_r value; read solution on V_1 line.

Figure I.5-5. Determination of lane 1 volume upstream of one-lane on-ramps on four-lane freeways (two lanes in each direction) with adjacent upstream on-ramps.



Equation: $V_1 = -121 + 0.244V_f - 0.085V_u + 640\ V_d/D_d$

Diagram:



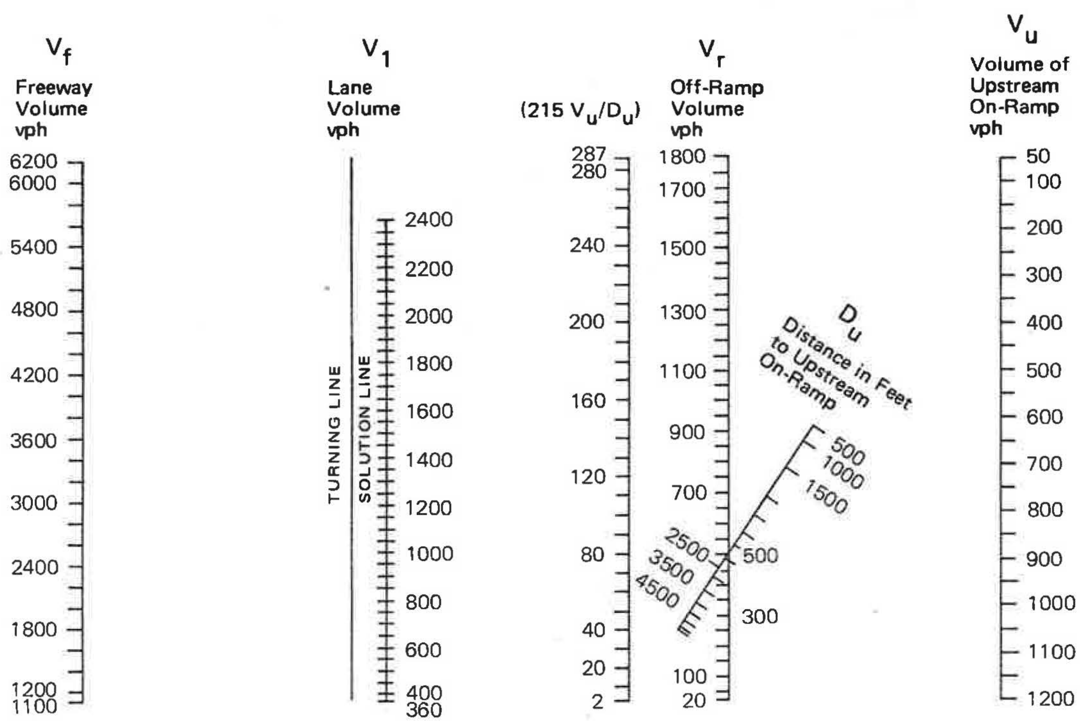
Conditions for Use:

1. Single-lane on-ramps on 6-lane freeways with or without upstream and/or downstream off-ramps, with or without acceleration lane.
2. If there is no upstream off-ramp within 2600 ft, use $V_u = 50$.
3. If there is no downstream off-ramp within 5700 ft, and $V_f \leqslant 5000$ vph, use 640 $V_d/D_d = 5$, and skip step 2 below.
4. Normal range of use: $V_f = 2400$ to 6200 vph
   $V_u = 50$ to 1100 vph
   $V_d = 50$ to 1300 vph
   $V_r = 100$ to 1700 vph
   $D_d = 900$ to 5700 ft
   $D_u = 900$ to 2600 ft

Steps in Solution:

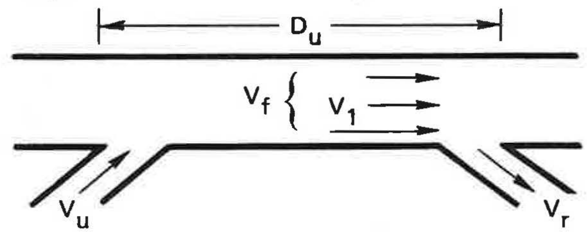
1. Draw a line from V_f value to V_u value, intersecting turning line 1.
2. Draw a line from V_d value to D_d value, intersecting 640 V_d/D_d line.
3. Draw a line from the step 1 intersection with turning line 1 to the 640 V_d/D_d value of step 2; read solution at intersection with V_1 line.

Figure I.5-6. Determination of lane 1 volume upstream of one-lane on-ramps on six-lane freeways (three lanes in each direction) with or without adjacent off-ramps.



Equation: $V_1 = 94 + 0.231V_f + 0.473V_r + 215\ V_u/D_u$

Diagram:



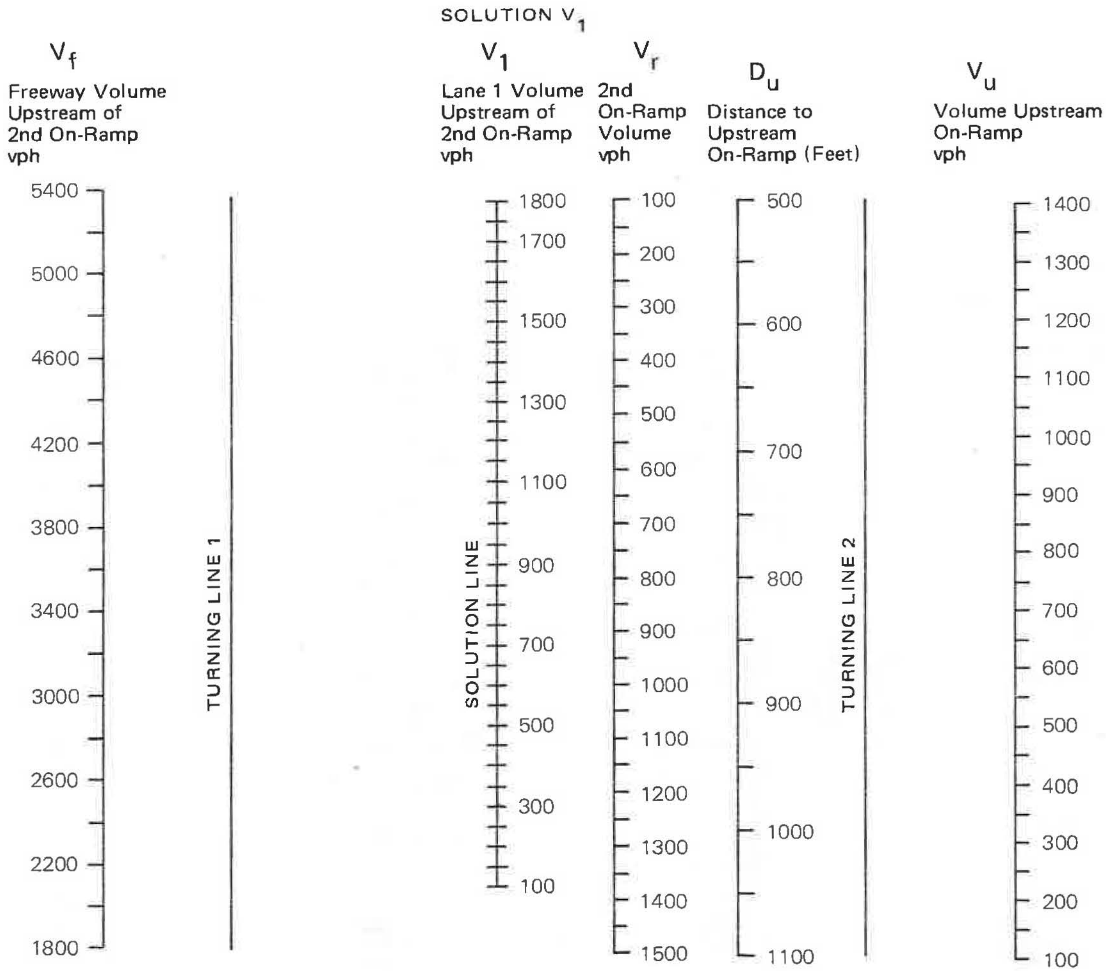
Conditions for Use:

1. Single-lane off-ramp on 6-lane freeway with or without upstream on-ramp with or without deceleration lane.
2. If there is no upstream on-ramp within 5700 ft, skip step 2 below, and set 215 V_u/D_u = 2.
3. Normal range of use: V_f = 1100 to 6200 vph
   V_r = 20 to 1800 vph
   V_u = 50 to 1200 vph
   D_u = 900 to 5700 ft

Steps in Solution:

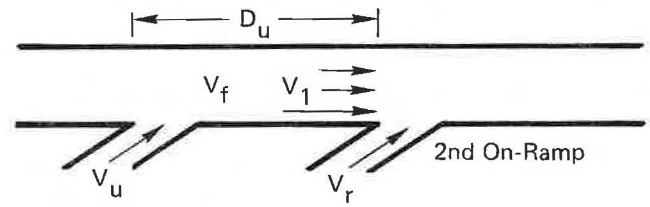
1. Draw a line from V_f value to V_r value, intersecting the turning line.
2. Draw a line from V_u value to D_u value, intersecting the 215 V_u/D_u line.
3. Draw a line from intersection point on the turning line of step 1 to the value on the 215 V_r/D_u line of step 2; read solution on V_1 line.

Figure I.5-7. Determination of lane 1 volume upstream of one-lane off-ramps on six-lane freeways (three lanes in each direction).



Equation: $V_1 = 574 + 0.228V_f - 0.194V_r - 0.714D_u + 0.274\,V_u$

Diagram:



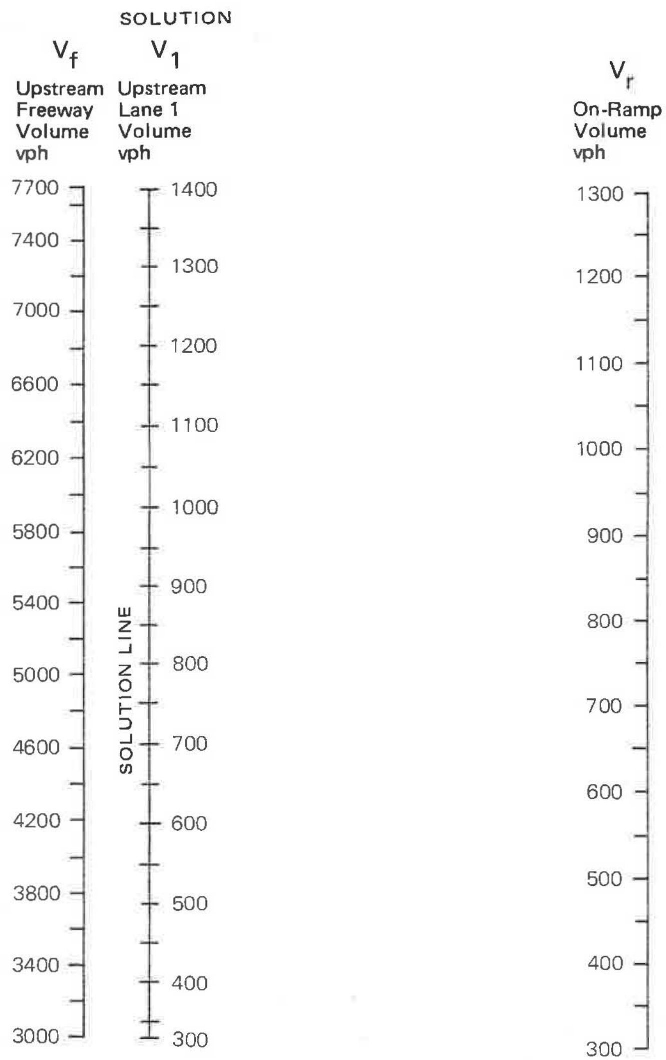
Conditions for Use:

1. Single-lane on-ramp on 6-lane freeways with adjacent upstream on-ramps, with or without acceleration lanes.
2. Normal range of use: V_f = 1800 to 5400 vph
   V_r = 100 to 1500 vph
   V_u = 100 to 1400 vph
   D_u = 500 to 1000 ft

Steps in Solution:

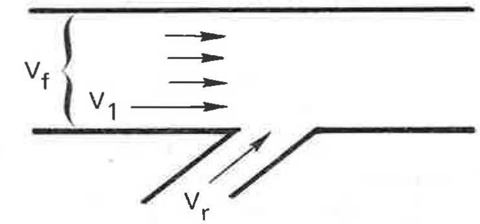
1. Draw a line from V_f value to V_r value, intersecting turning line 1.
2. Draw a line from V_u value to D_u value, intersecting turning line 2.
3. Draw a line from intersection on turning line 1 of step 1 to the intersection on turning line 2 of step 2; read solution on V_1 line.

Figure I.5-8. Determination of lane 1 volume upstream of one-lane on-ramps on six-lane freeways (three lanes in each direction) with upstream on-ramps.



Equation: $V_1 = -312 + 0.201V_f + 0.127V_r$

Diagram:



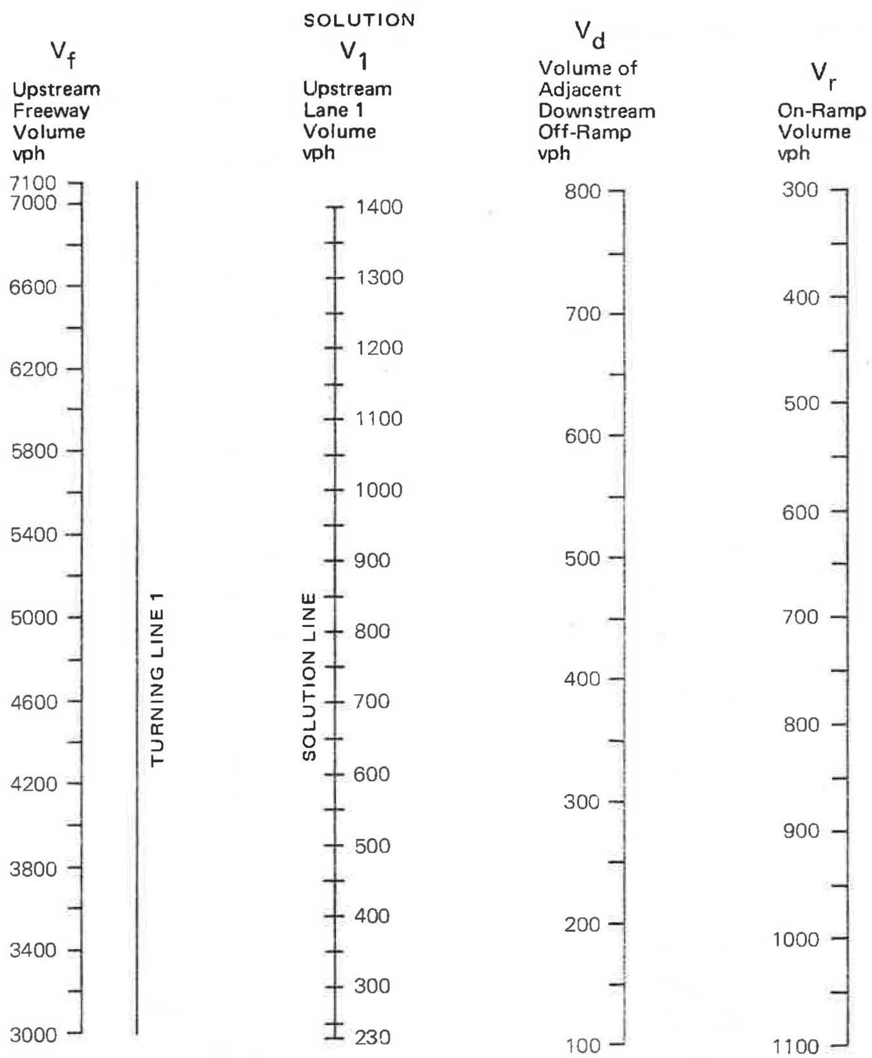
Conditions for Use:

1. Single-lane on-ramp on 8-lane freeway with or without acceleration lane.
2. Not for use if there is adjacent downstream off-ramp within 3000 ft.
3. Normal range of use: V_f = 3000 to 7700 vph
   V_r = 300 to 1300 vph

Steps in Solution:

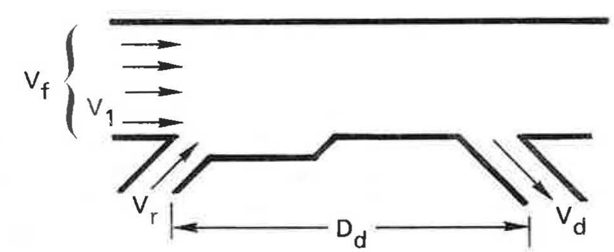
1. Draw a line from V_f value to V_r value; read solution on V_1 line.

Figure I.5-9. Determination of lane 1 volume upstream of one-lane on-ramps on eight-lane freeways (four lanes in each direction).



Equation: $-353 + 0.199V_f - 0.057V_r + 0.486V_d$

Diagram:



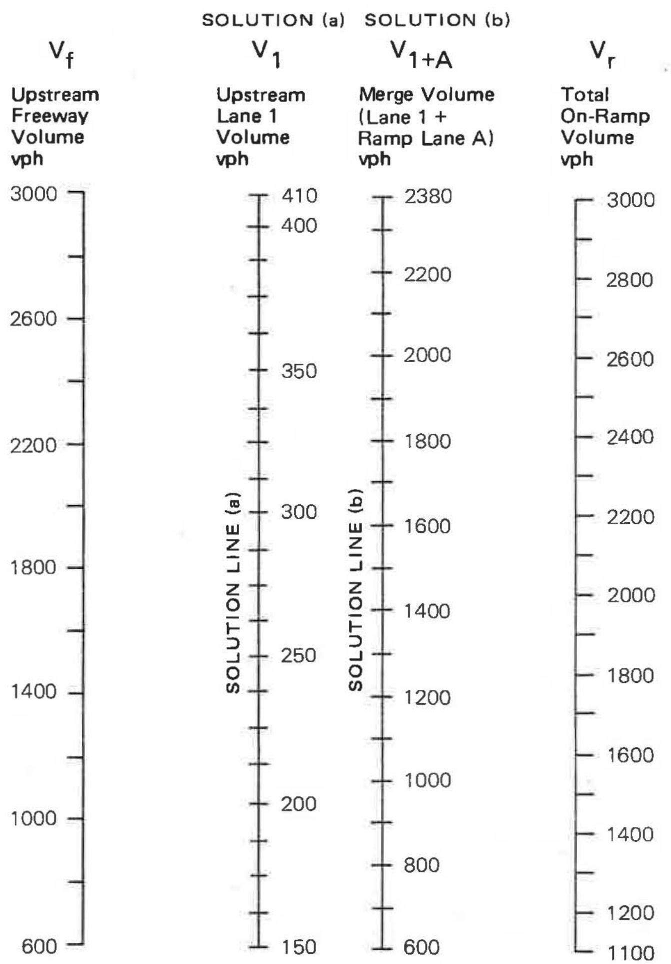
Conditions for Use:

1. Single-lane on-ramp on 8-lane freeway *with* acceleration lane, with adjacent downstream off-ramp within 1500 to 3000 ft.
2. Normal limits of use: V_f = 3000 to 7100 vph
   V_r = 300 to 1100 vph
   V_d = 100 to 800 vph
   D_d = 1500 to 3000 ft

Steps in Solution:

1. Draw a line from V_f value to V_r value; intersecting turning line.
2. Draw line from intersection of step 1 with turning line 1 to V_d value; read result on solution line.

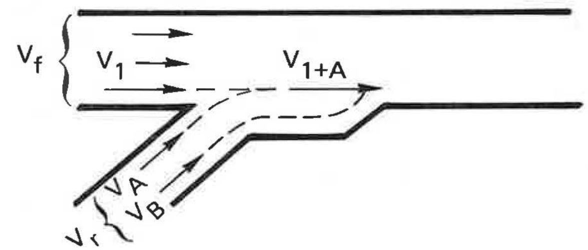
Figure I.5-10. Determination of lane 1 volume upstream of on-ramps on eight-lane freeways (four lanes in each direction) with adjacent downstream off-ramps.



Equation: (a) $V_1 = 54 + 0.070V_f + 0.049V_r$

(b) $V_{1+A} = -205 + 0.287V_f + 0.575V_r$

Diagram:



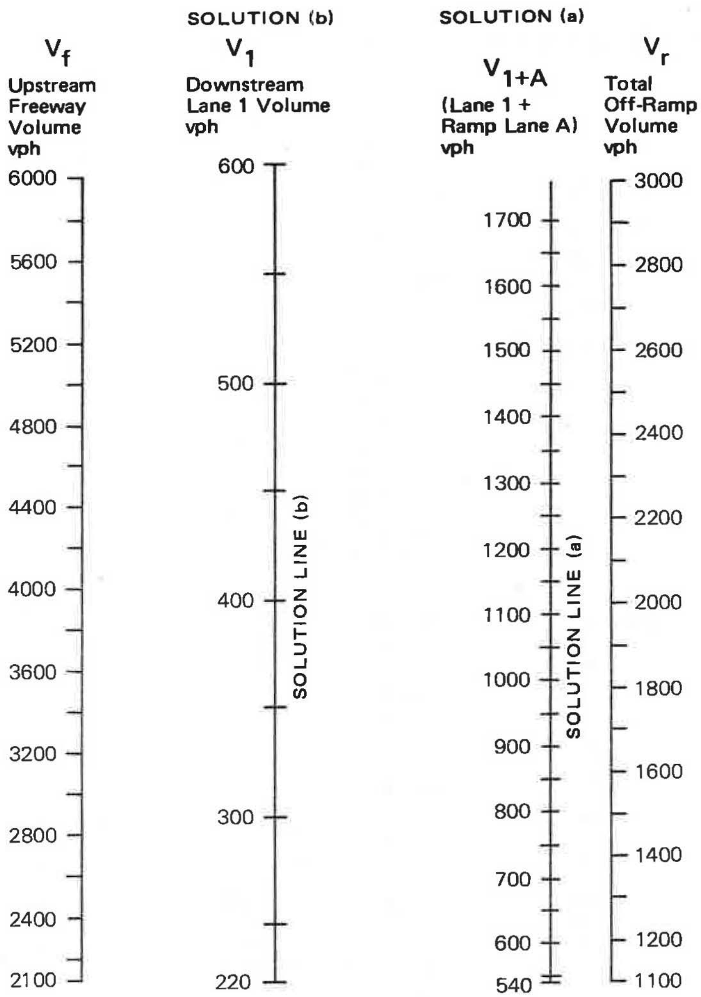
Conditions for Use:

1. Two-lane on-ramps on 6-lane freeways with acceleration lane of at least 800 ft in length.
2. Normal range of use: V_f = 600 to 3000 vph
   V_r = 1100 to 3000 vph

Steps in Solution:

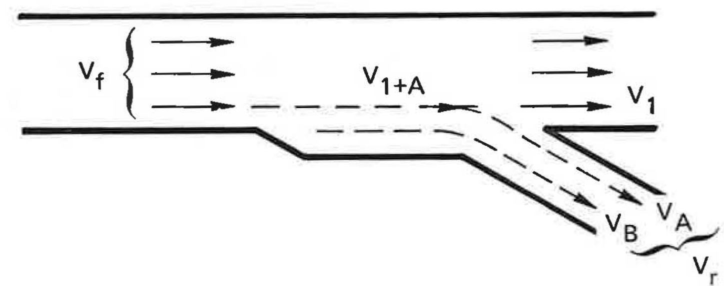
1. Draw line from V_f value to V_r value. Read V_1 on V_1 line, V_{1+A} on V_{1+A} line.
2. Compute $V_A = V_{1+A} - V_1$; $V_B = V_r - V_A$.
3. Check Level of Service for two merge points: $V_{m1} = V_1 + V_A$ and $V_{m2} = V_{1+A} + V_B$.

Figure I.5-11. Determination of lane 1 volume upstream of two-lane on-ramps on six-lane freeways (three lanes in each direction).



Equation: (a) $V_{1+A} = -158 + 0.035V_f + 0.567V_r$
(b) $V_1 = 18 + 0.060V_f + 0.072V_r$

Diagram:



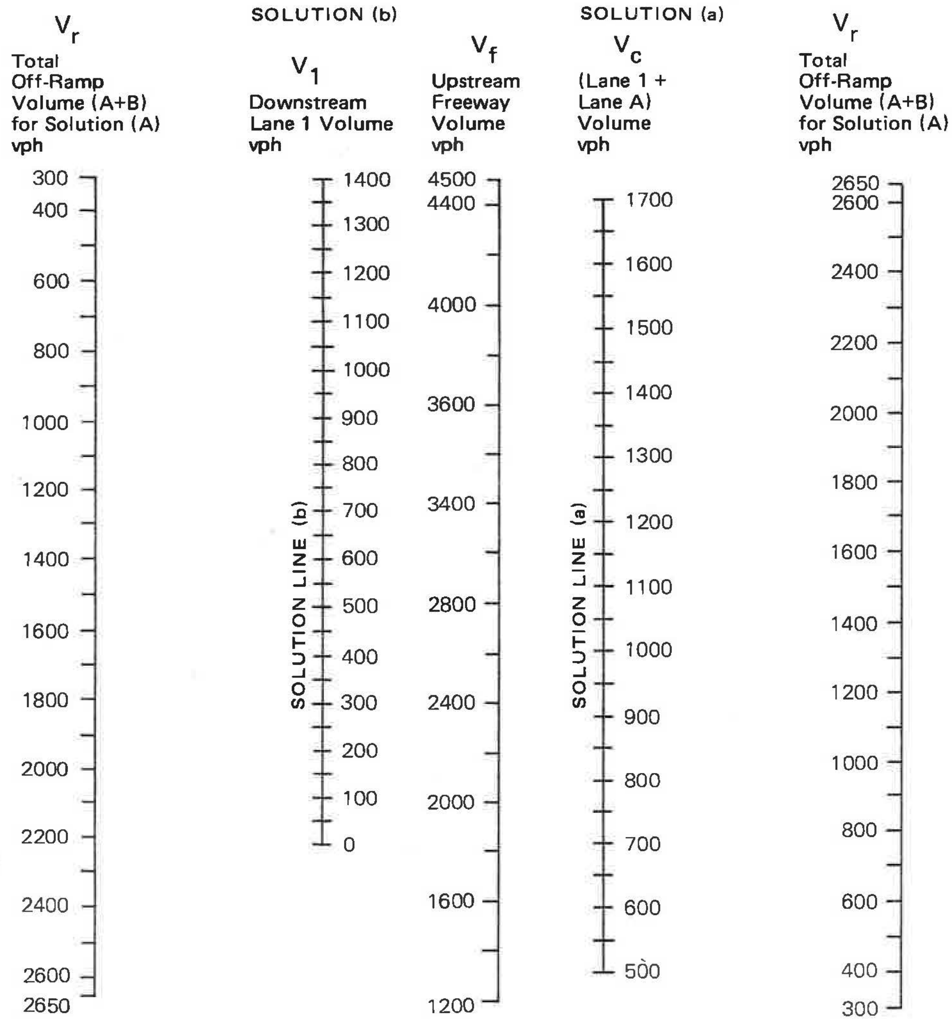
Conditions for Use:

1. Two-lane off-ramps on 6-lane freeways with deceleration lanes of at least 700 ft in length.
2. Normal range of use: V_f = 2100 to 6000 vph
   V_r = 1100 to 6000 vph

Steps in Solution:

1. Draw line from V_f value to V_r value; read V_1 solution on V_1 line, V_{1+A} solution on V_{1+A} line.
2. Compute: $V_A = V_{1+A} - V_1$; $V_B = V_r - V_A$.
3. Check level of service for two diverge volumes: $V_{d1} = V_{1+A}$ and $V_{d2} = V_B$.

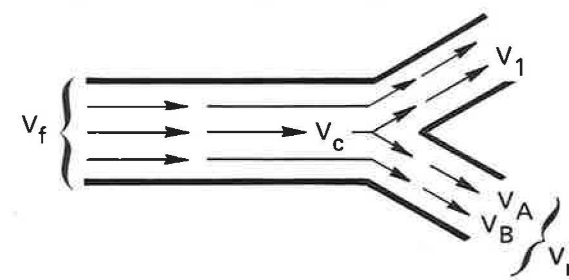
Figure I.5-12. Determination of lane 1 volume upstream of two-lane off-ramps on six-lane freeways (three lanes in each direction).



Equation: (a) $V_c = 64 + 0.285V_f + 0.141V_r$

(b) $V_1 = 173 + 0.295V_f - 0.320V_r$

Diagram:



Conditions for Use:

1. Major diverge junctions on a 6-lane freeway, with three lanes dividing to two 2-lane roadways.
2. Normal range of use: V_f = 1200 to 4500 vph
   V_r = 300 to 2650 vph

Steps in Solution:

1. Draw line from V_f value to V_r value on the far right-hand scale, read V_c on solution (a) line.
2. Draw a line from V_f value to V_r value on the far left-hand scale; read V_1 on solution (b) line.
3. Compute $V_A = V_c - V_1$ and $V_B = V_r - V_A$.
4. Check level of service for two diverge volumes: $V_{d1} = V_c$; $V_{d2} = V_B$.

Figure I.5-13. Determination of critical lane volumes at a major fork on a six-lane freeway (three lanes in each direction) which divides into two four-lane freeways (two lanes in each direction).

APPENDIX II

TABLES AND FIGURES FOR USE IN THE ANALYSIS OF RAMPS AND RAMP JUNCTIONS

TABLES *PAGE*

FIGURES

TABLE 5-1. LEVEL-OF-SERVICE CRITERIA FOR CHECKPOINT FLOW RATES AT RAMP-FREEWAY TERMINALS

LEVEL OF SERVICE	MERGE FLOW RATE (PCPH)[a] v_m	DIVERGE FLOW RATE (PCPH)[b] v_d	FREEWAY FLOW RATES (PCPH)[c], v_f								
			70-MPH DESIGN SPEED			60-MPH DESIGN SPEED			50-MPH DESIGN SPEED		
			4-LANE	6-LANE	8-LANE	4-LANE	6-LANE	8-LANE	4-LANE	6-LANE	8-LANE
A	≤ 600	≤ 650	≤ 1,400	≤ 2,100	≤ 2,800	d	d	d	d	d	d
B	≤ 1,000	≤ 1,050	≤ 2,200	≤ 3,300	≤ 4,400	≤ 2,000	≤ 3,000	≤ 4,000	d	d	d
C	≤ 1,450	≤ 1,500	≤ 3,100	≤ 4,650	≤ 6,200	≤ 2,800	≤ 4,200	≤ 5,600	≤ 2,600	≤ 3,900	≤ 5,200
D	≤ 1,750	≤ 1,800	≤ 3,700	≤ 5,550	≤ 7,400	≤ 3,400	≤ 5,100	≤ 6,800	≤ 3,200	≤ 4,800	≤ 6,400
E	≤ 2,000	≤ 2,000	≤ 4,000	≤ 6,000	≤ 8,000	≤ 4,000	≤ 6,000	≤ 8,000	≤ 3,800	≤ 5,700	≤ 7,600
F	WIDELY VARIABLE										

[a] Lane-1 flow rate plus ramp flow rate for one-lane, right-side on-ramps.
[b] Lane-1 flow rate immediately upstream of off-ramp for one-lane, right-side ramps.
[c] Total freeway flow rate in one direction upstream of off-ramp and/or downstream of on-ramp.
[d] Level of service not attainable due to design speed restrictions.

TABLE 5-2. INDEX TO THE USE OF NOMOGRAPHS AND APPROXIMATION PROCEDURE FOR THE COMPUTATION OF LANE 1 VOLUME

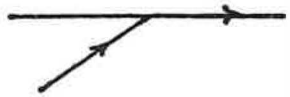
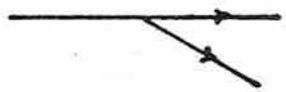
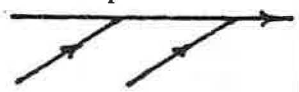
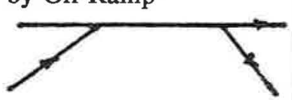
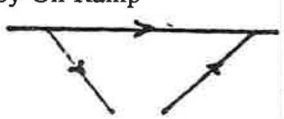
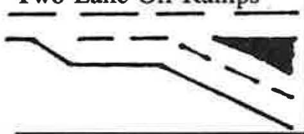
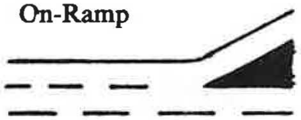
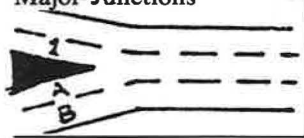
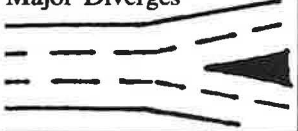
CONFIGURATION	4-LANE FREEWAY (2 LANES EACH DIRECTION)		6-LANE FREEWAY (3 LANES EACH DIRECTION)		8-LANE FREEWAY (4 LANES EACH DIRECTION)	
	1st RAMP	2nd RAMP	1st RAMP	2nd RAMP	1st RAMP	2nd RAMP
Isolated, One Lane 	Fig. I.5-1	—	Fig. I.5-6	—	Fig. I.5-9	—
Isolated, One Lane 	Fig. I.5-2	—	Fig. I.5-7	—	Approximate Using Table 5-3 and Fig. 5-5	—
Adjacent One-Lane On-Ramps 	Fig. I.5-1	Fig. I.5-5	Fig. I.5-6	Fig. I.5-8	Approximate Using Table 5-3 and Fig. 5-5	Approximate Using Table 5-3 and Fig. 5-5
Adjacent One-Lane Off-Ramps 	See Note 1	Fig. I.5-2	See Note 2	Fig. I.5-7	Approximate Using Table 5-3 and Fig. 5-5	Approximate Using Table 5-3 and Fig. 5-5
On-Ramp Followed by Off-Ramp 	Fig. I.5-1	Fig. I.5-3	Fig. I.5-6	Fig. I.5-7	Fig. I-5-10	Approximate Using Table 5-3 and Fig. 5-5
Off-Ramp Followed by On-Ramp 	Treat as Isolated Ramps			Fig. I.5-6	Treat as Isolated Ramps	

TABLE 5-2 (CONTINUED).

Loop Ramps 	Fig. I.5-4	Fig. I.5-3	Fig. I.5-6	Fig. I.5-7	Fig. I.5-10	Approximate Using Table 5-3 and Fig. 5-5
Two-Lane On-Ramps 	See Note 3	—	Fig. I.5-11	—	See Note 3	—
Two-Lane Off-Ramps 	See Note 4	—	Fig. I.5-12	—	See Note 4	—
Addition of Lane at On-Ramp 	Merge criteria in Table 5-1 may be applied directly to the on-ramp flow rate as a checkpoint.					
Dropping a Lane to the Off- at Off-Ramp 	Diverge criteria in Table 5-1 may be applied directly to the off-ramp flow rate as a checkpoint.					
Major Junctions 	Assume that lane B carries an amount of traffic equal to the merge checkpoint volume in Table 5-1 for the assumed level of service. Ramp lane A then carries the remaining ramp traffic. Compute lane 1 volume using Figure I.5-1 (4-lane freeway), Figure I.5-5 (6-lane freeway), or Figure I.5-9 (8-lane freeway), entering with ramp volume = lane A volume. Find checkpoint levels of service. Continue computations until assumed LOS agrees with results.					
Major Diverges 	Not Available	—	Fig. I.5-13	—	Not Available	—

NOTES:

1. Use Figure I.5-2 to find V_1 in advance of the first ramp, but enter with a V_r which is equal to the total volume on *both* ramps. This technique is valid where the distance between ramps is less than 800 ft. Where the distance betwen ramps is between 800 and 4,000 ft, use Table 5-3 and Figure 5-5 to approximate the situation. If the distance between ramps is greater than 4,000 ft, consider ramps to be isolated and consider separately.
2. Use Figure I.5-7 to find V_1 in advance of the first ramp, but enter with a V_r which is equal to the total volume on *both* off-ramps. This technique is valid where the distance between ramps is less than 800 ft. For other distance, see note 1.
3. Treat as two successive on-ramps separated by 400 ft; divide ramp volume equally between two ramp lanes.
4. Treat as two successive off-ramps separated by 400 ft; divide off-ramp volume equally between two ramp lanes.

TABLE 5-3. APPROXIMATE PERCENTAGE OF THROUGH TRAFFIC[a] REMAINING IN LANE 1 IN THE VICINITY OF RAMP TERMINALS

TOTAL THROUGH VOLUME, ONE DIRECTION (VPH)	THROUGH VOLUME REMAINING IN LANE 1 (%)		
	8-LANE FREEWAY	6-LANE FREEWAY	4-LANE FREEWAY
≥ 6500	10	—	—
6000 – 6499	10	—	—
5500 – 5999	10	—	—
5000 – 5499	9	—	—
4500 – 4999	9	18	—
4000 – 4499	8	14	—
3500 – 3999	8	10	—
3000 – 3499	8	6	40
2500 – 2999	8	6	35
2000 – 2499	8	6	30
1500 – 1999	8	6	25
≤ 1499	8	6	20

[a] Through traffic not involved in *any* ramp within 4,000 ft of the subject location.

TABLE 5-4. CONVERSION FACTORS FOR CONSIDERATION OF RAMPS ON FIVE-LANE SEGMENTS

RAMP TYPE	5-LANE FREEWAY VOLUME (VPH)	CONVERSION FACTOR
On-Ramp	All Volumes	0.78
Off-Ramp	≤ 4,000	1.00
	4,001 – 5,500	0.90
	5,501 – 7,000	0.85
	≥ 7,001	0.80

TABLE 5-5. APPROXIMATE SERVICE FLOW RATES FOR SINGLE-LANE RAMPS[a] (pcph)

LOS	RAMP DESIGN SPEED (MPH)				
	≤ 20	21-30	31-40	41-50	≥ 51
A	[b]	[b]	[b]	[b]	600
B	[b]	[b]	[b]	900	900
C	[b]	[b]	1,100	1,250	1,300
D	[b]	1,200	1,350	1,550	1,600
E	1,250	1,450	1,600	1,650	1,700
F	WIDELY VARIABLE				

[a] For two-lane ramps, multiply the values in the table by: 1.7 for ≤ 20 mph
1.8 for 21–30 mph
1.9 for 31–40 mph
2.0 for ≥ 41 mph

[b] Level of service not attainable due to restricted design speed.

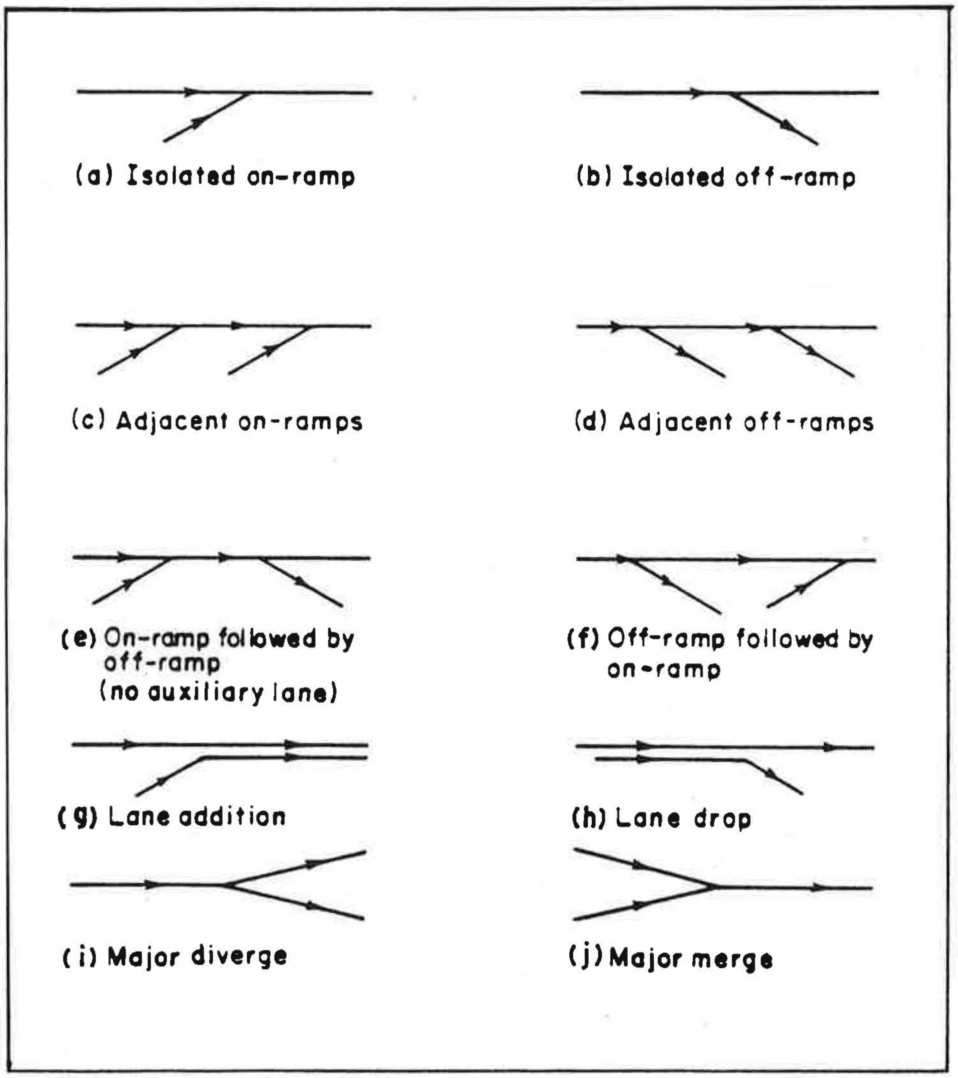


Figure 5-1. Ramp configurations covered by procedures.

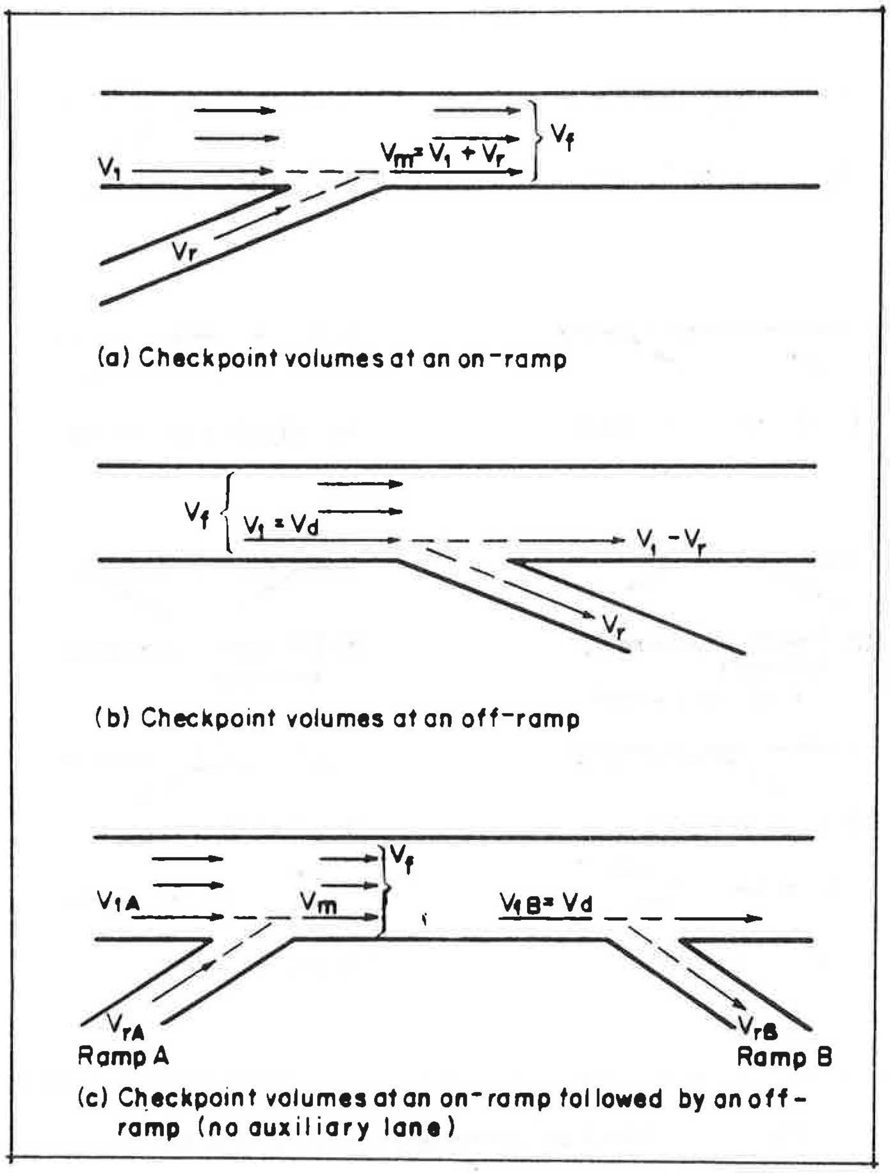
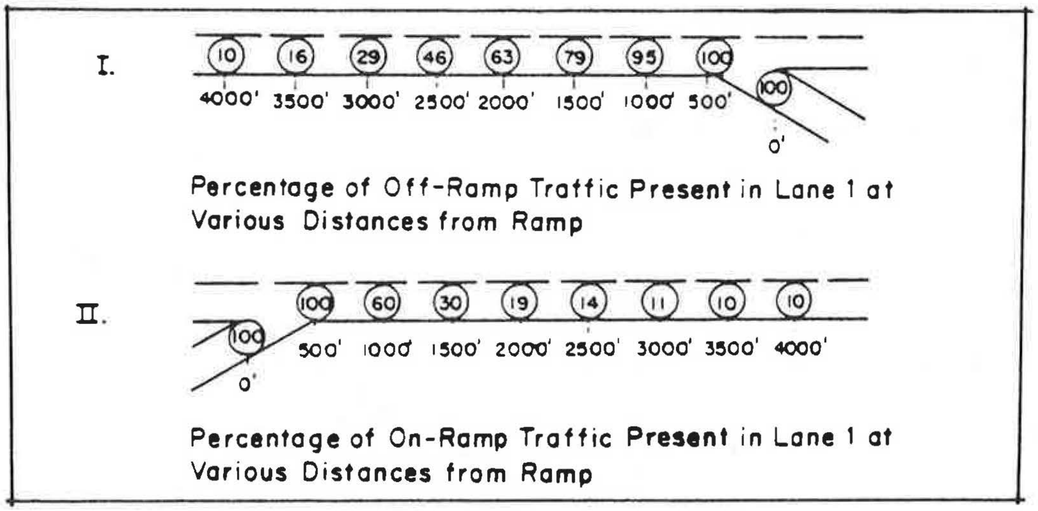


Figure 5-2. Checkpoint volumes for ramp-freeway terminals.



NOTE: If the percentage found in this figure is less than the percent of through volume in lane 1 from Table 5-3, use the percentage given for through volume in Table 5-3.

Figure 5-5. Percentage of ramp vehicles in lane 1.

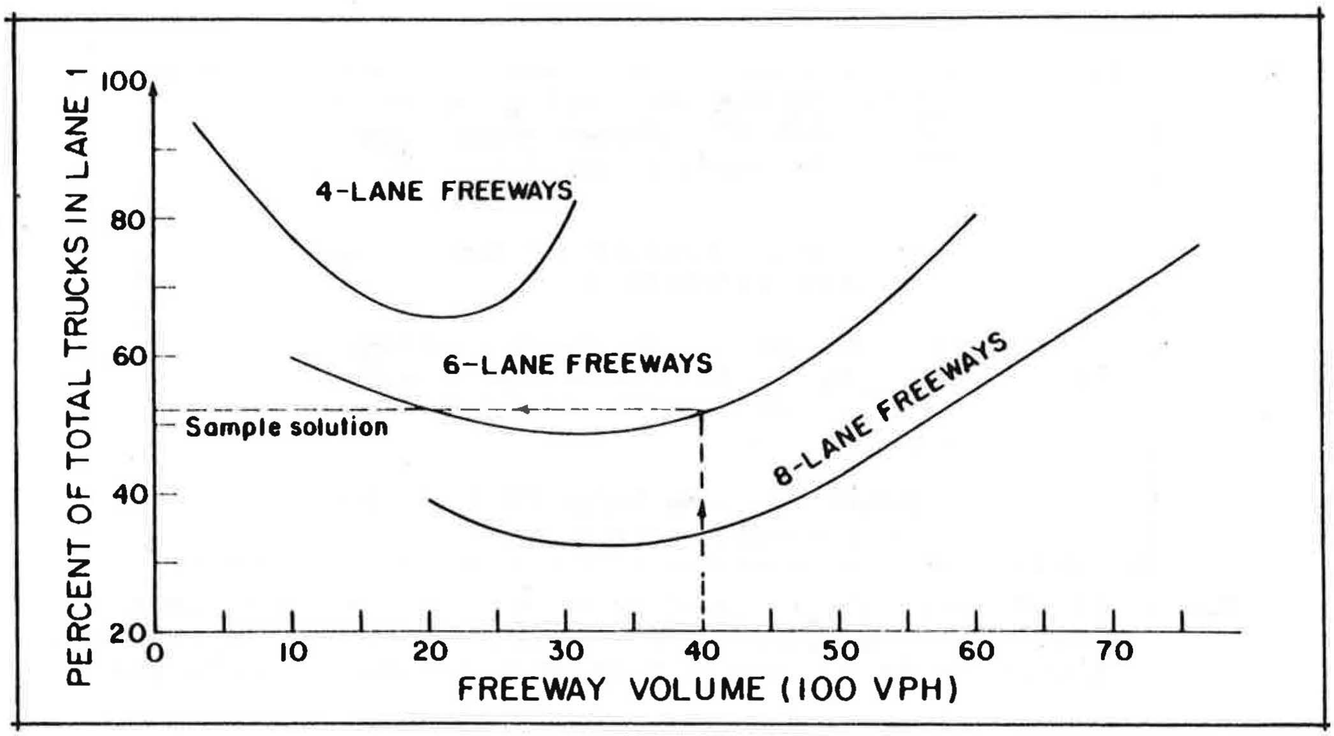
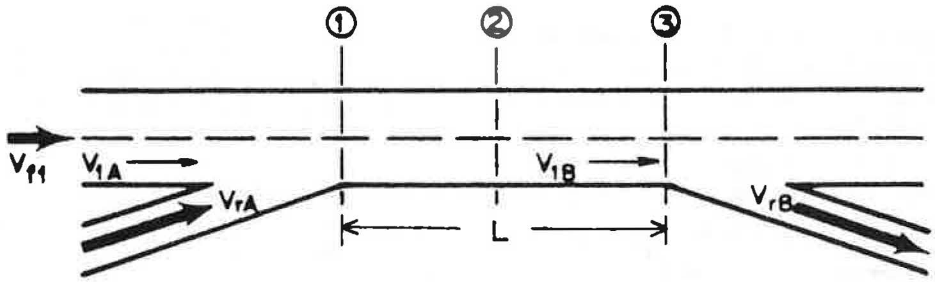


Figure 5-6. Truck presence in lane 1.



CHECKPOINTS

(1) Merge (immediately after on-ramp) at Point ①:

$$V_m = V_{1A} + V_{rA}$$

(2) Diverge (immediately before off-ramp) at Point ③:

$$V_d = V_{1B}$$

(3) Freeway Checkpoint Volume (upstream of off-ramp, downstream of on-ramp, between the ramps) at Point ②:

$$V_f = V_{f1} + V_{rA}$$

Figure 5-7. Computation of checkpoint volumes for an on-ramp followed by an off-ramp.

APPENDIX III

GLOSSARY AND SYMBOLS

GLOSSARY

direct ramp—A ramp roadway on which vehicles turn only in the direction of their intended directional change, i.e., a ramp providing a left-turn connection *would not* require vehicles to turn to the right, or vice-versa.

diverge—A movement in which a single lane of traffic separates into two separate lanes without the aid of traffic signals.

downstream—The direction to which traffic is flowing.

lane 1—The freeway lane adjacent to the shoulder.

loop ramp—A ramp serving a left-turn movement which requires vehicles to execute that movement by turning right; typically, a 90 deg left turn is made by turning 270 deg to the right.

merge—A movement in which two separate lanes of traffic combine to form a single lane without the aid of traffic signals or other right-of-way controls.

ramp—A short segment of roadway serving as a connection between two traffic facilities; usually services flow in one direction only.

ramp control—A system in which the entry of vehicles onto a freeway from a ramp is metered by a traffic signal; the signal allows one vehicle to enter on each green indication, or "green flash".

ramp-freeway junction—The roadway area over which an on- or off-ramp joins with the main line of a freeway.

ramp-street junction—The roadway area over which an on- or off-ramp joins with a surface street or arterial.

upstream—The direction from which traffic is flowing.

SYMBOLS

D_d distance to downstream adjacent ramp, in feet.

D_u distance to upstream adjacent ramp, in feet.

V_d diverge volume, in vehicles per hour.

V_f total freeway volume in the vicinity of the ramp, in vehicles per hour.

V_m merge volume, in vehicles per hour.

V_r ramp volume, in vehicles per hour.

V_u volume at an adjacent ramp upstream of the ramp in question, in vehicles per hour.

CHAPTER 7

MULTILANE HIGHWAYS

CONTENTS

I. INTRODUCTION

This chapter treats the capacity analysis of multilane highways that cannot be classified as freeways because they are undivided, lack full control of access, or both. Such highways exist in a variety of settings, from typical low-density rural environments to suburban areas, where development density is higher, and where traffic frictions due to turning vehicles and other factors also increase.

Between points of fixed interruptions, multilane highways operate under uninterrupted flow conditions. Such flow, however, is not as efficient as flow on freeways because of the various sources of side- and medial-frictions which exist on multilane highways, such as:

1. Vehicles enter and leave the roadside to access parking lots, driveways, unsignalized intersections, and other points; such movements may involve right or left turns, with left turns having a much greater negative impact on flow.
2. The friction due to opposing vehicles on undivided multilane roadways also impacts negatively on flow; on divided multilane highways, this impact is eliminated.
3. The visual impact of development fronting directly on the highway influences driver behavior, and contributes to its being less efficient than on comparable freeways.

The level of such interferences varies widely depending on the development environment served by the multilane highway. The principal determinants of the degree of such interferences are the type and density of land use along the roadway.

This chapter presents procedures for both divided and undivided multilane highways, in environments ranging from low-density rural areas to suburban areas of considerably higher development density. The procedures are generally applicable where the distance between signals on the multilane highway is 2 mi or greater. Where signal spacing is 1 mi or less, the procedures in Chapter 11, "Arterials," should be used.

Where signal spacing is between 1 and 2 mi, the user may wish to consider both the uninterrupted flow operations between signals using the methodology of this chapter, and the operations at each signalized intersection, using the procedures of Chapter 9. This will allow the consideration of speed and travel time between intersections and delay at individual intersections. It should be remembered, however, that flow on multilane highways with signal spacings under 2 mi is likely to be in platoons.

The procedures of this chapter are structurally similar to those for freeways, although specific values and flow characteristics differ. They treat the uninterrupted flow characteristics of multilane highways between fixed interruptions, and do not specifically account for conditions at signalized intersections.

MULTILANE HIGHWAY FEATURES REQUIRING CONSIDERATION

A number of aspects require consideration in the analysis of multilane highways:

1. *Facility classification*—Multilane highways exist in a wide variety of environments that cause substantial variations in the magnitude of frictions to uninterrupted flow. For the purposes of capacity analysis, multilane highways are classified into one of four basic types:

a. All multilane highways are classified as either *divided* or *undivided;* divided highways reduce the incidence of medial friction substantially by controlling and limiting points at which median crossings are permitted.

b. All multilane highways are classified as either *rural* or *suburban,* based on the density of land-use development; suburban highways are usually subject to substantially higher levels of side- and medial-friction than are rural highways.

c. The four basic classifications for multilane highways are, therefore: (1) rural, divided; (2) rural, undivided; (3) suburban, divided; and (4) suburban, undivided.

Illustrations 7-1 through 7-4 depict typical multilane highways in each of these four basic categories.

Multilane highway designs, however, cover a broad range of conditions, and not all facilities are simply categorized. *Median treatments* cover a substantial range of alternatives. A wide median providing left-turn lanes for all left-turn locations will produce less medial friction than a similar divided highway not having left-turn lanes, assuming similar flow levels. The number

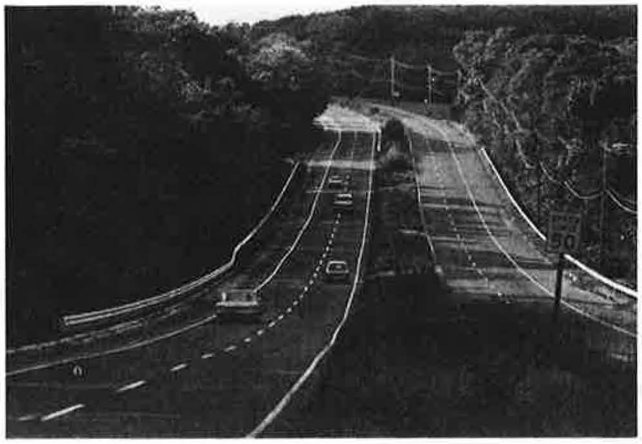


Illustration 7-1. A divided multilane highway in a rural environment.

Illustration 7-2. A divided multilane highway in a suburban environment.

Illustration 7-3. An undivided multilane highway in a rural environment.

Illustration 7-4. An undivided multilane highway in a suburban environment.

of median openings allowing crossings, and the number of such crossings, will also be a factor influencing the degree of friction present.

At the other extreme are undivided multilane highways that have only a centerline dividing opposing flow. In such cases, left turns are uncontrolled, and the presence of an opposing flow in adjacent lanes presents substantial friction as well.

There are also a variety of intermediate treatments including painted medians with or without left-turn lanes, and *continuous left-turn lanes* for both directions. This latter case is interesting in that it separates opposing flows by one full lane, but does not control or limit the number of left turns. Such cases generally provide for friction levels approximately midway between the levels provided by divided and undivided highways, for similar development environments.

The classification of highways as *rural* or *suburban* is also not a simple matter. The range of development environments is continuous, and reflects such variables as:

a. The frequency of unsignalized intersections.

b. The frequency of driveways and other uncontrolled access points.

c. The number of left turns into and out of these intersections, driveways, etc.

d. The number of right turns into and out of these intersections, driveways, etc.

Because data quantifying these variables, and relating them to specific aspects of multilane flow, are sparse, the chapter classifies multilane highways into one of the four categories previously noted. Judgment is required in making this classification. In very approximate terms, highways with more than 10 uncontrolled access points per mile (on one side) would be considered to be "suburban." Also, any highway on which left or right turns cause appreciable delay to through vehicles would also be classified as "suburban." The latter is somewhat dependent on how turns are handled in the facility design. Highways with turn lanes can accommodate more turns without influencing through movements than similar highways without such lanes.

2. *Uninterrupted flow segments*—Those multilane highway segments between fixed interruptions, such as signalized intersections, are analyzed as uninterrupted flow segments, using procedures specified in this chapter.

3. *Weaving areas*—Although quite rare, weaving sections may occur occasionally on multilane highways. While thère are no special procedures for the analysis of weaving areas on multilane highways, the procedures of Chapter 4 may be applied to such sections as an approximation.

4. *Ramp junctions*—Multilane highways often have high-speed on- and off-ramp junctions at interchanges with freeways, other multilane highways, or other roadway types. The procedures of Chapter 5 may be used to analyze such junctions.

5. *Signalized intersections*—Signalized intersections do exist at widespread intervals along most multilane highways. This chapter contains a short approximation technique for the capacity analysis of such intersections that may be used as a rough estimate of conditions. Procedures detailed in Chapter 9 should be applied for a precise analysis.

UNINTERRUPTED FLOW CHARACTERISTICS FOR MULTILANE HIGHWAYS

Figures 7-1 and 7-2 describe the speed-density and speed-flow relationships for a typical uninterrupted flow segment on a multilane highway under *ideal conditions*. Ideal conditions for multilane highways include:

1. Level terrain.
2. Twelve-ft lane widths.
3. A minimum of 6-ft lateral clearance between the edge of travel lanes and obstructions at the roadside or in the median.
4. Passenger cars only in the traffic stream.
5. A divided highway cross section in a rural environment.

Note that Figure 7-2 indicates that average travel speed is sensitive to flow levels throughout the full range of flow rates, although the degree of sensitivity increases as capacity is approached. This contrasts with speed-flow curves for freeway uninterrupted flow, which are virtually flat for flows up to 1,600 pcphpl, and is a reflection of the impact of side- and medial-frictions on normal multilane flow. As shown in Figure 7-1, density also varies with flow throughout the full range, a sensitivity which also increases as capacity is approached.

Figures 7-1 and 7-2 are indicative of average operating characteristics under the ideal conditions stated. Local driver habits vary somewhat from location to location, and the operating characteristics at any given location may vary somewhat from these averages.

FACTORS AFFECTING MULTILANE HIGHWAY FLOW UNDER IDEAL CONDITIONS

The characteristics depicted in Figures 7-1 and 7-2 are affected by prevailing conditions that are not "ideal." These effects are discussed in the following sections.

Lane Width and/or Lateral Clearance Restrictions

Ideal conditions call for 12-ft lanes and 6-ft lateral clearance at the roadside of multilane highways. Failure to provide either of these adversely affects operating conditions.

Narrow lanes force drivers to operate their vehicles closer to each other laterally than they would normally desire. They compensate for this by observing longer longitudinal headways than under ideal conditions at any given speed. Thus, for a given speed, narrow lanes cause a reduction in the flow rate that can be sustained. For a given flow rate, the speed of the traffic stream will be slower than if 12-ft lanes existed.

Roadside and median obstructions closer than 6 ft to the pavement edge have the same impact. Obstructions cause drivers to shift their position laterally in the traffic lane. They, in effect, "shy away" from the obstruction(s). This also results in placing vehicles laterally closer to one another than under ideal conditions, and drivers compensate as previously described.

Heavy Vehicles

"Heavy vehicles" are generally defined as any vehicle having more than two axles or four tires touching the pavement. They are divided into three broad categories: (1) trucks, (2) recreational vehicles, and (3) buses.

As in Chapter 3, "Basic Freeway Segments," typical truck streams are represented by a truck with an average weight-to-horsepower ratio 200 lb/hp. Options are provided for analysis of cases where trucks are either more or less powerful than the typical value.

Heavy vehicles have a detrimental effect on traffic flow for two reasons: (1) they are larger than passenger cars, and therefore occupy more roadway space; and (2) their performance characteristics are generally inferior to passenger cars, leading to the formation of gaps in the traffic stream which cannot always be effectively filled by normal passing maneuvers. The latter effect is particularly marked on grades. Heavy vehicles are often incapable of maintaining speed on upgrades of significant length. Thus, long gaps may form between passenger cars and heavy vehicles in the traffic stream. Because such gaps are continually lengthening and new gaps are forming, it generally is not possible for passenger cars to fill all of them using passing maneuvers. Because of this, roadway space is used far less efficiently than by a uniform traffic stream composed only of passenger cars.

Type of Multilane Highway

Ideal conditions for multilane highways refer to a divided highway in a rural environment. Additional side- and/or medial-frictions that occur on other categories of multilane highways have a further adverse effect on traffic flow characteristics.

Driver Population

Not all driver populations use multilane highways with the same efficiency. In general, commuters or other frequent users of a facility will use highways more efficiently than recreational or other occasional drivers. Capacity losses as high as 10 to 25 percent have been observed for recreational traffic streams as compared to commuters using the same facility.

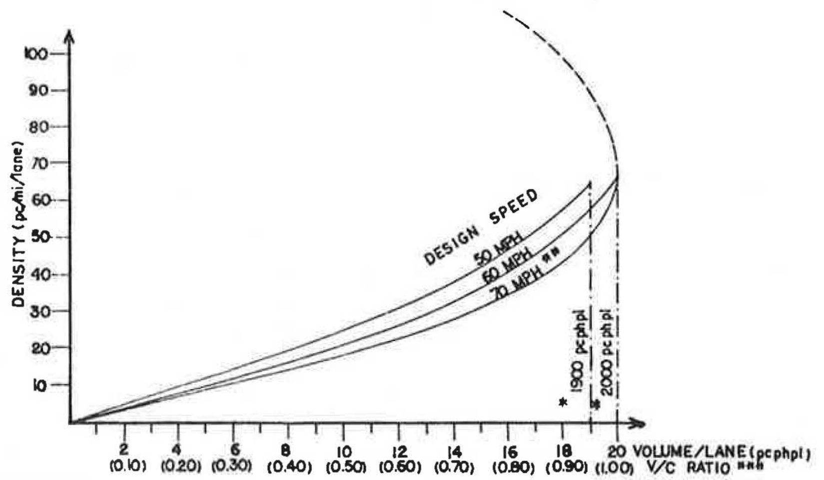
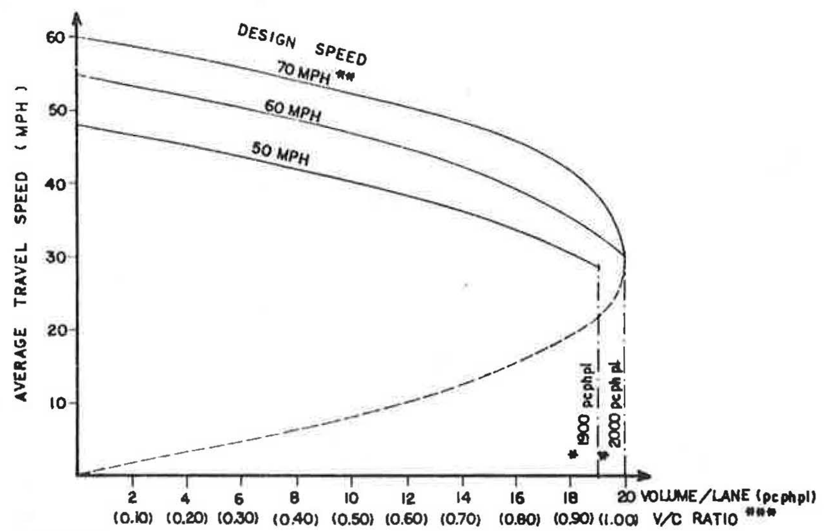


Figure 7-1. Density flow characteristics for uninterrupted flow segments of multilane highways.

* capacity

** reflects 55 MPH speed limit

*** v/c ratio based on capacity of 2000 pcphpl, applies only to 60 and 70 MPH design speeds



* capacity

** reflects 55 MPH speed limit

*** v/c ratio based on capacity of 2000 pcphpl, applies only to 60 and 70 MPH design speeds

Figure 7-2. Speed-flow characteristics for uninterrupted flow segments of multilane highways.

II. METHODOLOGY

LEVEL-OF-SERVICE CRITERIA

Level-of-service (LOS) criteria for multilane highways are defined in terms of density. Density is a measure which quantifies the proximity to other vehicles in the traffic stream. It expresses the degree of maneuverability within the traffic stream.

Boundary values of density are given, as follows, for the various levels of service. They are the same as the values used in Chapter 3 for freeways.

Level of Service	*Maximum Density (pc/mi/ln)*
A	12
B	20
C	30
D	42
E	67

Complete LOS criteria are given in Table 7-1. For 70-mph, 60-mph, and 50-mph design speed elements, the table gives the average travel speed, the maximum value of v/c, and the corresponding maximum service flow rate, *MSF,* for each level of service. The speeds, v/c ratios, and maximum service flow rates tabulated are expected to exist in traffic streams operating at the densities defined for each level of service under ideal conditions.

Level-of-service criteria depend on the design speed of the highway element being studied. A "highway element" can be an isolated geometric element, such as a curve or grade having a reduced design speed, or a series of such geometric elements that dominate the operation of a longer segment of highway. Straight and level highway segments are assumed to have a design speed of 70 mph.

Level-of-service A describes completely free-flow conditions. The operation of vehicles is virtually unaffected by the presence of other vehicles, and operations are constrained only by the geometric features of the highway and driver preferences. Vehicles are spaced at an average of 440 ft, or 22 car-lengths, at a maximum density of 12 pc/mi/ln. The ability to maneuver within the traffic stream is high. Minor disruptions to flow are easily absorbed at this level without causing significant delays or queuing.

Level-of-service B is also indicative of free flow, although the presence of other vehicles begins to be noticeable. Average travel speeds are somewhat diminished from LOS A, but are still generally over 53 mph on sections with 70-mph design speed. Vehicles are spaced at an average of approximately 264 ft, or 13 car-lengths, at a maximum density of 20 pc/mi/ln. Minor disruptions are still easily absorbed at this level, although local deterioration in LOS will be more obvious.

Level-of-service C represents a range in which the influence of traffic density on operations becomes marked. The ability to maneuver within the traffic stream, and to select an operating speed, is now clearly affected by the presence of other vehicles. Average travel speeds are reduced to about 50 mph on 70-mph design speed sections, and the average spacing of vehicles is reduced to approximately 175 ft, or 9 car-lengths, at a maximum density of 30 pc/mi/ln. Minor disruptions may be expected to cause serious local deterioration in service, and queues may form behind any significant traffic disruption. Severe or long-term disruptions may cause the facility to operate at LOS F.

Level-of-service D borders on unstable flow. Speeds and ability to maneuver are severely restricted because of traffic congestion. Average travel speeds are approximately 40 mph on 70-mph design speed sections, while the average spacing of vehicles is 125 ft, or 6 car-lengths, at a maximum density of 42 pc/mi/ln. Only the most minor of disruptions can be absorbed without the formation of extensive queues and the deterioration of service to LOS F.

Level-of-service E represents operations at or near capacity, and is quite unstable. At capacity, vehicles are spaced at only 80 ft, or 4 car-lengths, at a maximum density of 67 pc/mi/ln. This is the minimum spacing at which uniform flow can be maintained, and effectively defines a traffic stream with no usable gaps. Thus, disruptions cannot be damped or dissipated, and any disruption, no matter how minor, will cause queues to form and service to deteriorate to LOS F. Average travel speeds at capacity are approximately 30 mph.

Level-of-service F represents forced or breakdown flow. It occurs at a point where vehicles arrive either at a rate greater than that at which they are discharged or at a point on a planned facility where forecasted demand exceeds the computed capacity. While operations at such points (and on immediately downstream sections) will appear to be at capacity or better, queues will form behind these breakdowns. Operations within queues are highly unstable, with vehicles experiencing short spurts of movement followed by stoppages. Average travel speeds within queues are generally under 30 mph, with densities higher than 67 pc/mi/ln. Note that the term "LOS F" may be used to characterize both the point of the breakdown and the operating conditions within the queue. It must be remembered, however, that it is the point of breakdown that causes the queue to form, and that operations within the queue are generally not related to defects along the highway segment over which the queue extends. Chapters 3 and 6 contain more detailed discussions of the use and application of LOS F, and of the analysis of breakdown conditions.

The user should note that the level-of-service criteria of Table 7-1 are based on the typical speed-flow-density relationships depicted in Figures 7-1 and 7-2. The criteria reflect the shape of those curves—particularly the fact that both speed and density deteriorate rapidly as capacity is immediately approached. Thus, as LOS goes from A to E, the range of densities and speeds in each level becomes larger, while the corresponding range of maximum service flow rates is more stable.

As with other LOS criteria, the maximum service flow rates of Table 7-1 are stated in terms of rates of flow for the peak 15 min. Demand or forecasted volumes are generally divided by the peak hour factor (PHF) to reflect a maximum flow rate within the hour before comparing with the criteria of Table 7-1.

TABLE 7-1. LEVEL-OF-SERVICE CRITERIA FOR MULTILANE HIGHWAYS

LEVEL OF SERVICE	DENSITY (PC/MI/LN)	70 MPH DESIGN SPEED			60 MPH DESIGN SPEED			50 MPH DESIGN SPEED		
		SPEED[a] (MPH)	v/c	MSF[b] (PCPHPL)	SPEED[a] (MPH)	v/c	MSF[b] (PCPHPL)	SPEED[a] (MPH)	v/c	MSF[b] (PCPHPL)
A	≤ 12	≥ 57	0.36	700	≥ 50	0.33	650	—	—	—
B	≤ 20	≥ 53	0.54	1,100	≥ 48	0.50	1,000	≥ 42	0.45	850
C	≤ 30	≥ 50	0.71	1,400	≥ 44	0.65	1,300	≥ 39	0.60	1,150
D	≤ 42	≥ 40	0.87	1,750	≥ 40	0.80	1,600	≥ 35	0.76	1,450
E	≤ 67	≥ 30	1.00	2,000	≥ 28	1.00	2,000	≥ 30	1.00	1,900
F	> 67	< 30	c	c	< 28	c	c	< 30	c	c

[a] Average travel speed.
[b] Maximum rate of flow per lane under ideal conditions, rounded to the nearest 50 pcphpl.
[c] Highly variable.

BASIC RELATIONSHIPS

Table 7-1 gives the values of maximum service flow rate and v/c ratio for multilane highways. These values represent maximum flow rates that can be accommodated under ideal conditions. Equations 7-1 through 7-3 are used to compute service flow rate under prevailing roadway and traffic conditions.

$$SF_i = MSF_i \times N \times f_w \times f_{HV} \times f_E \times f_p \tag{7-1}$$
$$MSF_i = c_j \times (v/c)_i \tag{7-2}$$
$$SF_i = c_j \times (v/c)_i \times N \times f_w \times f_{HV} \times f_E \times f_p \tag{7-3}$$

where:

SF_i = service flow rate; the maximum flow rate that can be accommodated by the multilane highway segment under study, in one direction, under prevailing roadway and traffic conditions, while meeting the performance criteria of LOS i, in vph;

MSF_i = maximum service flow rate; the maximum rate of flow which can be accommodated by the multilane highway segment under study, per lane, under *ideal conditions*, while meeting the performance criteria of LOS i, in pcphpl;

c_j = capacity per lane for a multilane highway with design speed j; 2,000 pcphpl for j = 70 mph or 60 mph, 1,900 pcphpl for j = 50 mph; c_j may be obtained from Table 7-1 as the maximum service flow rate for LOS E;

N = number of lanes in one direction;

$(v/c)_i$ = maximum volume-to-capacity ratio allowable while maintaining the performance characteristics of LOS i;

f_w = adjustment factor for lane width and/or lateral clearance restrictions;

f_{HV} = adjustment factor for the presence of heavy vehicles in the traffic stream;

f_E = adjustment factor for the development environment and type of multilane highway; and

f_p = adjustment factor for driver population.

Equation 7-1 takes a value of MSF from Table 7-1 and adjusts it to reflect prevailing roadway and traffic conditions.

Equation 7-2 computes the MSF from the limiting value of v/c ratio for the specified LOS. Values of MSF in Table 7-1 are computed in this manner, and have been rounded to the nearest 50 pcphpl.

Equation 7-3 is a combination of Eqs. 7-1 and 7-2, and is useful when solving for v/c or N. It is the most frequently used form of these relationships.

ADJUSTMENTS TO MAXIMUM SERVICE FLOW RATE

Adjustment for Lane Width and Lateral Clearance Restrictions

Ideal conditions for multilane highways include the provision of 12-ft lanes and 6-ft lateral clearance, i.e., roadside obstructions must be located at least 6 ft from the edge of the travel lanes.

Designs that fail to meet either or both of these criteria will have an adverse impact on traffic flow. This effect is accounted for by the adjustment factor, f_w, given in Table 7-2.

"Lateral obstructions" may be objects periodically located at the roadside, such as light standards, signs, trees, abutments, bridge rails, or other objects. They may also be continuous fixtures, such as traffic barriers or retaining walls. In Table 7-2, "obstruction on both sides of roadway" refers to one roadside and the median of the roadway. This condition applies primarily to divided multilane highways which may have obstructions or barriers in the median. It may also apply to an undivided highway which periodically divides to pass around bridge abutments or other center objects.

As with other types of facilities, some judgment should be exercised in determining whether or not a "lateral obstruction" exists. In general, if the existence of roadside or median objects does not cause drivers to either "shy" away from them or slow down because of them, there will be no measurable impact on traffic flow.

Illustrations 7-5 through 7-8 depict various types of roadside and median treatments that can affect multilane highway flow.

Adjustment for the Presence of Heavy Vehicles

A second "ideal" condition incorporated into the basic LOS criteria for multilane highways is a traffic stream composed of only passenger cars. Rarely will such a traffic stream exist on multilane highways. Service flow rates must therefore be adjusted to reflect the actual traffic composition.

TABLE 7-2. ADJUSTMENT FACTOR FOR RESTRICTED LANE WIDTH AND LATERAL CLEARANCE

DISTANCE FROM EDGE OF TRAVELED WAY TO OBSTRUCTION[a]	ADJUSTMENT FACTOR, f_w							
	OBSTRUCTION ON ONE SIDE OF ROADWAY[b]				OBSTRUCTION ON BOTH SIDES OF ROADWAY[c]			
	LANE WIDTH (FT)							
(FT)	12	11	10	9	12	11	10	9
	4-LANE DIVIDED MULTILANE HIGHWAYS (2 LANES EACH DIRECTION)							
≥ 6	1.00	0.97	0.91	0.81	1.00	0.97	0.91	0.81
4	0.99	0.96	0.90	0.80	0.98	0.95	0.89	0.79
2	0.97	0.94	0.88	0.79	0.94	0.91	0.86	0.76
0	0.90	0.87	0.82	0.73	0.81	0.79	0.74	0.66
	6-LANE DIVIDED MULTILANE HIGHWAYS (3 LANES EACH DIRECTION)							
≥ 6	1.00	0.96	0.89	0.78	1.00	0.96	0.89	0.78
4	0.99	0.95	0.88	0.77	0.98	0.94	0.87	0.77
2	0.97	0.93	0.87	0.76	0.96	0.92	0.85	0.75
0	0.94	0.91	0.85	0.74	0.91	0.87	0.81	0.70
	4-LANE UNDIVIDED MULTILANE HIGHWAYS (2 LANES EACH DIRECTION)							
≥ 6	1.00	0.95	0.89	0.77	NA	NA	NA	NA
4	0.98	0.94	0.88	0.76	NA	NA	NA	NA
2	0.95	0.92	0.86	0.75	0.94	0.91	0.86	NA
0	0.88	0.85	0.80	0.70	0.81	0.79	0.74	0.66
	6-LANE UNDIVIDED MULTILANE HIGHWAYS (3 LANES EACH DIRECTION)							
≥ 6	1.00	0.95	0.89	0.77	NA	NA	NA	NA
4	0.99	0.94	0.88	0.76	NA	NA	NA	NA
2	0.97	0.93	0.86	0.75	0.96	0.92	0.85	NA
0	0.94	0.90	0.83	0.72	0.91	0.87	0.81	0.70

[a] Use the average distance to obstruction on "both sides" where the distance to obstructions on the left and right differs.
[b] Factors for one-sided obstructions allow for the effect of opposing flow.
[c] Two-sided obstructions include one roadside and one median obstruction. Median obstruction may exist in the median of a divided multilane highway or in the center of an undivided highway which periodically divides to go around bridge abutments or other center objects.
NA = Not applicable; use factor for one-sided obstruction.

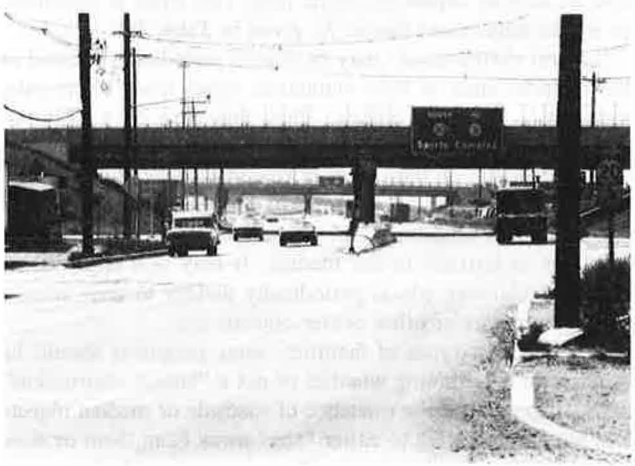


Illustration 7-5. Note the bridge pier located in the center of a normally undivided suburban multilane highway. Vehicles will tend to adjust their position in adjacent travel lanes to avoid traveling too closely to the abutment.

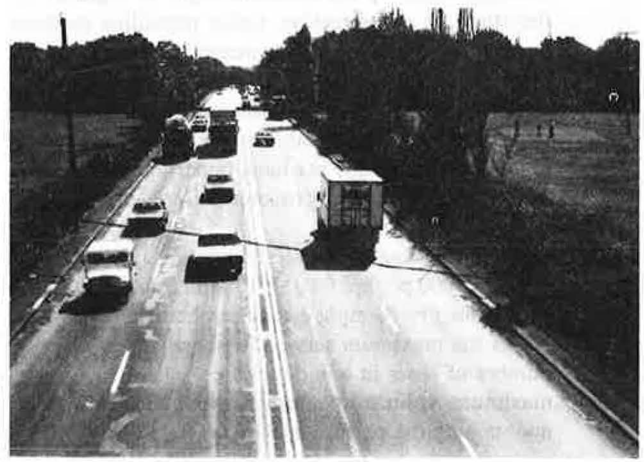


Illustration 7-6. The absence of a usable shoulder and the close proximity of obstructions to the edge of the traveled way on this highway will also influence driver behavior.

Illustration 7-7. This divided multilane highway displays "ideal" geometric conditions, with no median or roadside obstructions to influence flow.

Illustration 7-8. This undivided multilane highway has no obstructions at the roadside closer than 6ft to the travel lanes. The impact of opposing flow on median lanes is not a "lateral obstruction," and is accounted for elsewhere in the procedure.

The procedures and factors used to accomplish this are the same as those used in Chapter 3, "Basic Freeway Segments." For convenience, the procedure is briefly described herein, and the factors are repeated. For a more detailed discussion, refer to Chapter 3.

Adjustments for the presence of heavy vehicles in the traffic stream consider three types of vehicles: trucks, recreational vehicles (RV's), and buses. Finding the adjustment factor requires two steps, as follows:

1. Find the passenger-car equivalent (pce) for trucks, recreational vehicles (RV's), and buses, respectively, for the prevailing operating conditions.
2. Using the values found in step 1, compute an adjustment factor that corrects for all heavy vehicles in the traffic stream.

Each of these steps is briefly discussed in the following subsections.

1. *Finding passenger-car equivalents*—Values of passenger-car equivalents are selected from Tables 7-3 through 7-8 for a variety of basic conditions.

For long segments of highway over which no single grade has a significant impact on operations, Table 7-3 is used to select passenger-car equivalent values for trucks, E_T, recreational vehicles, E_R, and buses, E_B. A long multilane highway segment may be classified as a "general segment" if no one grade of 3 percent or less is more than 1 mi long and no one grade of more than 3 percent is more than ½ mi long. Such segments should be categorized as follows:

a. *Level terrain*—any combination of horizontal and vertical alignment permitting heavy vehicles to maintain approximately the same speed as passenger cars; this generally includes short grades of no more than 1 to 2 percent.

b. *Rolling terrain*—any combination of horizontal and vertical alignment causing heavy vehicles to reduce their speeds substantially below those of passenger cars, but NOT causing heavy vehicles to operate at crawl speeds for any significant length of time or at frequent intervals.

TABLE 7-3. PASSENGER-CAR EQUIVALENTS ON EXTENDED GENERAL MULTILANE HIGHWAY SEGMENTS

	TYPE OF TERRAIN		
FACTOR	LEVEL	ROLLING	MOUNTAINOUS
E_T for Trucks	1.7	4.0	8.0
E_B for Buses	1.5	3.0	5.0
E_R for RV's	1.6	3.0	4.0

c. *Mountainous terrain*—any combination of horizontal and vertical alignment causing heavy vehicles to operate at crawl speeds for significant distances or at frequent intervals.

For all such general highway segments, values of E_T, E_R, and E_B are selected from Table 7-3.

Any grade of 3 percent or less that is longer than 1 mi or any grade greater than 3 percent that is longer than ½ mi should be treated as an isolated significant grade. The *upgrade* and *downgrade* must be treated separately because the impact of heavy vehicles varies substantially for these two conditions.

Tables 7-4 through 7-8 give passenger-car equivalents for *upgrades*. Tables 7-4, 7-5, and 7-6 give values of E_T for various truck populations:

a. Table 7-4—"typical" truck populations (wt/hp ratio = 200 lb/hp).
b. Table 7-5—"light" truck populations (wt/hp ratio = 100 lb/hp).
c. Table 7-6—"heavy" truck populations (wt/hp ratio = 300 lb/hp).

These tables can be used to adjust an analysis to reflect the character of trucks at a given location. Note, however, that only *one* value is selected for E_T. The truck population *should not* be segmented into three parts. The value used should be selected from the table best representing the approximate average weight-to-horsepower ratio for prevailing conditions. The equivalents shown are designed to represent traffic streams with a broad

TABLE 7-4. PASSENGER-CAR EQUIVALENTS FOR TYPICAL TRUCKS (200 LB/HP)

GRADE (%)	LENGTH (MI)	PASSENGER-CAR EQUIVALENT, E_T															
		4-LANE HIGHWAYS								6-LANE HIGHWAYS							
PERCENT TRUCKS		2	4	5	6	8	10	15	20	2	4	5	6	8	10	15	20
<1	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
1	0–1/2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	1/2–1	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	≥1	4	3	3	3	3	3	3	3	4	3	3	3	3	3	3	3
2	0–1/4	4	4	4	3	3	3	3	3	4	4	4	3	3	3	3	3
	1/4–1/2	5	4	4	3	3	3	3	3	5	4	4	3	3	3	3	3
	1/2–3/4	6	5	5	4	4	4	4	4	6	5	5	4	4	4	4	4
	3/4–1½	7	6	6	5	4	4	4	4	7	5	5	5	4	4	4	4
	≥1½	8	6	6	6	5	5	4	4	8	6	6	5	4	4	4	4
3	0–1/4	6	5	5	5	4	4	4	3	6	5	5	5	4	4	4	3
	1/4–1/2	8	6	6	6	5	5	5	4	7	6	6	6	5	5	5	4
	1/2–1	9	7	7	6	5	5	5	5	9	7	7	6	5	5	5	5
	1–1½	9	7	7	7	6	6	5	5	9	7	7	6	5	5	5	5
	≥1½	10	7	7	7	6	6	5	5	10	7	7	6	5	5	5	5
4	0–1/4	7	6	6	5	4	4	4	4	7	6	6	5	4	4	4	4
	1/4–1/2	10	7	7	6	5	5	5	5	9	7	7	6	5	5	5	5
	1/2–1	12	8	8	7	6	6	6	6	10	8	7	6	5	5	5	5
	≥1	13	9	9	9	8	8	7	7	11	9	9	8	7	6	6	6
5	0–1/4	8	6	6	6	5	5	5	5	8	6	6	6	5	5	5	5
	1/4–1/2	10	8	8	7	6	6	6	6	8	7	7	6	5	5	5	5
	1/2–1	12	11	11	10	8	8	8	8	12	10	9	8	7	7	7	7
	≥1	14	11	11	10	8	8	8	8	12	10	9	8	7	7	7	7
6	0–1/4	9	7	7	7	6	6	6	6	9	7	7	6	5	5	5	5
	1/4–1/2	13	9	9	8	7	7	7	7	11	8	8	7	6	6	6	6
	1/2–3/4	13	9	9	8	7	7	7	7	11	9	9	8	7	6	6	6
	≥3/4	17	12	12	11	9	9	9	9	13	10	10	9	8	8	8	8

NOTE: If the length of grade falls on a boundary value, use the equivalent for the longer grade class. Any grade steeper than the percent stated must use the next higher grade category.

TABLE 7-5. PASSENGER-CAR EQUIVALENTS FOR LIGHT TRUCKS (100 LB/HP)

GRADE (%)	LENGTH (MI)	PASSENGER-CAR EQUIVALENT, E_T															
		4-LANE HIGHWAYS								6-LANE HIGHWAYS							
PERCENT TRUCKS		2	4	5	6	8	10	15	20	2	4	5	6	8	10	15	20
≤2	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
3	0–1/4	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	1/4–1/2	4	4	4	3	3	3	3	3	4	4	4	3	3	3	3	3
	1/2–3/4	4	4	4	4	3	3	3	3	4	4	4	3	3	3	3	3
	3/4–1	5	4	4	4	4	3	3	3	5	4	4	4	3	3	3	3
	>1	6	5	5	5	4	4	4	3	6	5	5	4	4	4	3	3
4	0–1/4	4	4	4	3	3	3	3	3	5	4	4	4	3	3	3	3
	1/4–1/2	5	5	5	4	4	4	4	4	5	4	4	4	4	4	4	4
	1/2–1	6	5	5	5	4	4	4	4	6	5	5	4	4	4	4	4
	>1	7	6	6	5	4	4	4	4	7	5	5	5	4	4	4	4
5	0–1/4	6	5	5	5	4	4	4	3	6	5	5	5	4	4	4	3
	1/4–1	8	7	7	6	5	5	5	5	8	7	7	6	5	5	5	5
	>1	9	7	7	6	5	5	5	5	8	7	7	6	5	5	5	5
6	0–1/4	7	5	5	5	4	4	4	4	7	5	5	5	4	4	3	3
	1/4–1	9	7	7	6	5	5	5	5	8	7	7	6	5	5	5	5
	>1	9	7	7	7	6	6	5	5	9	7	7	6	5	5	5	5

NOTE: If a length of grade falls on a boundary value, use the equivalent for the longer grade category. Any grade steeper than the percent shown must use the next higher grade category.

TABLE 7-6. PASSENGER-CAR EQUIVALENTS FOR HEAVY TRUCKS (300 LB/HP)

GRADE (%)	LENGTH (MI)	PASSENGER-CAR EQUIVALENT, E_T															
		4-LANE HIGHWAYS								6-LANE HIGHWAYS							
PERCENT TRUCKS		2	4	5	6	8	10	15	20	2	4	5	6	8	10	15	20
<1	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
1	0–1/4	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	1/4–1/2	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	1/2–3/4	4	4	4	4	3	3	3	3	4	4	4	3	3	3	3	3
	3/4–1	5	4	4	4	3	3	3	3	5	4	4	4	3	3	3	3
	1–1½	6	5	5	5	4	4	4	3	6	5	5	4	4	4	3	3
	>1½	7	5	5	5	4	4	4	3	7	5	5	5	4	4	3	3
2	0–1/4	4	4	4	3	3	3	3	3	4	4	4	3	3	3	3	3
	1/4–1/2	7	6	6	5	4	4	4	4	7	5	5	5	4	4	4	4
	1/2–3/4	8	6	6	5	5	4	4	4	8	6	6	6	5	5	4	4
	3/4–1	8	6	6	6	5	5	5	5	8	6	6	6	5	5	5	5
	1–1½	9	7	7	7	6	6	5	5	9	7	7	6	5	5	5	5
	>1½	10	7	7	7	6	6	5	5	10	7	7	6	5	5	5	5
3	0–1/4	6	5	5	5	4	4	4	3	6	5	5	5	4	4	4	3
	1/4–1/2	9	7	7	6	5	5	5	5	8	7	7	6	5	5	5	5
	1/2–3/4	12	8	8	7	6	6	6	6	10	8	7	6	5	5	5	5
	3/4–1	13	9	9	8	7	7	7	7	11	8	8	7	6	6	6	6
	>1	14	10	10	9	8	8	7	7	12	9	9	8	7	7	7	7
4	0–1/4	7	5	5	5	4	4	4	4	7	5	5	5	4	4	3	3
	1/4–1/2	12	8	8	7	6	6	6	6	10	8	7	6	5	5	5	5
	1/2–3/4	13	9	9	8	7	7	7	7	11	9	9	8	7	6	6	6
	3/4–1	15	10	10	9	8	8	8	8	12	10	10	9	8	7	7	7
	>1	17	12	12	10	9	9	9	9	13	10	10	9	8	8	8	8
5	0–1/4	8	6	6	6	5	5	5	5	8	6	6	6	5	5	5	5
	1/4–1/2	13	9	9	8	7	7	7	7	11	8	8	7	6	6	6	6
	1/2–3/4	20	15	15	14	11	11	11	11	14	11	11	10	9	9	9	9
	>3/4	22	17	17	16	13	13	13	13	17	14	14	13	12	11	11	11
6	0–1/4	9	7	7	7	6	6	6	6	9	7	7	6	5	5	5	5
	1/4–1/2	17	12	12	11	9	9	9	9	13	10	10	9	8	8	8	8
	>1/2	28	22	22	21	18	18	18	18	20	17	17	16	15	14	14	14

NOTE: If the length of grade falls on a boundary value, the equivalent corresponding to the longer grade category is used. Any grade steeper than the percent shown must use the next higher grade category.

mix of trucks having the average weight-to-horsepower ratios indicated.

Table 7-7 is used to find E_R, and Table 7-8 is used to find E_B.

Tables 7-4 through 7-8 give values of passenger-car equivalents for *uniform* upgrades. When several consecutive grades form a *composite grade,* an equivalent uniform grade is computed and used to enter the tables. The most common technique for making this determination is the Average Grade Technique. The average grade is computed as the total rise from the beginning of the grade divided by the total horizontal distance over which the rise was accomplished.

Consider the following example. Three consecutive upgrades, as follows, are to be analyzed:

a. 3 percent grade—1,000 ft long
b. 4 percent grade—2,000 ft long
c. 2 percent grade—1,000 ft long

The total rise of the 4,000-ft grade may be computed as:

$$\begin{aligned} 1{,}000 \times 0.03 &= 30 \text{ ft} \\ 2{,}000 \times 0.04 &= 80 \text{ ft} \\ 1{,}000 \times 0.02 &= \underline{20 \text{ ft}} \\ & 130 \text{ ft} \end{aligned}$$

The "average grade" may now be expressed as follows:

$$\text{Ave. Grade} = (130/4{,}000) \times 100 = 3.25 \text{ Percent}$$

Passenger-car equivalents would then be selected for a 6,000-ft grade of 3.25 percent.

The average grade approach is reasonably accurate for grades of 4,000 ft or less, or no greater than 4 percent. For steeper and longer grades, a more exact technique is described in Appendix I of Chapter 3.

Downgrade conditions are handled in a more approximate fashion. For grades of less than 4,000 ft and/or 4 percent, downgrade segments may be considered operationally similar to level terrain segments and are analyzed accordingly.

For longer or steeper downgrades, it is recommended that field measurements of downgrade heavy vehicle speeds be made and that an equivalent upgrade value be used. Where such field measurements are not practical, the downgrade passenger-car equivalent may be roughly approximated as one-half the corresponding upgrade value.

2. *Computing the heavy vehicle adjustment factor*—Once values for E_T, and E_R, and E_B are determined, the adjustment factor for heavy vehicles may be computed as follows:

TABLE 7-7. PASSENGER-CAR EQUIVALENTS FOR RECREATIONAL VEHICLES

GRADE (%)	LENGTH (MI)	E_R															
		4-LANE HIGHWAYS								6-LANE HIGHWAYS							
PERCENT RV'S		2	4	5	6	8	10	15	20	2	4	5	6	8	10	15	20
<2	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
3	0–1/2	3	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	≥ 1/2	4	3	3	3	3	3	3	3	4	3	3	3	3	3	3	3
4	0–1/4	3	2	2	2	2	2	2	2	3	2	2	2	2	2	2	2
	1/4–3/4	4	3	3	3	3	3	3	3	4	3	3	3	3	3	3	3
	≥ 3/4	5	4	4	4	3	3	3	3	4	4	4	4	3	3	3	3
5	0–1/4	4	3	3	3	3	3	3	3	4	3	3	3	2	2	2	2
	1/4–3/4	5	4	4	4	4	4	4	4	5	4	4	4	4	4	4	4
	≥ 3/4	6	5	4	4	4	4	4	4	5	5	4	4	4	4	4	4
6	0–1/4	5	4	4	4	3	3	3	3	5	4	4	3	3	3	3	3
	1/4–3/4	6	5	5	4	4	4	4	4	6	4	4	4	4	4	4	4
	≥ 3/4	7	6	6	6	5	5	5	5	6	5	5	5	4	4	4	4

NOTE: If a length of grade falls on a boundary condition, the equivalent from the longer grade class is used. Any grade steeper than the percent shown must use the next higher grade category.

TABLE 7-8. PASSENGER-CAR EQUIVALENTS FOR BUSES

GRADE	E_B
0–3	1.6
4[a]	1.6
5[a]	3.0
6[a]	5.5

[a]Use generally restricted to grades more than 1/4 mi long.

$$f_{HV} = 1/[1 + P_T(E_T - 1) + P_R(E_R - 1) + P_B(E_B - 1)] \quad (7\text{-}4)$$

where:

f_{HV} = adjustment factor for the presence of heavy vehicles in the traffic stream;

E_T, E_R, E_B = passenger-car equivalents for trucks, RV's, and buses, respectively; and

P_T, P_R, P_B = proportion of trucks, RV's, and buses, respectively, in the traffic stream (expressed as a decimal).

Where only one type of heavy vehicle is present in the traffic stream, Table 7-9 may be used to convert a passenger-car equivalent directly to the adjustment factor. Where the ratio of trucks in the traffic stream to the total number of buses and RV's is more than 5:1, all heavy vehicles may be treated as if they were trucks. Thus, a traffic stream consisting of 15 percent trucks, 2 percent RV's, and 1 percent buses may be analyzed as if it contained 18 percent trucks. This will allow the use of Table 7-9 to find f_{HV}.

Adjustment for Development Environment and Type of Multilane Highway

The base criteria for maximum service flow rate under ideal conditions apply to a divided multilane highway in a rural development environment. For undivided and/or suburban development environments, the adjustment factor f_E is selected from Table 7-10, and applied.

Undivided highways are those on which opposing flows are separated only by a centerline marking. Divided highways are those on which opposing flows are separated by a physical barrier. Multilane highways with painted medians may be classified as "divided" if the median is at least 10 ft in width, and if crossing prohibitions are well enforced. Where the painted median is narrower, or where crossings occur in significant numbers, the average of divided and undivided factors would be appropriate for use. The same average factor would be used for multilane highways with a continuous left-turn lane separating opposing flows.

The suburban/rural categorization is less precise, and depends on several factors including roadside development density, the frequency of unsignalized intersections and driveway entrances to the facility, and the number of vehicles turning into and out of such unsignalized locations. In general, any highway with more than 10 driveways and/or unsignalized intersections per mile on any one side of the highway would be classified as "suburban," as would any highway on which turning movements onto and/or off of the facility represented a cause of noticeable delay to through vehicles. Judgment is used in this classification because precise quantification of these factors is not yet available.

Any multilane facility with signalized intersections occurring at intervals of less than 1 mi should be classified as an "arterial" and analyzed using the procedures of Chapter 11.

Adjustment for Driver Population

The adjustment factor for driver population is given in Table 7-11. The selection of a value for traffic streams consisting

TABLE 7-9. ADJUSTMENT FACTOR FOR THE EFFECT OF TRUCKS, BUSES, OR RECREATIONAL VEHICLES IN THE TRAFFIC STREAM

PCE[a]	ADJUSTMENT FACTOR, f_{HV}														
E_T E_R E_B	PERCENTAGE OF TRUCKS, P_T; RV'S, P_R; OR BUSES, P_B														
	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
16	0.87	0.77	0.69	0.63	0.57	0.53	0.49	0.45	0.43	0.40	0.36	0.32	0.29	0.27	0.25
17	0.86	0.76	0.68	0.61	0.56	0.51	0.47	0.44	0.41	0.38	0.34	0.31	0.28	0.26	0.24
18	0.85	0.75	0.66	0.60	0.54	0.49	0.46	0.42	0.40	0.37	0.33	0.30	0.27	0.25	0.23
19	0.85	0.74	0.65	0.58	0.53	0.48	0.44	0.41	0.38	0.36	0.32	0.28	0.26	0.24	0.22
20	0.84	0.72	0.64	0.57	0.51	0.47	0.42	0.40	0.37	0.34	0.30	0.27	0.25	0.23	0.21
21	0.83	0.71	0.63	0.56	0.50	0.45	0.41	0.38	0.36	0.33	0.29	0.26	0.24	0.22	0.20
22	0.83	0.70	0.61	0.54	0.49	0.44	0.40	0.37	0.35	0.32	0.28	0.25	0.23	0.21	0.19
23	0.82	0.69	0.60	0.53	0.48	0.43	0.39	0.36	0.34	0.31	0.27	0.25	0.22	0.20	0.19
24	0.81	0.68	0.59	0.52	0.47	0.42	0.38	0.35	0.33	0.30	0.27	0.24	0.21	0.19	0.18
25	0.80	0.67	0.58	0.51	0.46	0.41	0.37	0.34	0.32	0.29	0.26	0.23	0.20	0.18	0.17

[a]Passenger-car equivalent, obtained from Table 7-3, 7-4, 7-5, or 7-6.

TABLE 7-10. ADJUSTMENT FACTOR FOR TYPE OF MULTILANE HIGHWAY AND DEVELOPMENT ENVIRONMENT, f_E

TYPE	DIVIDED	UNDIVIDED
Rural	1.00	0.95
Suburban	0.90	0.80

TABLE 7-11. ADJUSTMENT FACTOR FOR DRIVER POPULATION

DRIVER POPULATION	FACTOR, f_p
Commuter, or Other Regular Users	1.00
Recreational, or Other Nonregular Users	0.75–0.90

primarily of occasional users requires some judgment. The range of values given in Table 7-11 reflects varied observations throughout the United States. Local data should be consulted in selecting an exact value. Where such data are not available, general knowledge of local conditions should be applied.

Summary

The preceding discussion has presented the basic structure of capacity analysis procedures for multilane highways. Detailed applications of these in operational analysis, design, and planning, follow.

III. PROCEDURES FOR APPLICATION

The methodology described in the previous section may be applied in three ways:

1. *Operational analysis*—In operational analysis applications, known traffic and geometric conditions for an existing highway, or projections of these for a future highway, are analyzed to determine the existing or projected level of service, and the approximate speed and density of the traffic stream.

Operational analysis is the most detailed application of procedures, and requires detailed input information concerning both roadway and traffic conditions. It is also the most flexible use of procedures, and is useful in the evaluation of alternative improvements to existing highways. In such comparisons, the approximate operating conditions of the traffic stream resulting from several alternative improvements may be estimated and compared.

2. *Design*—In design applications, a forecast of traffic conditions is used with detailed information on geometric design standards and horizontal and vertical alignment to determine the number of lanes required to provide for a specified level of service. Where such determinations result in fractional lanes, alternative operational analyses may be carried out to compare the impacts of selecting either of the two integer values surrounding the fractional computation.

3. *Planning*—A planning analysis gives the same basic result as a design analysis: the determination of the number of lanes needed to provide for a specified level of service. At the planning stage of a project, however, this determination is a rough approximation based on the very general traffic forecasts and geometric information available at the time. A planning analysis yields a general guide to the size of facility to be anticipated, an estimate which must be checked on a segment-by-segment basis during the design process.

OPERATIONAL ANALYSIS

Objectives of Operational Analysis

Operational analysis is intended to predict the operating characteristics of an existing or planned roadway when subjected to a present or future demand. This is the most detailed type of analysis, and requires the most detailed input information. It results in an estimate of the prevailing or expected level of service, and of the approximate speed and density of the traffic stream.

Data Requirements

The following information must be available as inputs to the operational analysis procedure:

1. *Geometrics*—The geometrics of the facility should be specified in detail, including: (a) design speed, (b) lane widths, (c) shoulder and median clearances, (d) grades, (e) length of grades, (f) horizontal curvature, and (g) type of terrain (if applicable).

2. *Volumes*—The existing traffic volume, or the projected future volume, must be known, in vehicles per hour (vph) for the hour of interest (usually the peak hour).

3. *Traffic characteristics*—Detailed traffic characteristics are needed in operational analysis, including: (a) the PHF, (b) percent trucks, (c) percent RV's, and (d) percent buses.

4. *Facility environment*—The multilane highway must be classified as either divided or undivided, and as rural or suburban.

Segmenting the Facility

Analysis procedures are intended for use on multilane highway segments of more-or-less uniform characteristics. Thus, changes in the characteristics noted will require a new segment for analysis.

Significant changes in grade or terrain, in traffic demand, in development environment, and so forth, require establishing new analysis segments. Signalized intersections also serve as boundaries where new segments are often defined, because demand is subject to change at these locations. Careful dividing of the facility into uniform analysis segments will avoid the difficulty involved in classifying a long segment as level-of-service *i*, when various subsegments are experiencing different levels of service and different operating conditions.

Computational Steps

The general approach taken in operational analysis is to use Eq. 7-1 or Eq. 7-3 to solve for the effective value of *MSF* or v/c ratio. This is then used to find the level of service in Table 7-1, and to enter Figures 7-1 and 7-2 to find the likely density and speed of the traffic stream.

The following computational steps may be followed:

1. The volume for the hour of interest is converted to the peak flow rate within the hour, and for computational purposes, is set equal to the service flow rate, *SF*:

$$SF = V/\text{PHF} \tag{7-5}$$

where:

SF = service flow rate, in vph;

V = full hour volume, in vph; and

PHF = peak hour factor.

2. Adjustment factors f_w (lane width and lateral clearance), f_{HV} (heavy vehicles), f_E (development environment and type of highway), and f_p (driver population) are found from the appropriate tables:

f_w (Table 7-2)
E_T (Table 7-3, 7-4, 7-5, or 7-6)

E_R (Table 7-3 or 7-7)
E_b (Table 7-3 or 7-8)
f_{HV} (Table 7-9) or compute as:
$1/[1 + P_T(E_T - 1) + P_R(E_R - 1) - P_B(E_B - 1)]$
f_E (Table 7-10
f_p (Table 7-11)

3. Equation 7-3 is used with these factors to compute the effective v/c ratio:

$$v/c = SF/[c_j \times N \times f_w \times f_{HV} \times f_E \times f_p] \qquad (7\text{-}6)$$

Alternatively, Eq. 7-1 may be used to compute the effective maximum service flow rate, *MSF:*

$$MSF = SF/[N \times f_w \times f_{HV} \times f_E \times f_p] \qquad (7\text{-}7)$$

where all symbols are as previously defined.

4. Using either result from step 3, Table 7-1 is entered to determine the existing or anticipated level of service. Note that the values given in Table 7-1 are the *maximum* allowable values for the indicated levels of service.

5. Where a more detailed evaluation of operating conditions is desired, the v/c ratio or maximum service flow rate, *MSF,* determined in step 3 may be used to enter Figure 7-2 to determine the approximate average travel speed of the traffic stream, and Figure 7-1 to determine the approximate density of the traffic stream.

A worksheet for use in operational analysis is shown on Figure 7-3. It is similar to the worksheet for operational analysis of basic freeway segments, and is a useful format for the organization and display of computations.

Interpretation of Results

Operational analysis results in an approximate determination of the operating characteristics of the traffic stream for the segment under study. The densities and speeds estimated on the basis of Figures 7-1 and 7-2 represent average U.S. conditions, and local characteristics may vary somewhat from these values.

The densities drawn from Figure 7-1 are expressed as passenger cars per mile per lane. When field measurements of

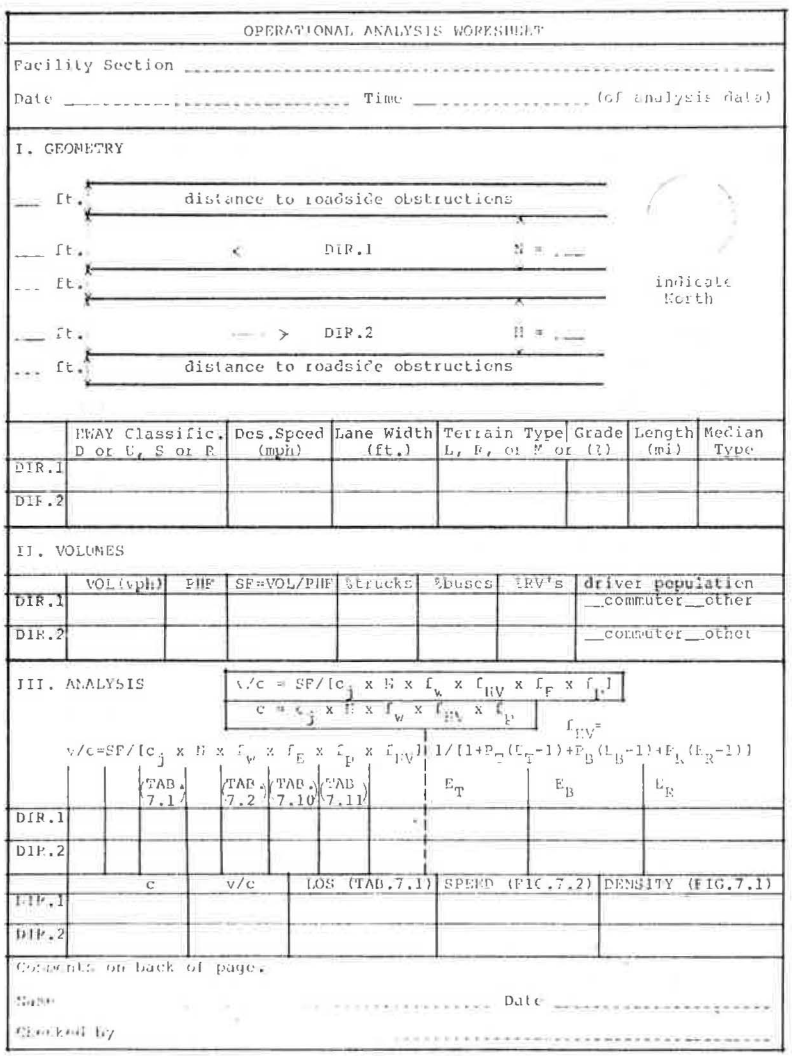


Figure 7-3. Worksheet for operational analysis.

density are used to determine level of service, data values in vehicles per mile per lane must be converted to passengers cars per mile per lane before comparing to the density criteria of Table 7-1. The average travel speeds drawn from Figure 7-2 are also based on all passenger cars in the traffic stream. Actual values for mixed traffic streams will be somewhat lower than Figure 7-2 values.

Where the analysis of a segment suggests that LOS F exists, it will often be useful to estimate the propagation of queues upstream of the breakdown. A detailed technique for such analyses is included in Chapter 6, "Freeway Systems."

DESIGN

Objectives of Design

The objective of a design analysis is straightforward: the determination of the number of lanes needed in each direction on a multilane highway.

"Design" applications suggest that related aspects of a highway are also in the design process and that details of the horizontal and vertical alignment are known, as well as details concerning the expected traffic demand.

Data Requirements

The design process requires less detailed data than operational analysis. Data are required on future traffic demand volumes, details of horizontal and vertical alignment, and general geometric standards.

1. *Geometric design standards*—(a) design speeds, (b) lane widths, (c) lateral clearances, and (d) median type.
2. *Details of horizontal and vertical alignment*—(a) type of terrain, (b) grades, (c) grade lengths, and (d) horizontal alignment elements requiring reduced design speed.
3. *Demand volumes*—(a) directional design hour volume, *DDHV*, (b) traffic composition, and (c) peak hour factor, PHF, for the design year.
4. *Environmental conditions*—(a) development environment, (b) type of multilane highway, and (c) driver population.

Many of these factors can be controlled in the design process, and the impacts of some design decisions on geometrics, and horizontal and vertical alignment may affect the number of lanes which must be provided.

Selecting a Design Value of *v/c* Ratio

Boundary values of v/c for use in design may be selected directly from Table 7-1. Design, however, need not be limited to boundaries between levels of service. Table 7-12 has been provided to assist designers in selecting appropriate values of v/c. It shows v/c ratios in increments of 0.10 for the range of 0.30 through 0.80, and gives the average travel speed, density, and level of service that result from their use. For convenience, boundary values of v/c are also shown in this table, so that Table 7-1 need not be consulted in addition to Table 7-12.

Relationship to AASHTO Design Criteria

It should be noted that the levels of service referred to in the current AASHTO policies are based on previous documents. The levels of service herein are *not* analogous, and should not be directly applied using AASHTO-recommended LOS values. AASHTO design criteria for multilane highways call for the following v/c values:

1. Rural design—0.50 (i.e., 1,000 pcphpl, max.).
2. Suburban design—0.75 (i.e., 1,500 pcphpl, max.).

Separating the Facility into Uniform Design Segments

The facility undergoing design must be separated into uniform segments for design. Changes in terrain, significant grades, major junctions at which demand volume changes significantly, changes in the development environment, and similar factors would indicate the need to begin a new segment for design analysis.

Computational Steps

The general approach to design uses Eq. 7-2 or Eq. 7-3 to solve for *N*, the number of required lanes. It should be noted that for significant grades, the upgrade and downgrade must be considered separately. The following computational steps are used:

1. The directional design hour volume must be converted to a peak flow rate, which is set equal to the service flow rate:

$$SF = DDHV/\mathrm{PHF} \quad (7\text{-}8)$$

where all values are as previously defined.

2. All adjustment factors for expected prevailing conditions are found from the appropriate tables, as follows:

f_w (Table 7-2)
E_T (Table 7-3, 7-4, 7-5, or 7-6)
E_R (Table 7-3 or 7-7)
E_B (Table 7-3 or 7-8)
f_{HV} (Table 7-9) or compute from:
$1/[1 + P_T(E_T - 1) + P_R(E_R - 1) + P_B(E_B - 1)]$
f_E (Table 7-10)
f_p (Table 7-11)

3. Using Eq. 7-3, the required number of lanes is computed:

$$N = SF/[c_j \times (v/c) \times f_w \times f_{HV} \times f_E \times f_p] \quad (7\text{-}9)$$

or alternatively, using Eq. 7-2:

$$N = SF/[MSF \times f_w \times f_{HV} \times f_E \times f_p] \quad (7\text{-}10)$$

where all terms are as previously defined.

TABLE 7-12. VOLUME-TO-CAPACITY VALUES FOR USE IN DESIGN OF MULTILANE HIGHWAYS

v/c RATIO	MSF[c]	RESULTING PERFORMANCE PARAMETERS		
		LOS[a]	DENSITY (PC/MI/LN)	SPEED (MPH)
		70-MPH ELEMENTS		
0.30	600	A	10.5	57
0.36[b]	700	A	12.0	57
0.40	800	B	14.0	56
0.50	1,000	B	18.0	54
0.54[b]	1,100	B	20.0	53
0.60	1,200	C	22.0	52
0.70	1,400	C	28.0	50
0.71[b]	1,400	C	30.0	50
0.80	1,600	D	34.0	47
		60-MPH ELEMENTS		
0.30	600	A	11.5	51
0.33[b]	650	A	12.0	50
0.40	800	B	15.5	49
0.50[b]	1,000	B	20.0	48
0.60	1,200	C	25.0	45
0.65[b]	1,300	C	30.0	44
0.70	1,400	D	32.0	42
0.80[b]	1,600	D	40.0	40
		50-MPH ELEMENTS		
0.30	550	B	13.0	43
0.40	750	B	17.5	42
0.45[b]	850	B	20.0	42
0.50	950	C	24.0	41
0.60[b]	1,150	C	30.0	39
0.70	1,350	D	38.0	37
0.76[b]	1,450	D	42.0	35
0.80	1,500	E	46.5	34

[a] Design may be within a LOS.
[b] Maximum threshold v/c for LOS shown.
[c] Rounded to the nearest 50 pcphpl.

Interpretation of Results

Design computations for N will generally result in fractional results. Because the number of lanes must be an integer value, the designer is faced with the decision of whether to reduce or increase the computed value to the nearest integer, a decision with large economic consequences. While there are no set "rule-of-thumb" guidelines for such decisions, analysts should perform an operational analysis on the possible choices to determine the LOS and approximate speed and density that would result. This allows such decisions to be made with some knowledge of the operational impacts—knowledge that must be weighted against the relative costs involved.

The decision on number of lanes in a specific segment of a multilane highway also depends on the continuity of lanes in adjacent segments and the rest of the highway system. Frequent addition and dropping of lanes along a highway are not practical, although either may be considered at critical locations.

On specific grades, the number of lanes required on the upgrade may be larger than the number required on the downgrade. This is a clear indication that a climbing lane is required. Chapter 3 contains a detailed procedure for the design and evaluation of climbing lanes that may be used for a more precise treatment of such cases.

Figure 7-4 illustrates a worksheet that may be used in conjunction with design analyses. It is a useful form for performing and summarizing the results of design computations.

PLANNING

Objectives of Planning

The objectives of a planning analysis are similar to those in design: determination of the likely number of lanes required for the multilane highway segment under consideration. The pri-

DESIGN WORKSHEET

Facility Section ______________________________

Date ______________________ Time ______________ (of analysis data)

I. DESIGN CRITERIA

	LOS	v/c (TAB. 7.12)	Highway Classification D or U, S or R	Design Speed	Lane Width	Lateral clear. (ft) roadside	Lateral clear. (ft) median	Terrain Type L, R, or M	Grade or (%)	Length (mi)
DIR.1										
DIR.2										

II. TRAFFIC FORECASTS

	DDHV (vph)	PHF	SF=(DDHV/PHF)	%trucks	%buses	%RV's	driver population
DIR.1							__commuter__other
DIR.2							__commuter__other

III. DESIGN ANALYSIS

$$\boxed{N = SF/[c_j \times v/c \times f_w \times f_{HV} \times f_E \times f_p]}$$

$f_{HV}{}^* = 1/[1+P_T(E_T-1)+P_B(E_B-1)+P_R(E_R-1)]$

	N=	SF/	c_j x (TAB. 7.1)	(v/c) x	f_w x (TAB. 7.2)	f_E x (TAB. 7.10)	f_p x (TAB. 7.11)	* f_{HV}	E_T	E_B	E_R
DIR.1											
DIR.2											

IV. SKETCH DESIGN

* Table 7-9 or compute as shown.

Name ______________________ Date ______________

Checked by ______________________

Figure 7-4. Worksheet for design analysis.

mary difference between design and planning is the detail of available information. In the planning stage, details of horizontal and vertical alignment, and even of final location, are not yet known. Thus, volume projections are less accurate, and general geometric parameters are a matter of assumption. Nevertheless, planning computations can assess the probable number of lanes that would be required, and more importantly, whether or not a multilane highway is appropriate for the expected conditions.

Data Requirements

The planning methodology assumes that ideal geometrics exist and that traffic streams consist only of passenger cars and trucks. The required input data are reasonably straightforward: (1) general terrain through which the highway will pass, (2) the *AADT* for the design year, (3) the PHF for the design year, (4) the percent trucks in the traffic stream, and (5) the type of multilane highway and anticipated development environment.

Computational Steps

The general computational approach in planning analysis is to convert the design year *AADT* to a *DDHV*, and apply a general estimate of service flow rate per lane to find *N*.

1. The *AADT* is converted to a *DDHV* using the following equation:

$$DDHV = AADT \times K \times D \tag{7-11}$$

where:

K = the percent of *AADT* occurring in the peak hour; and
D = the percent of traffic in the peak direction of flow.

The K-factor is dependent on the type and density of the development environment. If local data are unavailable, the following general average values may be used:

Type of Environment	*K-Factor*
Urban	0.07 to 0.10
Suburban	0.10 to 0.15
Rural	0.15 to 0.20

The D-factor is dependent on the type of route served by the highway in question. Where local data are not available, the following general average values may be used:

Type of Route	*D-Factor*
Rural	0.65
Suburban	0.60
Urban Radial	0.55
Urban Circumferential	0.50

These default values should be used with great caution. Small errors in these values can result in large errors in the estimated directional design hour volume. It is always preferable to base these values on local data concerning these characteristics.

2. Table 7-13 is used to find a value of SFL_i, the per lane service flow rate for LOS i, for prevailing conditions of terrain and percent trucks in the traffic stream.

3. The value of N is estimated as:

$$N = DDHV/(SFL_i \times f_E \times \text{PHF}) \qquad (7\text{-}12)$$

where all values are as previously defined, and the value of f_E is found in Table 7-10.

Interpretation of Results

Planning analysis results in a rough estimation of N, the number of lanes required in each direction, for the multilane highway in question. This estimate is based on very general input information, and planning computations must be refined during the design phase of a project.

Multilane highways of more than three lanes in each direction are rare, and more than four lanes, virtually nonexistent. Computations resulting in more than four lanes in each direction offer a good indication that a multilane highway may be inappropriate for the anticipated conditions and that a limited access facility should be considered.

INTERSECTIONS ON MULTILANE HIGHWAYS

Multilane highways will generally have signalized intersections at periodic intervals, occurring at major junction points that are not grade separated. These intersections may be subjected to a detailed analysis using the methodology of Chapter 9, "Signalized Intersections."

As a rough estimate, the capacity of a multilane highway intersection approach can be taken to be the capacity of the uninterrupted flow segment approaching the intersection (the service flow rate for LOS E) multiplied by the G/C ratio, i.e., the ratio of green time (in seconds) to the cycle length (in seconds).

When the ratio of the approach volume to the estimated approach capacity exceeds 0.50, a detailed analysis of conditions using the procedures of Chapter 9 should be conducted. This will allow a detailed evaluation of intersection delay.

It should be noted that this procedure provides only a rough estimate, and does not take into account special features, such as turn conflicts, turning lanes, multiple phasing, added through lanes approaching the intersection, and so on, all of which can have a drastic impact on intersection operations. For detailed analysis of such features, the procedure of Chapter 9 should be used.

THREE-LANE HIGHWAYS WITH PERMANENTLY ASSIGNED THIRD LANES

The use of three-lane highways, which declined in the late 1960's, has once again begun to be more common. Three-lane highways may be operated in a number of ways, the most common of which include:

1. Use of the center lane as a continuous left-turn lane (more common in suburban settings).
2. Alternate assignment of the center lane to one direction, then the other, providing exclusive passing lanes for each direction of flow at periodic intervals.
3. A long segment of three-lane highway, permanently operated with two lanes in one direction, and one in the other.

Although there are no specially designed methodologies for the capacity analysis of three-lane highways, techniques in this chapter and in Chapter 8, "Two-lane Highways," can be used to obtain approximate insight into their operation. Multilane highway techniques, for example, may be used to approximate operating conditions on segments of three-lane highway where two lanes are assigned for the exclusive use of one direction for a significant length (note that this is not the same as alternating assignment of the third lane for passing purposes). Criteria and factors for four-lane undivided highways would be used for this purpose.

The second lane in the preferred direction on a three-lane highway is generally used less efficiently than the second lane on a full four-lane facility, where it exists for only a short distance of less than 1 to 2 mi. The added lane is often used primarily to pass slower moving vehicles (particularly on long upgrades) and to execute left turns. The second lane adds to the capacity of the two-lane highway by providing more efficient passing and reducing left-turn conflicts, but would not approach that of a four-lane highway, even in the preferred direction.

Where the third lane of a three-lane highway is permanently assigned to one direction for a significant distance of several miles, the operation of the preferred direction can approach that of a four-lane highway. Procedures in this chapter can be used to analyze the two-lane direction in such cases. It is recommended, however, that the maximum service flow rates of Table 7-1 be reduced by 10 to 15 percent to reflect somewhat reduced efficiency compared to the full four-lane case.

Chapter 8 contains other suggestions for adapting two-lane highway analysis procedures to some other three-lane cases.

TABLE 7-13. SERVICE FLOW RATE PER LANE FOR PLANNING APPLICATIONS (DESIGN SPEED = 70 MPH ONLY)

LOS	PERCENT TRUCKS									
	0	2	4	5	6	8	10	12	15	20
	LEVEL TERRAIN[a]									
A	700	700	700	700	650	650	650	650	650	600
B	1,100	1,100	1,050	1,050	1,050	1,050	1,000	1,000	1,000	1,000
C	1,400	1,400	1,350	1,350	1,350	1,350	1,300	1,300	1,250	1,250
D	1,750	1,750	1,700	1,700	1,650	1,650	1,650	1,600	1,600	1,550
E	2,000	2,000	1,950	1,950	1,900	1,900	1,850	1,850	1,800	1,750
	ROLLING TERRAIN[a]									
A	700	650	600	600	600	550	550	500	500	500
B	1,100	1,050	1,000	950	950	900	850	800	800	700
C	1,400	1,300	1,250	1,200	1,200	1,150	1,100	1,050	1,000	900
D	1,750	1,650	1,550	1,500	1,500	1,400	1,350	1,300	1,250	1,100
E	2,000	1,900	1,800	1,750	1,700	1,600	1,550	1,500	1,450	1,250
	MOUNTAINOUS TERRAIN[a]									
A	700	600	550	500	500	450	400	400	350	300
B	1,100	950	850	800	700	700	650	600	550	450
C	1,400	1,250	1,100	1,050	1,000	900	850	750	700	600
D	1,750	1,550	1,350	1,300	1,250	1,100	1,050	950	850	750
E	2,000	1,750	1,550	1,500	1,400	1,250	1,200	1,100	1,000	850

[a] All values rounded to the nearest 50 vphpl.

IV. SAMPLE CALCULATIONS

CALCULATION 1—OPERATIONAL ANALYSIS OF A SUBURBAN UNDIVIDED HIGHWAY

1. *Description*—Consider the multilane highway segment illustrated in Figure 7-5, which shows an undivided, suburban multilane highway with light standards located 2 ft from the traveled way at both roadsides, and bridge abutments located in the center of the roadway at frequent intervals. The facility has 11-ft lanes which narrow to 10 ft at bridge abutments. The design speed of the segment is 60 mph, and the driver population consists primarily of commuters.

If the segment carries a peak hour demand of 2,000 vph, with 15 percent trucks and a PHF of 0.91, what LOS can be expected in this segment?

2. *Solution*—The primary judgment in this problem is the selection of the f_w-factor. The major constriction occurs at bridge abutments, where lane width is 10 ft, with a roadside obstruction at 2 ft and a center obstruction at 0 ft. Other sections of the roadway, however, have 11-ft lanes, with a roadside obstruction at 2 ft, and no center obstruction. Because the most conservative analysis would consider the abutments, and because these abutments are "frequent," the minimum condition will be used. Note also that f_w-factors for center obstructions are specifically intended for periodic isolated obstructions, which condition applies to the highway segment described.

Then:

$$v/c = SF/[c_j \times N \times f_w \times f_{HV} \times f_E \times f_p]$$

where:

SF = 2,000/0.91 = 2,198 vph (Given);
c_j = 2,000 pcphpl (Table 7-1);

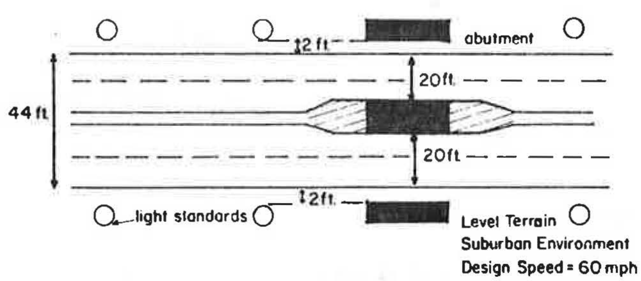


Figure 7-5. Multilane highway segment for Calculation 1.

N = 2 (Given);
f_w = 0.80 (Table 7-2, undivided highway, 10-ft lane, obstructions both sides at an average of 1 ft);
E_T = 1.7 (Table 7-3, level terrain);
f_{HV} = $1/[1 + 0.15(1.7 - 1)] = 0.90$;
f_E = 0.80 (Table 7-10, suburban undivided highway);
f_p = 1.00 (Table 7-11, commuters); and
v/c = $2,198/[2,000 \times 2 \times 0.80 \times 0.90 \times 0.80 \times 1.00]$
v/c = 0.95.

Entering Table 7-1 with a v/c of 0.95 for a highway with a 60-mph design speed, the LOS is found to be E.

Entering Figure 7-2 with a $v/c = 0.95$, the expected speed of the traffic stream is 33 mph. Entering Figure 7-1 with this value, the expected density is 59 pc/mi/ln.

Figure 7-6 illustrates these solutions, and shows the calculation as worked on the operational analysis worksheet.

OPERATIONAL ANALYSIS WORKSHEET

Facility Section CARLSON TURNPIKE

Date 10/18 Time 5-6 PM (of analysis data)

I. GEOMETRY

2 ft. distance to roadside obstructions

20 ft. ← DIR.1 N = 2

0 ft.

20 ft. → DIR.2 N = 2

2 ft. distance to roadside obstructions

indicate North

	HWAY Classific. D or U, S or R	Des.Speed (mph)	Lane Width (ft.)	Terrain Type L, R, or M or	Grade (%)	Length (mi)	Median Type
DIR.1	U, S	60	10	L	–	–	–
DIR.2	U, S	60	10	L	–	–	–

II. VOLUMES

	VOL(vph)	PHF	SF=VOL/PHF	%trucks	%buses	%RV's	driver population
DIR.1	2,000	0.91	2,198	15	–	–	X commuter __ other
DIR.2	2,000	0.91	2,198	15	–	–	X commuter __ other

III. ANALYSIS

$v/c = SF/[c_j \times N \times f_w \times f_{HV} \times f_E \times f_p]$

$c = c_j \times N \times f_w \times f_{HV} \times f_p$

$f_{HV} = 1/[1+P_T(E_T-1)+P_B(E_B-1)+P_R(E_R-1)]$

	v/c=SF/[		c_j (TAB 7.1)	x N	x f_w (TAB 7.2)	x f_E (TAB 7.10)	x f_p (TAB 7.11)	x f_{HV}]	E_T	E_B	E_R
DIR.1	0.95	2198	2000	2	0.80	0.80	1.00	0.90	1.7	–	–
DIR.2	0.95	2198	2000	2	0.80	0.80	1.00	0.90	1.7	–	–

	c	v/c	LOS (TAB.7.1)	SPEED (FIG.7.2)	DENSITY (FIG.7.1)
DIR.1	2,304	0.95	E	33 MPH	59 PC/MI/LN
DIR.2	2,304	0.95	E	33 MPH	59 PC/MI/LN

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Name JOHN JONES Date

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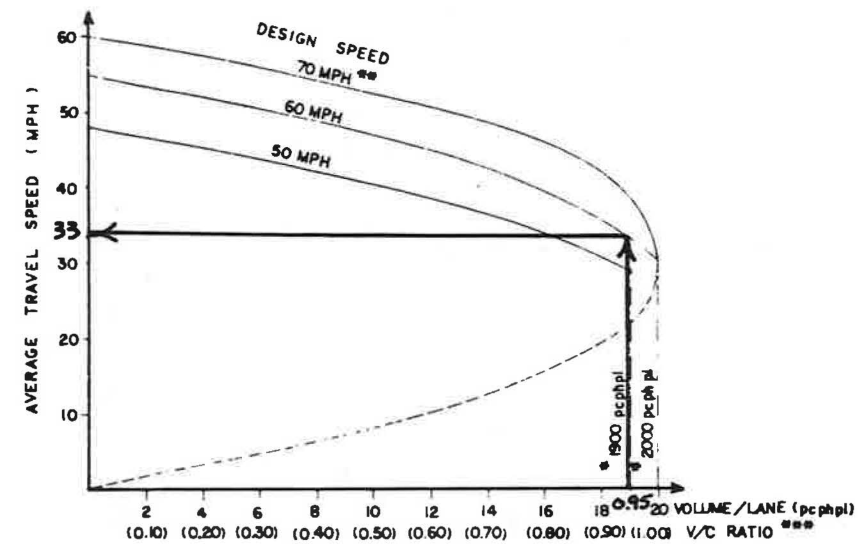
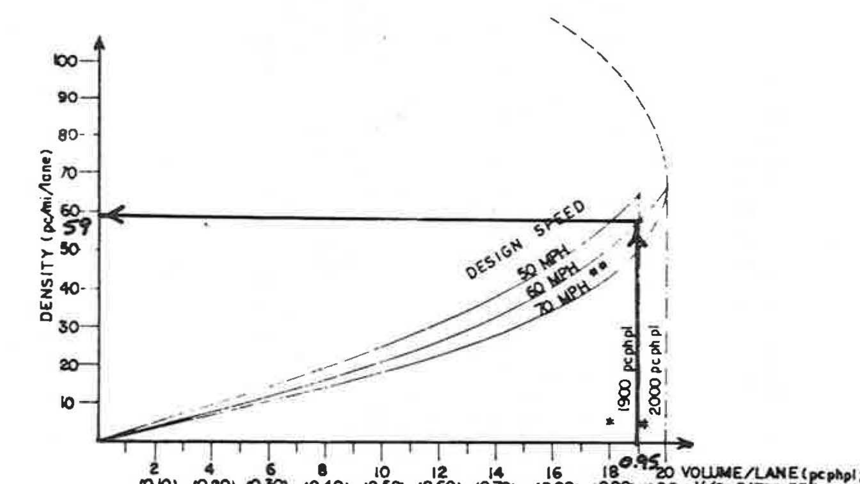


* capacity

** reflects 55 MPH Speed limit

*** v/c ratio based on capacity of 2000 pcphpl, applies only to 60 and 70 MPH design speeds



* capacity

** reflects 55 MPH Speed limit

*** v/c ratio based on capacity of 2000 pcphpl, applies only to 60 and 70 MPH design speeds

Figure 7-6. Illustration of solution to Calculation 1.

CALCULATION 2—OPERATIONAL ANALYSIS OF A RURAL DIVIDED HIGHWAY ON A SPECIFIC GRADE

1. *Description*—Consider the multilane highway segment illustrated in Figure 7-7. It depicts a rural, divided, multilane highway with an ideal cross section. An analysys of the existing level of service and operations is desired for a segment of this highway on a significant grade of 3 percent, 5,000 ft long.

The directional demand on this segment is 2,200 vph in peak periods, with 10 percent trucks, 5 percent RV's, and a PHF of 0.85. The segment serves primarily recreational traffic.

2. *Solution*—Because this segment is a significant grade, the upgrade and downgrade conditions must be considered separately. Lacking local data on downgrade speeds, downgrade values of E_T and E_R will be taken to be one-half the corresponding upgrade values, as recommended in the methodology.

Then:

$$v/c = SF/[c_j \times N \times f_w \times f_{HV} \times f_E \times f_p]$$

where:

SF = 2,200/0.85 = 2,588 vph (Given);
c_j = 2,000 pcphpl;
N = 3 (Given);
f_w = 1.00 (Table 7-2, ideal conditions);
E_T (Upgrade) = 5 (Table 7-4, 10 percent trucks, 3 percent grade, ½ to 1 mi, 6-lane highway);
E_T (Downgrade) = 2.5;
E_R (Upgrade) = 3 (Table 7-7, 5 percent RV's, 3 percent grade, ≥ ½ mi, 6-lane highway);
E_R (Downgrade) = 1.5;
f_{HV} (Upgrade) = $1/[1 + 0.10\,(5 - 1) + 0.05\,(3 - 1)] = 0.67$;
f_{HV} (Downgrade) = $1/[1 + 0.10\,(2.5 - 1) + 0.05\,(1.5 - 1)] = 0.85$;
f_E = 1.0 (Table 7-10, divided rural highway); and
f_p = 0.82 (Table 7-11, select value in middle of range given).

Then:

v/c (Upgrade) = $2{,}588/[2{,}000 \times 3 \times 1.0 \times 0.67 \times 1.0 \times 0.82] = 0.79$
v/c (Downgrade) = $2{,}588/[2{,}000 \times 3 \times 1.0 \times 0.85 \times 1.0 \times 0.82] = 0.62$

Checking Table 7-1, it is found that the downgrade segment operates at LOS C. The upgrade operates at LOS D.

The computed v/c ratios may be used to enter Figures 7-1 and 7-2 to determine the approximate operating conditions in the traffic stream. If this is done, the upgrade segment is expected to operate at 46 mph with a density of 33 pc/mi/ln, and the downgrade at 50 mph and 23 pc/mi/ln. These solutions, in addition to the analysis worksheet for this calculation, are shown in Figure 7-8.

The downgrade will operate better than the upgrade. Should additional demand cause further deterioration in upgrade operations, a truck climbing lane would be considered. Present operations, however, are stable, because density is still well below the 42-pc/mi/ln boundary for LOS E.

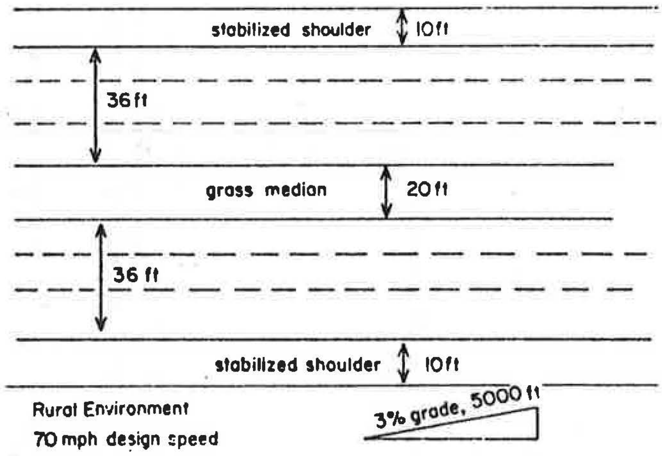


Figure 7-7. Multilane highway for Calculation 2.

CALCULATION 3—DESIGN OF A SUBURBAN MULTILANE HIGHWAY

1. *Description*—A suburban multilane highway is to be designed to carry an expected *DDHV* of 1,800 vph, with 5 percent trucks, and a PHF of 0.90. The driver population will consist primarily of commuters.

The highway is located in an area with generally rolling terrain. The objective is to design for a v/c of 0.75. This corresponds to the AASHTO design recommendation for suburban multilane highways, and is within LOS D (as described herein), but close to the LOS C boundary.

2. *Solution*—The following design standards are assumed for this solution: (a) 12-ft lanes, (b) adequate shoulder clearances, (c) divided highway, and (d) 70-mph design speed.

Then:

$$N = SF/[C_j \times (v/c) \times f_w \times f_{HV} \times f_E \times f_p]$$

where:

SF = 1,800/0.90 = 2,000 vph (Given);
v/c = 0.75 (Given);
c_j = 2,000 pcphpl;
f_w = 1.00 (Table 7-2, ideal conditions);
E_T = 4 (Table 7-3, rolling terrain);
f_{HV} = 0.87 (Table 7-9, $E_T = 4$, 5 percent trucks);
f_E = 0.90 (Table 7-10, suburban divided highway); and
f_p = 1.00 (Table 7-11, commuters).

Then:

$$N = 2{,}000/[2{,}000 \times 0.75 \times 1.0 \times 0.87 \times 0.90 \times 1.00] = 1.7 \text{ lanes}$$

OPERATIONAL ANALYSIS WORKSHEET

Facility Section ALLEN HIGHWAY

Date 1/7/83 Time 8-9 AM (of analysis data)

I. GEOMETRY

10 ft. distance to roadside obstructions

36 ft. ⟵ DIR.1 UP N = 3

20 ft.

36 ft. ⟶ DIR.2 DOWN N = 3

10 ft. distance to roadside obstructions

indicate North

	HWAY Classific. D or U, S or R	Des.Speed (mph)	Lane Width (ft.)	Terrain Type L, R, or M or	Grade (%)	Length (mi)	Median Type
DIR.1	D, R	70	12		3	0.95	GRASS
DIR.2	D, R	70	12		3	0.95	GRASS

II. VOLUMES

	VOL(vph)	PHF	SP=VOL/PHF	%trucks	%buses	%RV's	driver population
DIR.1	2,200	0.85	2,588	10	–	5	__commuter X other
DIR.2	2,200	0.85	2,588	10	–	5	__commuter X other

III. ANALYSIS

$v/c = SP/[c_j \times N \times f_w \times f_{HV} \times f_E \times f_p]$

$c = c_j \times N \times f_w \times f_{HV} \times f_p$

$f_{HV} = 1/[1+P_T(E_T-1)+P_B(E_B-1)+P_R(E_R-1)]$

	$v/c=SP/[c_j$	$\times N$	$\times f_w$	$\times f_E$	$\times f_p$	$\times f_{HV}]$					
			TAB 7.1		TAB 7.2	TAB 7.10	TAB 7.11		E_T	E_B	E_R
DIR.1	0.77	2588	2000	3	1.0	1.0	0.82	0.67	5	–	3
DIR.2	0.62	2588	2000	3	1.0	1.0	0.82	0.85	2.5	–	1.5

	c	v/c	LOS (TAB.7.1)	SPEED (FIG.7.2)	DENSITY (FIG.7.1)
DIR.1	3,296	0.78	D	46	33
DIR.2	4,182	0.62	C	50	23

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Name JENNIFER SMITH Date ______

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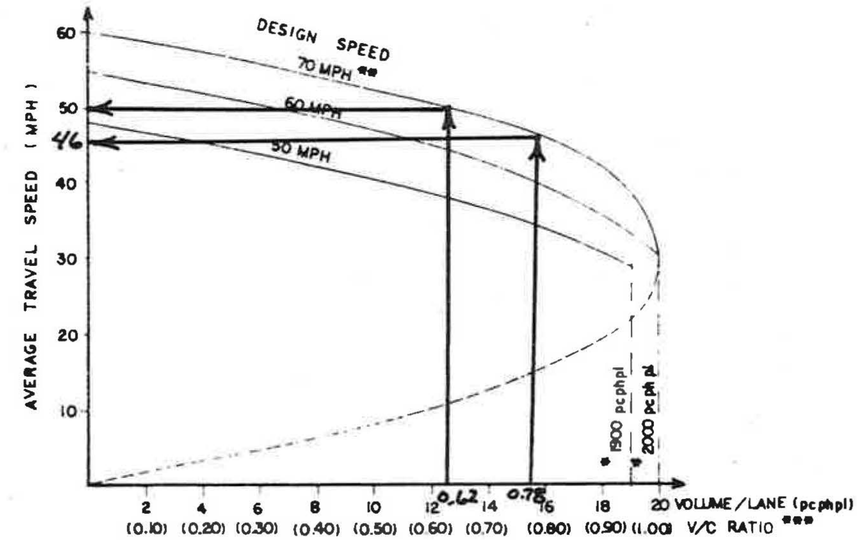
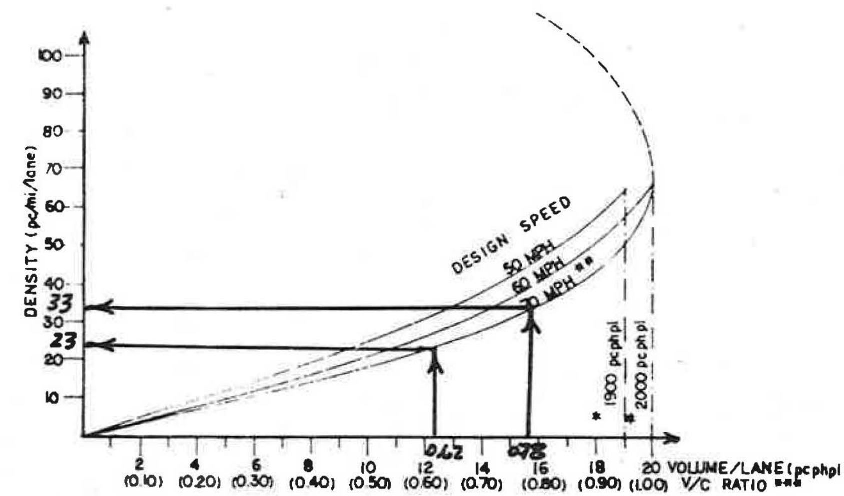


* capacity

** reflects 55 MPH Speed limit

*** v/c ratio based on capacity of 2000 pcphpl, applies only to 60 and 70 MPH design speeds



* capacity

** reflects 55 MPH Speed limit

*** v/c ratio based on capacity of 2000 pcphpl, applies only to 60 and 70 MPH design speeds

Figure 7-8. Illustration of solution to Calculation 2.

Two lanes would be provided in each direction, and a four-lane multilane highway would be built.

Because two lanes in each direction are more than required to meet a v/c objective of 0.75, it would be useful to determine what v/c will actually result. This is done using the operational analysis procedure, and:

$$v/c = SF/[c_j \times N \times f_w \times f_{HV} \times f_E \times f_p]$$
$$v/c = 2{,}000/[2{,}000 \times 2 \times 1.0 \times 0.87 \times 0.90 \times 1.00] = 0.64$$

This yields an operation well within LOS C boundaries, which is considerably better than the minimum originally anticipated.

The same computation could be repeated for an undivided cross section. The only value which changes is f_E, which becomes 0.80. Then:

$$N = 2{,}000/[2{,}000 \times 0.75 \times 1.0 \times 0.87 \times 0.80 \times 1.00] = 1.9 \text{ lanes}$$

Thus, a four-lane highway would still be acceptable. It would not, however, operate as well as the divided cross section:

$$v/c = 2{,}000/[2{,}000 \times 2 \times 1.0 \times 0.87 \times 0.80 \times 1.00] = 0.72$$

This is barely outside the LOS C boundary of 0.71, and is technically LOS D. Figures 7-1 and 7-2 can be entered to determine the difference in operating conditions expected for the divided and undivided alternatives: (a) The *divided* design would operate at an approximate speed of 51 mph and a density of 23 pc/mi/ln. (b) The *undivided* design would operate at an approximate speed of 49 mph and a density of 28 pc/mi/ln.

The estimated operating conditions taken from Figures 7-1 and 7-2 are based on passenger-car streams and on actual values that may vary somewhat because of the presence of trucks in the traffic stream. The comparisons, however, give the designer a good idea of how operations might be affected by the choice of a divided or an undivided cross section. The final decision on this point, of course, depends on many factors. An analysis such as the one illustrated here merely provides some additional information as an input to the decision.

Figure 7-9 illustrates these results, and shows the design worksheet for the problem.

CALCULATION 4—DESIGN OF A RURAL MULTILANE HIGHWAY

1. *Description*—A rural multilane highway segment on a long, steep grade must be designed to accommodate a *DDHV* of 1,000 vph, with 20 percent trucks, and a PHF of 0.85. The driver population is composed of regular users.

The grade in question is 6 percent, and is 1 mi long. Because of the terrain, the design speed on the segment is limited to 60 mph.

2. *Solution*—The grade will require separate analyses of upgrade and downgrade conditions. Lacking local data on downgrade truck speeds, values of E_T will be taken to be one-half the corresponding upgrade values.

The following design criteria are assumed for this problem: (a) 60-mph design speed (given), (b) 12-ft lanes, (c) adequate shoulder clearances, and (d) undivided cross section.

A design v/c should be selected from Table 7-12, with reference to AASHTO recommendations. AASHTO recommends a value of 0.50 for rural multilane highways. From Table 7-12, this is the maximum value for LOS B. This value will, therefore, be used in this design.

Then:

$$N = SF/[c_j \times (v/c) \times f_w \times f_{HV} \times f_E \times f_p]$$

where:

SF = 1,000/0.85 = 1,176 vph (Given);
c_j = 2,000 pcphpl;
v/c = 0.50 (From above discussion);
f_w = 1.00 (Table 7-2, ideal conditions);
E_T (Upgrade) = 9 (4-lane highways); 8 (6-lane highways);
E_T (Downgrade) = 4.5 (4-lane highways); 4 (6-lane highways)
(Table 7.4, 6 percent grade, 1 mi long, 20 percent trucks);
f_{HV} (Upgrade) = 0.38 (4-lane) or 0.42 (6-lane);
f_{HV} (Downgrade) = 0.59 (4-lane) or 0.63 (6-lane) (Table 7-9);
f_E = 0.95 (Table 7-10, rural undivided highway); and
f_p = 1.00 (Table 7-11, regular users).

The values of E_T are dependent on the number of lanes on the multilane highway. Because that is the factor to be determined, a trial-and-error solution is required. The first trial will use the values for a four-lane highway, which produces the most conservative values of f_{HV}.

Then:

$$N\text{ (Upgrade)} = 1{,}176/[2{,}000 \times 0.50 \times 1.0 \times 0.38 \times 0.95 \times 1.00] = 3.3 \text{ lanes}$$
$$N\text{ (Downgrade)} = 1{,}176/[2{,}000 \times 0.50 \times 1.0 \times 0.59 \times 0.95 \times 1.00] = 2.1 \text{ lanes}$$

As both these values are higher than the assumed four-lane highway (two in each direction), a second trial will use values for a six-lane highway (three in each direction).

Then:

$$N\text{ (Upgrade)} = 1{,}176/[2{,}000 \times 0.50 \times 1.0 \times 0.42 \times 0.95 \times 1.00] = 2.9 \text{ lanes}$$
$$N\text{ (Downgrade)} = 1{,}176/[2{,}000 \times 0.50 \times 1.0 \times 0.63 \times 0.95 \times 1.00] = 2.0 \text{ lanes}$$

DESIGN WORKSHEET

Facility Section WASHINGTON ROAD

Date 6/83 Time 5-6 PM (of analysis data)

I. DESIGN CRITERIA

	LOS	v/c (TAB. 7.12)	Highway Classification D or U, S or R	Design Speed	Lane Width	Lateral clear. (ft) roadside	Lateral clear. (ft) median	Terrain Type L, R, or M	Grade or (%)	Length (mi)
DIR.1	D	0.75	D, S	70	12	IDEAL	IDEAL	R		
DIR.2	D	0.75	D, S	70	12	IDEAL	IDEAL	R		

II. TRAFFIC FORECASTS

	DDHV(vph)	PHF	SF=(DDHV/PHF)	%trucks	%buses	%RV's	driver population
DIR.1	1,800	0.90	2,000	5	—	–	X commuter __ other
DIR.2	1,800	0.90	2,000	5	—	–	X commuter __ other

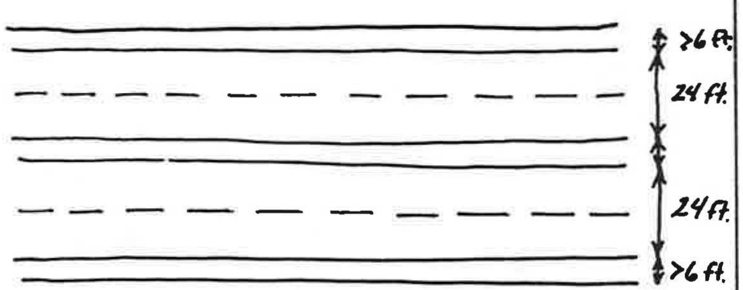
III. DESIGN ANALYSIS

$$\boxed{N = SF/[c_j \times v/c \times f_w \times f_{HV} \times f_E \times f_p]}$$

$$f_{HV} = 1/[1+P_T(E_T-1)+P_B(E_B-1)+P_R(E_R-1)]$$

	N=	SF/	c_j (TAB. 7.1)	x(v/c)	x f_w (TAB. 7.2)	x f_E (TAB. 7.10)	x f_p (TAB. 7.11)	x f_{HV}*	E_T	E_B	E_R
DIR.1	1.7	2000	2000	0.75	1.00	0.90	1.00	0.87	4		
DIR.2	1.7	2000	2000	0.75	1.00	0.90	1.00	0.87	4		

IV. SKETCH DESIGN



* Table 7-9 or compute as shown.

Name ARTHUR JONES Date ____

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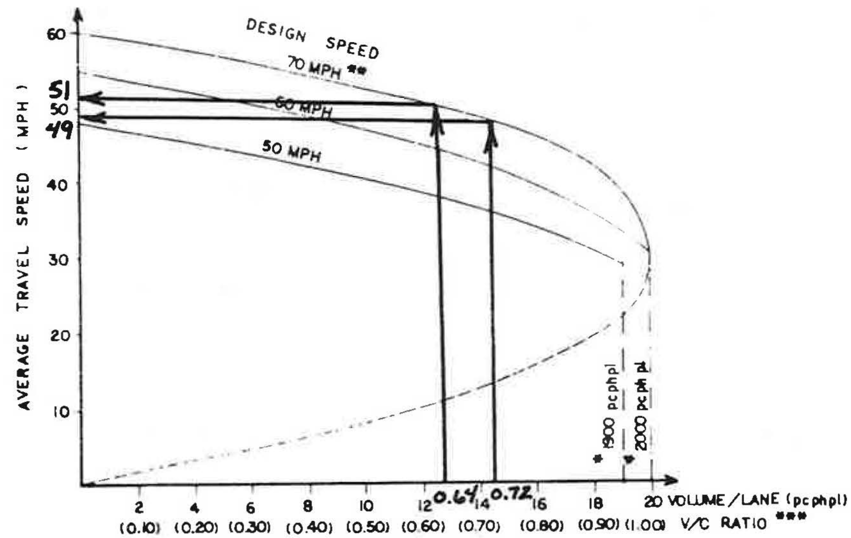
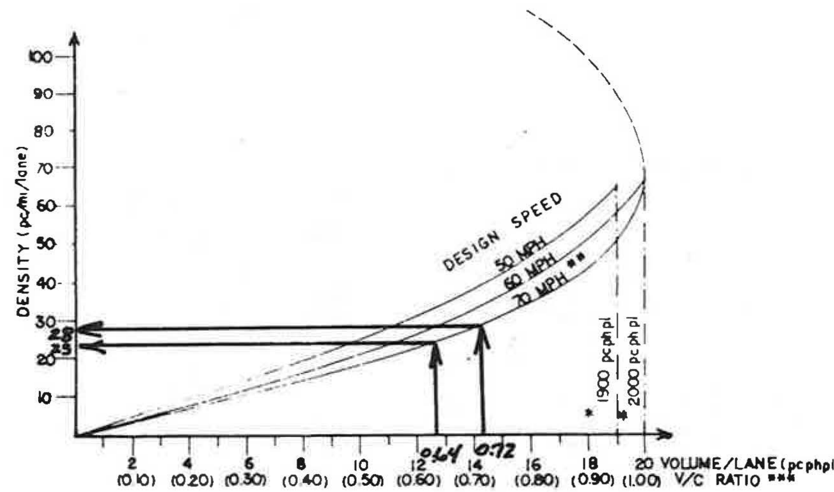


* capacity

** reflects 55 MPH Speed limit

*** v/c ratio based on capacity of 2000 pcphpl, applies only to 60 and 70 MPH design speeds



* capacity

** reflects 55 MPH Speed limit

*** v/c ratio based on capacity of 2000 pcphpl, applies only to 60 and 70 MPH design speeds

Figure 7-9. Illustration of solution to Calculation 3.

The upgrade, therefore, clearly requires three lanes. The downgrade computations are interesting: when two lanes were, in effect, assumed, the computation indicated a need for more than two lanes; when three lanes were assumed, the computation indicated that two lanes were adequate. Because in both cases the exact computation was close to two lanes, that result will be adopted.

Thus, the final design would have three lanes on the upgrade and two lanes on the downgrade, an indication that a truck-climbing lane should be used. If desired, a more precise analysis of the specifics of climbing lane operation may be performed using procedures detailed in Chapter 3.

Figure 7-10 illustrates the design worksheet for this calculation.

CALCULATION 5—A MULTILANE HIGHWAY INTERSECTION, APPROXIMATE ANALYSIS

1. *Description*—The multilane highway described in Calculation 1 and illustrated in Figure 7-5 has a major signalized intersection with a 60-sec cycle length, of which the multilane highway has 40 sec of "green time." Does this intersection appear to be a problem, given the demand described in Calculation 1?

2. *Solution*—This solution will utilize the approximate analysis method described in this chapter.

The capacity of the uninterrupted flow segment approaching the intersection is computed as:

$$c = SF_E = c_j \times N \times (v/c) \times f_w \times f_{HV} \times f_E \times f_p$$
$$c = SF_E = 2{,}000 \times 2 \times 1.00 \times 0.80 \times 0.90 \times 0.80 \times 1.00$$
$$c = 2{,}304 \text{ vph (All values as specified in Calculation 1)}$$

This assumes that there are no turning interferences beyond those normally present on multilane highways, and there are no special geometric features present at the intersection, such as additional through and/or turning lanes.

With these assumptions, the capacity of the intersection approach may be roughly estimated as the capacity of the uninterrupted flow segment times the G/C ratio:

$$c_I = 2{,}304 \times (40/60) = 1{,}536 \text{ vph}$$

The demand flow rate for the intersection approach is 2,198 vph, which greatly exceeds the approximated capacity of the approach.

This indicates that the intersection will present a problem. It also indicates the need for detailed analysis of the intersection, using the techniques presented in Chapter 9. With the more detailed techniques of Chapter 9, the addition of turning lanes, signal phasing, and other design specifics can be evaluated to improve the intersection capacity to required levels.

CALCULATION 6—THREE-LANE RURAL HIGHWAY

1. *Description*—A segment of three-lane highway on an extended grade of 4 percent, 2 mi long, is striped to permit vehicles to exclusively use two lanes in the upgrade direction. The unbalanced striping is continued for several miles beyond the grade. The upgrade carries 800 vph, with 20 percent trucks, and a PHF of 0.80. The segment has 12-ft lanes and adequate lateral clearances, but poor alignment reduces the design speed to 50 mph. The driver population is composed primarily of regular users. How well may the upgrade be expected to operate?

2. *Solution*—The upgrade segment will be approximated as a four-lane multilane highway, with the value of c_j reduced by 15 percent to reflect the reduced efficiency compared to a full four-lane case. The second lane for the upgrade is available for more than 2 mi, which may be considered to be a "significant" distance.

$$v/c = SF/[c_j \times N \times f_w \times f_{HV} \times f_E \times f_p]$$

where:

SF = 800/0.80 = 1,000 vph (Given);
c_j = 1,900 pcphpl (For 50-mph design speed) × 0.85 (The procedure recommends a 10–15 percent reduction in service volumes for three-lane computations);
c_j = 1,615 pcphpl (Assuming a 15 percent reduction);
N = 2;
f_w = 1.0 (Table 7-2, ideal conditions);
E_T = 7 (Table 7-4);
f_{HV} = 0.45 (Table 7-9);
f_E = 0.95 (Table 7-10); and
f_p = 1.00 (Table 7-11).

Then:

$$v/c = 1{,}000/[1{,}615 \times 2 \times 1.0 \times 0.45 \times 0.95 \times 1.00] = 0.72$$

From Table 7-1, this will provide for LOS C operation. It should be noted that this is an approximate analysis when applied to a three-lane highway cross section.

CALCULATION 7—PLANNING APPLICATION

1. *Description*—A planner must determine the most probable size of a multilane highway to be built through a rural area of rolling terrain. The *AADT* is expected to be 15,000 vpd, with 8 percent trucks. The PHF in the region is generally 0.92, and the desired LOS is B. It will be assumed that the highway will be divided.

2. *Solution*—The *AADT* is first converted to an expected *DDHV*. This is computed as:

$$DDHV = AADT \times K \times D$$

where K varies from 0.15 to 0.20 for rural areas (use 0.175), and D is approximately 0.65 for most rural roads.

$$DDHV = 15{,}000 \times 0.175 \times 0.65 = 1{,}706 \text{ vph}$$

From Table 7-13, the per lane service volume for multilane highways in rolling terrain, with 8 percent trucks, at LOS B is:

DESIGN WORKSHEET

Facility Section MAYWOOD ROAD

Date 9/9/83 Time 8-9 AM (of analysis data)

I. DESIGN CRITERIA

	LOS	v/c (TAB. 7.12)	Highway Classification D or U, S or R	Design Speed	Lane Width	Lateral clear. (ft) roadside	median	Terrain Type L, R, or M	Grade or (%)	Length (mi)
DIR.1	B	0.50	U, R	60	12	IDEAL			6	1
DIR.2	B	0.50	U, R	60	12	IDEAL			6	1

II. TRAFFIC FORECASTS

	DDHV(vph)	PHF	SF=(DDHV/PHF)	%trucks	%buses	%RV's	driver population
DIR.1	1,000	0.85	1,176	20	—	—	X commuter __ other
DIR.2	1,000	0.85	1,176	20	—	—	X commuter __ other

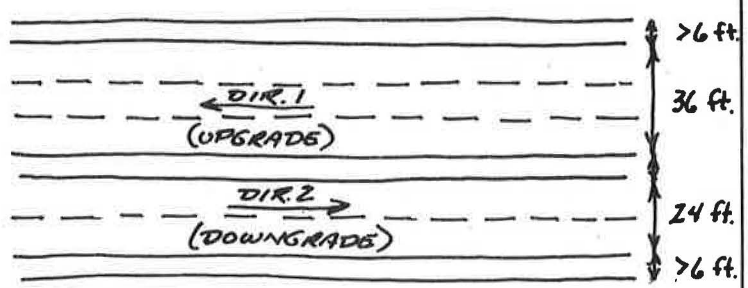
III. DESIGN ANALYSIS

$$N = SF/[c_j \times v/c \times f_w \times f_{HV} \times f_E \times f_p]$$

$$f_{HV} = 1/[1+P_T(E_T-1)+P_B(E_B-1)+P_R(E_R-1)]$$

	N=	SF/	c_j x (TAB. 7.1)	(v/c) x	f_w x (TAB. 7.2)	f_E x (TAB. 7.10)	f_p x (TAB. 7.11)	* f_{HV}	E_T	E_B	E_R
DIR.1	3.3 or 3.0	1176	2000	0.50	1.0	0.95	1.0	0.38 or 0.42	9 or 8	—	—
DIR.2	2.1 or 2.0	1176	2000	0.50	1.0	0.95	1.0	0.59 or 0.63	4.5 or 4	—	—

IV. SKETCH DESIGN



* Table 7-9 or compute as shown.

Name ELIZABETH DURAN Date

Checked by

Figure 7-10. Worksheet for Calculation 4.

$SFL_B = 900$ vphpl

and:

$$N = DDHV/[SFL \times f_E \times \text{PHF}]$$
$$N = 1{,}706/[900 \times 1.00 \times 0.92] = 2.1 \text{ lanes}$$

Therefore, to maintain a minimum of LOS B, three lanes would be needed in each direction. Since the requirement is only 0.1 lanes over 2, however, a four-lane highway would be seriously considered in the design process. At that point, specific operational analyses could be performed to evaluate the use of two vs. three lanes on a segment-by-segment basis.

APPENDIX I

TABLES, FIGURES, AND WORKSHEETS FOR USE IN THE ANALYSIS OF MULTILANE HIGHWAYS

TABLE 7-1. LEVEL-OF-SERVICE CRITERIA FOR MULTILANE HIGHWAYS

LEVEL OF SERVICE	DENSITY (PC/MI/LN)	70 MPH DESIGN SPEED			60 MPH DESIGN SPEED			50 MPH DESIGN SPEED		
		SPEED[a] (MPH)	v/c	*MSF*[b] (PCPHPL)	SPEED[a] (MPH)	v/c	*MSF*[b] (PCPHPL)	SPEED[a] (MPH)	v/c	*MSF*[b] (PCPHPL)
A	$\leq$ 12	$\geq$ 57	0.36	700	$\geq$ 50	0.33	650	—	—	—
B	$\leq$ 20	$\geq$ 53	0.54	1,100	$\geq$ 48	0.50	1,000	$\geq$ 42	0.45	850
C	$\leq$ 30	$\geq$ 50	0.71	1,400	$\geq$ 44	0.65	1,300	$\geq$ 39	0.60	1,150
D	$\leq$ 42	$\geq$ 40	0.87	1,750	$\geq$ 40	0.80	1,600	$\geq$ 35	0.76	1,450
E	$\leq$ 67	$\geq$ 30	1.00	2,000	$\geq$ 28	1.00	2,000	$\geq$ 30	1.00	1,900
F	$>$ 67	$<$ 30	[c]	[c]	$<$ 28	[c]	[c]	$<$ 30	[c]	[c]

[a] Average travel speed.
[b] Maximum rate of flow per lane under ideal conditions, rounded to the nearest 50 pcphpl.
[c] Highly variable.

Table 7-2. Adjustment Factor for Restricted Lane Width and Lateral Clearance

Distance from Edge of Traveled Way to Obstruction[a] (ft)	Adjustment Factor, f_w: Obstruction on One Side of Roadway[b] — Lane Width (ft) 12	11	10	9	Obstruction on Both Sides of Roadway[c] — Lane Width (ft) 12	11	10	9
4-Lane Divided Multilane Highways (2 Lanes Each Direction)								
≥ 6	1.00	0.97	0.91	0.81	1.00	0.97	0.91	0.81
4	0.99	0.96	0.90	0.80	0.98	0.95	0.89	0.79
2	0.97	0.94	0.88	0.79	0.94	0.91	0.86	0.76
0	0.90	0.87	0.82	0.73	0.81	0.79	0.74	0.66
6-Lane Divided Multilane Highways (3 Lanes Each Direction)								
≥ 6	1.00	0.96	0.89	0.78	1.00	0.96	0.89	0.78
4	0.99	0.95	0.88	0.77	0.98	0.94	0.87	0.77
2	0.97	0.93	0.87	0.76	0.96	0.92	0.85	0.75
0	0.94	0.91	0.85	0.74	0.91	0.87	0.81	0.70
4-Lane Undivided Multilane Highways (2 Lanes Each Direction)								
≥ 6	1.00	0.95	0.89	0.77	NA	NA	NA	NA
4	0.98	0.94	0.88	0.76	NA	NA	NA	NA
2	0.95	0.92	0.86	0.75	0.94	0.91	0.86	NA
0	0.88	0.85	0.80	0.70	0.81	0.79	0.74	0.66
6-Lane Undivided Multilane Highways (3 Lanes Each Direction)								
≥ 6	1.00	0.95	0.89	0.77	NA	NA	NA	NA
4	0.99	0.94	0.88	0.76	NA	NA	NA	NA
2	0.97	0.93	0.86	0.75	0.96	0.92	0.85	NA
0	0.94	0.90	0.83	0.72	0.91	0.87	0.81	0.70

[a] Use the average distance to obstruction on "both sides" where the distance to obstructions on the left and right differs.
[b] Factors for one-sided obstructions allow for the effect of opposing flow.
[c] Two-sided obstructions include one roadside and one median obstruction. Median obstruction may exist in the median of a divided multilane highway or in the center of an undivided highway which periodically divides to go around bridge abutments or other center objects.
NA = Not applicable; use factor for one-sided obstruction.

Table 7-3. Passenger-Car Equivalents on Extended General Multilane Highway Segments

Factor	Type of Terrain: Level	Rolling	Mountainous
E_T for Trucks	1.7	4.0	8.0
E_B for Buses	1.5	3.0	5.0
E_R for RV's	1.6	3.0	4.0

Table 7-4. Passenger-Car Equivalents for Typical Trucks (200 lb/hp)

Grade (%)	Length (mi)	Passenger-Car Equivalent, E_T: 4-Lane Highways								6-Lane Highways							
Percent Trucks		2	4	5	6	8	10	15	20	2	4	5	6	8	10	15	20
< 1	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
1	0–1/2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	1/2–1	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	≥ 1	4	3	3	3	3	3	3	3	4	3	3	3	3	3	3	3
2	0–1/4	4	4	4	3	3	3	3	3	4	4	4	3	3	3	3	3
	1/4–1/2	5	4	4	3	3	3	3	3	5	4	4	3	3	3	3	3
	1/2–3/4	6	5	5	4	4	4	4	4	6	5	5	4	4	4	4	4
	3/4–1½	7	6	6	5	4	4	4	4	7	5	5	5	4	4	4	4
	≥ 1½	8	6	6	6	5	5	4	4	8	6	6	5	4	4	4	4
3	0–1/4	6	5	5	5	4	4	4	3	6	5	5	5	4	4	4	3
	1/4–1/2	8	6	6	6	5	5	5	4	7	6	6	6	5	5	5	4
	1/2–1	9	7	7	6	5	5	5	5	9	7	7	6	5	5	5	5
	1–1½	9	7	7	7	6	6	5	5	9	7	7	6	5	5	5	5
	≥ 1½	10	7	7	7	6	6	5	5	10	7	7	6	5	5	5	5
4	0–1/4	7	6	6	5	4	4	4	4	7	6	6	5	4	4	4	4
	1/4–1/2	10	7	7	6	5	5	5	5	9	7	7	6	5	5	5	5
	1/2–1	12	8	8	7	6	6	6	6	10	8	7	6	5	5	5	5
	≥ 1	13	9	9	9	8	8	7	7	11	9	9	8	7	6	6	6
5	0–1/4	8	6	6	6	5	5	5	5	8	6	6	6	5	5	5	5
	1/4–1/2	10	8	8	7	6	6	6	6	8	7	7	6	5	5	5	5
	1/2–1	12	11	11	10	8	8	8	8	12	10	9	8	7	7	7	7
	≥ 1	14	11	11	10	8	8	8	8	12	10	9	8	7	7	7	7
6	0–1/4	9	7	7	7	6	6	6	6	9	7	7	6	5	5	5	5
	1/4–1/2	13	9	9	8	7	7	7	7	11	8	8	7	6	6	6	6
	1/2–3/4	13	9	9	8	7	7	7	7	11	9	9	8	7	6	6	6
	≥ 3/4	17	12	12	11	9	9	9	9	13	10	10	9	8	8	8	8

NOTE: If the length of grade falls on a boundary value, use the equivalent for the longer grade class. Any grade steeper than the percent stated must use the next higher grade category.

TABLE 7-5. PASSENGER-CAR EQUIVALENTS FOR LIGHT TRUCKS (100 LB/HP)

GRADE (%)	LENGTH (MI)	PASSENGER-CAR EQUIVALENT, E_T															
		4-LANE HIGHWAYS								6-LANE HIGHWAYS							
PERCENT TRUCKS		2	4	5	6	8	10	15	20	2	4	5	6	8	10	15	20
≤2	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
3	0–1/4	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	1/4–1/2	4	4	4	3	3	3	3	3	4	4	4	3	3	3	3	3
	1/2–3/4	4	4	4	4	3	3	3	3	4	4	4	3	3	3	3	3
	3/4–1	5	4	4	4	4	3	3	3	5	4	4	4	3	3	3	3
	> 1	6	5	5	5	4	4	4	3	6	5	5	4	4	4	3	3
4	0–1/4	4	4	4	3	3	3	3	3	5	4	4	4	3	3	3	3
	1/4–1/2	5	5	5	4	4	4	4	4	5	4	4	4	4	4	4	4
	1/2–1	6	5	5	5	4	4	4	4	6	5	5	4	4	4	4	4
	> 1	7	6	6	5	4	4	4	4	7	5	5	5	4	4	4	4
5	0–1/4	6	5	5	5	4	4	4	3	6	5	5	5	4	4	4	3
	1/4–1	8	7	7	6	5	5	5	5	8	7	7	6	5	5	5	5
	> 1	9	7	7	6	5	5	5	5	8	7	7	6	5	5	5	5
6	0–1/4	7	5	5	5	4	4	4	4	7	5	5	5	4	4	3	3
	1/4–1	9	7	7	6	5	5	5	5	8	7	7	6	5	5	5	5
	> 1	9	7	7	7	6	6	5	5	9	7	7	6	5	5	5	5

NOTE: If a length of grade falls on a boundary value, use the equivalent for the longer grade category. Any grade steeper than the percent shown must use the next higher grade category.

Table 7-6. Passenger-Car Equivalents for Heavy Trucks (300 lb/hp)

Grade (%)	Length (mi)	Passenger-Car Equivalent, E_T															
		4-Lane Highways								6-Lane Highways							
Percent Trucks		2	4	5	6	8	10	15	20	2	4	5	6	8	10	15	20
< 1	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
1	0–1/4	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	1/4–1/2	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
	1/2–3/4	4	4	4	4	3	3	3	3	4	4	4	3	3	3	3	3
	3/4–1	5	4	4	4	3	3	3	3	5	4	4	4	3	3	3	3
	1–1½	6	5	5	5	4	4	4	3	6	5	5	4	4	4	3	3
	> 1½	7	5	5	5	4	4	4	3	7	5	5	5	4	4	3	3
2	0–1/4	4	4	4	3	3	3	3	3	4	4	4	3	3	3	3	3
	1/4–1/2	7	6	6	5	4	4	4	4	7	5	5	5	4	4	4	4
	1/2–3/4	8	6	6	5	5	4	4	4	8	6	6	6	5	5	4	4
	3/4–1	8	6	6	6	5	5	5	5	8	6	6	6	5	5	5	5
	1–1½	9	7	7	7	6	6	5	5	9	7	7	6	5	5	5	5
	> 1½	10	7	7	7	6	6	5	5	10	7	7	6	5	5	5	5
3	0–1/4	6	5	5	5	4	4	4	3	6	5	5	5	4	4	4	3
	1/4–1/2	9	7	7	6	5	5	5	5	8	7	7	6	5	5	5	5
	1/2–3/4	12	8	8	7	6	6	6	6	10	8	7	6	5	5	5	5
	3/4–1	13	9	9	8	7	7	7	7	11	8	8	7	6	6	6	6
	> 1	14	10	10	9	8	8	7	7	12	9	9	8	7	7	7	7
4	0–1/4	7	5	5	5	4	4	4	4	7	5	5	5	4	4	3	3
	1/4–1/2	12	8	8	7	6	6	6	6	10	8	7	6	5	5	5	5
	1/2–3/4	13	9	9	8	7	7	7	7	11	9	9	8	7	6	6	6
	3/4–1	15	10	10	9	8	8	8	8	12	10	10	9	8	7	7	7
	> 1	17	12	12	10	9	9	9	9	13	10	10	9	8	8	8	8
5	0–1/4	8	6	6	6	5	5	5	5	8	6	6	6	5	5	5	5
	1/4–1/2	13	9	9	8	7	7	7	7	11	8	8	7	6	6	6	6
	1/2–3/4	20	15	15	14	11	11	11	11	14	11	11	10	9	9	9	9
	> 3/4	22	17	17	16	13	13	13	13	17	14	14	13	12	11	11	11
6	0–1/4	9	7	7	7	6	6	6	6	9	7	7	6	5	5	5	5
	1/4–1/2	17	12	12	11	9	9	9	9	13	10	10	9	8	8	8	8
	> 1/2	28	22	22	21	18	18	18	18	20	17	17	16	15	14	14	14

NOTE: If the length of grade falls on a boundary value, the equivalent corresponding to the longer grade category is used. Any grade steeper than the percent shown must use the next higher grade category.

Table 7-7. Passenger-Car Equivalents for Recreational Vehicles

Grade (%)	Length (mi)	E_R															
		4-Lane Highways								6-Lane Highways							
Percent RV's		2	4	5	6	8	10	15	20	2	4	5	6	8	10	15	20
< 2	All	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
3	0–1/2	3	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	≥ 1/2	4	3	3	3	3	3	3	3	4	3	3	3	3	3	3	3
4	0–1/4	3	2	2	2	2	2	2	2	3	2	2	2	2	2	2	2
	1/4–3/4	4	3	3	3	3	3	3	3	4	3	3	3	3	3	3	3
	≥ 3/4	5	4	4	4	3	3	3	3	4	4	4	4	3	3	3	3
5	0–1/4	4	3	3	3	3	3	3	3	4	3	3	3	2	2	2	2
	1/4–3/4	5	4	4	4	4	4	4	4	5	4	4	4	4	4	4	4
	≥ 3/4	6	5	4	4	4	4	4	4	5	5	4	4	4	4	4	4
6	0–1/4	5	4	4	4	3	3	3	3	5	4	4	3	3	3	3	3
	1/4–3/4	6	5	5	4	4	4	4	4	6	4	4	4	4	4	4	4
	≥ 3/4	7	6	6	6	5	5	5	5	6	5	5	5	4	4	4	4

NOTE: If a length of grade falls on a boundary condition, the equivalent from the longer grade class is used. Any grade steeper than the percent shown must use the next higher grade category.

Table 7-8. Passenger-Car Equivalents for Buses

Grade	E_B
0–3	1.6
4[a]	1.6
5[a]	3.0
6[a]	5.5

[a] Use generally restricted to grades more than 1/4 mi long.

TABLE 7-9. ADJUSTMENT FACTOR FOR THE EFFECT OF TRUCKS, BUSES, OR RECREATIONAL VEHICLES IN THE TRAFFIC STREAM

PCE[a]	ADJUSTMENT FACTOR, f_{HV}														
E_T E_R E_B	PERCENTAGE OF TRUCKS, P_T; RV'S, P_R; or BUSES, P_B														
	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
16	0.87	0.77	0.69	0.63	0.57	0.53	0.49	0.45	0.43	0.40	0.36	0.32	0.29	0.27	0.25
17	0.86	0.76	0.68	0.61	0.56	0.51	0.47	0.44	0.41	0.38	0.34	0.31	0.28	0.26	0.24
18	0.85	0.75	0.66	0.60	0.54	0.49	0.46	0.42	0.40	0.37	0.33	0.30	0.27	0.25	0.23
19	0.85	0.74	0.65	0.58	0.53	0.48	0.44	0.41	0.38	0.36	0.32	0.28	0.26	0.24	0.22
20	0.84	0.72	0.64	0.57	0.51	0.47	0.42	0.40	0.37	0.34	0.30	0.27	0.25	0.23	0.21
21	0.83	0.71	0.63	0.56	0.50	0.45	0.41	0.38	0.36	0.33	0.29	0.26	0.24	0.22	0.20
22	0.83	0.70	0.61	0.54	0.49	0.44	0.40	0.37	0.35	0.32	0.28	0.25	0.23	0.21	0.19
23	0.82	0.69	0.60	0.53	0.48	0.43	0.39	0.36	0.34	0.31	0.27	0.25	0.22	0.20	0.19
24	0.81	0.68	0.59	0.52	0.47	0.42	0.38	0.35	0.33	0.30	0.27	0.24	0.21	0.19	0.18
25	0.80	0.67	0.58	0.51	0.46	0.41	0.37	0.34	0.32	0.29	0.26	0.23	0.20	0.18	0.17

[a] Passenger-car equivalent, obtained from Table 7-3, 7-4, 7-5, or 7-6.

TABLE 7-10. ADJUSTMENT FACTOR FOR TYPE OF MULTILANE HIGHWAY AND DEVELOPMENT ENVIRONMENT, f_E

TYPE	DIVIDED	UNDIVIDED
Rural	1.00	0.95
Suburban	0.90	0.80

TABLE 7-11. ADJUSTMENT FACTOR FOR DRIVER POPULATION

DRIVER POPULATION	FACTOR, f_p
Commuter, or Other Regular Users	1.00
Recreational, or Other Nonregular Users	0.75–0.90

Table 7-12. Volume-to-Capacity Values for Use in Design of Multilane Highways

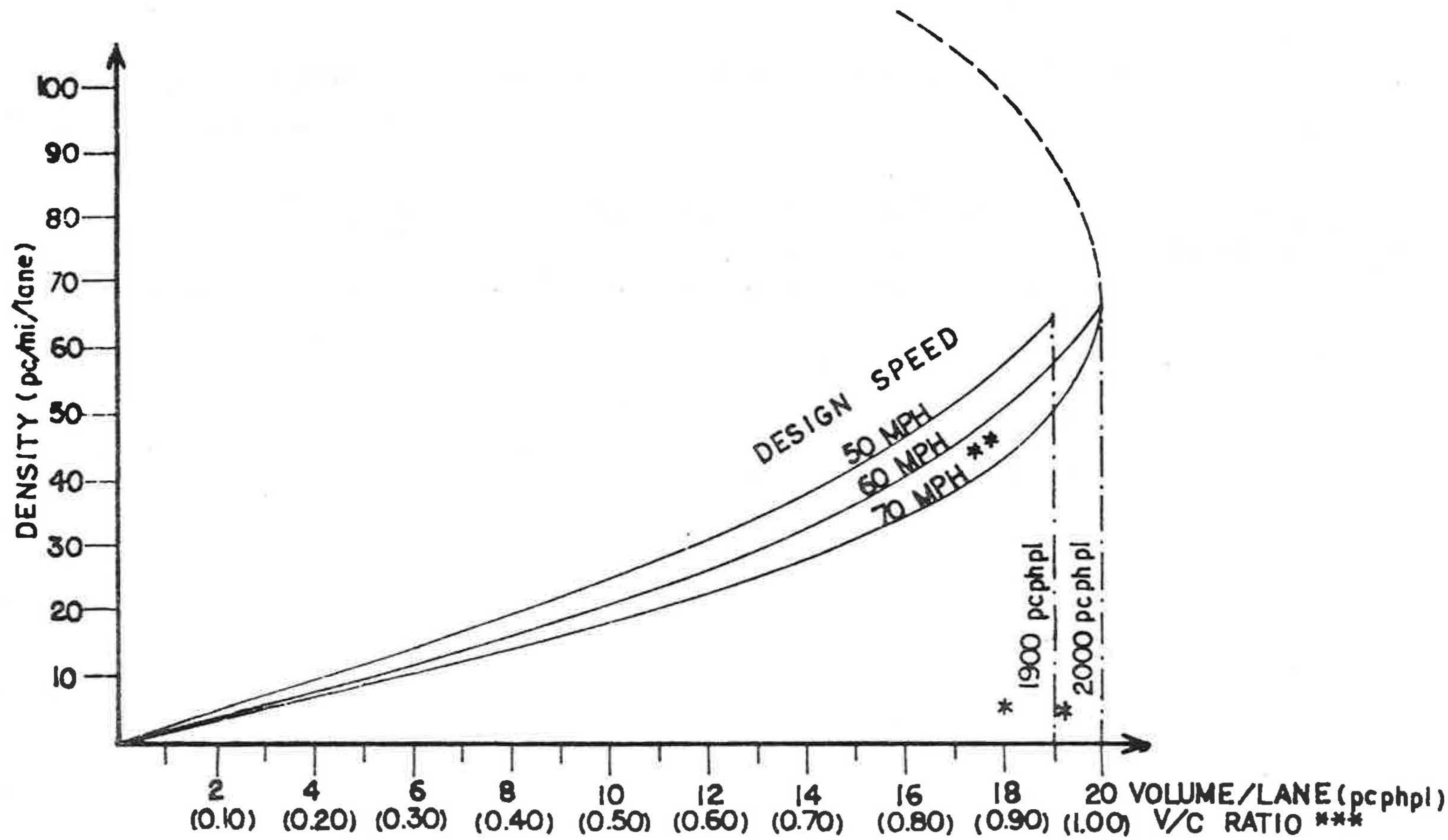
v/c Ratio	*MSF*[c]	Resulting Performance Parameters		
		LOS[a]	Density (pc/mi/ln)	Speed (mph)
		70-mph Elements		
0.30	600	A	10.5	57
0.36[b]	700	A	12.0	57
0.40	800	B	14.0	56
0.50	1,000	B	18.0	54
0.54[b]	1,100	B	20.0	53
0.60	1,200	C	22.0	52
0.70	1,400	C	28.0	50
0.71[b]	1,400	C	30.0	50
0.80	1,600	D	34.0	47
		60-mph Elements		
0.30	600	A	11.5	51
0.33[b]	650	A	12.0	50
0.40	800	B	15.5	49
0.50[b]	1,000	B	20.0	48
0.60	1,200	C	25.0	45
0.65[b]	1,300	C	30.0	44
0.70	1,400	D	32.0	42
0.80[b]	1,600	D	40.0	40
		50-mph Elements		
0.30	550	B	13.0	43
0.40	750	B	17.5	42
0.45[b]	850	B	20.0	42
0.50	950	C	24.0	41
0.60[b]	1,150	C	30.0	39
0.70	1,350	D	38.0	37
0.76[b]	1,450	D	42.0	35
0.80	1,500	E	46.5	34

[a] Design may be within a LOS.
[b] Maximum threshold v/c for LOS shown.
[c] Rounded to the nearest 50 pcphpl.

TABLE 7-13. SERVICE FLOW RATE PER LANE FOR PLANNING APPLICATIONS (DESIGN SPEED = 70 MPH ONLY)

LOS	PERCENT TRUCKS									
	0	2	4	5	6	8	10	12	15	20
	LEVEL TERRAIN[a]									
A	700	700	700	700	650	650	650	650	650	600
B	1,100	1,100	1,050	1,050	1,050	1,050	1,000	1,000	1,000	1,000
C	1,400	1,400	1,350	1,350	1,350	1,350	1,300	1,300	1,250	1,250
D	1,750	1,750	1,700	1,700	1,650	1,650	1,650	1,600	1,600	1,550
E	2,000	2,000	1,950	1,950	1,900	1,900	1,850	1,850	1,800	1,750
	ROLLING TERRAIN[a]									
A	700	650	600	600	600	550	550	500	500	500
B	1,100	1,050	1,000	950	950	900	850	800	800	700
C	1,400	1,300	1,250	1,200	1,200	1,150	1,100	1,050	1,000	900
D	1,750	1,650	1,550	1,500	1,500	1,400	1,350	1,300	1,250	1,100
E	2,000	1,900	1,800	1,750	1,700	1,600	1,550	1,500	1,450	1,250
	MOUNTAINOUS TERRAIN[a]									
A	700	600	550	500	500	450	400	400	350	300
B	1,100	950	850	800	700	700	650	600	550	450
C	1,400	1,250	1,100	1,050	1,000	900	850	750	700	600
D	1,750	1,550	1,350	1,300	1,250	1,100	1,050	950	850	750
E	2,000	1,750	1,550	1,500	1,400	1,250	1,200	1,100	1,000	850

[a] All values rounded to the nearest 50 vphpl.

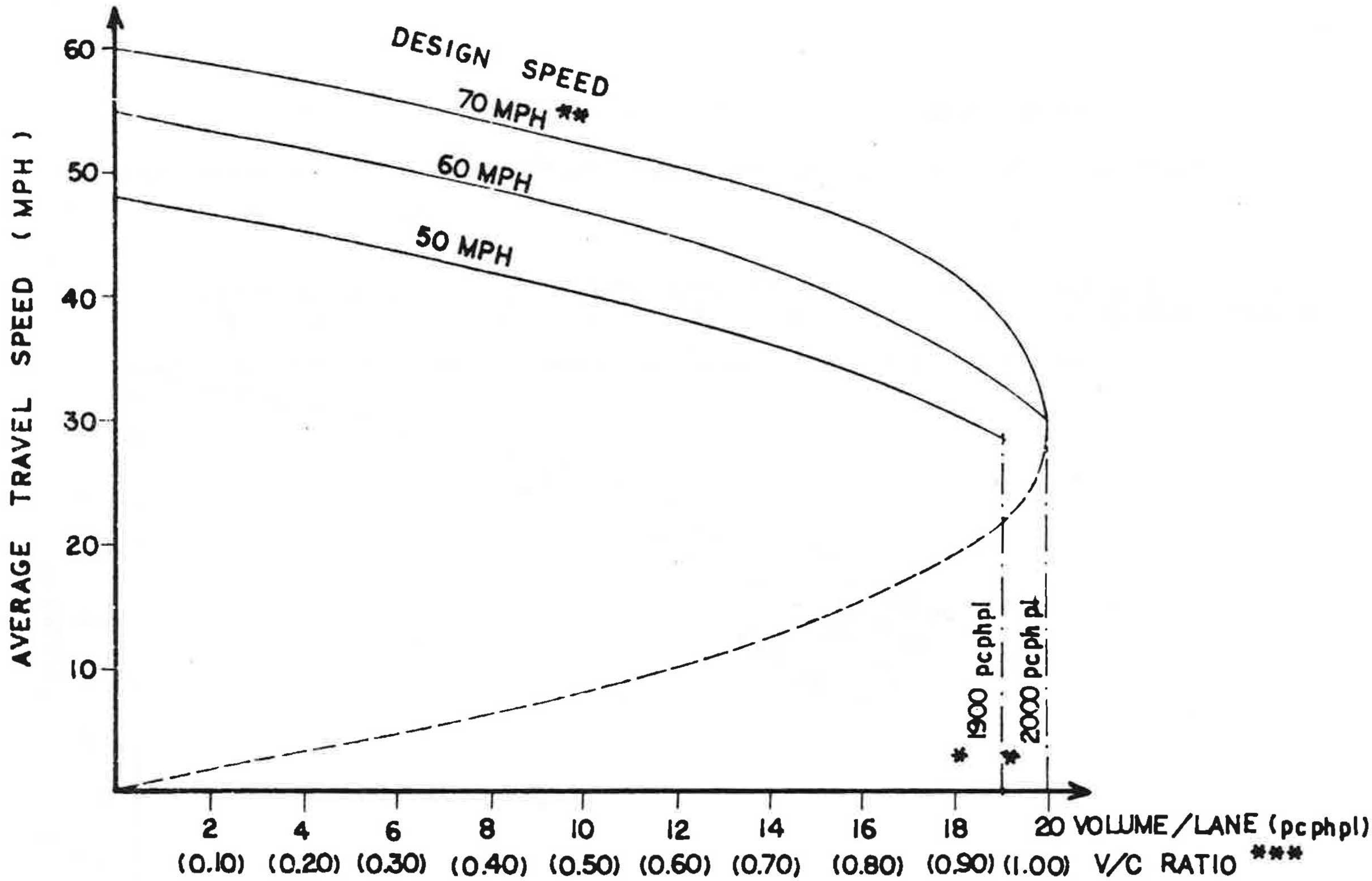


* capacity

** reflects 55 MPH speed limit

*** v/c ratio based on capacity of 2000 pcphpl, applies only to 60 and 70 MPH design speeds

Figure 7-1. Density flow characteristics for uninterrupted flow segments of multilane highways.



* capacity

** reflects 55 MPH Speed limit

*** v/c ratio based on capacity of 2000 pcphpl, applies only to 60 and 70 MPH design speeds

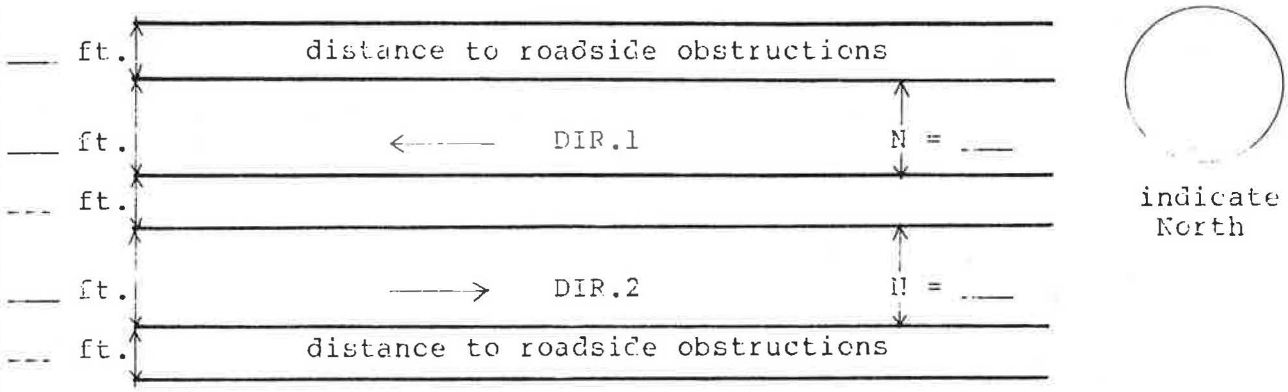
Figure 7-2. Speed-flow characteristics for uninterrupted flow segments of multilane highways.

OPERATIONAL ANALYSIS WORKSHEET

Facility Section ____

Date ______________________ Time ______________(of analysis data)

I. GEOMETRY



	HWAY Classific. D or U, S or R	Des.Speed (mph)	Lane Width (ft.)	Terrain Type L, R, or M or	Grade (%)	Length (mi)	Median Type
DIR.1							
DIR.2							

II. VOLUMES

	VOL(vph)	PHF	SF=VOL/PHF	%trucks	%buses	%RV's	driver population
DIR.1							__commuter__other
DIR.2							__commuter__other

III. ANALYSIS

$$v/c = SF/[c_j \times N \times f_w \times f_{HV} \times f_E \times f_P]$$

$$c = c_j \times N \times f_w \times f_{HV} \times f_P$$

$$v/c=SF/[c_j \times N \times f_w \times f_E \times f_p \times f_{HV}]$$

$$f_{HV}= 1/[1+P_T(E_T-1)+P_B(E_B-1)+P_R(E_R-1)]$$

	c_j (TAB. 7.1)	N	f_w (TAB. 7.2)	f_E (TAB. 7.10)	f_p (TAB 7.11)	f_{HV}	E_T	E_B	E_R
DIR.1									
DIR.2									

	c	v/c	LOS (TAB.7.1)	SPEED (FIG.7.2)	DENSITY (FIG.7.1)
DIR.1					
DIR.2					

Comments on back of page.

Name ______________________________ Date ________________

Checked by ____

DESIGN WORKSHEET

Facility Section ____

Date ______________________________ Time ______________ (of analysis data)

I. DESIGN CRITERIA

	LOS	v/c (TAB. 7.12)	Highway Classification D or U, S or R	Design Speed	Lane Width	Lateral clear. (ft) roadside	median	Terrain Type L, R, or M	Grade or (%)	Length (mi)
DIR.1										
DIR.2										

II. TRAFFIC FORECASTS

	DDHV (vph)	PHF	SF=(DDHV/PHF)	%trucks	%buses	%RV's	driver population
DIR.1							__commuter__other
DIR.2							__commuter__other

III. DESIGN ANALYSIS

$$\boxed{N = SF/[c_j \times v/c \times f_w \times f_{HV} \times f_E \times f_p]}$$

	N=	SF/	c_j x (TAB. 7.1)	(v/c) x	f_w x (TAB. 7.2)	f_E x (TAB. 7.10)	f_p x (TAB. 7.11)	* f_{HV}	f_{HV} = 1/[1+$P_T(E_T-1)+P_B(E_B-1)+P_R(E_R-1)$] E_T	E_B	E_R
DIR.1											
DIR.2											

IV. SKETCH DESIGN

* Table 7-9 or compute as shown.

Name ____ Date ________________

Checked by ____

APPENDIX II

GLOSSARY AND SYMBOLS

GLOSSARY

cycle length—The total time it takes a traffic signal to time through all of its phases and indications once; the time between the initiation of any given phase, and the initiation of that phase for a second time.

green time—The portion of a signal cycle which provides for movement of vehicles on a given approach or portion of an approach.

multilane highway—A highway with at least two lanes for the exclusive use of traffic in each direction, with no or partial control of access, that may have periodic interruptions to flow at signalized intersections.

SYMBOLS

c_I the approximate capacity of a multilane intersection approach, in vehicles per hour.

f_E the multiplicative adjustment factor for type of multilane highway and development environment.

G/C the ratio of green time to cycle length for a given intersection approach.

CHAPTER 10

UNSIGNALIZED INTERSECTIONS

CONTENTS

I. INTRODUCTION

This chapter contains procedures for the capacity analysis of unsignalized intersections. It presents a specific methodology for two-way STOP- and YIELD-controlled intersections. These procedures are *not* intended for use in the analysis of four-way STOP-controlled intersections or totally uncontrolled intersections. Because the procedure is based on the use of gaps in a major traffic stream by vehicles crossing or turning through that stream, it requires that the right-of-way be clearly assigned and that the movements seeking gaps remain unchanged. In uncontrolled or four-way STOP-controlled cases, each movement seeks gaps in other conflicting streams, creating a selection process quite different from that at two-way STOP- or YIELD-controlled intersections. Consequently, the methodology of this chapter is inappropriate for use in these cases. Capacity data and information concerning four-way STOP-controlled intersections are presented in a separate section of this chapter.

Procedures for the capacity analysis of two-way STOP- and YIELD-controlled intersections are based on a German method originally published in 1972 *(1)* and translated in a 1974 publication of the Organization for Economic Cooperation and Development (OECD) *(2)*. The method has been modified based on a limited number of validation studies, in the United States, conducted by the Unsignalized Intersection Subcommittee of the Highway Capacity and Quality of Service Committee of the Transportation Research Board.

Unsignalized intersections make up the vast majority of at-grade junctions in any street system. STOP and YIELD signs are used to assign the right-of-way to one street at such intersections. This designation forces drivers on the controlled street to judgmentally select gaps in the major street flow through which to execute crossing or turning maneuvers. Thus, the capacity of the controlled legs is based on two factors:

1. The distribution of gaps in the major street traffic stream.
2. Driver judgment in selecting gaps through which to execute their desired maneuvers.

Computational procedures depend on both factors: gap distributions in conflicting traffic streams and the gap acceptance behavior of drivers at such intersections.

It is assumed that gaps in conflicting streams are randomly distributed. For this reason, the procedure will be less reliable in situations in which conflicting flows are strongly platooned, as would be the case at many urban intersections where the major street is part of a signalized network.

The impact of progression on the gap distribution in a major traffic stream can vary substantially. On one-way arterials, there will be periodic large gaps between platoons through which minor street traffic may easily execute crossing and/or turning movements. Such a condition is likely to permit higher side-street capacities and better operations than the random arrivals assumed by the methodology of this chapter.

On two-way arterials, side street traffic may face a wide range of conditions. Platoons arrive in *two* directions on the major street. They may arrive such that considerable gaps exist between platoons, or they may arrive in a staggered fashion (first from one direction, then the other). In the former case, side street crossings will be easier to make than in the latter case, where the crossing vehicle is faced with a virtually endless platoon.

Consider the case shown on Figure 10-1. Presented in the form of a time-space diagram, the figure depicts two opposing platoons moving through a segment of an arterial. Depending on the position of the cross street with respect to these flows, the gap distribution differs substantially. Vehicle 1, on Figure 10-1, attempts to cross at a location where there is virtually no gap in arriving platoons. Just as the NB platoon ends, the beginning of the SB platoon arrives, and vice-versa. Vehicle 2 has a more favorable condition. At that location, NB and SB platoons arrive at the same time, and there are substantial gaps *between* platoon arrivals that crossing vehicles may use. The impacts of platoons can be quite complex, and they depend on the percentage of major street traffic arriving in platoons, the major street flow rates within and between platoons, and other factors.

The effects of platoon flow on the major street may be qualitatively considered when reviewing the results of analyses using the methodology presented in this chapter. Where greater precision is desired, Appendix I presents a rational procedure for extending this methodology to platoon flow that makes use of time-space diagrams and platoon flow rates.

Illustrations 10-1 through 10-3 depict typical intersections controlled by STOP and YIELD signs. The choice between STOP and YIELD control is generally specified by state and/or local standards, and is generally based on approach speed, sight distance considerations, and other factors.

This chapter introduces a variety of new terminology applying to the unique characteristics of unsignalized intersection capacity. For clarity, these terms are introduced and defined when used in the following sections.

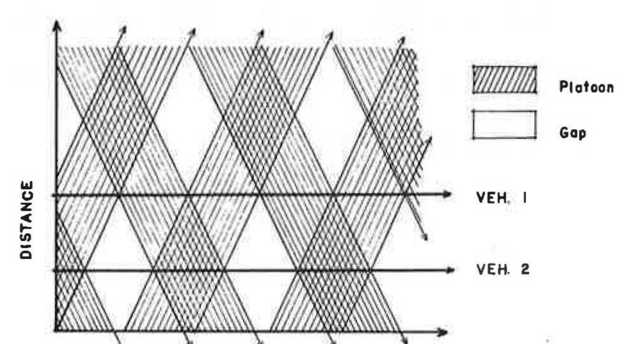


Figure 10-1. Impacts of platoon flow on gap distribution.

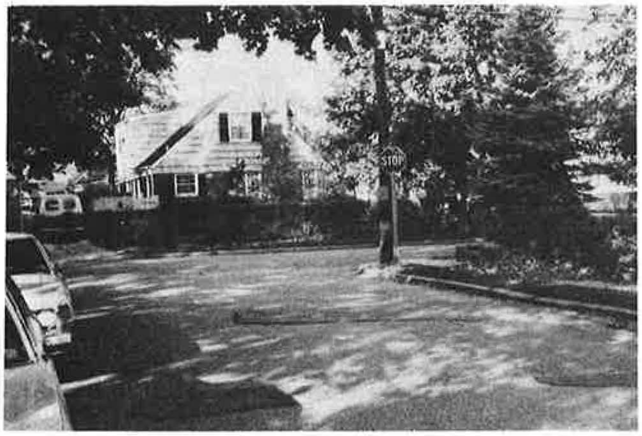


Illustration 10-1. STOP *signs control this intersection of two low-volume streets in a residential area.*

Illustration 10-2. STOP *signs control the intersection between a low-volume local street and a major arterial.*

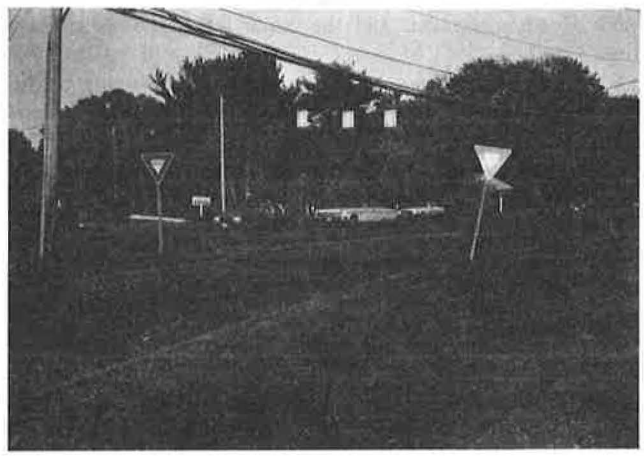


Illustration 10-3. YIELD *signs are used at this right-turn roadway where approach speeds and sight distance are such that vehicles need not come to a full stop to safely select a gap in the major street flow.*

II. METHODOLOGY

CONCEPTUAL APPROACH

The method generally assumes that major street traffic is not affected by minor street flows. This assumption is generally good for periods when the operation is smooth and uncongested. When congestion occurs, it is likely that major flows will experience some impedance due to minor street traffic. Left turns from the major street are assumed to be affected by the opposing major street flow, and minor street traffic is affected by all conflicting movements.

The methodology also adjusts for the additional impedance of minor street flows on each other, and accounts for the shared use of lanes by two or three minor street movements, for example, right-turn, through, and left-turn movements sharing a single minor street lane.

To properly account for mutual impedances, the method is based on a prioritized regime of gap utilization. Gaps in the major street traffic flow are used by a number of competing flows. A gap used by a vehicle from one of these flows is no longer available for use by another vehicle. Gaps are utilized by vehicles in the following priority order:

1. Right turns from the minor street.
2. Left turns from the major street.
3. Through movements from the minor street.
4. Left turns from the minor street.

For example, if a left-turning vehicle on the major street and a through vehicle from the minor street are waiting to cross the major traffic stream, the first available gap (of acceptable size) would be taken by the left-turning vehicle. The minor street through vehicle must wait for the second available gap. In aggregate terms, a large number of such left-turning vehicles could use up so many of the available gaps that minor street through vehicles are severely impeded or unable to make safe crossing movements.

Right-turning vehicles from the minor street are not assumed to "use up" available gaps. Because such vehicles merely merge into gaps in the right-hand lane of the stream into which they turn, they require only a gap in that lane, not in the entire major street traffic flow. Further, a gap in the overall major street traffic could be simultaneously used by another vehicle. For this reason, the method does not assume that right turns from the minor street impede any of the other flows using major street gaps.

The basic structure of the procedure is as follows:

1. Define existing geometric and volume conditions for the intersection under study.
2. Determine the "conflicting traffic" through which each minor street movement, and the major street left turn, must cross.
3. Determine the size of the gap in the conflicting traffic stream needed by vehicles in each movement crossing a conflicting traffic stream.
4. Determine the capacity of the gaps in the major traffic stream to accommodate each of the subject movements that will utilize these gaps.
5. Adjust the capacities so found to account for impedance and the use of shared lanes.

Each of these basic analysis steps is discussed in detail in the sections that follow.

INPUT DATA REQUIREMENTS

Basic data requirements for the unsignalized intersection methodology are similar to those for other capacity analysis techniques. Detailed descriptions of the geometrics, control, and volumes at the intersection are needed.

Key geometric factors include:

1. Number and use of lanes.
2. Channelization.
3. Percent grade.
4. Curb radii and approach angle.
5. Sight distances.

Each of these factors has a substantial impact on how gaps are utilized, and on the size of the gap that is required by the various movements. Sight distances, curb radii, and approach angles may be approximately evaluated.

The number and use of lanes is a critical factor. Vehicles in adjacent lanes can use the same gap in the traffic stream simultaneously (unless impeded by a conflicting user of the gap). When movements share lanes, only one vehicle from those movements may use each gap. Channelization is also important because it can be used to reduce impedance by separating conflicting flows from each other.

Volumes must be specified by movement. In general, full hour volumes are used in the analysis of unsignalized intersections because short-term fluctuations will generally not present major difficulties at such locations. The analyst may, however, choose to consider flow rates for the peak 15-min interval by dividing all volumes by the peak hour factor (PHF) before beginning computations. The volume for movement i is designated as V_i in this chapter. In cases where flow rates are used, the notation remains, but refers to the flow rate instead of volume.

By convention, subscripts 1 to 6 are used to define movements on the major street, and subscripts 7 to 12 to define movements on the minor street. Conversion of vehicles per hour to passenger cars per hour is accomplished using the passenger-car equivalent values given in Table 10-1. Note that the table accounts for both grade and vehicle type, and that even passenger cars must be adjusted if the intersection approach is on a grade.

In addition to the geometric and volume data noted above, it is necessary to record the average running speed of vehicles on the major roadway.

TABLE 10-1. PASSENGER-CAR EQUIVALENTS FOR UNSIGNALIZED INTERSECTIONS

TYPE OF VEHICLE	GRADE (%)				
	−4%	−2%	0%	+2%	+4%
Motorcycles	0.3	0.4	0.5	0.6	0.7
Passenger Cars	0.8	0.9	1.0	1.2	1.4
SU/RV's[a]	1.0	1.2	1.5	2.0	3.0
Combination Veh.	1.2	1.5	2.0	3.0	6.0
All Vehicles[b]	0.9	1.0	1.1	1.4	1.7

[a] Single-unit trucks and recreational vehicles.
[b] If vehicle composition is unknown, these values may be used as an approximation.

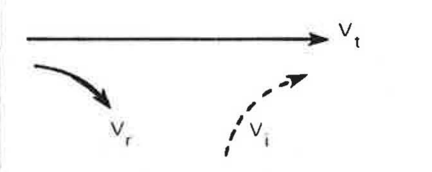
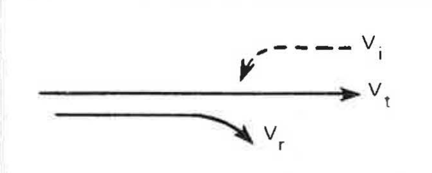
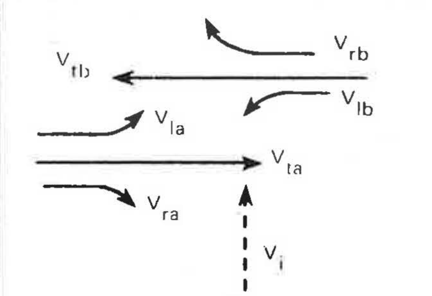
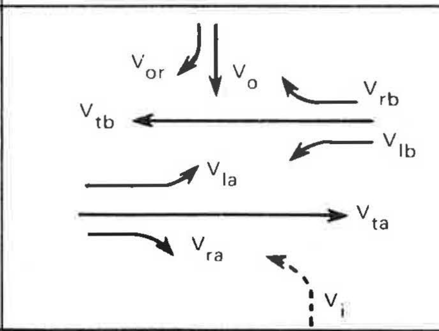
CONFLICTING TRAFFIC

The nature of conflicting movements at an unsignalized intersection is relatively complex. Each subject movement faces a different set of conflicts that are directly related to the nature of the movement. These conflicts are depicted in Figure 10-2, which illustrates the computation of the parameter:

V_{ci} = the "conflicting volume" for movement i, that is, the total volume which conflicts with movement i, expressed in vehicles per hour.

The right-turn movement from the minor street, for example, is in conflict with only the major street through movement in the right-hand lane into which right-turners will merge. Figure 10-2 includes one-half of the right-turn movement from the major street, because this flow has been found to have a somewhat inhibiting effect on the subject movement. This may be caused by such vehicles approaching without using their turn indicator, causing the driver of a waiting vehicle to believe it will travel straight through the intersection and/or side frictions created as they turn into a lane adjacent to waiting vehicles.

Left turns from the major street are in absolute conflict with the total opposing through and right-turn flows, because they

Subject Movement	Conflicting Traffic, V_{ci}	Illustration
1. RIGHT TURN from minor street.	$1/2(V_r)^{**} + V_t^{*}$	
2. LEFT TURN from major street.	$V_r^{***} + V_t$	
3. THROUGH MVT from minor street.	*** $1/2(V_{ra})^{**} + V_{ta} + V_{la}$ $+ V_{rb} + V_{tb} + V_{lb}$	
4. LEFT TURN from minor street.	$1/2(V_{ra})^{**} + V_{ta} + V_{la}$ $+ V_{rb}^{***} + V_{tb} + V_{lb}$ $+ V_o + V_{or}$	

* V_t includes only the volume in the right hand lane.

** Where a right-turn lane is provided on major street, eliminate V_r or V_{ra}.

*** Where the right-turn radius into minor street is large and/or where these movements are STOP/YIELD-controlled, eliminate V_r (Case 2), and V_{ra} and/or V_{rb} (Case 4). V_{rb} may also be eliminated on multilane major streets.

Figure 10-2. Definition and computation of conflicting traffic volumes.

must cross the through flow and merge with the right-turn flow. The method does not differentiate between crossing and merging conflicts. Left turns from the major street and the opposing right turns from the major street are considered to merge, regardless of the number of lanes provided in the exit roadway.

Minor street through movements have a direct crossing or merging conflict with all movements on the major street, as indicated in Figure 10-2, except the right turn into the subject approach. Only one-half of this movement is included in the computation, for the same reasons as discussed above.

The left turn from the minor street is the most difficult maneuver to execute from an unsignalized intersection, and it faces the most complex set of conflicting flows. Conflicting volumes include all major street flows, in addition to the opposing right turn and through movement on the minor roadway.

When using Figure 10-2 to compute conflicting volumes, the analyst should carefully consult the footnotes, which allow modifications to the equations shown in special cases.

Note that in the equations of Figure 10-2, the conflicting traffic volume for movement *i*, which is denoted as V_{ci}, is computed in *terms of an hourly volume in mixed vph.* Subscripts *r* denote right turns, *l* left turns, *t* through movements, and *o* opposing minor street flows.

CRITICAL GAP SIZE

The "critical gap" is defined as the median time headway between two successive vehicles in the major street traffic stream that is accepted by drivers in a subject movement that must cross and/or merge with the major street flow. It is denoted as T_c, and is expressed in seconds.

The critical gap depends on a number of factors, including:

1. The type of maneuver being executed.
2. The type of minor street control (STOP or YIELD).
3. The prevailing approach speed (average running speed) on the major street.
4. The number of lanes on the major street.
5. The geometrics and environmental conditions at the intersection.

The type of movement is a most significant factor. As the movement being executed becomes more complex, drivers will require longer gaps through which to make their maneuver. Thus, the required gap for a right turn from the minor street, which involves only a single merging conflict, is shorter than the gap required to execute a left turn from the minor street, which involves a variety of complex conflicts. The latter movement is facing a more complex conflict, and the driver's decision process in selecting the gap is more complex as a result.

The type of control is important as well. At STOP-controlled locations, drivers usually start from a stopped condition, while at a YIELD-controlled location, some proportion of vehicles starts from a low, but moving speed. It will obviously take longer to cross an intersection, on the average, when starting from a stopped condition than it will take from a slow-speed condition, and a longer critical gap is therefore required at STOP-controlled locations.

The speed of major street traffic has a major impact on required gap. When a driver selects a gap through which to execute a maneuver, judgment is based on the size of the gap available and the driver's confidence that the gap will remain stable as he or she crosses through it. As the speed of major street traffic increases, drivers tend to require longer gaps. In effect, as speeds increase, drivers become more conservative in their gap selection in reaction to the increased hazard of crossing a higher speed traffic stream.

As the number of lanes on the major street increases, the critical gap size also increases. Selecting and negotiating a gap in a multilane traffic stream is a more complex maneuver than the same process for a single lane, and drivers will require longer gaps.

Geometric conditions can also play a major role in determining the size of the critical gap. Such features as channelized turning lanes, large corner radii, and similar measures, make certain movements easier to execute, and thereby can reduce the critical gap size needed for those movements. For example, a channelized right-turn lane may effectively reduce the angle of the turn at the merge point from 90 deg to some shallower angle. Features such as acceleration and deceleration lanes will also have the same effect. On the other hand, geometric conditions that restrict sight distances will have an opposite impact, increasing critical gap size by making it more difficult for drivers to observe and select gaps.

Environmental conditions always affect traffic flow, and unsignalized intersections are no exception. Similarly to all capacity analysis procedures, the methods of this chapter assume good weather conditions, daylight, no traffic incidents, and good pavement conditions. Inclement weather, darkness, traffic incidents, and poor pavement conditions will all serve to decrease capacity and reduce level of service. In general, any of these conditions will cause drivers to require larger gaps to execute desired maneuvers, and while no quantitative criteria are provided, the user should be aware of these likely impacts of poor environmental conditions.

VALUES OF CRITICAL GAP

Values of critical gap are selected from Table 10-2 in a two-part process:

1. The basic critical gap size is selected from the first half of the table for the type of movement, type of control, and major street speed at the subject location.
2. Adjustments and modifications to the basic critical gap size are selected from the second half of the table for a variety of conditions, subject to the limitations given in the footnotes.

The population factor has been incorporated because field experience indicates that drivers familiar with more congested traffic environments tend to select smaller gaps. Analysts may wish to use some judgment in applying this adjustment, and should have knowledge of local driving habits.

The restrictive impact of poor sight distance is also a complex factor requiring some judgment. The user may wish to conduct a site examination before deciding either on a value for this adjustment, or on whether or not it should be utilized at all. Such factors as accident experience, driver response and gap acceptance, traffic volumes, and measured sight distances should be considered. Where such field examinations are not possible, computations should be done using a range of values to examine the sensitivity to this factor.

POTENTIAL CAPACITY FOR A MOVEMENT

The potential capacity of a movement is denoted as c_{pi} (for movement i), and is defined as the "ideal" capacity for a specific subject movement, assuming the following conditions:

1. Traffic on the major roadway does not block the minor road.
2. Traffic from nearby intersections does not back up into the intersection under consideration.
3. A separate lane is provided for the exclusive use of each minor street movement under consideration.
4. No other movements impede the subject movement.

The potential capacity is selected from Figure 10-3, and is based on the conflicting traffic volume, V_c, in vehicles per hour, and the critical gap, T_c, in seconds. The figure is entered on the horizontal axis with the value of V_c. A vertical line is drawn to the appropriate "critical gap" curve. A horizontal line is drawn from the intersection with the "critical gap" curve to the vertical axis, where the result is read, in passenger cars per hour.

IMPEDANCE EFFECTS

It has been noted that vehicles utilize gaps at an unsignalized intersection in a prioritized manner. When traffic becomes congested in a high-priority movement, it can impede lower priority

Table 10-2. Critical Gap Criteria for Unsignalized Intersections

BASIC CRITICAL GAP FOR PASSENGER CARS, SEC				
VEHICLE MANEUVER AND TYPE OF CONTROL	AVERAGE RUNNING SPEED, MAJOR ROAD 30 MPH		55 MPH	
	NUMBER OF LANES ON MAJOR ROAD 2	4	2	4
RT from Minor Road				
STOP	5.5	5.5	6.5	6.5
YIELD	5.0	5.0	5.5	5.5
LT from Major Road	5.0	5.5	5.5	6.0
Cross Major Road				
STOP	6.0	6.5	7.5	8.0
YIELD	5.5	6.0	6.5	7.0
LT from Minor Road				
STOP	6.5	7.0	8.0	8.5
YIELD	6.0	6.5	7.0	7.5

ADJUSTMENTS AND MODIFICATIONS TO CRITICAL GAP, SEC	
CONDITION	ADJUSTMENT
RT from Minor Street: Curb radius > 50 ft or turn angle < 60°	−0.5
RT from Minor Street: Acceleration lane provided	−1.0
All movements: Population ≥ 250,000	−0.5
Restricted sight distance.[a]	up to +1.0

NOTES: Maximum total decrease in critical gap = 1.0 sec.
Maximum Critical gap = 8.5 sec.
For values of average running speed between 30 and 55 mph, interpolate.
[a] This adjustment is made for the specific movement impacted by restricted sight distance.

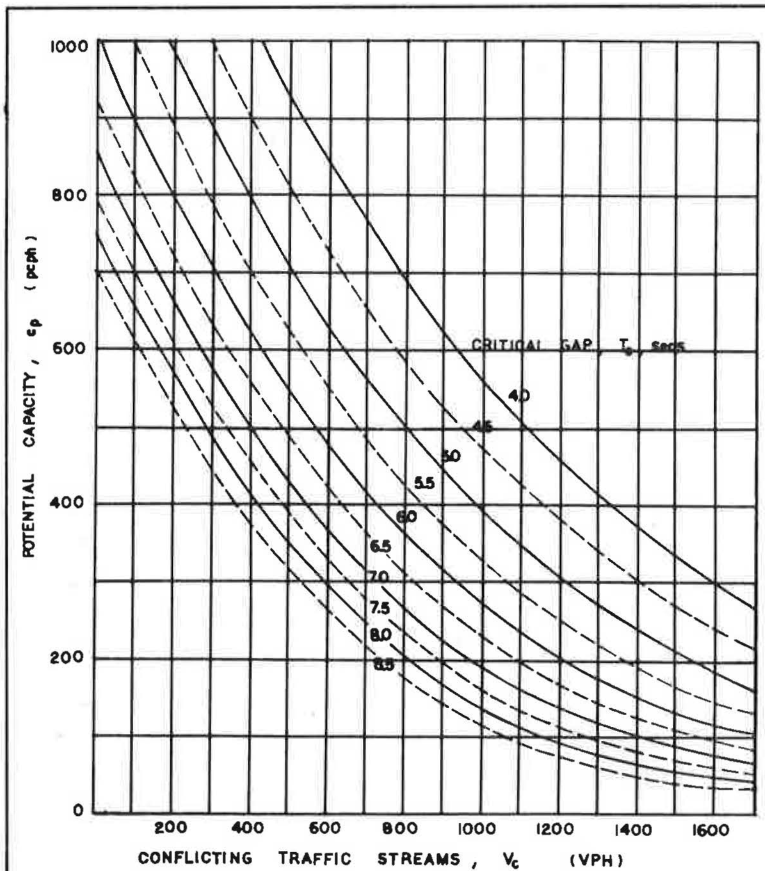


Figure 10-3. Potential capacity based on conflicting traffic volume and critical gap size.

movements from utilizing gaps in the traffic stream, and reduce the potential capacity of the movement. It should be noted that major street traffic is not assumed to be impeded at any time by minor street flows, and that "impedance" affects only minor street vehicles.

Right turns from the minor street do not generally impede other traffic elements, except for opposing left turns from the minor street where both movements are merging into the same traffic stream. Given the priority of gap usage:

1. Left turns from the major street impede both through movements and left turns from the minor street.
2. Through movements from the minor street impede left turns from the minor street.

In general, the impact of impedance is addressed by multiplying the potential capacity of a movement, c_{pi}, by a series of impedance factors, P_j, for each impeding movement j. These computations are illustrated in Figure 10-4, and result in the finding of the movement capacity, c_{mi}, which is the adjusted capacity of the movement. The "movement capacity" still assumes that the movement has exclusive use of a separate lane.

Impedance factors, P_j, are found from Figure 10-5. They are based solely on the percent of potential capacity of the impeding movement used by existing demand. Consider the following example. A left-turn movement from a minor street at a T-intersection is impeded by the left turn from the major street. The latter movement has a potential capacity of 500 pcph and

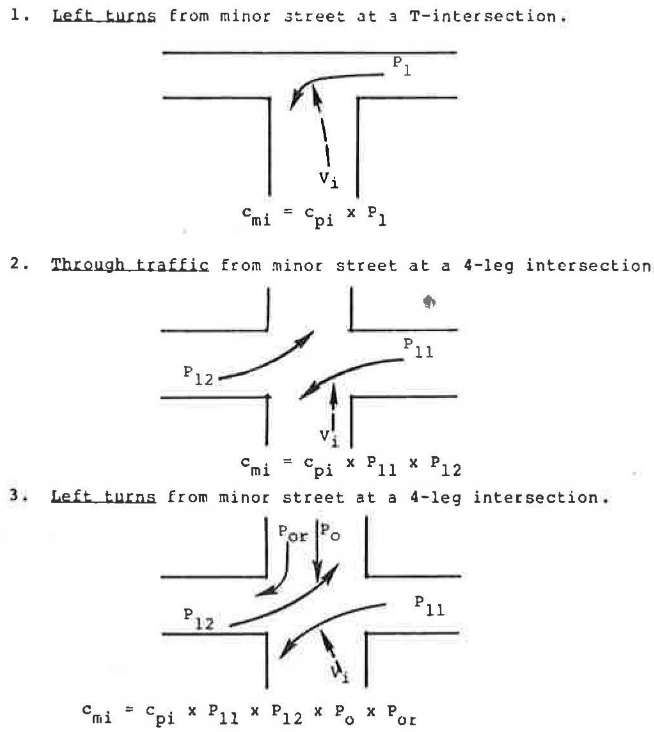


Figure 10-4. Illustration of impedance computations.

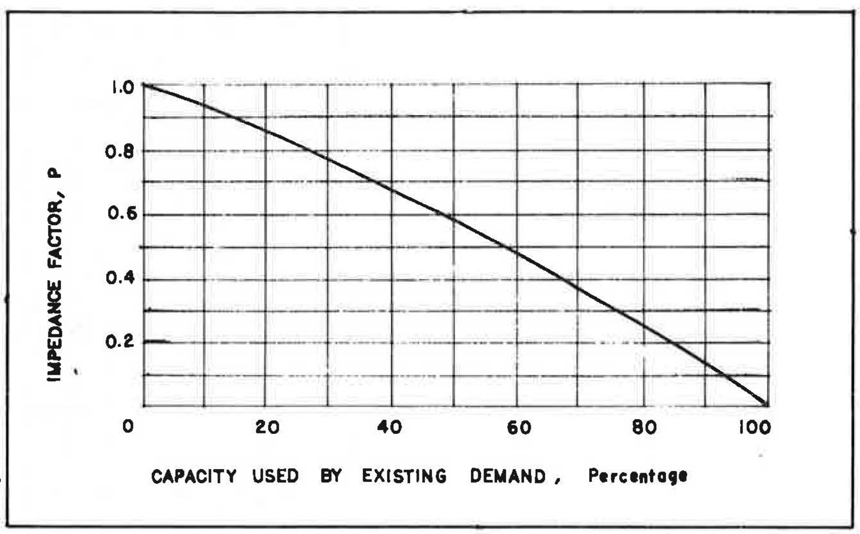


Figure 10-5. Impedance factors as a result of congested movements.

a demand of 200 pcph. Thus, the major street left turn uses 200/500 = 0.40, or 40 percent, of its available capacity. Figure 10-5 is entered with this value, and an impedance factor of 0.68 is read. The potential capacity for the minor street left turn must then be multiplied by 0.68 to account for the impedance of the major street left turn.

Essentially, the computation of potential capacity assumes that all movements have exclusive access to available gaps. The availability of these gaps to lower priority movements is reduced as they are utilized by higher priority movements. This reduction is computationally represented in the impedance factors.

SHARED-LANE CAPACITY

Up to this point, the methodology has assumed that each minor street movement has the exclusive use of a lane. This is often not the case, and frequently two or three movements share a single lane on the minor approach. When this occurs, vehicles from different movements do not have simultaneous access to gaps, nor can more than one vehicle from the sharing movements utilize the same gap.

Occasionally, an intersection with wide corner radii will allow vehicles approaching in the same lane to stop side-by-side. This will act to reduce or eliminate the adverse impact of the shared lane. Where several movements share the same lane, and cannot stop side-by-side at the stop line of the intersection, the following equation is used to compute the capacity of the shared lane:

$$c_{SH} = \frac{v_l + v_t + v_r}{[v_l/c_{ml}] + [v_t/c_{mt}] + [v_r/c_{mr}]}$$

where:

c_{SH} = capacity of the shared lane, in pcph;
v_l = volume of left-turn movement in shared lane, in pcph;
v_t = volume of through movement in shared lane, in pcph;
v_r = volume of right-turn movement in shared lane, in pcph;
c_{ml} = movement capacity of the left-turn movement in shared lane, in pcph;
c_{mt} = movement capacity of the through movement in shared lane, in pcph; and
c_{mr} = movement capacity of the right-turn movement in shared lane, in pcph.

Only those movements included in the shared lane are included in the equation. If the shared lane includes only right-turn and through movements, both numerator and denominator terms for left-turners are deleted in the equation.

LEVEL-OF-SERVICE CRITERIA

The computations described above result in a solution for the capacity of each lane on the minor approaches to a STOP- or YIELD-controlled intersection. Level-of-service criteria for this methodology are stated in very general terms, and are related to general delay ranges. The criteria are given in Table 10-3, and are based on the reserve, or unused, capacity of the lane in question. This value is computed as:

$$c_R = c_{SH} - v$$

where:

c_R = reserve or unused capacity of the lane, in pcph;
c_{SH} = shared-lane capacity of the lane, in pcph; and
v = total volume using the lane, in pcph.

TABLE 10-3. LEVEL-OF-SERVICE CRITERIA FOR UNSIGNALIZED INTERSECTIONS

RESERVE CAPACITY (PCPH)	LEVEL OF SERVICE	EXPECTED DELAY TO MINOR STREET TRAFFIC
≥ 400	A	Little or no delay
300–399	B	Short traffic delays
200–299	C	Average traffic delays
100–199	D	Long traffic delays
0– 99	E	Very long traffic delays
*	F	*

* When demand volume exceeds the capacity of the lane, extreme delays will be encountered with queuing which may cause severe congestion affecting other traffic movements in the intersection. This condition usually warrants improvement to the intersection.

Caution should be used in the interpretation of these criteria. They are stated in general terms, without specific numeric values. It is, therefore, not possible to directly compare an unsignalized LOS with a signalized intersection analysis LOS (Chapter 9) in terms of specific delay values without collecting delay data directly at the subject site. The levels of service in this chapter *are not* associated with the delay values cited for signalized intersections in Chapter 9.

Because the basic criteria for LOS are given in terms of a general delay description, an unusual result sometimes occurs. A movement, most often a left-turn movement, can have a poorer LOS if it is given a separate lane than if it shares a lane with another movement (usually a through movement). This is not inconsistent in terms of the stated criteria. Left-turn movements will generally experience longer delays than other movements because of the nature and priority of the movement. If left turns are placed in a shared lane, the *average delay to vehicles in that lane* may indeed be less than the average delay to left turns in a separate lane. However, all vehicles in the shared lane experience increased delay over the condition in which left turns have a separate lane. Consider the following:

1. Ten left-turners will experience an average delay of 10 sec if they have an exclusive lane, and 15 sec if they share a lane with a through movement.

2. Fifty through-vehicles will experience an average delay of 5 sec if they have an exclusive lane, and 6 sec if they share a lane with the above left-turners.

If the vehicles are forced to share a lane, the average delay to a vehicle in the shared lane will be:

$$[(10 \times 15) + (50 \times 6)]/[10 + 50] = 450/60 = 7.5 \text{ sec/veh}$$

Table 10-4 illustrates this comparison. While each vehicle experiences *increased* delay when placed in a shared lane, the average delay in the shared lane is *less* than the average delay to left-turners in an exclusive lane and *more* than the average delay to through vehicles in an exclusive lane. Thus, the LOS in the exclusive LT lane may be poorer than that for the mixed lane. The analyst, however, may wish to carefully consider the aggregate impact on delay which takes place. In general, expanding a one-lane STOP- or YIELD-controlled approach to include an exclusive LT or RT lane will decrease the aggregate delay, regardless of level-of-service designations.

Level-of-service F exists when there are insufficient gaps of suitable size to allow a side street demand to safely cross through a major street traffic stream. This is generally evident from extremely long delays experienced by side street traffic, and by queuing on the minor approaches. The method, however, is based on a constant critical gap size; that is, the critical gap remains constant, no matter how long the side street motorist waits. Level-of-service F may also appear in the form of side street vehicles selecting smaller than usual gaps. In such cases, safety may be a problem, and some disruption to the major traffic stream may result. It is important to note that LOS F may not always result in long queues, but may result in adjustments to a normal gap acceptance behavior. The latter is more difficult to observe in the field than queuing, which is more obvious.

POTENTIAL IMPROVEMENTS

It should be noted that this methodology is not a formal warrant for considerating signalization. Where unacceptable levels of service are present at an unsignalized location, a range of improvements may be considered, including such measures as channelization, lane use controls, sight distance improvements, multiway STOP control, and so on. Within this context, the possibility of signalization should also be considered, and the standard data generally collected for such consideration should be obtained and examined. This methodology should *not* be used as a de facto signal warrant without further study of the location in question.

TABLE 10-4. ILLUSTRATION OF DELAY EXAMPLE

MOVEMENT	VOLUME (VEH)	SEPARATE LANE CASE		SHARED LANE CASE	
		DELAY/VEH (SEC/VEH)	TOTAL DELAY (SEC)	DELAY/VEH (SEC/VEH)	TOTAL DELAY (SEC)
LT	10	10	100	15 } 7.5	150
TH	50	5	250	6 }	300
			350		450

III. PROCEDURES FOR APPLICATION

The analysis of unsignalized intersections is generally applied to existing locations either to evaluate existing operational conditions under present demands, or to estimate the impacts of anticipated new demands. The methodology is specifically structured to yield a level of service and an estimate of reserve capacity for an existing case. Thus, operational analysis is the mode in which it is used. Design applications are treated as trial-and-error computations based on anticipated improvements to an existing intersection or on the projected design of a new intersection. The procedure, however, is easily manipulated to investigate the impact of key design features on probable operation.

FIELD DATA REQUIREMENTS

As noted previously, computations require several types of data as inputs to the methodology. These include:

1. Volumes by movement for the hour of interest.
2. Vehicle classification for the hour of interest.
3. Peak hour factor (if peak flow rates are being used as the basis for analysis).
4. Prevailing (average running) speed of traffic on the major street.

5. Number of lanes on the major street.
6. Number and use of lanes on the minor street approaches.
7. Grade of all approaches.
8. Other geometric features of interest: channelization, angle of intersection, sight distance, etc.
9. Type of control on the minor approaches.

Because the methodology herein results in a qualitative evaluation of delay, it is also recommended, if possible, that some delay data be collected with the above information. This will allow for a better quantification and description of existing operating conditions at the location under study. It would also allow for a more precise comparison with a signalized intersection analysis (Chapter 9), for which precise delay estimates are generated.

SEQUENCE OF COMPUTATIONS

As the methodology is based on a prioritized use of gaps by vehicles at an unsignalized intersection, it is important that computations be made in a precise order. The computational sequence is the same as the priority of gap use, and movements are considered in the following order:

1. *Right turns* from the minor street.
2. *Left turns* from the major street.
3. *Through movements* from the minor street.
4. *Left turns* from the minor street.

To assist in maintaining the proper order of computations, worksheets are provided for the two principal types of intersections which are generally the subject of such analyses: four-leg intersections and T-intersections. The use of each of these in computational analysis is described in the sections below.

ANALYSIS OF FOUR-LEG INTERSECTIONS

Figure 10-6 illustrates the worksheet for four-leg intersections, and is a 3-page form. The following steps describe how computations are made and summarized using this form.

Volume Summary and Adjustment

The first page of the worksheet consists of the summarization and adjustment of demand volumes. Basic geometric data are also summarized on this page.

1. *Hourly volumes* are summarized on the top portion of the form on the diagram provided. A "north" indication should be inserted to ensure proper orientation of the intersection and of the demand volumes.

Note the notation convention which utilizes V_1 to V_6 to denote major street flows, and V_7 to V_{12} to denote minor street flows. The flows should be carefully entered, because the worksheet refers to these flows by their worksheet designation to ease computations.

In addition to volume summaries, the number of lanes on each approach should be indicated (notations on their use may be added, as well). The type of control is indicated by checking the appropriate box, and the prevailing (average running) speed on the major street and the PHF are listed as indicated.

In some cases, an intersection will have special geometric and/or other features that cannot be adequately illustrated on the worksheet. In these cases, it is recommended that a schematic drawing be made and attached to the worksheet for clarity.

2. *Volume adjustments* are made to convert vph to pcph. In general, analysis will be on the basis of full hour volumes. Should the analyst wish to examine flow during the peak 15-min period, all volumes should be divided by the PHF *before* entering them on the "vph" diagram of the worksheet.

The conversion from vph to pcph is made using the passenger-car equivalent values given in Table 10-1. Recall that the table assesses the impact of both vehicle type and grade, and that even passenger cars are subject to adjustment if a grade of more then +2 percent or −2 percent is present. Volume in pcph is computed by multiplying the number of vehicles in each category by the appropriate equivalent from Table 10-1 and adding to find the total volume for all categories.

For example, if an approach on a level grade had a volume of 150 passenger cars, 30 single-unit trucks, and 5 combination vehicles, the total equivalent volume in pcph would be:

$$\begin{aligned} 150 \times 1.0 &= 150 \text{ pcph} \\ 30 \times 1.5 &= 45 \text{ pcph} \\ 5 \times 2.0 &= \underline{10 \text{ pcph}} \\ & 205 \text{ pcph} \end{aligned}$$

where 1.0, 1.5, and 2.0 are the passenger-car equivalents for cars, single-unit trucks, and combination vehicles found in Table 10-1.

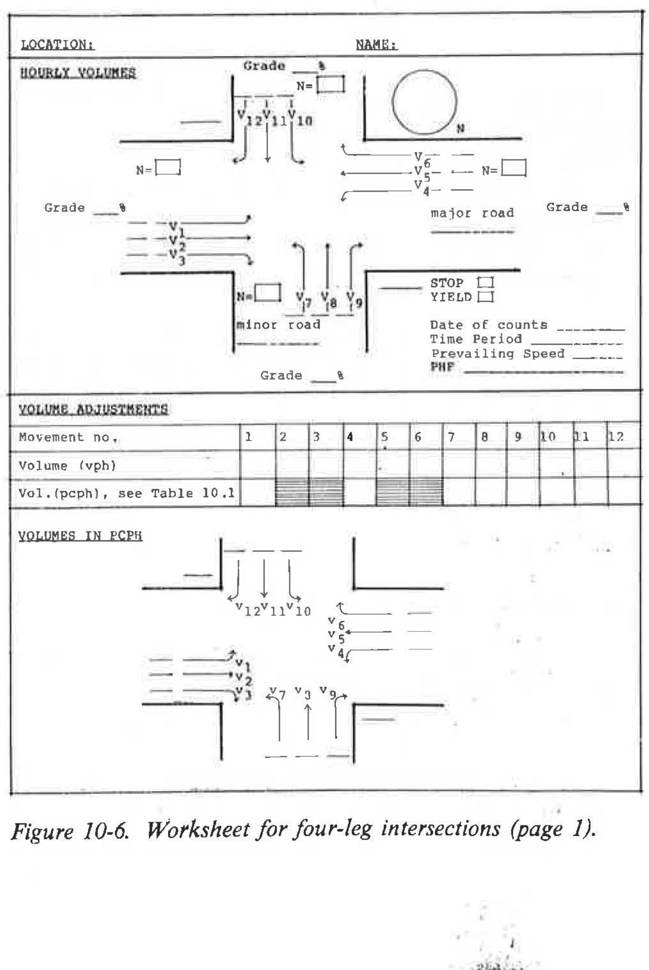


Figure 10-6. Worksheet for four-leg intersections (page 1).

STEP 1 : RT From Minor Street	v_9	v_{12}
Conflicting Flows, V_c	$1/2\ V_3+V_2=V_{c9}$ ___+___=_____vph	$1/2\ V_6+V_5=V_{c12}$ ___+___=_____vph
Critical Gap, T_c (Tab.10.2)	_____(secs.)	_____(secs.)
Potential Capacity, c_p (Fig.10.3)	c_{p9}=_____pcph	c_{p12}=_____pcph
% of c_p utilized	(v_9/c_{p9})x100=_____%	(v_{12}/c_{p12})x100=_____%
Impedance Factor, P (Fig.10.5)	P_9=_____	P_{12}=_____
Actual Capacity, c_m	$c_{m9}=c_{p9}$=_____pcph	$c_{m12}=c_{p12}$=_____pcph
STEP 2 : LT From Major Street	v_4	v_1
Conflicting Flows, V_c	$V_3+V_2=V_{c4}$ ___+___=_____vph	$V_6+V_5=V_{c1}$ ___+___=_____vph
Critical Gap, T_c (Tab.10.2)	_____(secs.)	_____(secs.)
Potential Capacity, c_p (Fig.10.3)	c_{p4}=_____pcph	c_{p1}=_____pcph
% of c_p utilized	(v_4/c_{p4})x100=_____%	(v_1/c_{p1})x100=_____%
Impedance Factor, P (Fig.10.5)	P_4=_____	P_1=_____
Actual Capacity, c_m	$c_{m4}=c_{p4}$=_____pcph	$c_{m1}=c_{p1}$=_____pcph
STEP 3 : TH From Minor Street	v_8	v_{11}
Conflicting Flows, V_c	$1/2V_3+V_2+V_1+V_6+V_5+V_4=V_{c8}$ __+__+__+__+__+__=___vph	$1/2V_6+V_5+V_4+V_3+V_2+V_1=V_{c11}$ __+__+__+__+__+__=___vph
Critical Gap, T_c (Tab.10.2)	_____(secs.)	_____(secs.)
Potential Capacity, c_p (Fig.10.3)	c_{p8}=_____pcph	c_{p11}=_____pcph
% of c_p utilized	(v_8/c_{p8})x100=_____%	(v_{11}/c_{p11})x100=_____%
Impedance Factor, P (Fig.10.5)	P_8=_____	P_{11}=_____
Actual Capacity, c_m	$c_{m8}=c_{p8}xP_1xP_4$ _____=___x___x___(pcph)	$c_{m11}=c_{p11}xP_1xP_4$ _____=___x___x___(pcph)
STEP 4 : LT From Minor Street	v_7	v_{10}
Conflicting Flows, V_c	V_8(step3)$+V_{11}+V_{12}=V_{c7}$ ____+____+____=_____vph	V_{c11}(step3)$+V_7+V_8=V_{c10}$ ____+____+____=_____vph
Critical Gap, T_c (Tab.10.2)	_____(secs.)	_____(secs.)
Potential Capacity, c_p (Fig.10.3)	c_{p7}=_____pcph	c_{p10}=_____pcph
Actual Capacity, c_m	$c_{m7}=c_{p7}xP_1xP_4xP_{11}xP_{12}$ ____=__x__x__x__(pcph)	$c_{m10}=c_{p10}xP_4xP_1xP_8xP_9$ ____=__x__x__x__(pcph)

Figure 10-6. Worksheet for four-leg intersections (page 2).

SHARED LANE CAPACITY

$$c_{SH} = \frac{v_i + v_j}{(v_i / c_{mi}) + (v_j / c_{mj})}$$ where 2 movements share a lane

$$c_{SH} = \frac{v_i + v_j + v_k}{(v_i / c_{mi}) + (v_j / c_{mj}) + (v_k / c_{mk})}$$ where 3 movements share a lane

APPROACH MOVEMENTS 7, 8, 9

Movement	v(pcph)	c_m(pcph)	c_{SH}(pcph)	$c_R = c_{SH} - v$	LOS
7					
8					
9					

APPROACH MOVEMENTS 10, 11, 12

Movement	v(pcph)	c_m(pcph)	c_{SH}(pcph)	$c_R = c_{SH} - v$	LOS
10					
11					
12					

MAJOR STREET LEFT TURNS 1, 4

Movement	v (pcph)	c_m (pcph)	$c_R = c_m - v$	LOS
1				
4				

COMMENTS:

Figure 10-6. Worksheet for four-leg intersections (page 3).

Where no specific vehicle classification is given, volumes are generally multiplied by 1.1 (for no grade) to reflect "normal" traffic composition, which consists of 5 percent combination vehicles and nominal numbers of other vehicle types (other than passenger cars).

Conversion computations are summarized in the "Volume Adjustments" section of the worksheet in the middle of the first page.

Through and right-turning volumes on the major street are not converted to pcph. This is because they are only utilized in the computation of "conflicting traffic volume," which is done in terms of vph.

The diagram on the lower half of the first worksheet page can be used to summarize the converted volumes for ease of reference.

Computation of Movement Capacities

The second page of the worksheet is for the computation of movement capacities for each subject movement. All equations are shown on the worksheet, so that the user need not refer back to other sections of this chapter, and volumes are keyed to the diagrams on page 1 of the worksheet. Note that volumes denoted with a capital V refer to volumes in vph, while those denoted with a small v refer to converted volumes in pcph.

Computations proceed in the prescribed order, considering first the right turns from the minor street, followed by left turns from the major street, through movements from the minor street, and left turns from the minor street. The user should solve pairs of movements before proceeding to the next step, i.e., *both* right turns in Step 1 should be computed before proceeding to Step 2.

For each movement, the following sequence of computations is followed:

1. Compute conflicting flows, V_{ci}, in vph. Figure 10-1 may be consulted if a further explanation of this computation is desired.
2. Find the critical gap, T_c, in sec, from Table 10-2.
3. Find the potential capacity, c_{pi}, in pcph, from Figure 10-3.
4. Compute the percent of potential capacity used by the movement.
5. Find the impedance factor, P_i, from Figure 10-5. NOTE: This factor will be used in later steps to adjust the capacity of lower priority movements for impedance.
6. Compute the movement capacity, c_{mi}.

At a four-leg intersection, with all movements permitted, there are 8 separate computations of this type to be made and summarized on page 2 of the worksheet.

Computation of Shared-Lane Capacity and Level of Service

The third page of the worksheet is used to compute shared-lane capacities, reserve capacities, and level of service. The user will have to determine from field data or available design plans the movements that share a lane. The appropriate computations for shared-lane capacity are made (equations are shown on the worksheet). Reserve capacity is then computed for each lane, and the level of service is determined from Table 10-3.

It is often useful to also compute the reserve capacity and level of service for each movement *as if it had a separate lane.* This will provide useful information in the consideration of providing such lanes as a potential improvement to the location under study.

ANALYSIS OF T-INTERSECTIONS

The analysis of T-intersections follows the same general steps as those described above for four-leg intersections. They are, however, very much simplified, because many of the movements and the majority of the conflicts present in a four-leg intersection are removed. Because of this, a simplified worksheet is provided for T-intersection computations, and is shown on Figure 10-7.

The upper portion of the sheet provides for the summarizing of volume and geometric data, and for the adjustment of volumes, as described for four-leg intersections. Note that there are only six volumes to be considered, and only three of these need be converted to pcph. Again, if the intersection contains unusual geometric elements that are difficult to show on the worksheet, a schematic sketch should be developed and attached for clarity.

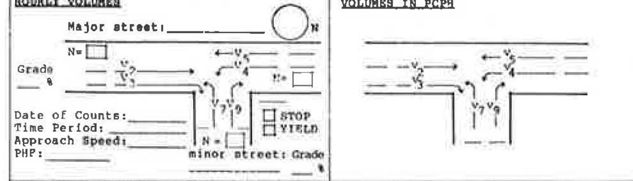
The middle portion of the form is for the computation of movement capacities. Again, note that there are only three movements to be considered, as opposed to eight for a four-leg intersection. Further, there is only one impedance element to consider: the left turn from the major street (V_4) impedes the left turn from the minor street (V_7).

The lower portion of the form provides for the computation of shared-lane capacities, which is also simplified. Because there are only two minor street flows, they either do or do not share a lane.

As was the case for four-leg intersections, it is often useful to compute the reserve capacity of each movement as if each had a separate lane, even where a lane is shared. This will assist in the assessment of possible lane additions as a solution to a substandard operation.

The sample problems illustrated later in the chapter detail the use of these procedures and worksheets as described.

LOCATION:	NAME:
HOURLY VOLUMES	VOLUMES IN PCPH



VOLUME ADJUSTMENTS

Movement no.	2	3	4	5	7	9
Volume (vph)						
Vol. (pcph), see Table 10.1						

STEP 1 : RT From Minor Street	V_9
Conflicting Flow, V_c	1/2 V_3+V_2=___+___=___vph (V_{c9})
Critical Gap, T_c and Potential Capacity, c_p	T_c=___secs (Tab. 10.2) c_{p9}=___pcph (Fig. 10.3)
Actual Capacity, c_m	$c_{m9}=c_{p9}$=______pcph
STEP 2 : LT From Major Street	V_4
Conflicting Flow, V_c	V_3+V_2=___+___=___vph (V_{c4})
Critical Gap, T_c and Potential Capacity, c_p	T_c=___secs (Tab. 10.2) c_{p4}=___pcph (Fig. 10.3)
% of c_p utilized and Impedance Factor (Fig. 10.5)	(v_4/c_{p4})x100=______ P_4=______
Actual Capacity, c_m	$c_{m4}=c_{p4}$=______pcph
STEP 3 : LT From Minor Street	V_7
Conflicting Flow, V_c	1/2 $V_3+V_2+V_5+V_4$=___+___+___+___=___vph (V_{c7})
Critical Gap, T_c and Potential Capacity, c_p	T_c=___secs (Tab. 10.2) c_{p7}=___pcph (Fig. 10.3)
Actual Capacity, c_m	$c_{m7}=c_{p7}$xP_4=___x___=______pcph

SHARED-LANE CAPACITY $SH = \frac{v_7 + v_9}{(v_7/c_{m7}) + (v_9/c_{m9})}$ if lane is shared

Movement no.	v (pcph)	c_m (pcph)	c_{SH} (pcph)	c_R	LOS
7					
9					
4					

Figure 10-7. Worksheet for analysis of T-intersections.

MULTIWAY STOP CONTROL

Multiway STOP control is a useful and appropriate type of intersection control for certain unsignalized intersections. Under multiway STOP control, all vehicles are stopped, with vehicles intended to depart in a counter-clockwise rotation regime under the basic rules of the road, wherein the "vehicle on the right" has the right-of-way. Multiway STOP control can be a low-cost solution at uncontrolled or two-way STOP or YIELD intersections where poor level of service is experienced.

Because vehicles at multiway STOP-controlled intersections are intended to depart in a strict rotational order as long as one vehicle is waiting on an approach, this type of control is most effective where demand on the several approaches is approximately equal.

It should be noted, however, that failure of drivers to observe the intended right-of-way discharge regime will result in poor levels of operation.

The capacity of multiway STOP-controlled intersections is a function of the number of approach lanes, and of the departure headways of vehicles crossing from a stopped position. At capacity, operations are relatively predictable, with queues developing along each approach, and vehicles discharging in a regular manner as described above.

Table 10-5 gives typical capacity values for a two-lane by two-lane four-way STOP-controlled intersection. As the table indicates, capacity is greatest when demand volume is evenly split between the crossing facilities. Capacities as high as 1,900 vph can be achieved at such intersections. A characteristic of intersections with a 50/50 demand is that vehicle delay tends to be uniform, and, because of the regular discharge pattern, is tolerated by most drivers. Lesser capacities and more variable distribution of delay occurs where demand is not as evenly split among the approaches.

The number of approach lanes also affects the capacity of multiway STOP-controlled intersections. Simultaneous movements from a two-lane approach can occur, increasing the overall capacity. Table 10-6 shows the capacity of four-way STOP-controlled intersections with a 50/50 demand split for a range of approach lane configurations.

Table 10-7 gives volume levels which can be accommodated at four-way STOP-controlled intersections under reasonable operating conditions. Although levels of service for such intersections are not specifically defined, Table 10-7 volumes are approximately indicative of LOS C.

TABLE 10-5. CAPACITY OF A TWO-BY-TWO LANE FOUR-WAY STOP-CONTROLLED INTERSECTION FOR VARIOUS DEMAND SPLITS

DEMAND SPLIT	CAPACITY[a] (VPH)
50/50	1,900
55/45	1,800
60/40	1,700
65/35	1,600
70/30	1,500

[a] Total capacity, all legs.
SOURCE: Ref. 9

TABLE 10-6. CAPACITY OF FOUR-WAY STOP-CONTROLLED INTERSECTIONS WITH 50/50 DEMAND SPLIT FOR VARIOUS APPROACH WIDTHS

INTERSECTION TYPE	CAPACITY[a] (VPH)
2-lane by 2-lane	1,900
2-lane by 4-lane	2,800
4-lane by 4-lane	3,600

[a] Total capacity, all legs.
SOURCE: Ref. 9

TABLE 10-7. APPROXIMATE LEVEL-OF-SERVICE C SERVICE VOLUMES FOR FOUR-WAY STOP-CONTROLLED INTERSECTIONS

DEMAND SPLIT	LOS C SERVICE VOLUME, VPH		
	NUMBER OF LANES		
	2 BY 2	2 BY 4	4 BY 4
50/50	1,200	1,800	2,200
55/45	1,140	1,720	2,070
60/40	1,080	1,660	1,970
65/35	1,010	1,630	1,880
70/30	960	1,610	1,820

SOURCE: Ref. 10

IV. SAMPLE CALCULATIONS

CALCULATION 1—A T-INTERSECTION

1. *Description*—This example concerns the intersection of Market Street with Jones Street, which is located in an urban area with a population of 100,000. Market Street is a two-lane collector, and Jones Street is a two-lane local street serving a residential development. It is controlled with a STOP sign. There is no widening in the vicinity of the intersection, and corner radii are 20 ft. The intersection is depicted in Figure 10-8.

Residents of the area have complained that there is substantial delay experienced in the late afternoon turning right into Market Street. They claim that this is due to the need for right and left turners to share a lane, and have requested that a right-turn-only lane be provided.

2. *Solution*—The T-intersection worksheet will be used for summarizing and organizing computations concerning this problem. The problem is to evaluate whether or not the requested improvement will achieve any reasonable reduction in the delay experienced by local residents traversing this location.

The computations on the worksheet (Figure 10-8) are described and discussed below:

1. Existing peak hour volumes for the afternoon period were collected, and are summarized as indicated on the upper-left diagram. The approach speed of major street traffic was also observed, and found to be 30 mph.

2. Since no classification of vehicles is given, nor is any grade present, volumes 4, 7, and 9 (which must be adjusted) are multiplied by 1.1 to reflect normal traffic composition. The adjusted volumes are entered on the upper-right diagram for easy reference in later computational steps.

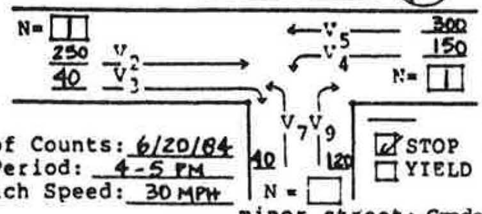
3. The RT from the minor street is the first movement considered. The conflicting volume is computed as one-half the major street right-turn volume, plus the through volume with which the minor street RT will merge. The conflicting traffic is thus found to be 270 vph.

The critical gap is selected from Table 10-2 for an RT from minor street, two-lanes on the major street, and prevailing speed of 30 mph. The critical gap is found to be 5.5 sec. There are no conditions which would allow adjusting this critical gap.

LOCATION:	NAME:

HOURLY VOLUMES

Major street: MARKET N



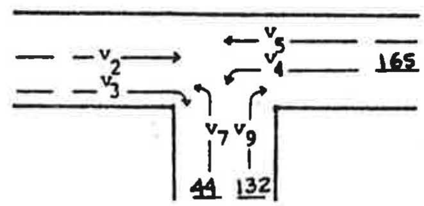
Grade ___ %

Date of Counts: 6/20/84
Time Period: 4-5 PM
Approach Speed: 30 MPH
PHF: ________
POP. 100,000

☑ STOP
☐ YIELD

minor street: JONES Grade ___ %

VOLUMES IN PCPH



VOLUME ADJUSTMENTS

Movement no.	2	3	4	5	7	9
Volume (vph)	250	40	150	300	40	120
Vol.(pcph), see Table 10.1			165		44	132

STEP 1 : RT From Minor Street	V_9
Conflicting Flow, V_c	$1/2\ V_3+V_2$ = 20 + 250 = 270 vph (V_{c9})
Critical Gap, T_c and Potential Capacity, c_p	T_c = 5.5 secs (Tab.10.2) c_{p9} = 825 pcph (Fig.10.3)
Actual Capacity, c_m	$c_{m9}=c_{p9}$ = 825 pcph

STEP 2 : LT From Major Street	V_4
Conflicting Flow, V_c	V_3+V_2 = 40 + 250 = 290 vph (V_{c4})
Critical Gap, T_c and Potential Capacity, c_p	T_c = 5.0 secs (Tab.10.2) c_{p4} = 900 pcph (Fig.10.3)
% of c_p utilized and Impedance Factor (Fig.10.5)	$(v_4/c_{p4})\times100$ = 18.3% P_4 = 0.88
Actual Capacity, c_m	$c_{m4}=c_{p4}$ = 900 pcph

STEP 3 : LT From Minor Street	V_7
Conflicting Flow, V_c	$1/2\ V_3+V_2+V_5+V_4$ = 20 + 250 + 300 + 150 = 720 vph (V_{c7})
Critical Gap, T_c and Potential Capacity, c_p	T_c = 6.5 secs (Tab.10.2) c_{p7} = 350 pcph (Fig.10.3)
Actual Capacity, c_m	$c_{m7}=c_{p7}\times P_4$ = 350 x 0.88 = 308 pcph

SHARED-LANE CAPACITY

$$SH = \frac{v_7 + v_9}{(v_7/c_{m7})+(v_9/c_{m9})}$$ if lane is shared

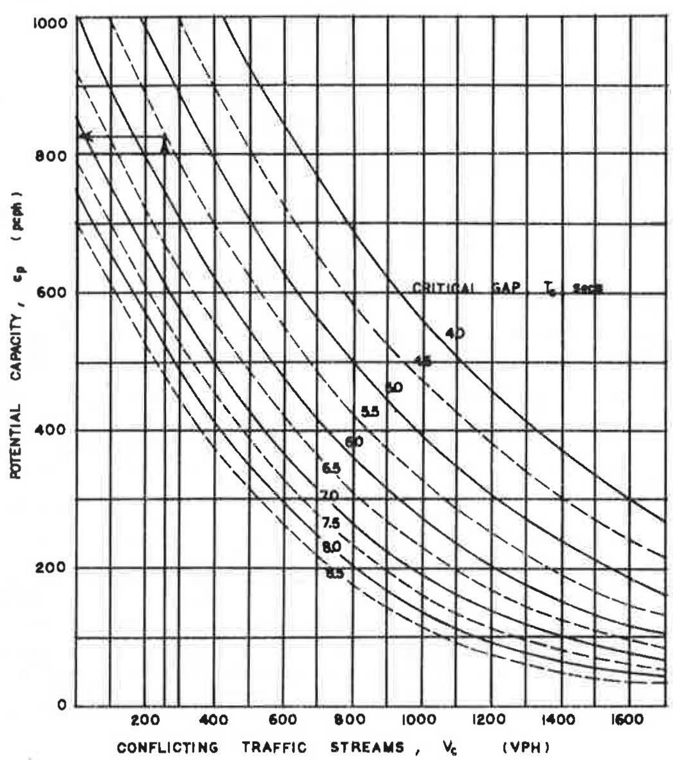
Movement no.	v(pcph)	c_m (pcph)	c_{SH} (pcph)	c_R	LOS
7	44	308	~~581~~ 308	~~405~~ 264	~~A~~ C
9	132	825	825	693	A
4	165	900	—	735	A

Figure 10-8. Worksheet for Calculation 1.

The potential capacity of the movement is found by entering Figure 10-3 with conflicting traffic of 270 vph and a critical gap of 5.5 sec. The potential capacity is found to be 825 pcph.

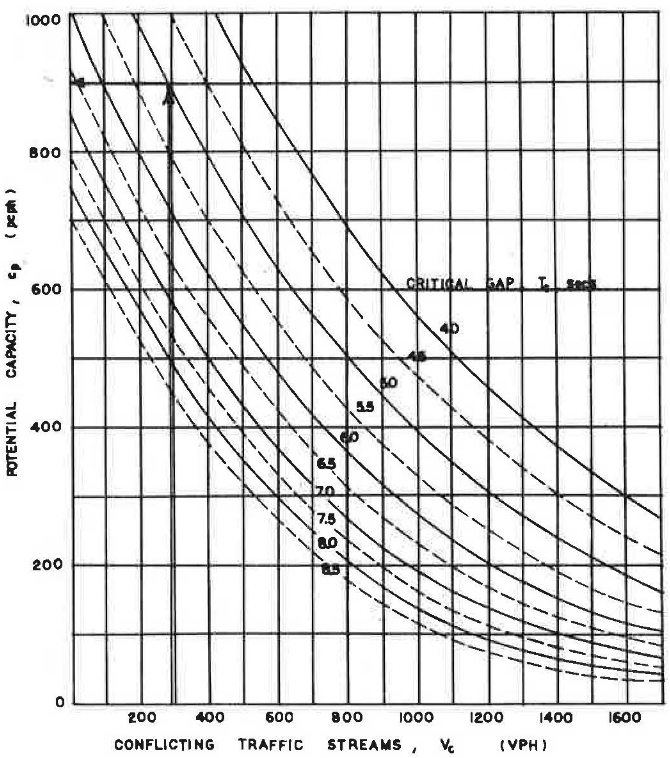
Because there are no movements which impede the minor street right turn, the movement capacity is the same as the potential capacity for this movement, or 825 pcph.

	BASIC CRITICAL GAP FOR PASSENGER CARS, SEC			
VEHICLE MANEUVER AND TYPE OF CONTROL	AVERAGE RUNNING SPEED, MAJOR ROAD 30 MPH		55 MPH	
	NUMBER OF LANES ON MAJOR ROAD 2	4	2	4
RT from Minor Road				
STOP	5.5	5.5	6.5	6.5
YIELD	5.0	5.0	5.5	5.5
LT from Major Road	5.0	5.5	5.5	6.0
Cross Major Road				
STOP	6.0	6.5	7.5	8.0
YIELD	5.5	6.0	6.5	7.0
LT from Minor Road				
STOP	6.5	7.0	8.0	8.5
YIELD	6.0	6.5	7.0	7.5



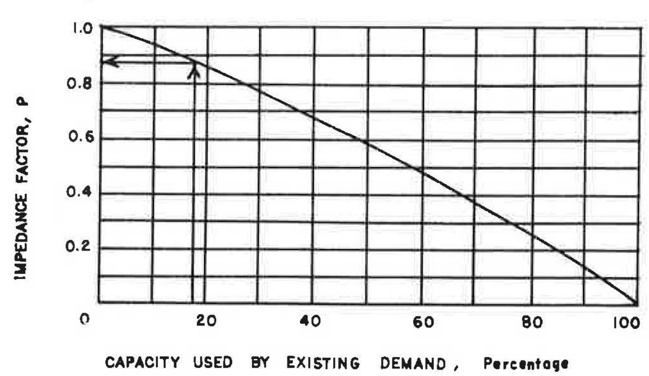
4. The second movement considered is the LT from the major street. The conflicting flow is computed as indicated on the sheet, and is found to be 290 vph. The critical gap is 5.0 sec (from Table 10-2, for major street LT, two lanes on the major street and 30-mph prevailing speed), and potential capacity is found to be 900 from Figure 10-3.

VEHICLE MANEUVER AND TYPE OF CONTROL	BASIC CRITICAL GAP FOR PASSENGER CARS, SEC — AVERAGE RUNNING SPEED, MAJOR ROAD: 30 MPH — NUMBER OF LANES ON MAJOR ROAD: 2	30 MPH: 4	55 MPH: 2	55 MPH: 4
RT from Minor Road				
STOP	5.5	5.5	6.5	6.5
YIELD	5.0	5.0	5.5	5.5
LT from Major Road	5.0	5.5	5.5	6.0
Cross Major Road				
STOP	6.0	6.5	7.5	8.0
YIELD	5.5	6.0	6.5	7.0
LT from Minor Road				
STOP	6.5	7.0	8.0	8.5
YIELD	6.0	6.5	7.0	7.5



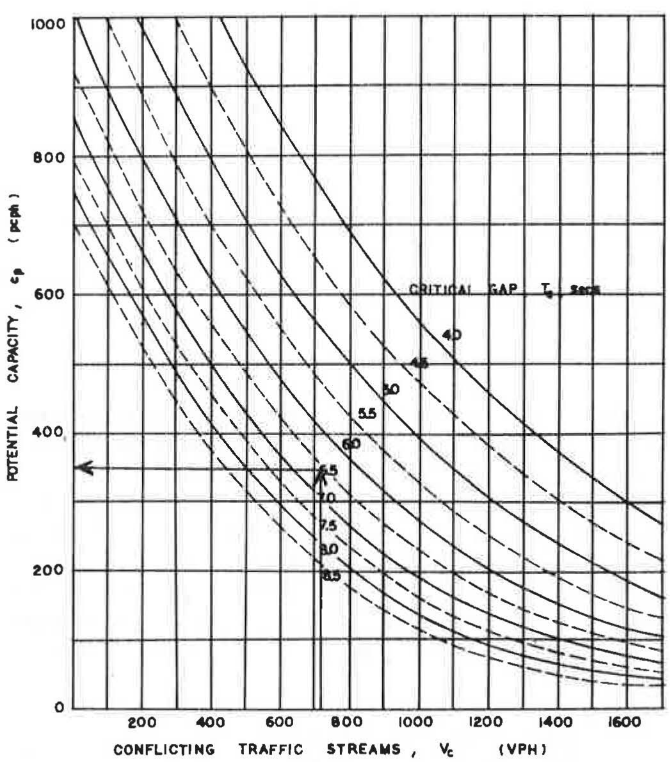
Again, there are no movements which impede the major street LT, and the movement capacity is the same as the potential capacity, 900 pcph.

An impedance factor, however, must be computed for this movement, because it, in turn, impedes the left turn from the minor street. The adjusted volume for movement 4 (the major street LT) is 165 pcph. Thus, the percent of potential capacity utilized is 165/900 = 0.183 (18.3 percent). This value is used to enter Figure 10-5 to obtain an impedance factor of 0.88.



5. The LT from the minor street is the last movement considered. The conflicting volume is computed to be 720 vph, as shown in Figure 10-8. The critical gap is found as illustrated from Table 10-2; and the potential capacity from Figure 10-3, as 350 pcph.

VEHICLE MANEUVER AND TYPE OF CONTROL	BASIC CRITICAL GAP FOR PASSENGER CARS, SEC — AVERAGE RUNNING SPEED, MAJOR ROAD: 30 MPH — NUMBER OF LANES ON MAJOR ROAD: 2	30 MPH: 4	55 MPH: 2	55 MPH: 4
RT from Minor Road				
STOP	5.5	5.5	6.5	6.5
YIELD	5.0	5.0	5.5	5.5
LT from Major Road	5.0	5.5	5.5	6.0
Cross Major Road				
STOP	6.0	6.5	7.5	8.0
YIELD	5.5	6.0	6.5	7.0
LT from Minor Road				
STOP	6.5	7.0	8.0	8.5
YIELD	6.0	6.5	7.0	7.5



The minor street LT is impeded by the major street LT (see Figure 10-4). The movement capacity is therefore found by multiplying the potential capacity by the impedance factor for the major street LT, which was found in the previous step to be 0.88. The movement capacity is therefore 350 × 0.88, or 308 pcph.

6. The final step is to determine the shared-lane capacity, reserve capacity, and LOS for the intersection. Since the problem is to examine the impact of a proposed exclusive right-turn lane, reserve capacities are computed and noted for both the existing shared-lane case and the proposed case in which each of the movements has a separate lane.

For the shared-lane case:

c_{SH} = [44 + 132]/[(44/308) + (132/825)] = 581 pcph
c_R = 581 − (44 + 132) = *405 pcph*
LOS = A

For the separate lane case:

c_R (Right Turn) = 825 − 132 = *693 pcph;* LOS = A
c_R (Left Turn) = 308 − 44 = *264 pcph;* LOS = C

The solution indicates that right-turn vehicles will not be significantly better served by an exclusive lane. The fact that left-turners seem to experience a *decrease* in LOS if given an exclusive lane must be considered in light of the discussion in the "Methodology" section of this chapter and the illustration of Table 10-4. While each left-turner will actually experience reduced delay due to the exclusive lane, their delay will be larger than the *average delay* for vehicles in the shared lane, which is dominated by right-turn movements.

All vehicles will experience some decrease in delay if a separate lane is provided for left and right turners. What this analysis suggests is that the decrease in delay will not be significant, and would not be expected to provide substantial relief to resident's complaints.

CALCULATION 2—A FOUR-LEG INTERSECTION

1. *Description*—This example concerns the intersection of Walnut Street, a four-lane arterial, and Elm Street, a two-lane collector street, in an area of population 150,000. Elm Street is STOP-controlled, and the northbound approach has recently been

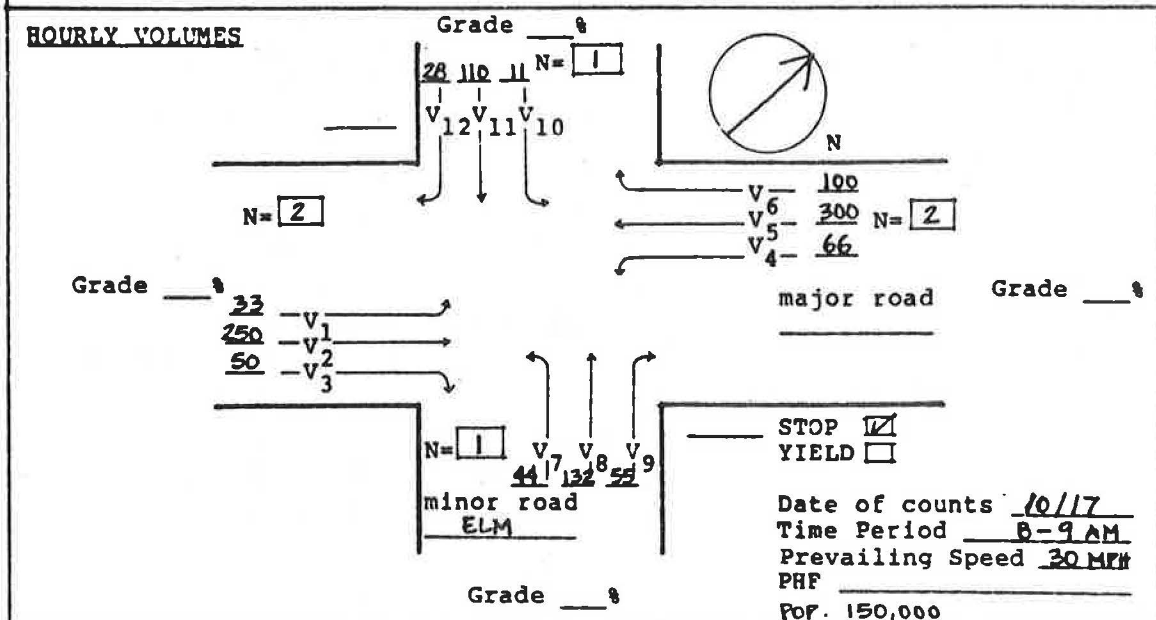
widened to add an LT turn lane. Local residents still complain that delays are excessive at this location. The intersection is shown on the worksheet for the problem, Figure 10-9.

2. *Solution*—This problem calls for a thorough evaluation of current operations and consideration of any possible improvements that might alleviate existing difficulties. All computations are performed on the four-leg intersections worksheet, and are illustrated in Figure 10-9. These computations are discussed in the items below.

1. Existing traffic volumes are shown on the diagram on page 1 of the worksheet. The critical period was determined to be the AM peak, and the volumes were obtained by taking a count during the 7 AM to 10 AM period, and identifying the peak hour, which occurred between 8 AM and 9 AM. Critical geometric features are also noted on the diagram. It should be noted that "critical period" may be the period of maximum demand on the minor legs, of maximum demand on the major street, or some other period when the combination of side street and

LOCATION: NAME:

HOURLY VOLUMES



VOLUME ADJUSTMENTS

Movement no.	1	2	3	4	5	6	7	8	9	10	11	12
Volume (vph)	33	250	50	66	300	100	44	132	55	11	110	28
Vol.(pcph), see Table 10.1	36			73			48	145	60	12	121	31

VOLUMES IN PCPH

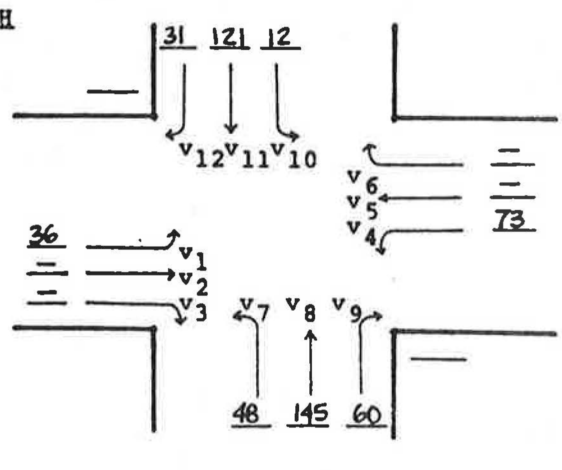


Figure 10-9. Worksheet for Calculation 2.

STEP 1 : RT From Minor Street	$\uparrow$ V_9	$\downarrow$ V_{12}
Conflicting Flows, V_c	$1/2\ V_3+V_2=V_{c9}$ 25 + 250/2 = 150 vph	$1/2\ V_6+V_5=V_{c12}$ 50 + 300/2 = 200 vph
Critical Gap, T_c (Tab.10.2)	5.5 (secs.)	5.5 (secs.)
Potential Capacity, c_p (Fig.10.3)	c_{p9} = 940 pcph	c_{p12} = 880 pcph
% of c_p utilized	(v_9/c_{p9})x100 = 6.4 %	(v_{12}/c_{p12})x100 = 3.5 %
Impedance Factor, P (Fig.10.5)	P_9 = 0.96	P_{12} = 0.98
Actual Capacity, c_m	$c_{m9}=c_{p9}$ = 940 pcph	$c_{m12}=c_{p12}$ = 880 pcph
STEP 2 : LT From Major Street	V_4	V_1
Conflicting Flows, V_c	$V_3+V_2=V_{c4}$ 50 + 250 = 300 vph	$V_6+V_5=V_{c1}$ 100 + 300 = 400 vph
Critical Gap, T_c (Tab.10.2)	5.5 (secs.)	5.5 (secs.)
Potential Capacity, c_p (Fig.10.3)	c_{p4} = 790 pcph	c_{p1} = 695 pcph
% of c_p utilized	(v_4/c_{p4})x100 = 9.2 %	(v_1/c_{p1})x100 = 5.2 %
Impedance Factor, P (Fig.10.5)	P_4 = 0.93	P_1 = 0.97
Actual Capacity, c_m	$c_{m4}=c_{p4}$ = 790 pcph	$c_{m1}=c_{p1}$ = 695 pcph
STEP 3 : TH From Minor Street	V_8	V_{11}
Conflicting Flows, V_c	$1/2V_3+V_2+V_1+V_6+V_5+V_4=V_{c8}$ 25+250+33+100+300+66 = 774 vph	$1/2V_6+V_5+V_4+V_3+V_2+V_1=V_{c11}$ 50+300+66+50+250+33 = 749 vph
Critical Gap, T_c (Tab.10.2)	6.5 (secs.)	6.5 (secs.)
Potential Capacity, c_p (Fig.10.3)	c_{p8} = 330 pcph	c_{p11} = 340 pcph
% of c_p utilized	(v_8/c_{p8})x100 = 43.9 %	(v_{11}/c_{p11})x100 = 35.6 %
Impedance Factor, P (Fig.10.5)	P_8 = 0.65	P_{11} = 0.72
Actual Capacity, c_m	$c_{m8}=c_{p8}xP_1xP_4$ 298 = 330x0.97x0.93 (pcph)	$c_{m11}=c_{p11}xP_1xP_4$ 307 = 340x0.97x0.93 (pcph)
STEP 4 : LT From Minor Street	V_7	V_{10}
Conflicting Flows, V_c	V_8(step3)$+V_{11}+V_{12}=V_{c7}$ 774 + 110 + 28 = 912 vph	V_{c11}(step3)$+V_7+V_8=V_{c10}$ 749 + 44 + 132 = 925 vph
Critical Gap, T_c (Tab.10.2)	7.0 (secs.)	7.0 (secs.)
Potential Capacity, c_p (Fig.10.3)	c_{p7} = 230 pcph	c_{p10} = 205 pcph
Actual Capacity, c_m	$c_{m7}=c_{p7}xP_1xP_4xP_{11}xP_{12}$ 146 = 230x0.97x.93x.72x.98 (pcph)	$c_{m10}=c_{p10}xP_4xP_1xP_8xP_9$ 115 = 205x.93x.97x.65x.96 (pcph)

Figure 10-9. Worksheet for Calculation 2 (Continued).

major street flows is critical. If the analysis is unclear as to the "critical period," several appropriate periods should be subjected to analysis.

As no specifics of vehicle classification are given, and no grade is present, all subject volumes are multiplied by 1.1 to reflect normal traffic composition.

Adjusted volumes are shown on the lower diagram of page 1 for convenience in their use later in the problem.

2. Movements are now considered in priority sequence, in pairs. The first movements to be analyzed are the right turns from the minor street, noted as movements 9 and 12 on the worksheet.

In computing conflicting volumes, note that only one-half of the major street through volumes are included. This is because the major street has two lanes in each direction, and right-turning vehicles merge with a traffic stream consisting of only approximately half these movements. See Figure 10-2 and discussion for a fuller explanation of this effect.

SHARED LANE CAPACITY

$$c_{SH} = \frac{v_i + v_j}{(v_i / c_{mi}) + (v_j / c_{mj})}$$ where 2 movements share a lane

$$c_{SH} = \frac{v_i + v_j + v_k}{(v_i / c_{mi}) + (v_j / c_{mj}) + (v_k / c_{mk})}$$ where 3 movements share a lane

APPROACH MOVEMENTS 7, 8, 9

Movement	v(pcph)	c_m(pcph)	c_{SH}(pcph)	$c_R = c_{SH} - v$	LOS
7	48	146	146	98	E
8	145	298	~~372~~	~~167~~ 153	~~D~~ D
9	60	940		880	A

APPROACH MOVEMENTS 10, 11, 12

Movement	v(pcph)	c_m(pcph)	c_{SH}(pcph)	$c_R = c_{SH} - v$	LOS
10	12	115	115	103	D
11	121	307	308 307	144 186	D D
12	31	880	880	849	A

MAJOR STREET LEFT TURNS 1, 4

Movement	v (pcph)	c_m (pcph)	$c_R = c_m - v$	LOS
1	695	36	659	A
4	790	73	717	A

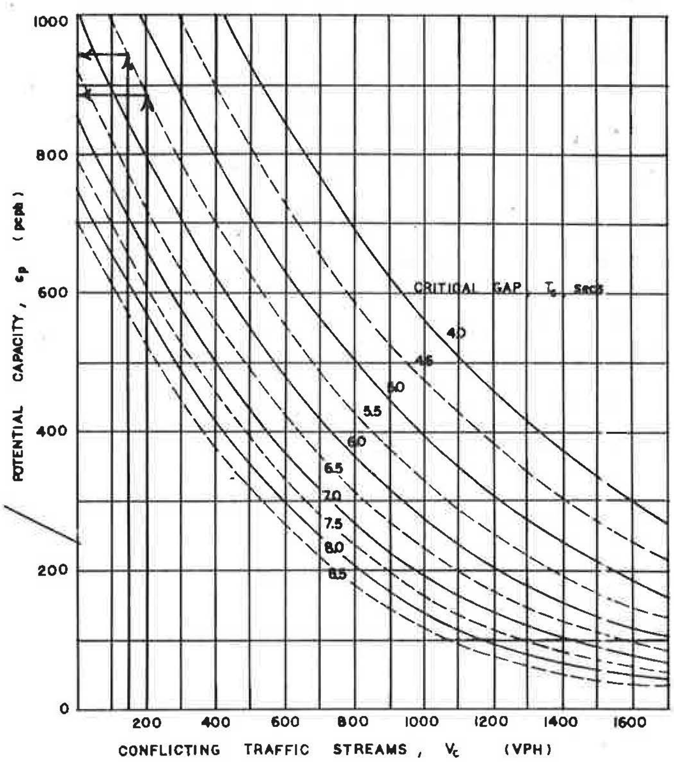
COMMENTS:

RT LANE WOULD SIGNIFICANTLY IMPROVE RT OPERATION, BUT WOULD NOT HAVE GREAT IMPACT ON THE REST OF THE INTERSECTION.

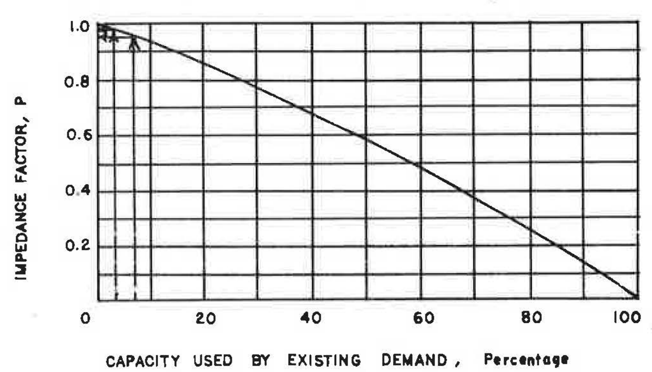
Figure 10-9. Worksheet for Calculation 2 (Continued).

The determination of critical gaps for the right-turn movements from Table 10-2 and the solution of potential capacity for these movements are illustrated below. There are no conditions warranting an adjustment in the basic critical gap determination.

BASIC CRITICAL GAP FOR PASSENGER CARS, SEC				
VEHICLE MANEUVER AND TYPE OF CONTROL	AVERAGE RUNNING SPEED, MAJOR ROAD			
	30 MPH		55 MPH	
	NUMBER OF LANES ON MAJOR ROAD			
	2	4	2	4
RT from Minor Road				
STOP	5.5	5.5	6.5	6.5
YIELD	5.0	5.0	5.5	5.5
LT from Major Road	5.0	5.5	5.5	6.0
Cross Major Road				
STOP	6.0	6.5	7.5	8.0
YIELD	5.5	6.0	6.5	7.0
LT from Minor Road				
STOP	6.5	7.0	8.0	8.5
YIELD	6.0	6.5	7.0	7.5



Impedance factors are also computed, as these right turns will impede opposing left turns from the minor street. Movement 9 utilizes 60/940, or 0.064 (6.4 percent), of its potential capacity. Movement 12 utilizes 31/880, or 0.035 (3.5 percent), of its capacity. These values are used to enter Figure 10.5 to determine the respective impedance factors that are listed on the worksheet, 0.96 for movement 9 and 0.98 for movement 12.

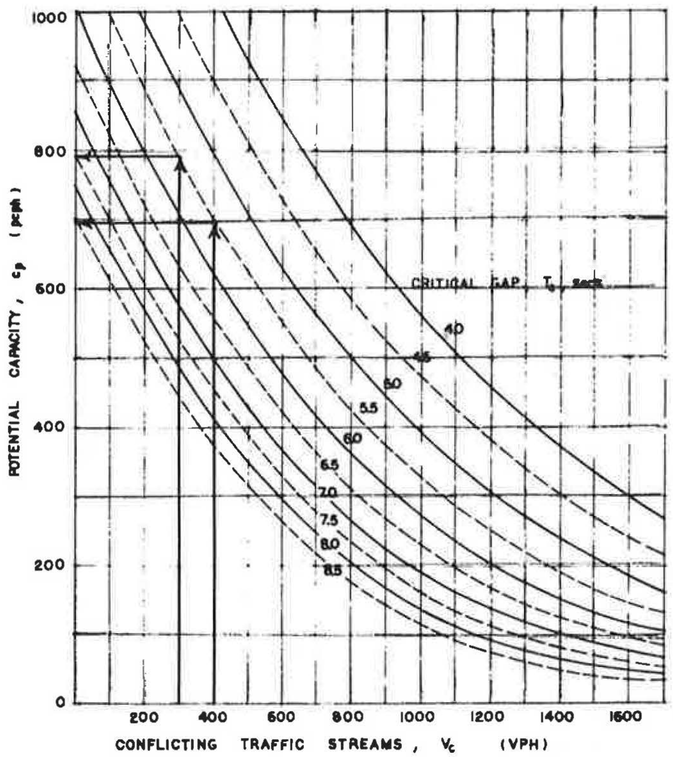


Because the right turns from the minor street are not impeded by other movements, the movement capacities are the same as the potential capacities.

3. Left turns from the major street are the next movements to be considered. These conflict with the total opposing through and right-turn movements, as indicated by the conflicting volume computations on the worksheet.

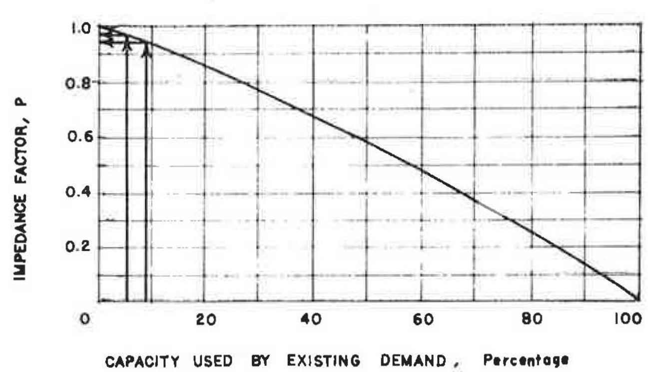
The solution of critical gaps and potential capacity for these movements is illustrated below.

BASIC CRITICAL GAP FOR PASSENGER CARS, SEC				
VEHICLE MANEUVER AND TYPE OF CONTROL	AVERAGE RUNNING SPEED, MAJOR ROAD			
	30 MPH		55 MPH	
	NUMBER OF LANES ON MAJOR ROAD			
	2	4	2	4
RT from Minor Road				
STOP	5.5	5.5	6.5	6.5
YIELD	5.0	5.0	5.5	5.5
LT from Major Road	5.0	5.5	5.5	6.0
Cross Major Road				
STOP	6.0	6.5	7.5	8.0
YIELD	5.5	6.0	6.5	7.0
LT from Minor Road				
STOP	6.5	7.0	8.0	8.5
YIELD	6.0	6.5	7.0	7.5



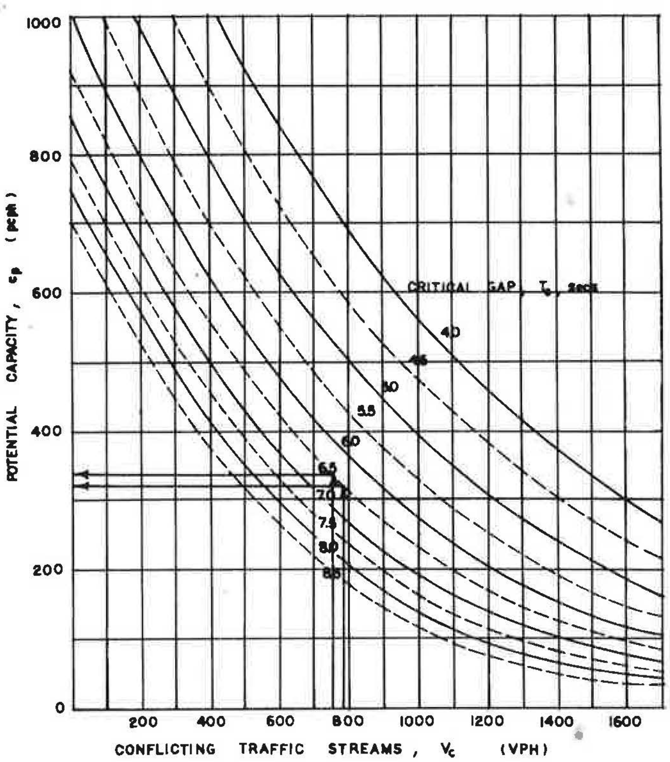
Again, impedance factors will be computed, because the LT from the major street will impede all lower priority movements. Movement 4 utilizes 73/790, or 0.092 (9.2 percent), of its potential capacity, and movement 1 utilizes 36/695, or 0.052 (5.2 percent), of its capacity. These values are used to enter Figure 10-5 to find the impedance factors listed in the worksheet, 0.93 for movement 4 and 0.97 for movement 1.

4. The minor street through movements (Nos. 8 and 11) are the next to be considered in the computational process. Again, conflicting traffic volumes are computed as shown, in accordance with the specifications of Figure 10-2. The critical gap for

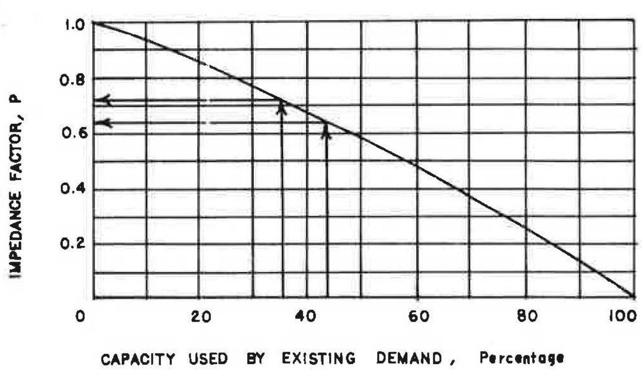


these through movements is found to be 6.5 sec from Table 10-2, and the unadjusted capacities to be 330 pcph and 340 pcph, respectively. These findings are illustrated below.

BASIC CRITICAL GAP FOR PASSENGER CARS, SEC				
VEHICLE MANEUVER AND TYPE OF CONTROL	AVERAGE RUNNING SPEED, MAJOR ROAD			
	30 MPH		55 MPH	
	NUMBER OF LANES ON MAJOR ROAD			
	2	4	2	4
RT from Minor Road				
STOP	5.5	5.5	6.5	6.5
YIELD	5.0	5.0	5.5	5.5
LT from Major Road	5.0	5.5	5.5	6.0
Cross Major Road				
STOP	6.0	6.5	7.5	8.0
YIELD	5.5	6.0	6.5	7.0
LT from Minor Road				
STOP	6.5	7.0	8.0	8.5
YIELD	6.0	6.5	7.0	7.5



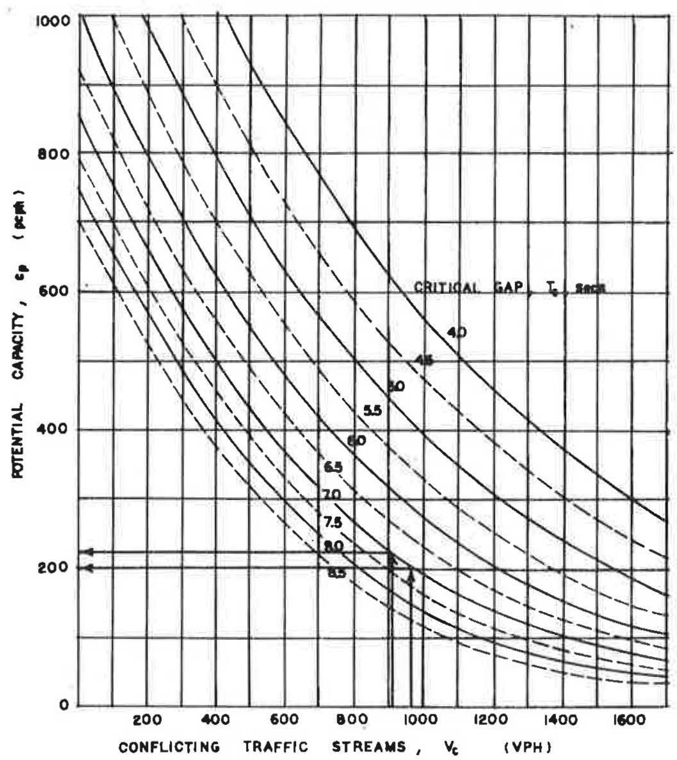
Impedance factors are also computed for these movements, as they will impede left turns from the minor street, a lower priority movement. Movement 8 uses 145/330, or 0.439 (43.9 percent), of its potential capacity, and movement 11 uses 121/340, or 0.356 (35.6 percent), of its potential capacity. Entering Figure 10-5 with these values, the impedance factors shown on the worksheet are found, 0.65 and 0.72 respectively.



From Figure 10-4, it is apparent that the through movements from the minor street are impeded by left turns from the major street. Thus, movement capacities are computed as shown by multiplying by the appropriate impedance factors.

5. The final movements to be considered are the lowest priority movements: left turns from the minor street. Computations for conflicting volumes are as shown, in accordance with Figure 10-2. The critical gap for these movements is 7.0 sec, and the potential capacities are as shown below.

VEHICLE MANEUVER AND TYPE OF CONTROL	BASIC CRITICAL GAP FOR PASSENGER CARS, SEC			
	AVERAGE RUNNING SPEED, MAJOR ROAD			
	30 MPH		55 MPH	
	NUMBER OF LANES ON MAJOR ROAD			
	2	4	2	4
RT from Minor Road				
STOP	5.5	5.5	6.5	6.5
YIELD	5.0	5.0	5.5	5.5
LT from Major Road	5.0	5.5	5.5	6.0
Cross Major Road				
STOP	6.0	6.5	7.5	8.0
YIELD	5.5	6.0	6.5	7.0
LT from Minor Road				
STOP	6.5	7.0	8.0	8.5
YIELD	6.0	6.5	7.0	7.5



Left turns from the minor street are impeded by left turns from the major street and the opposing through and right-turn movements from the minor street. Thus, movement capacities must be reduced by four different impedance factors as shown on the worksheet.

6. The third page of the worksheet shown in Figure 10-9 illustrates the shared-lane computations for this intersection. Movements 8 and 9 currently share a northbound lane, while movement 7 has an exclusive lane. Southbound movements 10, 11, and 12 all share a single lane. Note that reserve capacity computations are carried out for the shared-lane condition, and for each movement as if it had its own separate lane.

Shared-lane capacities were computed as follows:

$$c_{SH}(8,9) = [145 + 60]/[(145/298) + (60/940)] = 372 \text{ pcph}$$

$$c_{SH}(10,11,12) = [12 + 121 + 31]/[(12/115) + (121/307) + (31/880)] = 308 \text{ pcph}$$

The results shown on the worksheet tend to justify the residents complaints. Levels-of-service D and E prevail, indicating long or very long delays. Some interesting points are seen, however. Provision of exclusive right-turn lanes on each approach would significantly improve the operation of those movements, but would not have a great impact on the majority of vehicles. Provision of a left-turn lane for the southbound approach would not yield significant improvements.

It would therefore be advisable for a more exhaustive field study to be made in conjunction with serious consideration of signalization and/or other measures for this location. As part of the consideration of signalization, it would be important to collect field data on existing delays. These could be compared to predicted values if signals were installed (Chapter 9), so that the values could be compared. Signalization could either increase or decrease delays, depending on timing, geometrics, volumes, and other local conditions.

CALCULATION 3—A SUBURBAN INTERSECTION WITH HIGH APPROACH SPEEDS

1. *Description*—This intersection is an intersection of two-lane Benton Highway and a local street, Mill Road. Mill Road is on a 2 percent grade, and has a traffic composition of 85 percent passenger cars, 12 percent single-unit trucks, and 3 percent combination vehicles. Each approach has two lanes, an LT lane and an RT-TH lane. The intersection is an area of population 300,000, the PHF is 0.88, and approach speeds on Benton Highway are 55 mph. The problem is to evaluate current operations at the intersection. The intersection is YIELD-controlled.

2. *Solution*—

a. Volumes in vph are given as shown on Figure 10-10 (page 1). These must be adjusted to reflect the grade and vehicle mix stated in the problem. From Table 10-1, the following pce values are found:

Grade (%)	*Single Unit Trucks*	*Combination Vehicles*	*Passenger Cars*
−2	1.2	1.5	0.9
+2	2.0	3.0	1.2
0	1.5	2.0	1.0

Passenger-car equivalent computations are illustrated in the following table:

Volume (vph)	*PC*	*SUT*	*CV*	*Volume (pcph)*
V_1	60 × 0.85 × 1.0 +	60 × 0.12 × 1.5 +	60 × 0.03 × 2.0 =	65
V_4	40 × 0.85 × 1.0 +	40 × 0.12 × 1.5 +	40 × 0.03 × 2.0 =	44
V_7	20 × 0.85 × 1.2 +	20 × 0.12 × 2.0 +	20 × 0.03 × 3.0 =	27
V_8	40 × 0.85 × 1.2 +	40 × 0.12 × 2.0 +	40 × 0.03 × 3.0 =	54
V_9	10 × 0.85 × 1.2 +	10 × 0.12 × 2.0 +	10 × 0.03 × 3.0 =	14
V_{10}	10 × 0.85 × 0.9 +	10 × 0.12 × 1.2 +	10 × 0.03 × 1.5 =	10
V_{11}	20 × 0.85 × 0.9 +	20 × 0.12 × 1.2 +	20 × 0.03 × 1.5 =	19
V_{12}	120 × 0.85 × 0.9 +	120 × 0.12 × 1.2 +	120 × 0.03 × 1.5 =	115

These volumes are illustrated on the lower portion of Figure 10-10 (page 1).

b. The first movements to be considered are the right turns from the minor street. (It is helpful to refer to figures and tables in the text when reviewing this problem because they are not repeated here.) Conflicting volumes, potential capacities, and impedance factors are all selected according to normal procedures. *Note, however, that the critical gap of 5.5 sec selected from Table 10-2 may be reduced by 0.5 sec due to the population of the area, which exceeds 250,000 persons.* All critical gaps selected in this problem are subject to the same reduction.

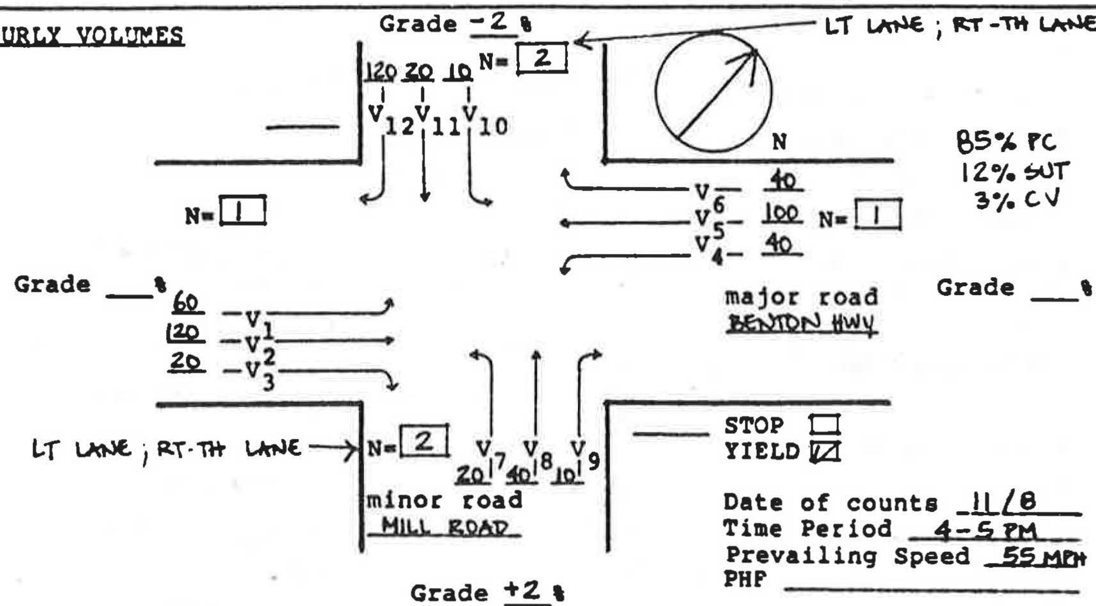
ADJUSTMENTS AND MODIFICATIONS TO CRITICAL GAP, SEC

CONDITION	ADJUSTMENT
RT from Minor Street: Curb radius > 50 ft or turn angle < 60°	−0.5
RT from Minor Street: Acceleration lane provided	−1.0
All movements: Population ≥ 250,000	−0.5
Restricted sight distance.[a]	up to +1.0

NOTES: Maximum total decrease in critical gap = 1.0 sec.
Maximum Critical gap = 8.5 sec.
For values of average running speed between 30 and 55 mph, interpolate.
[a] This adjustment is made for the specific movement impacted by restricted sight distance.

LOCATION: **NAME:**

HOURLY VOLUMES



VOLUME ADJUSTMENTS

Movement no.	1	2	3	4	5	6	7	8	9	10	11	12
Volume (vph)	60	120	20	40	100	40	20	40	10	10	20	120
Vol.(pcph), see Table 10.1	65			44			27	54	14	10	19	115

VOLUMES IN PCPH

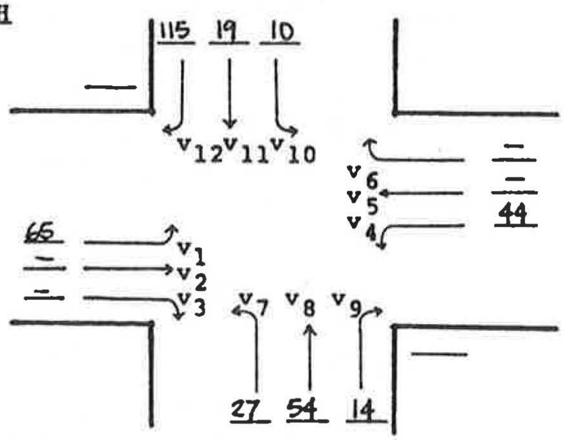


Figure 10-10. Worksheet for Calculation 10.

STEP 1 : RT From Minor Street	V_9	V_{12}
Conflicting Flows, V_c	$1/2\ V_3+V_2=V_{c9}$ 10 + 120 = 130 vph	$1/2\ V_6+V_5=V_{c12}$ 20 + 100 = 120 vph
Critical Gap, T_c (Tab.10.2)	(5.5 − 0.5) 5.0 (secs.)	(5.5 − 0.5) 5.0 (secs.)
Potential Capacity, c_p (Fig.10.3)	c_{p9} = 1000 pcph	c_{p12} = 1000 pcph
% of c_p utilized	$(v_9/c_{p9})\times100$ = 1.4 %	$(v_{12}/c_{p12})\times100$ = 11.5 %
Impedance Factor, P (Fig.10.5)	P_9 = 0.99	P_{12} = 0.91
Actual Capacity, c_m	$c_{m9}=c_{p9}$ = 1000 pcph	$c_{m12}=c_{p12}$ = 1000 pcph
STEP 2 : LT From Major Street	V_4	V_1
Conflicting Flows, V_c	$V_3+V_2=V_{c4}$ 20 + 120 = 140 vph	$V_6+V_5=V_{c1}$ 40 + 100 = 140 vph
Critical Gap, T_c (Tab.10.2)	(5.5 − 0.5) 5.0 (secs.)	(5.5 − 0.5) 5.0 (secs.)
Potential Capacity, c_p (Fig.10.3)	c_{p4} = 1000 pcph	c_{p1} = 1000 pcph
% of c_p utilized	$(v_4/c_{p4})\times100$ = 4.4 %	$(v_1/c_{p1})\times100$ = 6.5 %
Impedance Factor, P (Fig.10.5)	P_4 = 0.98	P_1 = 0.97
Actual Capacity, c_m	$c_{m4}=c_{p4}$ = 1000 pcph	$c_{m1}=c_{p1}$ = 1000 pcph
STEP 3 : TH From Minor Street	V_8	V_{11}
Conflicting Flows, V_c	$1/2V_3+V_2+V_1+V_6+V_5+V_4=V_{c8}$ 10 + 120 + 60 + 40 + 100 + 40 = 370 vph	$1/2V_6+V_5+V_4+V_3+V_2+V_1=V_{c11}$ 20 + 100 + 40 + 20 + 120 + 60 = 360 vph
Critical Gap, T_c (Tab.10.2)	(6.5 − 0.5) 6.0 (secs.)	(6.5 − 0.5) 6.0 (secs.)
Potential Capacity, c_p (Fig.10.3)	c_{p8} = 640 pcph	c_{p11} = 670 pcph
% of c_p utilized	$(v_8/c_{p8})\times100$ = 8.4 %	$(v_{11}/c_{p11})\times100$ = 2.8 %
Impedance Factor, P (Fig.10.5)	P_8 = 0.95	P_{11} = 0.99
Actual Capacity, c_m	$c_{m8}=c_{p8}\times P_1\times P_4$ 608 = 640 x .97 x .98 (pcph)	$c_{m11}=c_{p11}\times P_1\times P_4$ 637 = 670 x .97 x .98 (pcph)
STEP 4 : LT From Minor Street	V_7	V_{10}
Conflicting Flows, V_c	V_8 (step3) $+V_{11}+V_{12}=V_{c7}$ 370 + 20 + 120 = 510 vph	V_{c11} (step3) $+V_7+V_8=V_{c10}$ 360 + 20 + 40 = 420 vph
Critical Gap, T_c (Tab.10.2)	(7.0 − 0.5) 6.5 (secs.)	(7.0 − 0.5) 6.5 (secs.)
Potential Capacity, c_p (Fig.10.3)	c_{p7} = 475 pcph	c_{p10} = 540 pcph
Actual Capacity, c_m	$c_{m7}=c_{p7}\times P_1\times P_4\times P_{11}\times P_{12}$ 407 = 475 x .97 x .98 x .99 x .91 (pcph)	$c_{m10}=c_{p10}\times P_4\times P_1\times P_8\times P_9$ 463 = 540 x .98 x .97 x .95 x .99 (pcph)

Figure 10-10. Worksheet for Calculation 10 (Continued).

SHARED LANE CAPACITY

$$c_{SH} = \frac{v_i + v_j}{(v_i / c_{mi}) + (v_j / c_{mj})}$$ where 2 movements share a lane

$$c_{SH} = \frac{v_i + v_j + v_k}{(v_i / c_{mi}) + (v_j / c_{mj}) + (v_k / c_{mk})}$$ where 3 movements share a lane

APPROACH MOVEMENTS 7, 8, 9

Movement	v(pcph)	c_m(pcph)	c_{SH}(pcph)	$c_R = c_{SH} - v$	LOS
7	27	407	407	380	B
8	54	608	661 608	593 554	A A
9	14	1000	1000	986	A

APPROACH MOVEMENTS 10, 11, 12

Movement	v(pcph)	c_m(pcph)	c_{SH}(pcph)	$c_R = c_{SH} - v$	LOS
10	10	483	483	481	A
11	19	637	925 637	791 618	A A
12	115	1000	1000	885	A

MAJOR STREET LEFT TURNS 1, 4

Movement	v (pcph)	c_m (pcph)	$c_R = c_m - v$	LOS
1	1000	65	935	A
4	1000	44	956	A

COMMENTS:

THE INTERSECTION OPERATES ACCEPTABLY

Figure 10-10. Worksheet for Calculation 10 (Continued).

c. Left turns from the major street, through movements from the minor street, and left turns from the minor street, are all considered in sequence on page 2 of Figure 10-10. Values are found from the tables and figures listed on the worksheet.

d. Movements 8 and 9 share a lane, and movements 11 and 12 share a lane. Movements 7 and 10 have exclusive use of a left-turn lane. Shared-lane capacity computations are as follows:

$$c_{SH}(8,9) = [54 + 14]/[(54/608) + (14/1{,}000)] = 661 \text{ pcph}$$
$$c_{SH}(11,12) = [19 + 115]/[(19/637) + (115/1{,}000)] = 925 \text{ pcph}$$

As is seen from the results shown on Figure 10-10, this intersection operates at acceptable levels of service (A and B) during the period of interest. It is also seen that giving through and right-turning vehicles exclusive lanes would not meaningfully improve operations.

CALCULATION 4—AN OBTUSE-ANGLE CHANNELIZED INTERSECTION

1. *Description*—Calculation 4 concerns the intersection of Jerico Drive and Main Street, a suburban intersection in an area of population 150,000. As the T-intersection contains several important geometric features, a schematic sketch of the intersection is shown in Figure 10-11. Note that the intersection is channelized and that right turns from Jerico Drive are made at a shallow angle. The right turn from Jerico Drive is YIELD-controlled, while the left-turn is STOP-controlled. The problem is to evaluate the operation of the intersection.

2. *Solution*—The solution to this problem is done on the T-intersection worksheet, and is shown in Figure 10-12. As individual steps have been discussed in detail in previous problems, only the unique analysis points of this problem are highlighted below:

a. Demand volumes are shown on the worksheet. No grades are present, and no traffic composition is given. An adjustment factor of 1.1 is taken from Table 10-1 to reflect normal traffic distribution.

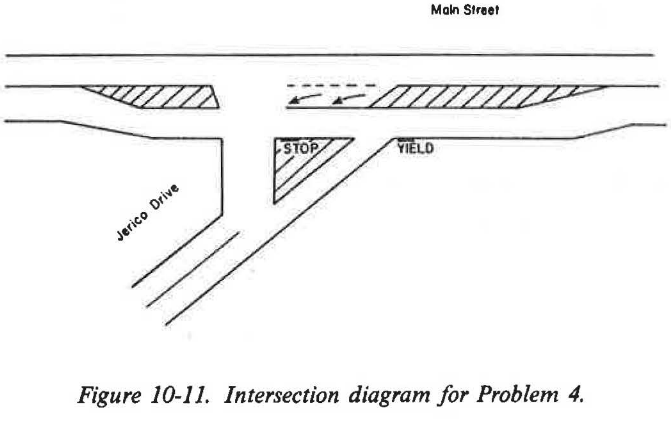


Figure 10-11. Intersection diagram for Problem 4.

b. In the selection of critical gaps, note that the right turn from the minor street is YIELD-controlled, and that the basic critical gap from Table 10-2 may be reduced by 0.5 sec due to the shallow angle of the turn. The left turn from the minor street is STOP-controlled.

c. In the computation of conflicting volume for the right turn from the minor street (Step 1), the right turn from the major street is *not* included, as the intersection channelization separates these two movements by a considerable distance.

d. There are no shared-lane computations to be made because each subject movement has its own lane.

The results indicate that the right-turn movement operates at LOS A and the left-turn movement at LOS E, even though the right-turn movement is the far heavier of the two. There is little that can be done to alleviate conflicts for the left turns, so that consideration might be given to signalizing this movement, perhaps with an actuated signal, despite its low volume. A thorough study on this point, however, should be made, including consideration of accidents, a traffic conflict study and analysis, and observation of delays and gap acceptance behavior. The right-turn movement should remain as at present.

V. REFERENCES

As noted earlier, the methodology presented in this chapter is based on a publication of the OECD *(2)*, which is a translation of an earlier methodology developed in Germany *(1)*. The Swedish Capacity Manual also contains a methodology for unsignalized intersections *(3)*. A number of interesting studies have also treated various aspects of unsignalized intersections, and may be of interest to the user *(4–8)*. Material on multiway STOP-controlled intersections is taken from Refs. *9* and *10*.

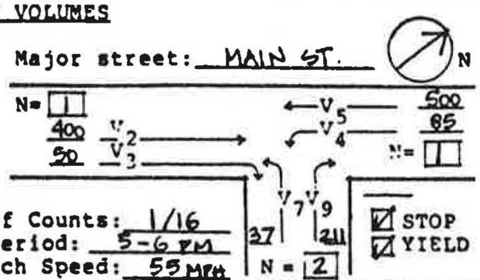
1. "Merkblatt for Lichtsignalanlagen an Landstrassen Ausgasbe." *Forschungsgesellschaft fur das Strassenwesen,* Forschungsgesellschaft fur das Strassenwesen, Koln, Germany (1972).
2. *Capacity of At-Grade Junctions.* Organization for Economic Cooperation and Development, Paris, France (1974).
3. HANNSON, ET AL., "Capacity of Unsignalized Intersections—Swedish Capacity Manual." *Transportation Research Record 667,* Transportation Research Board, Washington, D.C. (1980).
4. UBER, M., "Start-Up Times and Queue Acceptance of Large Gaps at T-Junctions." *Transportation Engineering,* Institute of Transportation Engineers, Washington, D.C. (April 1978).
5. SANDERSON, "Priority Rules at Uncontrolled Intersections." *Traffic Research Report 6,* New Zealand Ministry of Transport, Wellington, New Zealand (1974).

Figure 10-12. *Worksheet for Problem 4.*

LOCATION:	NAME:

HOURLY VOLUMES

Major street: MAIN ST.



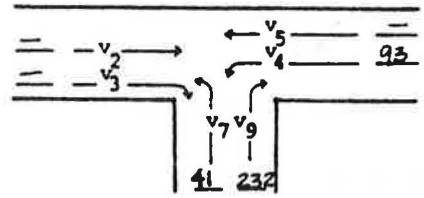
Grade ___ %

Date of Counts: 1/16
Time Period: 5-6 PM
Approach Speed: 55 MPH
PHF: ______
POP. 150,000

☑ STOP
☑ YIELD

minor street: JERICO DR. Grade ___ %

VOLUMES IN PCPH



VOLUME ADJUSTMENTS

Movement no.	2	3	4	5	7	9
Volume (vph)	400	50	85	500	37	211
Vol.(pcph), see Table 10.1			94		41	232

STEP 1 : RT From Minor Street	V_9
Conflicting Flow, V_c	$1/2\ V_3+V_2=$ — $+$ 400 $=$ 400 vph (V_{c9})
Critical Gap, T_c and Potential Capacity, c_p	$T_c=$ 5.0 secs (Tab.10.2) $c_{p9}=$ 795 pcph (Fig.10.3)
Actual Capacity, c_m	$c_{m9}=c_{p9}=$ 795 pcph
STEP 2 : LT From Major Street	V_4
Conflicting Flow, V_c	$V_3+V_2=$ 50 $+$ 400 $=$ 450 vph (V_{c4})
Critical Gap, T_c and Potential Capacity, c_p	$T_c=$ 5.5 secs (Tab.10.2) $c_{p4}=$ 650 pcph (Fig.10.3)
% of c_p utilized and Impedance Factor (Fig.10.5)	$(v_4/c_{p4})\times 100=$ 14.3% $P_4=$ 0.90
Actual Capacity, c_m	$c_{m4}=c_{p4}=$ 650 pcph
STEP 3 : LT From Minor Street	V_7
Conflicting Flow, V_c	$1/2\ V_3+V_2+V_5+V_4=$ 25 $+$ 400 $+$ 500 $+$ 85 $=$ 1010 vph (V_{c7})
Critical Gap, T_c and Potential Capacity, c_p	$T_c=$ 8.0 secs (Tab.10.2) $c_{p7}=$ 140 pcph (Fig.10.3)
Actual Capacity, c_m	$c_{m7}=c_{p7}\times P_4=$ 145 $\times$ 0.90 $=$ 126 pcph

SHARED-LANE CAPACITY

$$SH = \frac{v_7 + v_9}{(v_7/c_{m7}) + (v_9/c_{m9})}$$ if lane is shared

Movement no.	v (pcph)	c_m (pcph)	c_{SH} (pcph)	c_R	LOS
7	41	126	126	85	E
9	232	795	795	563	A
4	94	650	650	556	A

6. VASARHELYI, "Stochastic Simulation of the Traffic of an Uncontrolled Road Intersection." *Transportation Research,* Pergamon Press, Elmsford, N.Y. (Aug. 1976).
7. BLUMENFELD and WEISS. "The Effects of Gap Acceptance Criteria on Merging Delay and Capacity at an Uncontolled Intersection." *Traffic Engineering and Control,* Printerhall Ltd., London, England (Jan. 1979).
8. FRY, ET AL., "Delay and Interference in Combined Lanes at Unsignalized Intersections." *Australian Road Research Board Conference Proceedings,* Australian Road Research Board, Australia (1970).
9. HERBERT, J., "A Study of Four-Way Stop Intersection Capacities." *Highway Research Record 27,* Highway Research Board, Washington, D.C. (1963).
10. *Energy Saving Traffic Operations Project Manual.* Barton-Aschman Associates Inc., Evanston, Ill. (Sept. 1981).

APPENDIX I

APPLICATION OF PROCEDURES TO PLATOON FLOW ON THE MAJOR STREET

The procedures of this chapter assume that arrivals within the major street traffic stream are random. While this is a reasonable assumption for rural intersections, arrivals along major urban streets are rarely random. The existence of signal systems in urban street networks creates traffic streams which are organized into platoons of vehicles. Platoon flow is substantially different from a random pattern, because intermittent groups of vehicles arrive, followed by substantial gaps between groups (or platoons) in which flow is light.

The procedures of this chapter can be applied to the analysis of platoon flow along the major street by examining the time-space diagram for the street, and considering the location of the STOP- or YIELD-controlled intersection within the platoon flow pattern created by signalization.

In general, the following information is needed for this application:

1. The location of signalized intersections adjacent to the STOP- or YIELD-controlled intersection under study.
2. The timing of adjacent signals and the offset between them.
3. The average running speed of vehicles in platoons on the major street.
4. The percentage of major street flow which takes place in platoons.
5. All other information normally required for an unsignalized intersection analysis.

The application uses a traditional time-space diagram to identify the relative arrival pattern for platoons in both directions on the major street. Time is subdivided into discrete intervals during which the side-street vehicle is faced with crossing uniform conflicting flows. Separate analyses are done for each discrete interval, using the procedures of this chapter. After all intervals have been analyzed, the results are combined to determine the capacity of the minor street approaches. This application assumes that flow within platoons is random.

The application is best illustrated by example. Consider the intersection shown in Figure I.10-1. It shows a two-lane minor street with an NB volume of 200 vph and an SB volume of 100 vph. It intersects a two-lane major street with volumes of 500 vph in each direction. The intersection is located between two signalized intersections on the major street. For the sake of simplicity, all volumes include only passenger cars, there are no grades, and all minor street traffic is crossing straight through the intersection.

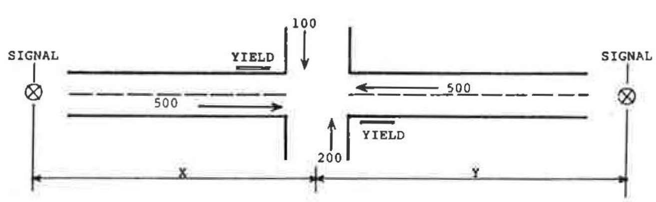


Figure I.10-1. Problem for illustration of platoon flow application.

Figure I.10-2 shows the time-space diagram for the major street. The two signalized intersections are spaced 1,320 ft apart. A 60-sec cycle is used, and each intersection has a 50-50 split of green time. An alternating progression is used, i.e., when one intersection is RED, the other is GREEN, and vice-versa. This progression provides for perfect progression in both directions at a speed of 44 fps, or 30 mph.

Depending on the location of the side street between the two signalized intersections, a crossing vehicle is faced with varied flow situations. For example, if the intersection were located at 990 ft, a crossing vehicle (Vehicle 1 on Figure I.10-2) would be faced with alternating platoons. Platoons from the left and from the right arrive in a perfectly alternating pattern. Thus, the crossing vehicle must always cross through *one* of these platoons. There are no gaps *between* the arrival of platoons, when both directions are considered. On the other hand, a vehicle at this location *will not* be faced with crossing simultaneous platoons in *both* directions.

If the intersection were located at 660 ft, the crossing vehicle (Vehicle 2 on Figure I.10-2) faces a different situation. At this location, platoons from the left and from the right arrive simultaneously. Thus, in every 60-sec cycle, the side-street vehicle faces 30 sec in which *both* platoons would have to be crossed, and 30 sec comprising a gap between platoon arrivals.

A vehicle (Vehicle 3 on Figure I.10-2) at an intersection located at 210 ft faces yet another flow situation. As shown in Figure I.10-2, a crossing vehicle here faces 10 sec during which *neither* platoon is present, 19 sec during which *one* platoon is present, 19 sec during which *the other* platoon is present, and 12 sec during which *both* platoons are present.

No matter what the location of the side street in the signalized platoon pattern, the time-space diagram can be used to identify discrete periods of flow, each of which can be analyzed separately.

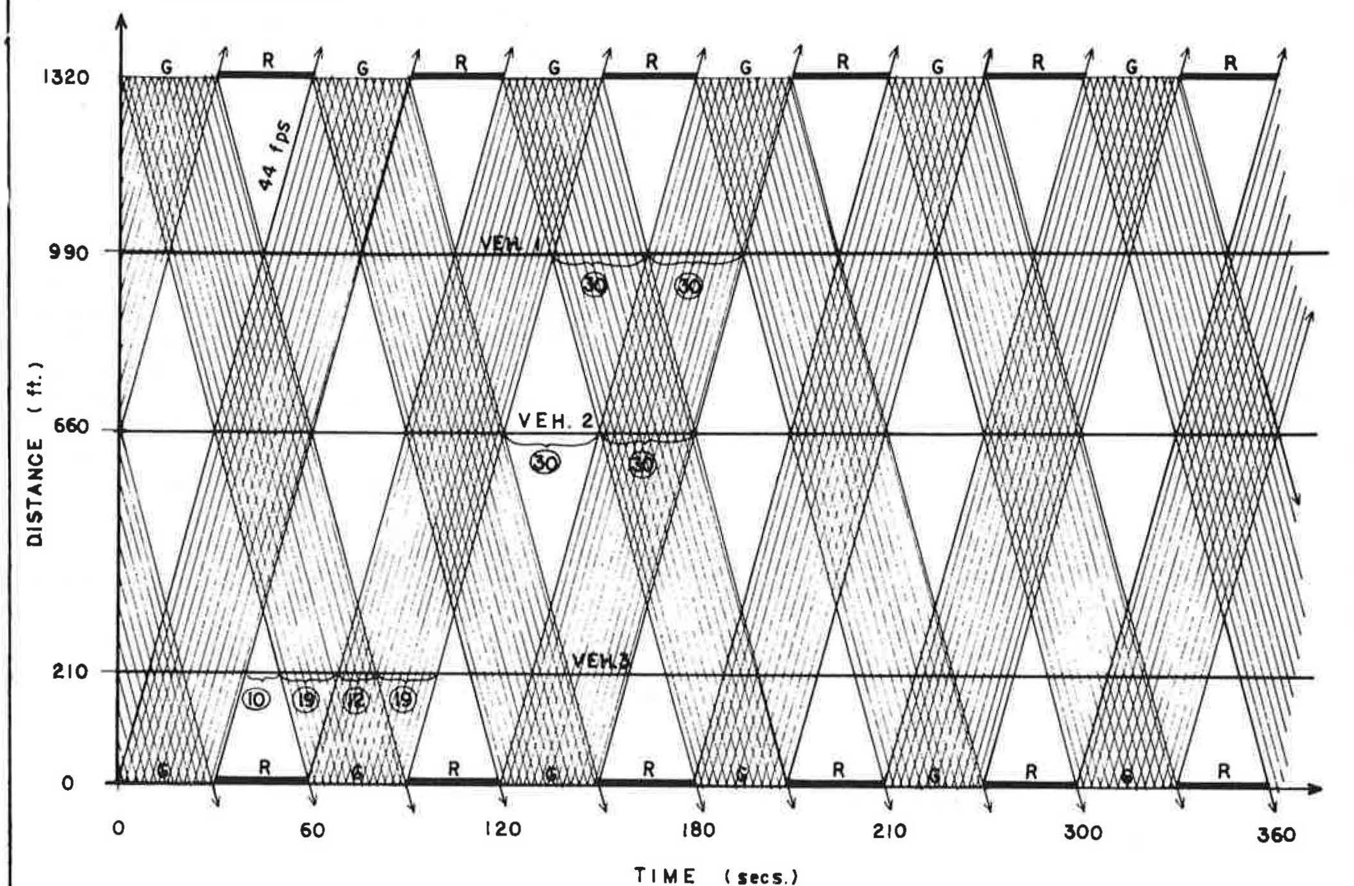


Figure I.10-2. Time-space diagram for illustrative problem.

In analyzing each discrete interval, the flow rates within platoons and between platoons must be identified. Even on arterials with strongly platooned flow, not all flow occurs within platoons. As vehicles enter and/or leave the major street from a variety of sources, such as unsignalized intersections, driveways, parking lanes, right-turn-on-red, etc., some volume between platoons occurs. Field studies should be conducted to identify the approximate percentage of total major street volume which occurs within platoons. For the illustrative problem, it is assumed that 80 percent of total volume occurs within platoons. Thus, in each direction of the major street, 400 vph occurs within platoons, and 100 vph between platoons. As the platoon flow occurs within 30 sec of each 60-sec cycle, and flow between platoons likewise, the effective flow rate within these periods is:

Within Platoons $= 400 \times (60/30) = 800$ vph
Between Platoons $= 100 \times (60/30) = 200$ vph

These flow rates will be used in subsequent computations.

For the three vehicles shown in Figure I.10-2, capacity computations are illustrated in Figure I.10-3.

Vehicle 1 faces alternating platoons from each direction. Thus, for 30 sec, side-street vehicles must cross a flow of 800 vph in one direction and 200 vph in the other. For the other 30 sec of the cycle, the flows are the same, but the directions are reversed. Figure I.10-3 shows the computation for each direction, which results in a capacity of 330 vph for each of the two 30-sec intervals. The total capacity is found by taking the capacity of each interval, and multiplying it by the proportion of total time that each interval exists. In this case, each interval exists for 30/60 ths of total time, and the total capacity is 330 vph. This results in LOS D and C for the NB and SB minor street flows respectively.

Vehicle 2 faces 30 sec during which flow is 200 vph in each direction, and 30 sec during which flow is 800 vph in each direction. The capacities for each interval are found to be 700 vph and 140 vph, respectively. The combined capacity is 420 vph, 90 vph more than the same intersection used by Vehicle 1, 330 ft away. The resulting NB and SB levels of service are improved by one level compared to the first computation, and are C and B, respectively.

For Vehicle 3, there are *four* discrete flow intervals to be considered: (1) the flow rate in each direction is 200 vph, (2) the flow rate is 200 vph in one direction and 800 vph in the other one, (3) the flow rates are as in 2, but the directions are reversed, and (4) the flow rate is 800 vph in each direction. The total capacity for this case is computed as 354 vph, and the NB and SB levels of service are D and C, respectively.

Finally, Figure I.10-3 also shows the results of a simple computation assuming random arrivals. This solution shows a capacity of 330 vph and a LOS D and C for NB and SB minor street flows respectively.

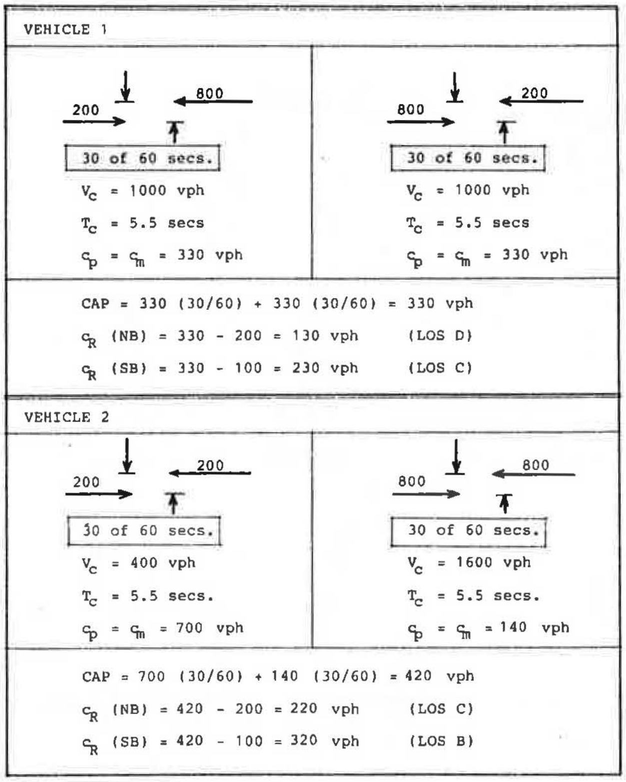


Figure I.10-3. Capacity computations for sample problem.

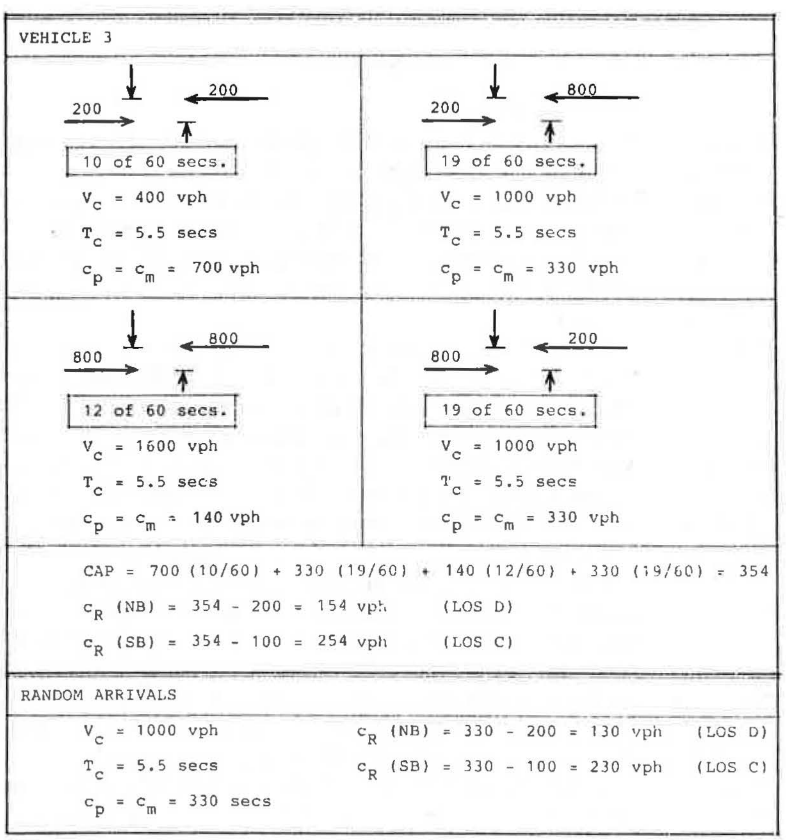


Figure I.10-3. Capacity computations for sample problem (Continued).

Note that the assumption of random arrivals would not have altered the result substantially for two of the three test cases, but would have underestimated the capacity of the intersection located at 660 ft by 25 percent. Note also that the impact of platoon flow is positive in this case and that the magnitude of the positive impact is large for Vehicle 2.

In general, negative impacts of platoon flow on unsignalized intersection capacity will not occur. While the intensity of flow within platoons is far greater than that for random arrivals, platooning either separates platoons in the two major street directions and/or provides periodic gaps between platoons during which the intensity of flow is far less than that for random arrivals. Where the gaps between platoons are substantial, the existence of platoon flow on the major street can provide more side-street capacity than would exist for random major street flow.

Computations can become more complex where multiple side-street movements are considered, and where the progression plan is more complex. The application, however, does not change. A complete analysis of each discrete interval is completed, with the results being combined as illustrated herein. Although this application still involves some assumptions regarding flow within and between platoons, it does allow for the approximate investigation of the impacts of platoon flow on unsignalized intersection capacity.

APPENDIX II

TABLES AND WORKSHEETS FOR USE IN THE ANALYSIS OF UNSIGNALIZED INTERSECTIONS

TABLES *PAGE*

WORKSHEETS

Table 10-1. Passenger-Car Equivalents for Unsignalized Intersections

TYPE OF VEHICLE	GRADE (%)				
	−4%	−2%	0%	+2%	+4%
Motorcycles	0.3	0.4	0.5	0.6	0.7
Passenger Cars	0.8	0.9	1.0	1.2	1.4
SU/RV's[a]	1.0	1.2	1.5	2.0	3.0
Combination Veh.	1.2	1.5	2.0	3.0	6.0
All Vehicles[b]	0.9	1.0	1.1	1.4	1.7

[a] Single-unit trucks and recreational vehicles.
[b] If vehicle composition is unknown, these values may be used as an approximation.

Table 10-2. Critical Gap Criteria for Unsignalized Intersections

BASIC CRITICAL GAP FOR PASSENGER CARS, SEC				
VEHICLE MANEUVER AND TYPE OF CONTROL	AVERAGE RUNNING SPEED, MAJOR ROAD			
	30 MPH		55 MPH	
	NUMBER OF LANES ON MAJOR ROAD			
	2	4	2	4
RT from Minor Road				
STOP	5.5	5.5	6.5	6.5
YIELD	5.0	5.0	5.5	5.5
LT from Major Road	5.0	5.5	5.5	6.0
Cross Major Road				
STOP	6.0	6.5	7.5	8.0
YIELD	5.5	6.0	6.5	7.0
LT from Minor Road				
STOP	6.5	7.0	8.0	8.5
YIELD	6.0	6.5	7.0	7.5

ADJUSTMENTS AND MODIFICATIONS TO CRITICAL GAP, SEC	
CONDITION	ADJUSTMENT
RT from Minor Street: Curb radius > 50 ft or turn angle < 60°	−0.5
RT from Minor Street: Acceleration lane provided	−1.0
All movements: Population ≥ 250,000	−0.5
Restricted sight distance.[a]	up to +1.0

NOTES: Maximum total decrease in critical gap = 1.0 sec.
Maximum Critical gap = 8.5 sec.
For values of average running speed between 30 and 55 mph, interpolate.
[a] This adjustment is made for the specific movement impacted by restricted sight distance.

TABLE 10-3. LEVEL-OF-SERVICE CRITERIA FOR UNSIGNALIZED INTERSECTIONS

RESERVE CAPACITY (PCPH)	LEVEL OF SERVICE	EXPECTED DELAY TO MINOR STREET TRAFFIC
$\geq$ 400	A	Little or no delay
300–399	B	Short traffic delays
200–299	C	Average traffic delays
100–199	D	Long traffic delays
0– 99	E	Very long traffic delays
[a]	F	[a]

[a] When demand volume exceeds the capacity of the lane, extreme delays will be encountered with queuing which may cause severe congestion affecting other traffic movements in the intersection. This condition usually warrants improvement to the intersection.

TABLE 10-5. CAPACITY OF A TWO-BY-TWO LANE FOUR-WAY STOP-CONTROLLED INTERSECTION FOR VARIOUS DEMAND SPLITS

DEMAND SPLIT	CAPACITY[a] (VPH)
50/50	1,900
55/45	1,800
60/40	1,700
65/35	1,600
70/30	1,500

[a] Total capacity, all legs.
SOURCE: Ref. 9

TABLE 10-6. CAPACITY OF FOUR-WAY STOP-CONTROLLED INTERSECTIONS WITH 50/50 DEMAND SPLIT FOR VARIOUS APPROACH WIDTHS

INTERSECTION TYPE	CAPACITY[a] (VPH)
2-lane by 2-lane	1,900
2-lane by 4-lane	2,800
4-lane by 4-lane	3,600

[a] Total capacity, all legs.
SOURCE: Ref. 9

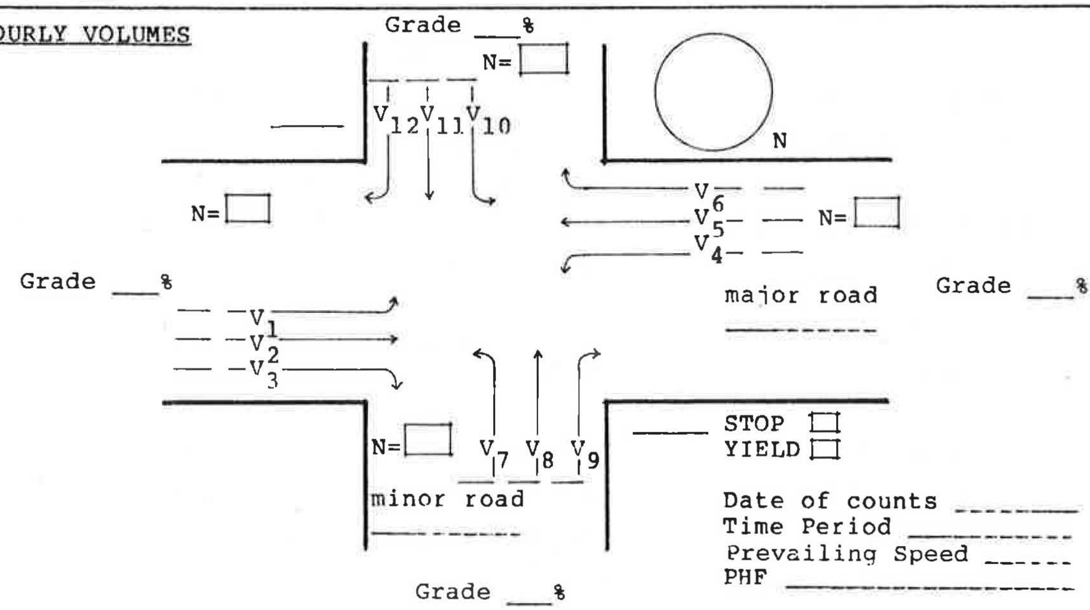
TABLE 10-7. APPROXIMATE LEVEL-OF-SERVICE C SERVICE VOLUMES FOR FOUR-WAY STOP-CONTROLLED INTERSECTIONS

DEMAND SPLIT	LOS C SERVICE VOLUME, VPH — NUMBER OF LANES: 2 BY 2	2 BY 4	4 BY 4
50/50	1,200	1,800	2,200
55/45	1,140	1,720	2,070
60/40	1,080	1,660	1,970
65/35	1,010	1,630	1,880
70/30	960	1,610	1,820

SOURCE: Ref. 10

LOCATION: NAME:

HOURLY VOLUMES



VOLUME ADJUSTMENTS

Movement no.	1	2	3	4	5	6	7	8	9	10	11	12
Volume (vph)												
Vol.(pcph), see Table 10.1												

VOLUMES IN PCPH

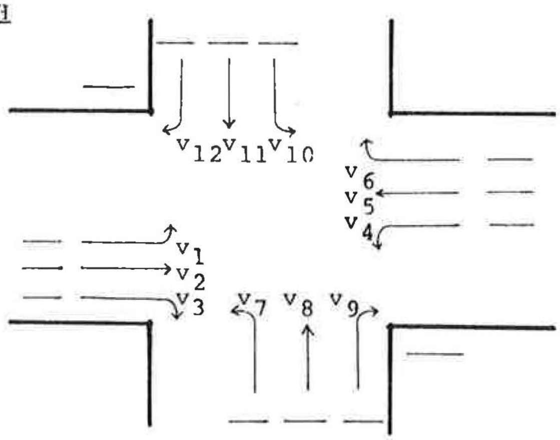


Figure 10-6. Worksheet for four-leg intersections (page 1).

STEP 1 : RT From Minor Street	V_9	V_{12}
Conflicting Flows, V_c	$1/2\ V_3+V_2=V_{c9}$ ____+____=______vph	$1/2\ V_6+V_5=V_{c12}$ ____+____=______vph
Critical Gap, T_c (Tab.10.2)	______(secs.)	______(secs.)
Potential Capacity, c_p (Fig.10.3)	c_{p9}=______pcph	c_{p12}=______pcph
% of c_p utilized	(v_9/c_{p9})x100=______%	(v_{12}/c_{p12})x100=______%
Impedance Factor, P (Fig.10.5)	P_9=______	P_{12}=______
Actual Capacity, c_m	$c_{m9}=c_{p9}$=______pcph	$c_{m12}=c_{p12}$=______pcph
STEP 2 : LT From Major Street	V_4	V_1
Conflicting Flows, V_c	$V_3+V_2=V_{c4}$ ____+____=______vph	$V_6+V_5=V_{c1}$ ____+____=______vph
Critical Gap, T_c (Tab.10.2)	______(secs.)	______(secs.)
Potential Capacity, c_p (Fig.10.3)	c_{p4}=______pcph	c_{p1}=______pcph
% of c_p utilized	(v_4/c_{p4})x100=______%	(v_1/c_{p1})x100=______%
Impedance Factor, P (Fig.10.5)	P_4=______	P_1=______
Actual Capacity, c_m	$c_{m4}=c_{p4}$=______pcph	$c_{m1}=c_{p1}$=______pcph
STEP 3 : TH From Minor Street	V_8	V_{11}
Conflicting Flows, V_c	$1/2V_3+V_2+V_1+V_6+V_5+V_4=V_{c8}$ __+__+__+__+__+__=___vph	$1/2V_6+V_5+V_4+V_3+V_2+V_1=V_{c11}$ __+__+__+__+__+__=___vph
Critical Gap, T_c (Tab.10.2)	______(secs.)	______(secs.)
Potential Capacity, c_p (Fig.10.3)	c_{p8}=______pcph	c_{p11}=______pcph
% of c_p utilized	(v_8/c_{p8})x100=______%	(v_{11}/c_{p11})x100=______%
Impedance Factor, P (Fig.10.5)	P_8=______	P_{11}=______
Actual Capacity, c_m	$c_{m8}=c_{p8}xP_1xP_4$ ______=___x___x___(pcph)	$c_{m11}=c_{p11}xP_1xP_4$ ______=___x___x___(pcph)
STEP 4 : LT From Minor Street	V_7	V_{10}
Conflicting Flows, V_c	V_8(step3)$+V_{11}+V_{12}=V_{c7}$ ____+____+____=______vph	V_{c11}(step3)$+V_7+V_8=V_{c10}$ ____+____+____=______vph
Critical Gap, T_c (Tab.10.2)	______(secs.)	______(secs.)
Potential Capacity, c_p (Fig.10.3)	c_{p7}=______pcph	c_{p10}=______pcph
Actual Capacity, c_m	$c_{m7}=c_{p7}xP_1xP_4xP_{11}xP_{12}$ ____=__x__x__x__x__(pcph)	$c_{m10}=c_{p10}xP_4xP_1xP_8xP_9$ ____=__x__x__x__x__(pcph)

Figure 10-6. Worksheet for four-leg intersections (page 2).

SHARED LANE CAPACITY

$$c_{SH} = \frac{v_i + v_j}{(v_i / c_{mi}) + (v_j / c_{mj})}$$ where 2 movements share a lane

$$c_{SH} = \frac{v_i + v_j + v_k}{(v_i / c_{mi}) + (v_j / c_{mj}) + (v_k / c_{mk})}$$ where 3 movements share a lane

APPROACH MOVEMENTS 7, 8, 9

Movement	v(pcph)	c_m(pcph)	c_{SH}(pcph)	$c_R = c_{SH} - v$	LOS
7					
8					
9					

APPROACH MOVEMENTS 10, 11, 12

Movement	v(pcph)	c_m(pcph)	c_{SH}(pcph)	$c_R = c_{SH} - v$	LOS
10					
11					
12					

MAJOR STREET LEFT TURNS 1, 4

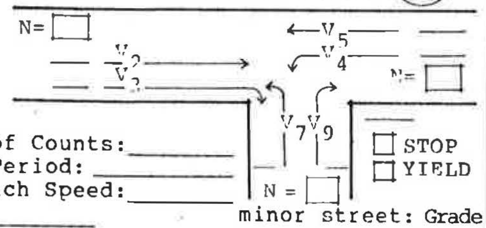
Movement	v (pcph)	c_m (pcph)	$c_R = c_m - v$	LOS
1				
4				

COMMENTS:

Figure 10-6. Worksheet for four-leg intersections (page 3).

LOCATION:	NAME:
HOURLY VOLUMES	VOLUMES IN PCPH

Major street:____________ N

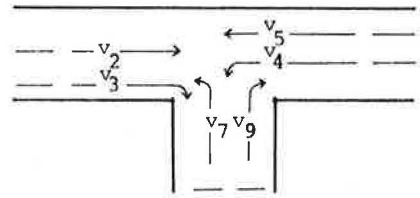


Grade ___ %

Date of Counts:________
Time Period: ________
Approach Speed:________
PHF: ________

☐ STOP
☐ YIELD

minor street: Grade __________ ___ %



VOLUME ADJUSTMENTS

Movement no.	2	3	4	5	7	9
Volume (vph)						
Vol.(pcph), see Table 10.1						

STEP 1 : RT From Minor Street	↱ V_9
Conflicting Flow, V_c	1/2 V_3+V_2=___+___=___vph (V_{c9})
Critical Gap,T_c and Potential Capacity,c_p	T_c=___secs (Tab.10.2) c_{p9}=___pcph (Fig.10.3)
Actual Capacity, c_m	$c_{m9}=c_{p9}$=_____pcph
STEP 2 : LT From Major Street	↲ V_4
Conflicting Flow, V_c	V_3+V_2=___+___=___vph (V_{c4})
Critical Gap,T_c and Potential Capacity,c_p	T_c=___secs (Tab.10.2) c_{p4}=___pcph (Fig.10.3)
% of c_p utilized and Impedance Factor (Fig.10.5)	(v_4/c_{p4})x100=_____ P_4=_____
Actual Capacity, c_m	$c_{m4}=c_{p4}$=_____pcph
STEP 3 : LT From Minor Street	↰ V_7
Conflicting Flow, V_c	1/2 $V_3+V_2+V_5+V_4$=___+___+___+___=___vph (V_{c7})
Critical Gap,T_c and Potential Capacity,c_p	T_c=___secs (Tab.10.2) c_{p7}=___pcph (Fig.10.3)
Actual Capacity, c_m	$c_{m7}=c_{p7}$xP_4=___x___=_____pcph

SHARED-LANE CAPACITY

$$SH = \frac{v_7 + v_9}{(v_7/c_{m7})+(v_9/c_{m9})}$$ if lane is shared

Movement no.	v(pcph)	c_m(pcph)	c_{SH}(pcph)	c_R	LOS
7					
9					
4					

Figure 10-7. Worksheet for analysis of T-intersections.

APPENDIX III

GLOSSARY AND SYMBOLS

GLOSSARY

basic critical gap—The median time headway between vehicles in a major traffic stream which will permit side-street vehicles at a STOP-or YIELD-controlled intersection to cross through or merge with the major traffic stream, unadjusted for geometric and other site-specific characteristics, in seconds.

conflicting traffic volume—The volume of traffic, in vehicles per hour, which conflicts with a specific movement at an unsignalized intersection.

critical gap—The median time headway between vehicles in a major traffic stream which will permit side-street vehicles at a STOP- or YIELD-controlled intersection to cross through or merge with the major traffic stream, in seconds.

impedance—The effect of congestion in higher priority movements at an unsignalized intersection on lower priority movements, which reduces the capacity of the lower priority movements.

movement capacity—The capacity of a specific movement at an unsignalized intersection, assuming that the movement has exclusive use of a separate lane, in passenger cars per hour.

potential capacity—The capacity of a specific movement at an unsignalized intersection, assuming that the movement is unimpeded by other movements and has exclusive use of a separate lane, in passenger cars per hour.

reserve capacity—The capacity of a lane at an unsignalized intersection minus the demand volume for that lane, where all terms are stated in passenger cars per hour.

shared-lane capacity—The capacity of a lane at an unsignalized intersection which is shared by two or three movements, in passenger cars per hour.

unsignalized intersection—An intersection controlled by two-way STOP signs or YIELD signs.

SYMBOLS

c_{mi} movement capacity for movement *i*, in passenger cars per hour.
c_{pi} potential capacity for movement *i*, in passenger cars per hour.
c_R reserve capacity, in passenger cars per hour.
c_{SH} shared-lane capacity, in passenger cars per hour.
pce passenger car equivalent.
v total volume or flow rate in a shared lane, in passenger cars per hour.
v_i full-hour volume or peak 15-min flow rate for movement *i*, in equivalent passenger cars per hour.
V_{ci} conflicting volume for movement *i*, in vehicles per hour.
V_i volume for movement *i*, in vehicles per hour for full hour.
T_c critical gap at an unsignalized intersection, in seconds.