R.B. Manvbay

HIGHWAY CAPACITY MANUAL

Special Report 209

TRANSPORTATION RESEARCH BOARD
National Research Council
Washington, D.C. 1985

Transportation Research Board Special Report 209

modes
1 highway transportation
2 public transit

subject areas
12 planning
21 facilities design
54 operations and traffic control
55 traffic flow, capacity, and measurements

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The project that is the subject of this report was approved by the Governing Board of the National Research Council, whose members are drawn from the councils of the National Academy of Sciences, the National Academy of Engineering, and the Institute of Medicine. The members of the committee responsible for the report were chosen for their special competence and with regard for appropriate balance.

This report has been reviewed by a group other than the authors according to procedures approved by a Report Review Committee consisting of members of the National Academy of Sciences, the National Academy of Engineering, and the Institute of Medicine.

Library of Congress Cataloging in Publication Data
National Research Council. Transportation Research Board.
Highway Capacity Manual
(Special report; 209)
1. Highway capacity—Handbooks, manuals, etc. I. Series: Special report (National Research Council (U.S.). Transportation Research Board); 209.
HE336.H48H54 1985 388.3'14 85-15331
ISBN 0-309-03821-9 ISSN 0360-859X

Printed in the United States of America
First Printing, August 1985
Second Printing, December 1985

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1985 HIGHWAY CAPACITY MANUAL

ERRATA 1

Page	Location	Change
iii	6th line from bottom	Change "some of" to "such as"
1-11	Col 2, 4th paragraph from bottom	Change "lightly loaded vans and panel trucks" to "lightly loaded farm vehicles"
2-3	Table 2-1, Urban 4-lane Freeways	Change "I-55, Jackson, Michigan" to "I-55, Jackson, Mississippi"
2-13	Col 2, 3rd line from bottom	Change "peak hour" to "design hour"
2-14	Col 1	In definition for K, change "peak direction" to "design hour"
2-26	Col 1, 5th line from bottom	Change "(*23*)" to "(*32*)"
2-26	Col 2, last full paragraph	Change paragraph to read: "Data from a single two-lane intersection approach in Evanston, Illinois, are given in Table 2-13. The data include headways across both lanes rather than a single lane, which make the data closely comparable to that of Figure 2-21."
2-27	Table 2-13	Remove "Arrival" from table title and heading of column 4 Change reference "*23*" to "*32*"
2-28	Reference	Add reference: "32. Berry, D.S., Discussion of King, G., and Wilkinson, M. (ref. *26*), *Transportation Research Record 615*, Transportation Research Board, Washington, D.C. (1976)."
3-4	Figure 3-3	Change "Design Flow" to "Design Speed"
3-8	Col 1, columnar data labeled "Level of Service" and "Density"	Change "Density" to "Maximum Density" Add "F" and ">67" at bottom of respective columns
3-14	Col 2, 5th line from bottom	Change "4 percent" to "3 percent"
3-16	Col 2, 2nd line from top	Change "3,000 ft" to "4,000 ft"
3-20	Col 2, after last complete paragraph	Add new paragraph: "In some cases, in the vicinity of Level-of-Service boundaries, the LOS as determined by v/c ratio is different from that determined by density. This is due to the rounding-off of boundary values. In such cases, density is the determinant of LOS, consistent with the Level-of-Service definition."
3-20	Figure 3-6	Change "Design Flow" to "Design Speed"
3-24	Col 1	Change K value of "0.09" to "0.07"
3-26	Figure 3-8, bottom paragraph	Change "Design Flow" to "Design Speed"
3-29	Figure 3-11, bottom paragraph	Change "Design Flow" to "Design Speed"
3-30	Figure 3-12, bottom paragraph	Change "Design Flow" to "Design Speed"
3-31	Col 2, Calculation 5	In Step 1, change "Descirption" to "Description" and "deisred" to "desired"
3-34	Col 1, Appendix 1, 3rd line	Change "or 3,000 ft in length" to "or the total length of the composite grade is less than 4,000 ft in length"
3-39	Figure 3-3	Change "Design Flow" to "Design Speed"
3-44	Operation Analysis Worksheet, near bottom	Add "*" to "LOS (Table 3-1)" and add footnote: "* LOS must be consistent with density criteria in Table 3-1."
4-4	Table 4-1	Add "Minimum" to table title after "vs." and as the first word in each column head
4-13	Col 1, Step e	Change "constrained" to "unconstrained"
4-18	Col 2, table	Columnar values for weaving speeds (Sw) beginning with a 3-lane Type A section 1,000 feet long should be "37[a], 38[a], 42[a], - , 30[a], 34[a], 37[a], 42[a], 45[a], - , 32[a], 37[a], 40[a], 44[a], 48[a]" (instead of "36, 23[a], 27[a], - , 15[a], 19[a], 22[a], 27[a], 30[a], - , 17[a], 22[a], 25[a], 29[a], 33[a]," respectively) Change value for nonweaving speed (S_{nw}) for a 3-lane Type A section 1,000 feet long from "40" to "41"

December 1986

Page	Location	Change
5-8	Table 5-2, Major Junctions	Change "Figure I.5-5" to "Figure I.5-6"
5-17	Col 2, calculation 2, Step 2. Solution for Ramp 2, 2nd paragraph, lines 2 and 3	Change "1,303 for ramp 2" to "1,300 for ramp 2"
5-19	Col 2, 3rd line from bottom	Change "0.249" to "0.250"
5-20	Col 1, Solution for v_m	Change "(LOS D, Table 5-1)" to "(LOS E, Table 5-1)"
5-20	Col 1, 1st and 2nd lines before Calculation 4	Change sentence to: "In this case, level-of-service E will prevail."
5-23	Col 1	Calculation for V_1 should be: V_1 = (5,360 x 0.09) + (1.00 x 400) = 882 vph
5-23	Col 1, Table	For V_1, change "936" to "882" and change "1,932" to "1,858"
5-23	Col 1, 4th line before Calculation 7	Change "1,932" to "1,858"
5-25	Figure I.5-1, V_1 under Solution	Change the "200" between 1,100 and 1,300 to "1,200"
5-34	Figure I.5-10	Equation should read: $V_1 = -353 + 0.199V_f - 0.057V_r + 0.486V_d$
6-5	Col 1, 1st line	Change "1.00" to "0.98"
6-7	Col 1, 5th line from bottom	Change "information" to "formation"
7-4	Col 2, 2nd line in Heavy Vehicles section	Delete "two axles or"
7-11	Col 2, 12th line from bottom	Change "4,000 ft and/or 4 percent" to "3,000 ft or 3 percent"
7-12	Col 2, 5th line from bottom	Change "1 mi" to "2 mi"
7-15	Figure 7-3, Section III. Analysis	Add "f_E" as a multiplicative factor to the equation for capacity
7-21	Figure 7-6, Worksheet, Section III. Analysis	Add "f_E" as a multiplicative factor to the equation for capacity
7-23	Figure 7-8, Worksheet, Section III. Analysis	Add "f_E" as a multiplicative factor to the equation for capacity
7-31	Section III. Analysis	Add "f_E" as a multiplicative factor to the equation for capacity
8-5	Table 8-1	Delete "Criteria" from table title Change footnote b to read: "These speeds are provided for information only and apply to roads with design speeds of 60 mph or higher."
8-9	Table 8-5	Change footnote b to read: "For analysis of specific grades, use LOS E factors for all speeds less than 45 mph." Also, move the reference "b" from LOS in the column headings to the word "LANES^b" in the column heading for each lane width
8-10	Table 8-7	Change value for 6% grade, 45 mph average upgrade speed, and 0% no passing zones from "0.49" to "0.85"
8-11	Table 8-8	In column titled "Percent of Traffic on Upgrade" change "30" to "$\leq$30"
8-13	Figure 8-3	Change "800" on the horizontal scale to "500"
8-25	Figure 8-9, Section VI. Level of Service Analysis	Change "980" for LOS E to "950"
8-30	Figure 8-3	Change "800" on the horizontal scale to "500"
9-9	Col 2, Equation for V_{LE}	Change "V" to "V_a" and change "$(N - \ell)$" to "$(N - 1)$"
9-15	Table 9-12	Add to footnote a: "In many instances, it is advisable to treat this case as a separate protected and separate permitted phasing."
9-22	Col 2, Step 1, 3rd line	Add "*" after "right-turn"
9-22	Col 2, bottom	Add footnote: "* Where RTOR is permitted, the right-turn volume may be reduced by the volume of right-turning vehicles moving on the red phase."
9-25	Figure 9-7	In column 9, Lane Utilization Factor, place "1.0" for all LT and RT movements on all approaches
9-25	Col 1, Step 6	Change paragraph to read: "The lane utilization factor for each lane group may be found from Table 9-4 to account for unequal use of available lanes by vehicles. Otherwise, enter 1.0."
9-28	Col 1, Step 1.a	Change sentence to read: "As the signal timing was unknown, this value was taken to be 90 sec . . ."
9-30	Col 1, Step 9 equation	Change "x_c" to "X_c"

Page	Location	Change
9-40	Col 1, 2nd equation for v_{LE}	Change "v" to "v_e"
9-42	Col 1, 4th line from bottom	Change "0.437" to "0.461"
9-42	Col 1, 3rd line from bottom	Change "0.437" to "0.461" and change "0.806" to "0.830"
9-42	Col 2, equation for X_c	Change "0.806" to "0.830" and change "0.881" to "0.908"
9-63	Figure I.9-1, horizontal scale title	Change "LEFT-TURN VOLUME (ECV)" to "LEFT-TURN VOLUME (PCE)"
9-70	Col 2, top line	Change equation to: "= 0.26(95/0.95) ="
9-76	Worksheet, col 9	In Lane Utilization Factor column, place "1.0" for all LT and RT movements on all approaches
10-13	Figure 10-7	In Shared-Lane Capacity section, change "SH" to "C_{SH}"
10-15	Figure 10-8	In Shared-Lane Capacity section, change "SH" to "C_{SH}"
10-27	Figure 10-12	In Shared-Lane Capacity section, change "SH" to "C_{SH}"
10-37	Worksheet	In Shared-Lane Capacity section, change "SH" to "C_{SH}"
11-30	Col 1, end of Section 4.a.	Add "Record the location (street name) of all desired checkpoints and their distance from the starting point of the study. The test car should be driven as if it is a through vehicle (i.e., in each available through lane)."
11-30	Col 1, Section 4.b.	Replace with "Record the cumulative travel time (CUM TT) between the exit end of the signalized intersections and other checkpoints such as stop signs. Record the cumulative stop delay (and cause) at each checkpoint location. Also record the principal cause and duration of midblock stops. Midblock cumulative stop delay is entered on the top line in the STOP TIME column; checkpoint cumulative stop delay is entered on the lower line. Although the stop delay data are not critical to the Chapter 11 procedure, they are useful for engineering purposes and as a bridge to Chapter 9, Signalized Intersections."
11-30	Table I.11-1	See changes to Worksheet on p. 11-33.
11-31	Worksheet	Change col heading "Random Arrival Delay" to "Initial Stopped Delay" Change col heading "Estimated Stopped Delay" to "Adjusted Stopped Delay" Footnote a should read: "Initial estimate of stopped delay assuming random vehicle arrivals (from Equation 11-3)." Footnote c should read: "Multiply (Initial Stopped Delay) times (Progression factor PF) to correct the initial delay estimate for the effect of progression."
11-33	Worksheet	In "Signal Location" col heading, delete "Signal" Delete top 0.4 in. of vertical line between "Run No.__ " and "Time" and blank space between cols 3 and 4, 5 and 6, and 7 and 8 Change "CUM TT" in cols 3, 5, and 7 to "STOP TIME" and change "STOP TIME" in cols 4, 6, and 8 to "CUM TT" Extend horizontal line under "Time" to span two cols; that is

Run No.__ Time		Run No.__ Time		Run No.__ Time	
STOP TIME (sec)	CUM TT (sec)	STOP TIME (sec)	CUM TT (sec)	STOP TIME (sec)	CUM TT (sec)

Page	Location	Change
12-8	Table 12-4	For a 40.0 ft transit bus, change the number of seats from "83" to "53"
12-8	Table 12-5	In table title, change "(5-Seat, . . .)" to "(50-Seat, . . .)"
12-10	Col 1, 7th line of Study 3.	Change "940" to "1,000"
12-28	Col 2, 2nd line from top	Change "Table 12-4" to "Table 12-17"
12-31	Table 12-28	In Item 1.B.4, change "toil plaza" to "toll plaza"
12-35	Table 12-30	In Equation 12-1, change "1.5f_1)" to "1.5 f')"

Page	Location	Change
12-43	Table 12-36	In col 5 heading, change "Tab. 12-34" to "Tab. 12-35"
12-45	Col 2, Calculation 11, Step 2, 1st and 2nd lines of 2nd paragraph	Change "Table 12-18" to "Table 12-20"
12-46	Col 1, for N_b	Change "Table 12-18" to "Table 12-19"
12-47	Col 2, equation for C_p	Change equation to read: $c_p = nSc_v = \frac{(g/C)\ 3{,}600\ nSR}{(g/C)D + t_c}$
12-55	Table I.12-7	Change first column heading to read: "TERMINAL AND CITY"
12-55	Table II.12-1	Change col 1 heading from "CITY" to "TRANSIT SYSTEM TYPE" and col 2 heading from "LOCATION" to "CITY"
12-58	Table II.12-3, Section B	In definition for a, change "sec^2" to "sec/sec" In definition for b_2, b_n, and b_e, add units "ft/sec/sec"
12-58	Table II.12-3, Section C	Change S.I.U. value for t_r from "5.0 sec" to "3.0 sec" Change S.I.U. unit for L from "ft" to "m" Change English units for V from "29.4-44.1" to "29.3-40.0 ft/sec" and add S.I.U. units "32.2-48.3 kpm" and "8.9-13.4 mps" Change S.I.U. units for a from "sec^2" to "sec/sec" Change S.I.U. units for b_n from "3.0m/sec^2" to "1.3m/sec/sec" Add S.I.U. units for b_e of "3.0m/sec/sec"

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ADDRESS CORRECTION REQUESTED

Foreword

Efforts to understand highway capacity started soon after the advent of automotive travel. Individual researchers were studying traffic flow and isolating the complex relationships of highway capacity more than 60 years ago. Dr. Bruce D. Greenshields (1894–1979) was among the early scientific observers. Another pioneer was Olav K. Normann (1906–1964) to whom the 1965 *Highway Capacity Manual* was dedicated. As an engineer-scientist and Director of Research for the U.S. Bureau of Public Roads, Normann had a powerful influence on the evolution of highway capacity practices. Nevertheless, despite these and other outstanding individual contributors, a hallmark of the highway capacity field has been the strong influence of collaborative professional processes.

This edition of the *Highway Capacity Manual* is a further step in that collaborative process. The Transportation Research Board and its Committee on Highway Capacity and Quality of Service provided the environment in which professional judgments could be made and tested among peers. Other institutions made important contributions: the Federal Highway Administration sponsored and conducted important research, and the National Cooperative Highway Research Program, supported by the individual states through the American Association of State Highway and Transportation Officials, funded investigations whose results were incorporated into this manual. A list of the contracts and contractors involved in this effort is provided in the list of Contributors and Acknowledgments.

The result of all these efforts is a collection of techniques for estimating highway capacity that have been judged, through consensus, as the best available at the time of publication. This manual describes techniques for computing highway capacity. It is neither designed as, nor does it establish, a legal standard for highway construction. Users should recognize that certain situations may call for variation from its provisions, subject to sound engineering judgment.

From the earliest studies, it has been apparent that a road's ultimate capacity potential was far less important than its carrying capacity associated with a particular quality of service. Unlike the physical capacities of beams to carry loads or pipes to accommodate fluids, highway capacity involves human beings who are sensitive to the quality of the service they are receiving and capable of reacting to it. Knowledgeable professionals, acting in concert, have provided the value judgments needed to quantify these flow-quality relationships and have established the common vocabulary and techniques for estimating the effect of one on the other. The application of these relationships to the determination of highway capacity is the subject of this Manual.

Throughout, the contents of the Manual represent changes from the 1965 edition. The material in Chapter 2, as an example, updates earlier information on traffic characteristics and performance. Other chapters, like those on freeways and rural highways, offer significantly revised procedures for capacity analysis. Still others, some of those on urban streets, present procedures quite different from those given earlier. Among the material entirely new to the Manual is that on pedestrians and bicycles. The sources of the changes can be traced directly to documents from research centers, consulting firms, and individuals from the United States, Australia, Canada, Great Britain, and other countries. There is no single author, *per se.* We cannot always identify and therefore do not attempt to specifically recognize each contribution, but we gratefully acknowledge them all. Assembling all these parts into a coherent and technically sound whole was the Committee's responsibility—

one that was greatly assisted by the dedicated and perceptive work of the researchers. Final synthesis of the manual was completed under the sponsorship of the National Cooperative Highway Research Program.

The Committee views this publication as a milestone in the growing body of knowledge of highway capacity—not the conclusion. Research will continue. In fact, at least one new major project was underway, at the time this publication went to press, to improve the understanding of capacity relationships on multilane rural highways. Further research is needed on urban intersections and arterials and other areas. As such research is completed and the findings are accepted, appropriate changes to the Manual will be issued on an annual basis.

In the years ahead, the Committee urges all readers to contribute to, as well as draw from, the reservoir of knowledge represented by this document.

For the Committee on Highway Capacity and
Quality of Service

Carlton C. Robinson
Chairman

Contributors and Acknowledgments

This report is a result of the coordinated efforts of many individuals, groups, research organizations, and government agencies. While responsibility for the content of the Highway Capacity Manual *lies with the Committee on Highway Capacity and Quality of Service, its preparation was accomplished by a research team under the sponsorship of the National Cooperative Highway Research Program involving the following principals:*

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM—PROJECT 3-28B
THE NEW HIGHWAY CAPACITY MANUAL

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Subcontractor

Texas Transportation Institute, *Texas A&M University System.*

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Last to be acknowledged among the volunteer contributors to this edition are the unnamed users of draft materials and TRB Circulars published and distributed during the period of the Manual's development. Their interest and support were a constant stimulus to both committee and research activities.

Perhaps last in the process, but not least among those who made this document possible, are staff members of the Transportation Research Board. Helen Mack, NCHRP Editor, diligently and meticulously worked with the researchers to produce the final camera-ready manuscript. Other design and production supervision was provided by Nancy A. Ackerman, Publications Manager. Robert E. Spicher, CRP Deputy Director and NCHRP Projects Engineer, and David K. Witheford, Engineer of Traffic and Operations, provided indispensable staff support to the Panel and Committee.

Contents

FIGURES

PHOTOGRAPHS

TABLES

INTRODUCTION, DEFINITIONS AND CONCEPTS, APPLICATIONS

CONTENTS

I. INTRODUCTION

This document is the third edition of the Highway Capacity Manual, originally published in 1950 by the then Bureau of Public Roads as a guide to the design and operational analysis of highway facilities. The second edition was published in 1965 by the then Highway Research Board, under the guidance of its Highway Capacity Committee. This third edition reflects over two decades of comprehensive research conducted by a variety of research agencies, with the sponsorship of a number of agencies, primarily the National Cooperative Highway Research Program and the Federal Highway Administration. Its development has been guided by the Transportation Research Board's Committee on Highway Capacity and Quality of Service.

The procedures and methodologies of this manual have been developed from a wide range of empirical research conducted since the mid-1960's. Procedures reflect North American operating experience, and may not be representative of operations in other parts of the world.

The fourteen chapters in this Third Edition of the HCM represent revisions and updates of material contained in the earlier editions, and new material reflecting the many changes in the characteristics of travel and in the information needed to conduct highway capacity analyses.

Chapter 2, "Traffic Characteristics," presents and discusses values observed throughout North America for many of the parameters and variables introduced herein.

Chapters 3 through 14 are the basic procedural chapters of the manual. They are organized according to the facility types referred to in Table 1-1. Chapters 3 to 8 cover uninterrupted flow facilities, with Chapters 3 to 6 treating freeways and their components, and Chapters 7 and 8 treating multilane and two-lane rural highways. Chapters 9 to 14 focus on interrupted flow facilities and their components, including signalized and unsignalized intersections, arterials, transit facilities, pedestrian facilities, and bicycle facilities. Chapter 6, "Freeway Systems," treats the coordinated analysis of a continuous series of individual components consisting of basic freeway sections, weaving areas, and ramp junctions. Chapter 6 emphasizes the impact of operations in one segment of the system on adjacent segments and on overall system evaluations.

Each of the procedural chapters is generally organized in four distinct parts:

1. *Introduction*—This section describes the basic characteristics, concepts, and philosophies of capacity analysis as applied to the subject type of facility.
2. *Methodology*—This material presents the basic components of the analysis procedure to be applied to the subject facility type. Included here are equations together with tabular and graphic information needed to complete an analysis.
3. *Procedures for application*—Step-by-step instructions for applying capacity analysis computations are included in this section. Procedures are specified for operational analysis, design, and planning, although not all chapters contain three distinct analysis levels. Worksheets are provided for most computational procedures, and they are explained in detail in this section.
4. *Sample calculations*—The "sample calculations" section presents a variety of example applications, showing all computations required for analysis, and detailed discussions of results and interpretations. Sample calculations are provided for the full range of potential applications in each chapter.

Many chapters have separate sections headed by the foregoing titles. In some chapters, sections are combined for clarity of presentation. Where this is done, section titles clearly indicate where material is located.

The organization of the procedural chapters in this fashion will allow frequent users to focus on step-by-step instructions, without having to read or scan an entire chapter. All users of this manual, however, should read the entire chapter being used, at least once, to become familiar with all of the concepts, applications, and interpretations of the procedures in the chapter.

As an additional convenience for frequent users, most chapters contain an appendix in which many figures and worksheets are reproduced (some to a larger scale than that appearing in the text) for use, without the need to flip through pages of the manual text.

II. DEFINITIONS AND CONCEPTS

TYPES OF FACILITIES

The analysis techniques provided in this manual cover a broad range of facilities, including streets and highways, transit facilities, pedestrian facilities, and bicycle facilities.

Facilities may generally be classified into one of two categories:

1. *Uninterrupted flow*—Uninterrupted flow facilities have no fixed elements, such as traffic signals, external to the traffic stream that cause interruptions to traffic flow. Traffic flow conditions are the result of interactions among vehicles in the traffic stream, and between vehicles and the geometric and environmental characteristics of the roadway.
2. *Interrupted flow*—Interrupted flow facilities have fixed elements causing periodic interruptions to traffic flow. Such elements include traffic signals, stop signs, and other types of controls. These devices cause traffic to periodically stop (or significantly slow) irrespective of how much traffic exists.

Uninterrupted and interrupted flow are terms describing the type of facility, not the quality of traffic flow at any given time. Thus, a freeway experiencing extreme congestion is still an "uninterrupted flow facility," as the causes of congestion are internal to the traffic stream.

The analysis of these types of facilities varies considerably. The analysis of interrupted flow facilities must account for the impact of fixed interruptions. A traffic signal, for example, limits the portion of time that is available to various movements in an intersection. Capacity is limited not only by the physical space provided, but the time of use that is available to various component movements in the traffic stream. Uninterrupted flow facilities have no fixed interruptions, and therefore have no time limitations on the use of roadway space.

Table 1-1 gives the types of facilities for which capacity analysis procedures are provided in this manual, and the chapters in which the procedures are found. It should be noted that the chapters on Transit, Pedestrians, and Bicycles focus on elements of those modes that interact with street traffic.

Freeways, and their components, operate under the purest form of uninterrupted flow. Not only are there no fixed interruptions to traffic flow, but access is controlled and limited to ramp locations. Multilane highways and two-lane highways may also operate under uninterrupted flow in long segments between points of fixed interruptions. In general, where signal spacing exceeds two miles, uninterrupted flow may exist between the signals. Where signal spacing is less than two miles, the facility is classified as an arterial, and flow is considered to be interrupted. On multilane and two-lane highways, it is often nec-

TABLE 1-1. TYPES OF FACILITIES

FACILITY	CHAPTER
Uninterrupted Flow Facilities	
Freeways	
Basic freeway segments	3
Weaving areas	4
Ramps and ramp junctions	5
Freeway systems	6
Multilane Highways	7
Two-Lane Highways	8
Interrupted Flow Facilities	
Signalized Intersections	9
Unsignalized Intersections (2-way STOP-YIELD-controlled approaches; 4-way STOP-controlled intersections)	10
Arterials	11
Transit	12
Pedestrians	13
Bicycles	14

essary to examine points of fixed interruption as well as uninterrupted flow segments.

Pedestrian and transit flows are generally considered to be interrupted. Uninterrupted flow can exist under certain circumstances, such as in a long busway without stops or a long pedestrian corridor.

CAPACITY AND LEVEL-OF-SERVICE CONCEPTS

A principal objective of capacity analysis is the estimation of the maximum amount of traffic that can be accommodated by a given facility. Capacity analysis would, however, be of limited utility if this were its only focus. Traffic facilities generally operate poorly at or near capacity, and facilities are rarely designed or planned to operate in this range. Capacity analysis is also intended to estimate the maximum amount of traffic that can be accommodated by a facility while maintaining prescribed operational qualities.

Capacity analysis is, therefore, a set of procedures used to estimate the traffic-carrying ability of facilities over a range of defined operational conditions. It provides tools for the analysis and improvement of existing facilities, and for the planning and design of future facilities.

The definition of operational criteria is accomplished using *levels of service.* Ranges of operating conditions are defined for each type of facility, and are related to amounts of traffic that can be accommodated at each level.

The following sections present and define the two principal concepts of this manual: *capacity* and *level of service.*

Capacity

In general, the *capacity* of a facility is defined as the maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of a lane or roadway during a given time period under prevailing roadway, traffic, and control conditions.

The *time period* used in most capacity analysis is 15-min, which is considered to be the shortest interval during which stable flow exists.

Capacity is defined for *prevailing roadway, traffic, and control conditions,* which should be reasonably uniform for any section of facility analyzed. Any change in the prevailing conditions will result in a change in the capacity of the facility. The definition of capacity assumes that good weather and pavement conditions exist.

1. *Roadway conditions*—Roadway conditions refer to the geometric characteristics of the street or highway, including: the type of facility and its development environment, the number of lanes (by direction), lane and shoulder widths, lateral clearances, design speed, and horizontal and vertical alignments.

2. *Traffic conditions*—Traffic conditions refer to the characteristics of the traffic stream using the facility. This is defined by the distribution of vehicle types in the traffic stream, the amount and distribution of traffic in available lanes of a facility, and the directional distribution of traffic.

3. *Control conditions*—Control conditions refer to the types and specific design of control devices and traffic regulations present on a given facility. The location, type, and timing of traffic signals are critical control conditions affecting capacity. Other important controls include STOP and YIELD signs, lane use restrictions, turn restrictions, and similar measures.

These and other factors affecting capacity are discussed in greater detail in a subsequent section of this chapter.

It is also important to note that *capacity* refers to a *rate* of vehicular or person flow during a specified period of interest, which is most often a peak 15-min. period. This recognizes the potential for substantial variations in flow during an hour, and focuses analysis on intervals of maximum flow.

Levels of Service

The concept of *levels of service* is defined as a qualitative measure describing operational conditions within a traffic stream, and their perception by motorists and/or passengers. A level-of-service definition generally describes these conditions in terms of such factors as speed and travel time, freedom to maneuver, traffic interruptions, comfort and convenience, and safety.

Six levels of service are defined for each type of facility for which analysis procedures are available. They are given letter designations, from A to F, with level-of-service A representing the best operating conditions and level-of-service F the worst.

1. *Level-of-service definitions*—In general, the various levels of service are defined as follows for uninterrupted flow facilities:

- *Level-of-service A* represents free flow. Individual users are virtually unaffected by the presence of others in the traffic stream. Freedom to select desired speeds and to maneuver within the traffic stream is extremely high. The general level of comfort and convenience provided to the motorist, passenger, or pedestrian is excellent.
- *Level-of-service B* is in the range of stable flow, but the presence of other users in the traffic stream begins to be noticeable. Freedom to select desired speeds is relatively unaffected, but there is a slight decline in the freedom to maneuver within the traffic stream from LOS A. The level of comfort and convenience provided is somewhat less than at LOS A, because the presence of others in the traffic stream begins to affect individual behavior.

• *Level-of-service C* is in the range of stable flow, but marks the beginning of the range of flow in which the operation of individual users becomes significantly affected by interactions with others in the traffic stream. The selection of speed is now affected by the presence of others, and maneuvering within the traffic stream requires substantial vigilance on the part of the user. The general level of comfort and convenience declines noticeably at this level.

• *Level-of-service D* represents high-density, but stable, flow. Speed and freedom to maneuver are severely restricted, and the driver or pedestrian experiences a generally poor level of comfort and convenience. Small increases in traffic flow will generally cause operational problems at this level.

• *Level-of-service E* represents operating conditions at or near the capacity level. All speeds are reduced to a low, but relatively uniform value. Freedom to maneuver within the traffic stream is extremely difficult, and it is generally accomplished by forcing a vehicle or pedestrian to "give way" to accommodate such maneuvers. Comfort and convenience levels are extremely poor, and driver or pedestrian frustration is generally high. Operations at this level are usually unstable, because small increases in flow or minor perturbations within the traffic stream will cause breakdowns.

• *Level-of-service F* is used to define forced or breakdown flow. This condition exists wherever the amount of traffic approaching a point exceeds the amount which can traverse the point. Queues form behind such locations. Operations within the queue are characterized by stop-and-go waves, and they are extremely unstable. Vehicles may progress at reasonable speeds for several hundred feet or more, then be required to stop in a cyclic fashion. Level-of-service F is used to describe the operating conditions within the queue, as well as the point of the breakdown. It should be noted, however, that in many cases operating conditions of vehicles or pedestrians discharged from the queue may be quite good. Nevertheless, it is the point at which arrival flow exceeds discharge flow which causes the queue to form, and level-of-service F is an appropriate designation for such points.

These definitions are general and conceptual in nature, and they apply primarily to uninterrupted flow. Levels of service for interrupted flow facilities vary widely in terms of both the user's perception of service quality and the operational variables used to describe them. Each chapter of the manual contains more detailed descriptions of the levels of service as defined for each facility type.

2. *Service flow rates*—The procedures of this manual attempt to establish or predict the maximum rate of flow which can be accommodated by various facilities at each level of service, except level-of-service F, for which flows are unstable. Thus, each facility has five service flow rates, one for each level of service (A through E), defined as follows.

The *service flow rate* is the maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of a lane or roadway during a given time period under prevailing roadway, traffic, and control conditions while maintaining a designated level of service. As to capacity, the service flow rate is generally taken for a 15-min time period.

Note that service flow rates are discrete values, while the levels of service represent a range of conditions. Because the service flow rates are defined as maximums for each level of service, they effectively define flow boundaries between the various levels of service.

3. *Measures of effectiveness*—For each type of facility, levels of service are defined based on one or more operational parameters which best describe operating quality for the subject facility type. While the concept of level of service attempts to address a wide range of operating conditions, limitations on data collection and availability make it impractical to treat the full range of operational parameters for every type of facility. The parameters selected to define levels of service for each facility type are called "measures of effectiveness," and represent those available measures that best describe the quality of operation on the subject facility type. Table 1-2 gives the measures of effectiveness used to define levels of service for each facility type.

Each level of service represents a range of conditions, as defined by a range in the parameter(s) given in Table 1-2. Thus, a level of service is not a discrete condition, but rather a range of conditions for which boundaries are established.

TABLE 1-2. MEASURES OF EFFECTIVENESS FOR LEVEL OF SERVICE DEFINITION

TYPE OF FACILITY	MEASURE OF EFFECTIVENESS
Freeways	
Basic freeway segments	Density (pc/mi/ln)
Weaving areas	Average travel speed (mph)
Ramp junctions	Flow rates (pcph)
Multilane Highways	Density (pc/mi/ln)
Two-Lane Highways	Percent time delay (%)
	Average travel speed (mph)
Signalized Intersections	Average individual stopped delay (sec/veh)
Unsignalized Intersections	Reserve capacity (pcph)
Arterials	Average travel speed (mph)
Transit	Load factor (pers/seat)
Pedestrians	Space (sq ft/ped)

BASIC PRINCIPLES OF TRAFFIC FLOW

Traffic Flow Measures

The operational state of any given traffic stream is defined by three primary measures:

1. Speed.
2. Volume and/or rate of flow.
3. Density.

1. *Speed* is defined as a rate of motion expressed as distance per unit time, generally as miles per hour (mph) or kilometers per hour (km/h). In characterizing the speed of a traffic stream, some representative value must be used, as there is generally a broad distribution of individual speeds that may be observed in the traffic stream. For the purposes of this manual, the speed measure used is *average travel speed.* This measure is used because it is easily computed from observation of individual vehicles within the traffic stream, and because it is the most statistically relevant measure in relationships with other varia-

bles. Average travel speed is computed by taking the length of the highway or street segment under consideration and dividing it by the average travel time of vehicles traversing the segment. Thus, if travel times $t_1, t_2, t_3, \ldots, t_n$ are measured for n vehicles traversing a segment of length L, the average travel speed would be:

$$S = \frac{L}{\sum_{i=1}^{n} t_i/n} = \frac{nL}{\sum_{i=1}^{n} t_i} \quad (1\text{-}1)$$

where:

S = average travel speed, in mph;
L = length of the highway segment, in mi;
t_i = travel time of the ith vehicle to traverse the section, in hr; and
n = number of travel times observed.

Consider the following travel times observed for vehicles traversing a one-mile segment of highway:

1.0 min	(0.0167 hr)
1.2 min	(0.0200 hr)
1.7 min	(0.0283 hr)
1.1 min	(0.0183 hr)

The average travel time is found as (0.0167 + 0.0200 + 0.0283 + 0.0183)/4 = 0.0208 hr. The average travel speed is the distance (1 mi) divided by this time, or: S = 1.0 mi/0.0208 hr = 48 mph.

It should be noted that the travel times used in this computation include stopped delays due to fixed interruptions or traffic congestion. They are total travel times to traverse the defined segment. Average travel speed should not be confused with another similar measure, *average running speed,* which is defined as the distance divided by the average running time to traverse the distance. "Average running time" includes only the time that the vehicle is in motion. For uninterrupted flow facilities operating under uncongested conditions, average travel speed and average running speed are equal.

Both average travel speed and average running speed may also be referred to as *space mean speed.* This term is a statistical term frequently used in the literature to denote an average speed based on the average travel time of vehicles to traverse a segment of roadway. It is called a "space" mean speed as the use of average travel time essentially weights the average according to the length of time each vehicle spends in the defined roadway segment or "space."

For capacity analysis, speeds are best measured by observing travel times over a known length of highway. For uninterrupted flow facilities operating in the range of stable flow, the length taken may be as short as several hundred feet for ease of observation. For interrupted flow facilities, segments should be long enough to include those points of fixed interruption of interest.

Radar meters or other devices can be used to measure speeds at a point. Such speeds may be averaged to yield a *time mean speed.* Time mean speeds are usually 1 to 3 mph higher than the corresponding space mean speed. Time mean speeds are generally not relevant in the evaluation of interrupted flow facilities, as the travel time lost to interruptions is a major component of the evaluation. It is possible to compute a space mean speed for a short segment of highway using radar or other observations of individual vehicles speeds by calculating the harmonic, rather than the arithmetic, mean of the observations. Chapter 2 contains further discussion of the relationships between time mean and space mean speeds.

2. *Volume and rate of flow* are two measures that quantify the amount of traffic passing a point on a lane or roadway during a designated time interval. These terms are defined as follows:

- Volume—The total number of vehicles that pass over a given point or section of a lane or roadway during a given time interval; volumes may be expressed in terms of annual, daily, hourly, or subhourly periods.
- Rate of flow—The equivalent hourly rate at which vehicles pass over a given point or section of a lane or roadway during a given time interval less than one hour, usually 15 min.

The distinction between volume and rate of flow is an important one. Volume is an actual number of vehicles observed or predicted to be passing a point during a time interval. Rate of flow represents the number of vehicles passing a point during a time interval less than one hour, but expressed as an equivalent hourly rate. A rate of flow is found by taking the number of vehicles observed in a subhourly period and dividing it by the time (in hours) over which they were observed. Thus, a *volume* of 100 vehicles observed in a 15-min period implies a *rate of flow* of 100 veh/0.25 hr or 400 vph.

The following example further illustrates the difference between the two measures. The following traffic counts were made during an hour-long study period:

Time Period	Volume (veh)	Rate of Flow (vph)
5:00–5:15	1,000	4,000
5:15–5:30	1,200	4,800
5:30–5:45	1,100	4,400
5:45–6:00	1,000	4,000
5:00–6:00	4,300	

Volumes were observed for four consecutive 15-min periods. The total *volume* for the hour is the sum of these counts or 4,300 veh, or 4,300 vph (because they were observed for an hour). The *rate of flow,* however, varies within each 15-min period.

During the 15-min period of maximum flow, the rate of flow is 1,200 veh/0.25 hr, or 4,800 vph. Note that 4,800 vehicles do not pass the point in question during the study hour, but they do pass the point at that rate for 15 min.

Consideration of peak flow rates is of critical importance in capacity analysis. If the capacity of the above segment of highway were 4,500 vph, it would break down during the peak 15-min period of flow, when vehicles arrive at a rate of 4,800 vph—even though over the full hour, volume is less than capacity. This is a serious situation, because the dynamics of dissipating a breakdown may extend congestion up to several hours beyond the time of the breakdown. Chapter 6 discusses these dynamics in greater detail.

Peak rates of flow are related to hourly volumes through the use of the *peak-hour factor,* which is defined as the ratio of total hourly volume to the maximum 15-min rate of flow within the hour:

$$\text{PHF} = \frac{\text{Hourly volume}}{\text{Peak rate of flow (within the hour)}} \quad (1\text{-}2)$$

Then, if 15-min periods are used, the PHF may be computed as:

$$\text{PHF} = V/(4 \times V_{15}) \quad (1\text{-}3)$$

where:

PHF = the peak-hour factor;
V = hourly volume, in vph; and
V_{15} = volume during the peak 15 min of the peak hour, in veh/15 min.

The procedures of this manual most often focus on the analysis of either a peak 15-min period or another 15-min period of interest. Thus, many analyses will be based on rates of flow for such a period. Where the peak-hour factor is known, it may be used to convert a peak-hour volume to a peak rate of flow, as follows:

$$v = V/\text{PHF} \quad (1\text{-}4)$$

where:

v = rate of flow for a peak 15-min period, in vph;
V = peak-hour volume, in vph; and
PHF = peak-hour factor.

Equation 1-4 need not be used to estimate peak flow rates where 15-min counts are available, as the rate may be directly computed as 4 times the maximum 15-min count.

Many of the procedures of this manual use this conversion to allow computations to focus on the peak flow period within the peak hour. For some types of facilities, notably rural highways and unsignalized intersections, the analyst has the option of analyzing either full peak hours or peak 15-min flow periods.

3. *Density*—Density is defined as the number of vehicles occupying a given length of a lane or roadway, averaged over time, usually expressed as vehicles per mile (vpm).

Direct measurement of density in the field is difficult, requiring a vantage point from which significant lengths of highway can be photographed, videotaped, or observed. It can be computed, however, from the average travel speed and rate of flow, which are more easily measured.

$$v = S \times D \quad (1\text{-}5)$$

where:

v = rate of flow, in vph;
S = average travel speed, in mph; and
D = density, in vpm.

Thus, a highway segment with a rate of flow of 1,000 vph and an average travel speed of 50 mph would have a density of: D = 1,000 vph/50 mph = 20 vpm.

Density is a critical parameter describing traffic operations. It describes the proximity of vehicles to one another, and reflects the freedom to maneuver within the traffic stream.

Characteristics of Uninterrupted Flow

Equation 1-5 cites the basic relationship among the three parameters describing an uninterrupted traffic stream. Although the relationship $v = S \times D$ algebraically allows for a given rate of flow to occur at an infinite number of combinations of speed and density, there are additional relationships which restrict the variety of flow conditions that may exist at any given location.

Figure 1-1 shows the general form of these relationships, which are the philosophical basis for the capacity analysis of uninterrupted flow facilities. Although the form of these relationships is similar for all uninterrupted flow facilities, the exact shape of these curves and their numeric calibration depend on the prevailing traffic and roadway conditions existing on the highway segment under study. It should also be noted that calibrated curves for specific facilities may be discontinuous near capacity.

The curves of Figure 1-1 illustrate a number of significant points. Note that a zero rate of flow occurs under two very different conditions:

1. When there are no cars on the facility, density is zero, and rate of flow is also zero. Speed is purely theoretical for this condition, and it would be whatever the first driver would select—presumably a high value.
2. When density becomes so high that all vehicles stop (speed is zero), the rate of flow is also zero, because there is no movement and vehicles cannot "pass" a point on the roadway. The density at which all movement stops is called *jam density.*

Between these two extreme points, the dynamics of traffic flow produce a maximizing effect. As density increases from zero, rate of flow also increases because more vehicles are on the roadway. While this is happening, speed begins to decline (due to the interaction of vehicles). This decline is virtually negligible at low densities and rates of flow. As density continues to increase, however, a point is reached at which speed declines precipitously. The maximum rate of flow is reached when the product of increasing density and decreasing speed results in reduced flow.

The maximum rate of flow for any given facility is its capacity. The density at which this occurs is referred to as *critical density,* and the speed at which it occurs is called *critical speed.* As capacity is approached, flow becomes more unstable because available gaps in the traffic stream are fewer. At capacity, there are no usable gaps in the traffic stream, and any perturbation from vehicles entering or leaving the facility, or from internal lane changing maneuvers, creates a disturbance that cannot be effectively damped or dissipated. Thus, operation at or near capacity is difficult to maintain for long periods of time without the formation of upstream queues, and forced or breakdown flow becomes almost unavoidable. For this reason, most facilities are designed to operate at volumes less than capacity.

As illustrated in Figure 1-1, any rate of flow other than capacity can occur under two different conditions—one with a high speed and low density, the other with high density and low speed. The entire high-density, low-speed side of the curves

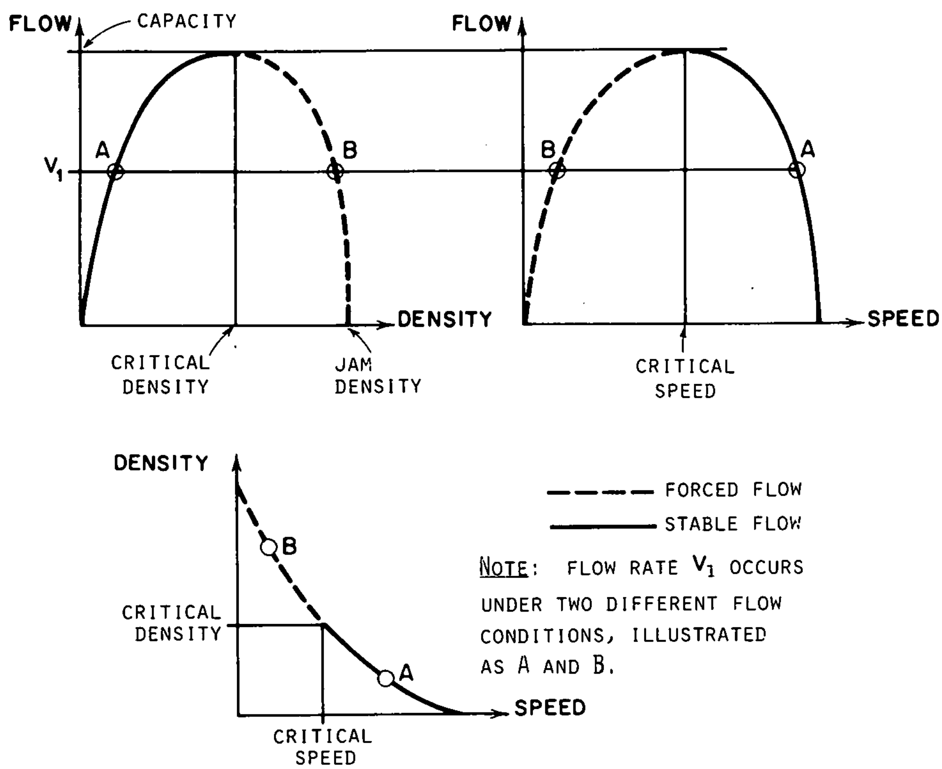


Figure 1-1. Relationships among speed, density, and rate of flow on uninterrupted flow facilities.

is considered to be unstable. This represents forced or breakdown flow. The low-density, high-speed side of the curves is the stable flow region. It is this flow region on which capacity analysis focuses. Levels-of-service A through E are defined on the stable side of the curves, with the maximum flow boundary of level-of-service E placed at capacity for uninterrupted flow facilities.

Characteristics of Interrupted Flow

Interrupted flow is far more complex than uninterrupted flow. Flow on an interrupted flow facility is usually dominated by points of fixed operation, such as traffic signals, STOP, and YIELD signs. These all operate quite differently, and have differing impacts on overall flow. Chapter 9 contains a detailed discussion of flow at signalized intersections, and Chapter 10 contains similar information for STOP and YIELD signs. Chapter 11 discusses arterial flow.

1. *The concept of green time at signalized intersections*—The most significant source of fixed interruptions on interrupted flow facilities is traffic signals. At traffic signals, flow in each movement or set of movements is periodically halted. Thus, movement on a given set of lanes is only possible for a portion of total time, because the signal prohibits movement during some periods. Only the time during which the signal is effectively green is available for movement. For example, if one set of lanes at a signalized intersection receives a 30-sec green phase out of a 90-sec total cycle, only 30/90 or one-third of total time is available for movement on the subject lanes. Thus, out of each hour of real time, only 20 min are available for flow on the lanes. If the lanes could accommodate a maximum rate of flow of 3,000 vph when the signal is green, they could accommodate a total rate of flow of only 1,000 vph, as only one-third of each hour is available as green.

As signal timings are subject to change, it is convenient to express capacities and service flow rates for signalized intersections in terms of "vehicles per hour of green" (vphg). In the previous example, the maximum rate of flow would be stated as 3,000 vphg. This can be converted to a real-time value by multiplying by the ratio of effective green time to cycle length for the signal.

2. *Saturation flow rate and lost times at signalized intersections*—At signalized intersections, traffic on all lanes will be periodically stopped. When the signal turns green, the dynamics of starting a standing queue of vehicles must be considered. Figure 1-2 illustrates a queue of vehicles stopped at a signal. When the signal turns green, the queue begins to move. The headways between vehicles can be observed as they cross the curb line of the intersection. The first headway would be the elapsed time, in seconds, between the initiation of the green and the crossing of the rear of the first vehicle over the curb line. The second headway would be the elapsed time between the crossing of rear of the first and second vehicles over the curb line. Subsequent headways would be similarly measured.

The driver of the first vehicle in the queue must observe the signal change to green and react to the change by taking his/her foot off the brake, and accelerating through the intersection. The first headway will be comparatively long as a result of this process. The second vehicle in the queue follows a similar process, except that the reaction and acceleration period can partially occur while the first vehicle is beginning to move. The second vehicle will be moving faster than the first as it crosses the curb line, because it has an additional vehicle length in

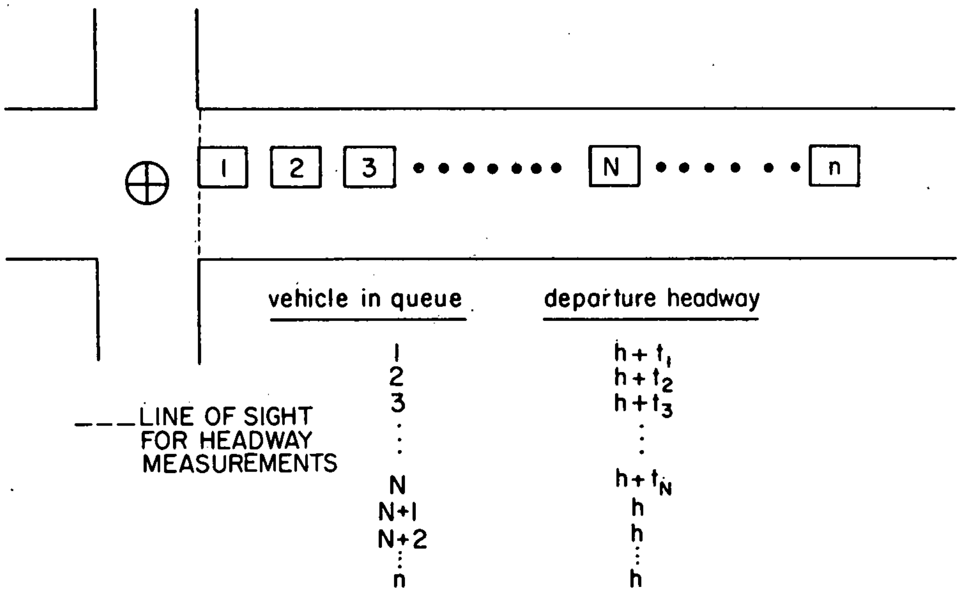


Figure 1-2. Conditions at a traffic interruption.

which to accelerate. Its headway will still be comparatively long, but is generally less than that of the first vehicle. The third and fourth vehicles follow a similar procedure, each achieving a slightly lower headway than the preceding vehicle. After some number of vehicles, "N" in Figure 1-2, the effect of the start-up reaction and acceleration has dissipated. Successive vehicles now move through past the curb line at their desired speed as a uniform moving queue until the last vehicle in the original queue has passed. The headway for these vehicles will be relatively constant.

In Figure 1-2, this constant average headway is denoted as "h" and is achieved after "N" vehicles. The headways for the first N vehicles are, on the average, greater than h, and are expressed as $h + t_i$, where t_i is the incremental headway for the ith vehicle due to the start-up reaction and acceleration. As i increases from 1 to N, t_i decreases.

Figure 1-3 shows a conceptual plot of headways measured as described previously. For purpose of illustration only, N is assumed to = 6, i.e., the start-up and acceleration increment disappears after the 6th vehicle.

The value h is defined as the *saturation headway*, and is estimated as the constant average headway between vehicles which occurs after the 6th vehicle in the queue and continues until the last vehicle in the initial queue clears the intersection. The saturation headway is the amount of time consumed by a vehicle in a stable moving queue as it passes through a signalized intersection on the green, assuming that a continuous queue of vehicles is available to move through the intersection.

Saturation flow rate is defined as the flow rate per lane at which vehicles can pass through a signalized intersection in such a stable moving queue. By definition, it is computed as:

$$s = 3{,}600/h \qquad (1\text{-}6)$$

where:

s = saturation flow rate, in vphgpl;
h = saturation headway, in sec; and
3,600 = number of seconds per hour.

The saturation flow rate represents the number of vehicles per hour per lane that can pass through an intersection if the green signal were available for the full hour, and the flow of vehicles were never halted. This assumes that in addition to a full hour of green being available, the average headway of all vehicles entering the interection is h seconds.

The reality of flow at a signalized intersection is that flow is periodically halted. Each time flow is halted, it must be started again, and it will experience start-up reaction and acceleration headways illustrated in Figure 1-3 for the first N vehicles. In Figure 1-3, the first six vehicles in the queue experience headways longer than h. The increments, t_i, are called *start-up lost times*. The total start-up lost time for these vehicles is the sum of these increments, or:

$$l_1 = \sum_{i=1}^{N} t_i \qquad (1\text{-}7)$$

where:

l_1 = total start-up lost time, sec; and
t_i = lost time for the ith vehicle in queue, in sec.

Each time a queue of vehicles receives a green signal, it will consume h seconds per vehicle, plus the start-up lost time, l_1, assuming that there are at least N vehicles in the queue.

Each time a stream of vehicles is stopped, another source of lost time is experienced. As one stream of vehicle stops, safety requires that there be some clearance time before a conflicting

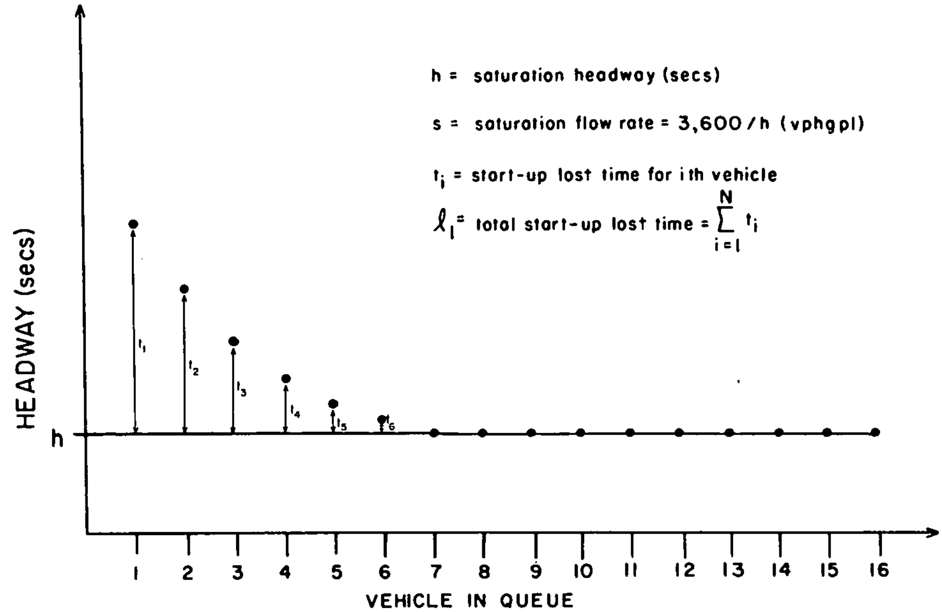


Figure 1-3. Saturation flow rate and lost time.

stream of traffic is allowed to enter the intersection. During this period, no vehicles use the intersection. This interval is called *clearance lost time,* l_2.

In practice, signal cycles provide for this clearance through the use of "change intervals," that may include yellow and/or all red indications. Drivers generally do not observe this entire interval and do use the intersection during some portion of it. The clearance lost time, l_2, is the portion of this change interval that is *not used* by motorists.

The relationship between saturation flow rate and lost times is a critical one. For any given lane or movement, vehicles use the intersection at the saturation flow rate for a period of time equaling *the available green time plus the change interval minus the start-up and clearance lost times.* As the lost times are experienced each time a movement is started and stopped, the total amount of time lost over an hour is related to the signal timing. If a signal has a 60-sec cycle length, it will start and stop each movement 60 times per hour, and the total lost time per movement will be $60(l_1 + l_2)$. If the signal has a 30-sec cycle, each movement will be stopped and started 120 times per hour, and the total lost time per movement will be $120(l_1 + l_2)$, twice as much as for the 60-sec cycle.

The amount of lost time impacts capacity. The foregoing logic suggests that the capacity of the intersection increases with increasing cycle length. This is somewhat offset by observations that the saturation headway, *h,* may increase if the length of continuous green indication becomes very long. Other intersection features may offset the reductive capacity impact of short cycles, such as turning lanes. Where left-turn lanes and phases exist, longer cycle lengths may cause the left-turn lane to overflow, thus reducing capacity by blocking through lanes.

As cycle length is increased, the average stopped-time delay per vehicle also tends to increase, assuming that adequate capacity is provided. Delay, however, is a complex variable that is affected by many variables, of which cycle length is only one.

Chapter 9 contains a complete discussion and presentation of analytic relationships among saturation headway, saturation flow rate, lost times, signal timing parameters, and delay.

3. *Flow at* STOP *and* YIELD *signs*—A driver at a STOP or YIELD sign faces a judgmental task. A gap must be selected in the major street flow through which to execute the desired movement. Thus, the capacity of STOP- or YIELD-controlled intersection approaches depends on two critical factors:

a. The distribution of available gaps in the major street traffic stream.
b. The distribution of gaps acceptable to minor street drivers.

The distribution of available gaps in the major street traffic stream depends on the total volume on the street, its directional distribution, the number of lanes on the major street, and the degree and type of platooning which exists in the traffic stream.

Gap acceptance characteristics depend on the type of maneuever (left, through, right) which must be executed by the minor street vehicle, the number of lanes on the major street, the speed of major street traffic, the sight distances, the length of time the minor street vehicle has been waiting, and the driver characteristics (eyesight, reaction time, age, etc.).

Chapter 10 describes flow at STOP- and YIELD-controlled interesection approaches, and analytic relationships relating critical variables to capacity.

4. *Delay*—A critical performance measure on interrupted flow facilities is delay. Delay is a general term that can be interpreted to mean a number of things. Average stopped-time delay is the principal measure of effectiveness used in evaluating level of service at signalized intersections.

Stopped-time delay is the time an individual vehicle spends stopped in a queue while waiting to enter an intersection.

Average stopped-time delay is the total stopped delay experienced by all vehicles in an approach or lane group during a designated time period divided by the total volume entering the intersection in the approach or lane group during the same time period, expressed in seconds per vehicle.

This parameter is also referred to in a general way in the determination of level of service for unsignalized intersections. The measure of effectiveness used is "reserve capacity," i.e., the capacity of the unsignalized intersection approach minus the demand. The procedure assumes, however, that this parameter is related to delay.

Analysis procedures for arterials (Chapter 11) consider both the travel time between signalized intersections and the delay encountered at intersections.

Stopped-time delay is used because it is a reasonably easy parameter to measure, and is conceptually simple. Delay due to traveling at speeds slower than desired is difficult to establish because it requires the setting of a reasonable desirable speed for each highway segment.

Transit and Pedestrian Measures

Principles of transit and pedestrian flow are described in Chapters 12 and 13, respectively. Measures of effectiveness used to analyze level of service for transit and pedestrian facilities were given in Table 1-2, and are defined in the following paragraphs.

1. *Load factor*—Load factor is a transit performance measure defined as the number of passengers on a transit vehicle divided by the number of seats on the vehicle. It is generally applied on a vehicle-by-vehicle basis, i.e., it is not averaged over a group of vehicles. It is a measure of the in-vehicle environment provided to the transit user.
2. *Service frequency*—The frequency of service is an important level-of-service measure for transit. Frequency of service is defined by the number of buses or trains provided per hour for a designated period of time. Service frequency impacts waiting and transfer times, and it determines the ease and convenience with which a service can be used for particular trips.
3. *Space*—Space is the measure of effectiveness used to define pedestrian level of service. It is defined as the average area provided each pedestrian in a pedestrian stream or queue, and is expressed as square feet per pedestrian.

Level-of-service criteria are not defined for bicycles, and the treatment of bicycles herein is limited to their impact on other vehicular flow at critical points in the street and highway system.

FACTORS AFFECTING CAPACITY, SERVICE FLOW RATE, AND LEVEL OF SERVICE

Ideal Conditions

Many of the procedures of this manual provide simple tabular or graphic presentations for a set of specified standard conditions, which must be adjusted to account for any prevailing conditions not matching those specified. Often, the conditions so defined are "ideal conditions."

In principle, an ideal condition is one for which further improvements will not achieve any increase in capacity. These conditions are specified in each chapter. Examples of ideal conditions are given below for uninterrupted flow facilities and for signalized intersections.

Ideal conditions for uninterrupted flow facilities include:

1. Twelve-foot lane widths.
2. Six-foot clearance between the edge of the travel lanes and the nearest obstructions or objects at the roadside and in the median.
3. Seventy-mile per hour design speed for multilane highways; 60 mph design speed for two-lane highways.
4. All passenger cars in the traffic stream.

Ideal conditions for signalized intersection approaches include:

1. Twelve-foot lane widths.
2. Level grade.
3. No curb parking on the intersection approaches.
4. All passenger cars in the traffic stream, including no local transit buses stopping within the intersection.
5. All vehicles traveling straight through the intersection.
6. Intersection located in a non-CBD area.
7. Green signal available at all times.

In most capacity analyses, prevailing conditions are not ideal, and computations of capacity, service flow rate, or level of service must include adjustments to reflect this. Prevailing conditions are generally categorized as roadway, traffic, or control conditions.

Roadway Conditions

Roadway factors include all of the geometric parameters describing the roadway, including:

1. The type of facility and its development environment.
2. Lane widths.
3. Shoulder widths and/or lateral clearances.
4. Design speed.
5. Horizontal and vertical alignments.

The type of facility is critical. Whether or not uninterrupted flow exists, whether or not directional flows are separated by medians, and other major facility type factors significantly affect flow characteristics and capacity. The development environment has also been found to affect the performance of multilane highways and signalized intersections.

Lane and shoulder widths can have a significant impact on traffic flow. Narrow lanes cause vehicles to travel closer to each other laterally than most drivers would prefer. Motorists compensate by slowing down or by observing larger longitudinal spacing for a given speed. This effectively reduces capacity and/or service flow rates.

Narrow shoulders and lateral obstructions have two important impacts. Many drivers will "shy away" from roadside or median objects they perceive to pose a hazard. This brings them laterally closer to vehicles in adjacent lanes and causes the same reactions as for narrow lanes. On two-lane highways in many areas, shoulders are used to allow for the passing of slow vehicles, and narrow shoulders may adversely affect flow.

Restricted design speeds affect operations and level of service, because drivers are forced to travel at somewhat reduced speeds, and to be more vigilant in reaction to the harsher horizontal and vertical alignments reflected by reduced design speed. In extreme cases, the capacity of multilane facilities has been found to be affected by low design speeds.

The horizontal and vertical alignment of a highway is a product of the design speed used and the topography through which the roadway must be constructed. Procedures for uninterrupted flow facilities categorize the general terrain of a highway as follows:

1. *Level terrain*—Any combination of grades and 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.

2. *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.

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

Heavy vehicles are defined as any vehicles having more than four tires touching the pavement. *Crawl speed* is the maximum sustained speed which heavy vehicles can maintain on an extended upgrade of a given percent.

These definitions are general, and they depend on the particular mix of heavy vehicles in the traffic stream. In general, as terrain becomes more severe, capacity and service flow rates are reduced. This impact is significant for two-lane rural highways, where the severity of terrain not only affects the operating capabilities of individual vehicles in the traffic stream, but also restricts the opportunities to pass slow-moving vehicles in the traffic stream.

In addition to the general impacts of terrain, isolated upgrades of significant length can have a significant effect on operations. Heavy vehicles slow significantly on such upgrades, creating operational difficulties in the traffic stream, and inefficient use of the roadway.

Grades can also have a major impact on the operation of intersection approaches, because vehicles must overcome both the inertia of starting from a stopped condition and the grade at the same time.

Traffic Conditions

1. *Vehicle types*—The principal characteristic of the traffic stream influencing capacity, service flow rate, and level of service is the distribution of vehicle types. Heavy vehicles, defined previously, adversely impact traffic in two critical ways:

a. They are larger than passenger cars and, therefore, occupy more roadway space than passenger cars.

b. They have poorer operating capabilities than passenger cars, particularly with respect to acceleration, deceleration, and the ability to maintain speed on upgrades.

The latter impact is the most critical. Because heavy vehicles cannot "keep up" with passenger cars in many situations, large gaps form in the traffic stream that are difficult to fill by passing maneuvers. This creates inefficiencies in the use of roadway space that cannot be completely overcome. This effect is particularly deleterious on sustained, steep upgrades, where the difference in operating capabilities is most pronounced, and on two-lane highways, where passing must be accomplished using the opposing travel lane.

Heavy vehicles may also impact downgrade operations, particularly where downgrades are steep enough to require such vehicles to operate in a low gear. In such cases, heavy vehicles again must operate at speeds slower than those of passenger cars, and gaps in the traffic stream will form.

Heavy vehicles are generally grouped into one of three categories:

a. Trucks—A truck is defined as a heavy vehicle involved primarily in the transport of goods, or in the delivery of a service (other than public transportation).

b. Recreational vehicles—A recreational vehicle (RV) is defined as a heavy vehicle, operated by a private motorist, and involved in the transport of recreational equipment or facilities.

c. Buses—A bus is a heavy vehicle involved in the transportation of groups of people on a for-hire, charter, or franchised transit basis. Buses are further categorized as intercity or local transit buses. Intercity (or "through") buses operate in a traffic stream without making stops to pick up or discharge passengers on the subject facility. Local transit buses make such stops within the confines of the subject facility.

There is considerable variation in the characteristics and performance capabilities of vehicles within each class of heavy vehicle, just as there is among passenger cars.

Trucks cover a particularly wide range of vehicles, however, from lightly loaded vans and panel trucks to the most heavily loaded coal, timber, and gravel haulers. Individual trucks will have widely varying operational characteristics based on how heavily they are loaded. Analysis procedures for each type of facility discuss the mix of trucks on each in some detail. Some analysis procedures allow the user to select various "typical" trucks, based on the prevailing mix. The typical truck in most traffic streams has an average weight-to-horsepower ratio of 200 lb/hp.

Facilities dominated by heavy farm trucks, gravel haulers, or similar vehicles would have an average weight-to-horsepower ratio in the 300 to 400 lb/hp range. Analysis procedures for trucks generally assume the characteristics of the typical truck in the traffic stream, and apply these characteristics to all trucks in the stream. None of the procedures segregate the truck population into subcategories for separate computational consideration.

Recreational vehicles also cover a broad range of vehicle types, including campers, both self-propelled and towed, motorhouses, and passenger cars or small trucks towing a variety of recreational equipment, such as boat, snowmobile, and motorcycle trailers. Weight-to-horsepower ratios in this category generally range from 30 to 60 lb/hp. While recreational vehicles have considerably better operating capabilities than trucks, the relative inexperience of many recreational vehicle drivers accentuates the impact of such vehicles' deficiencies.

Intercity buses are relatively uniform in their performance capabilities. Their weight-to-horsepower ratios are in the 70 to 100 lb/hp range, and they are generally capable of maintaining speed in level and rolling terrain, except on isolated long or steep grades.

Transit buses are generally not as powerful as intercity buses. Their most severe impact on traffic, however, is due to the discharge and pick-up of passengers on the roadway. Local transit buses make such stops at the curb, usually at intersections, along multilane suburban highways, arterials, and city streets. Where there is no curb parking on the roadway, the stopped bus blocks a travel lane. Where curb parking does exist, the bus disrupts flow in adjacent travel lanes as it enters and leaves the bus stop.

2. *Lane use and directional distribution*—In addition to the distribution of vehicle types, there are two other traffic characteristics that affect capacity, service flow rates, and level of service. *Directional distribution* has a dramatic impact on two-lane rural highway operation. Optimum conditions occur when the split of traffic is about 50 percent in each direction. Capacity declines as the directional split becomes more unbalanced. Capacity analysis procedures for multilane highways focus on a single direction of flow. Nevertheless, each direction of the facility is usually designed to accommodate the peak rate of flow in the peak direction. Typically, morning peak traffic occurs in one direction, while in the evening, it occurs in the opposite direction. *Lane distribution* is also a factor on multilane facilities. Typically, the shoulder lane of a multilane facility carries less traffic than other lanes. Analysis procedures assume typical lane distributions for most types of facilities.

Control Conditions

For interrupted flow facilities, the control of the time available for movement of specific traffic flows is a critical element affecting capacity, service flow rates, and level of service. The most critical type of control on such facilities is the traffic signal. Operations are affected by the type of control in use, the signal phasing, the allocation of green time, and the cycle length. All of these terms are defined and discussed in detail in Chapter 9, "Signalized Intersections." For this introduction, it suffices to note that the traffic signal determines the amount of time available for movement on various lanes of the intersection.

STOP and YIELD signs also affect capacity, but in a less deterministic way. While the signal positively assigns designated times when each movement is permitted, the STOP or YIELD sign merely assigns the right-of-way permanently to a major street. Minor street vehicles must find gaps in the major traffic flow through which to execute maneuvers. Thus, the capacity of such approaches is dependent on traffic conditions on the major street.

Four-way STOP control forces drivers to alternately enter the intersection from a standing stop in rotation. Such control limits capacity, and operational characteristics may vary widely depending on traffic demands on the various approaches.

There are other types of controls and regulations that can significantly affect capacity, service flow rates, and level of service. Restriction of curb parking can increase the number of available lanes available on a street or highway. Turn restrictions can eliminate conflicts at intersections, and increase capacity. Lane use controls can positively allocate available roadway space to component movements, and they can be used at intersections; they also can be used to create reversible lanes on critical arterials.

Summary

The preceding sections have emphasized the number of characteristics that have an effect on capacity. The importance of these is twofold. First, the variables discussed are important factors involved in the capacity analysis computations described in this manual. Second, these conditions define the parameters that planners and engineers can consider changing to provide for improvements to capacity and level of service. The engineer has, to varying degrees, control over the geometric and control parameters discussed. Through construction, reconstruction, or spot improvements, lane widths, shoulder widths, the number of lanes, horizontal and vertical alignment, and other geometric factors can be improved.

Through regulation and signalization, all of the control variables are subject to alteration. These, then, are the tools with which the engineer addresses capacity or service deficiencies. One of the most important uses of the procedures of this manual is in the evaluation of alternative improvement plans based on such changes.

III. APPLICATIONS

LEVELS OF ANALYSIS

Most of the procedural chapters address three different computational applications: operational analysis, design, and planning.

1. *Operational analysis*—Operational analysis is the most detailed and flexible application of capacity analysis techniques. In this application, known or projected traffic flow rates and characteristics are compared with known or projected highway descriptions to estimate the level of service that is expected to prevail.

For existing facilities, this requires detailed input information on traffic characteristics, including volumes, peak hour factors, directional distributions, and vehicle type distributions. All geometric conditions for the facility must also be known, including number and width of lanes, shoulder clearances, design speeds, grades, and horizontal and vertical alignments.

Where traffic controls exist, such as at a signalized intersection, they must be completely specified, including the type of control, cycle length, phasing, green time allocation, and other factors. All other types of controls must also be specified. For planned or future facilities, the same type of information is required. It would, however, be based on traffic projections and planned facilities rather than on field-measured data.

Operational analysis allows for an evaluation of level of service on an existing facility. This, however, is not its most powerful use. Operational analysis can be used to evaluate the level of service that would result from alternative spot and section improvements to an existing facility. The operational impacts of various improvement measures can be estimated and compared, and a rational decision made using the results and other relevant information. Alternative designs for new facilities can be similarly evaluated using the operational analysis approach.

Most of the procedures in this manual allow not only a determination of level of service, but an estimation of the value of critical performance parameters as well. Thus, for a freeway segment, density and speed of the traffic stream can be estimated, and for a signalized intersection, average individual stopped time delay can be estimated. Thus, an operational analysis not only yields a determination of the level of service (which covers a range of conditions), but it provides precise values of operational parameters as well.

An alternative use of operational analysis is to determine the service flow rates allowable under varying operational (LOS) assumptions. Such analyses are extremely useful in evaluating the sensitivity of service flow rates to various design or LOS assumptions.

2. *Design*—The design application of computations has a specific objective: to determine the number of lanes required on a particular facility to provide for a specified level of service. The design application of capacity analysis procedures treats this aspect of the overall "design process," and can also be used to assess the impact of such design variables as lane and shoulder width, lateral clearance, grades, lane use allocations, and other features. Detailed data on expected traffic volumes and characteristics are required, as is the assumption of geometric standards to be used in the design: lane widths, lateral clearances, design speeds, and horizontal and vertical alignment. Design of signal timings can also be accomplished using the procedures presented in Chapter 9, "Signalized Intersections."

Design computations are generally limited in scope, and often result in the generation of alternatives that are subsequently subjected to detailed operational analyses.

3. *Planning*—Planning computations have the same objective as design computations: determination of the number of lanes required to provide for a given level of service. The planning application, however, is intended for rough estimates at the earliest stages of planning when the amount, detail, and accuracy of information are limited. Planning procedures are often based on forecasts of average annual daily traffic (AADT), and on assumed traffic, roadway, and control conditions. The assumed "normal" characteristics are specified in each chapter. All planning computations must be refined as more information becomes available later in the planning and design processes.

The selection of a level of analysis depends on the intended use of results and also on the availability of data on which to base computations. All applications are basically variations of the same methodology. As the applications get less detailed, more "average" values are assumed and used in computations. It must be recognized, however, that the use of such values can lead to errors where prevailing conditions vary substantially from those assumed.

PRECISION

In making capacity computations, it should be remembered that results can be expected to be no more precise or accurate than the information or data used as inputs to the analysis. Thus, where traffic counts are only accurate to + or − 5 percent, or where projections are subject to even larger errors, computations cannot be expected to be accurate to the nearest vehicle per hour or mile per hour.

All tabulated service flow rates in this manual have been rounded to the nearest 50 vph, and analysts may wish to round all computational results in this manner as well.

FIELD DATA

The basic traffic data required to conduct any level of analysis are volume, either existing or forecast, and traffic characteristics: PHF, vehicle types, directional distribution. Procedures of this manual have been calibrated to estimate performance parameters such as speed, density, delay, and others, based on specified volumes and traffic characteristics.

This is the most prevalent use of analysis procedures, in that volume is the most readily and often measured traffic stream parameter. The procedures that predict performance, however, are based on average observed conditions throughout North America. The relationships between volume and performance are subject to variance due to local driving habits and other factors. Thus, estimations of operational criteria will never exactly duplicate field-measured values at specific locations.

It is possible, on existing facilities, to measure operational variables directly. When this is done, level-of-service determinations may be made by comparing field-measured values against the defined criteria. This is discussed in each chapter, and must be done with some care, as criteria are often defined for ideal or other specified conditions. For example, the densities defining level of service for freeways and multilane highways are specified in passenger cars per mile per lane. Field-measured values in vehicles per mile per lane would have to be converted to passenger car units before comparison to the established criteria.

Where local data are available in sufficient quantity, and in an acceptable form, they may be used to "fine tune" the procedures presented herein. Several chapters contain specific recommendations on when and how this should be done. Procedures specify certain average relationships and values, calibrated for average U.S. conditions. The procedures can often be made more accurate by substituting local calibrations for these. Some examples of local calibrations that could be used include flow-density-speed relationships for multilane facilities and saturation flow rates for signalized intersections.

Where such substitutions are made, care must be taken that local data and calibrations are for the same base conditions as described in the manual. A saturation flow rate for a 10-ft lane should not be substituted for a manual value applied to a 12-ft lane, without considering the impact on lane width adjustment factors, for example.

There is no substitute for accurately collected and presented field data. A capacity analysis based on inaccurate roadway, traffic, and control information will produce erroneous results.

The results of computations will not be more accurate than the input data on which they were based.

SUMMARY

The Highway Capacity Manual is a set of analysis procedures which provides information on and estimates of the performance of a variety of traffic facilities, based on known or projected roadway, traffic, and control conditions. Performance criteria can also be set at desired levels, and selected traffic, roadway, or control conditions estimated.

The results of the procedures provide critical comparative information to the engineer or planner. These results should be used along with other relevant information to formulate recommendations on highway, transit, and pedestrian improvements. No computation based on these procedures should be construed as mandating or requiring the implementation of a particular improvement or design alternative. The professional judgment of the engineer or planner is a necessary input to such decisions. The results of capacity analysis do not replace the need for the exercise of professional judgment, but rather provide additional information on which to partially base such judgments.

TRAFFIC CHARACTERISTICS

CONTENTS

I. INTRODUCTION

Chapter 1 introduced the basic concepts of capacity and level of service, as well as the generic characteristics of uninterrupted and interrupted flow.

The procedures of this manual are based on calibrated "national average" traffic characteristics observed over a range of facilities of each type. Observations of these characteristics at specific locations will vary somewhat from national averages because of local driving habits and unique features of the local driving environment. This chapter addresses the range of characteristics that have been observed, and relates them to the values used in the capacity analysis procedures of the subsequent chapters. The chapter also presents information on traffic parameters not explicitly used in analysis procedures, but whose impact on capacity and level of service is important.

The focus of this chapter is on highway traffic characteristics. Transit, pedestrian, and bicycle characteristics are discussed in Chapters 12, 13, and 14, respectively.

This chapter presents and discusses a sampling of national observations of key capacity and level-of-service parameters, including:

1. Volume and rate of flow.
2. Speed.
3. Density.
4. Spacing and headway.
5. Saturation flow rates.
6. Lost times.

Also discussed are relationships among these parameters and their variation in time and space. It is important to recognize the impact of these characteristics on highway operation and, therefore, on highway planning and design requirements, and to note, as well, the variations from national averages that can and do occur because of unique local conditions.

II. MAXIMUM OBSERVED VOLUMES AND FLOW RATES

Capacity is defined in terms of the maximum rate of flow that can be accommodated by a given traffic facility under prevailing conditions. The determination of capacity involves the observation of highways of various types operating under high-volume conditions.

The direct observation of absolute capacity is difficult to achieve for several reasons. The recording of a high, or even a maximum, volume or rate of flow for a given facility does not assure that a higher flow could not be accommodated at another time. Further, capacity is sometimes not a stable operating condition. It is often estimated by calibrating a speed-flow and/or density-flow curve for a given highway throughout the stable and unstable flow region. The peak of this curve, when fitted, would define capacity.

The following sections present and discuss maximum reported volume and flow rate observations on various types of facilities throughout the United States and Canada. It is noted that the reported observations may or may not represent the absolute capacities of the subject highways, but that they are reflective of prevailing conditions at the locations in question. These observations are a sample of high volumes recorded by state and local highway agencies, and should not be construed as suggesting that there are no other facilities experiencing similar or even higher volumes.

The data were collected from the literature, and from a survey conducted by the Committee on Highway Capacity and Quality of Service of the Transportation Research Board in 1984. The latter survey produced responses from 57 highway agencies.

FREEWAYS

Table 2-1 gives a sample of maximum observed volumes on rural and urban freeways in the United States. The table indicates average volume per lane and the volume in the peak lane.

The freeway and multilane highway capacity analysis procedures of this manual retain the use of 2,000 pcphpl as the basic capacity of such facilities under ideal conditions. This represents the average per lane capacity for multilane uninterrupted flow facilities across all lanes in a given direction. Table 2-1 contains numerous observations of values higher than this standard, but it should be remembered that these are *maximums* observed throughout a state or region.

A broad view of field-measured capacities suggests that values over 2,000 pcphpl still represent unusual occurrences, and that other facilities will not quite reach this level (such as noted in Table 2-1, the Northern and Southern State Parkways in New York). Thus, the recommended value of 2,000 pcphpl represents a national average, around which there is expected to be some variation from region-to-region.

It is also of interest that an individual lane of a multilane freeway can carry volumes well in excess of 2,000 pcphpl, with a maximum observation of 2,907 vphpl, in Table 2-1, occurring on a four-lane urban freeway in West Virginia. This same facility, however, had a total volume in one direction of 4,152 vph during the same period, and displays an unusually skewed lane distribution.

RURAL TWO-LANE HIGHWAYS

High-volume data on two-lane, two-way rural highways in the United States and Canada are difficult to obtain. Rarely do such highways operate at volumes approaching capacity, and thus the observation of capacity operations in the field is extremely complex.

A sampling of high-volume observations is given in Table 2-2, but it is emphasized that none of these may be taken to represent absolute capacity for the facilities shown. In several cases, the volumes noted were accompanied by good operating conditions.

European observations on two-lane, two-way rural highways have been reported at far higher volumes. Volumes of more than 2,700 vph have been observed in Denmark, 2,800 vph in France, 3,000 vph in Japan, and 2450 in Norway. Some of these volumes have contained significant numbers of trucks, some as high as 30 percent of the traffic stream (*4*).

The difficulty in observing capacity operations on two-lane highways in North America presents problems in terms of suggesting a standard value for use in computational procedures. The procedures for such highways, presented in Chapter 8, are based on a combination of field observations and simulation,

TABLE 2-1. MAXIMUM OBSERVED ONE-WAY HOURLY VOLUMES ON FREEWAYS

LOCATION	TOTAL VOLUME (VPH)	AVG. VOL. PER LANE (VPHPL)	VOL. IN PEAK LANE (VPHPL)
RURAL 4-LANE FREEWAYS			
I-93, Windham, New Hampshire	3510	1755	—
I-93, Tilton, New Hampshire	3096	1548	—
Calif. 4, Contra Costa City, California	4342	2171	—
Glenn Hwy, Anchorage, Alaska	3910	1455	—
RURAL 6-LANE FREEWAYS			
I-90, N/W Tollway, Illinois	6120	2040	—
I-94, Tri State Tollway, Illinois	8127	2709	—
URBAN 4-LANE FREEWAYS			
I-35W, Minneapolis, Minnesota	4690	2345	—
I-64, Charleston, West Virginia	4152	2077	2907
I-71, Kansas City, Missouri	5256	2628	—
I-70, Wheeling, West Virginia	3645	1823	2552
I-64, Charleston, West Virginia	3586	1793	2510
I-59, Birmingham, Alabama	4802	2401	—
I-295, Washington, D.C.	4480	2240	—
I-35, Kansas City, Kansas	4398	2199	—
I-45, Houston, Texas	4240	2120	—
I-55, Jackson, Michigan	3733	1867	2106
Northern State Pkwy, New York	3840	1920	—
URBAN 6-LANE FREEWAYS			
I-40, Nashville, Tennessee	6104	2035	2664
I-5, Seattle, Washington	—	—	2630
I-5, Seattle, Washington	—	—	2575
I-25, Denver, Colorado	6477	2159	2342
I-495, Prince George County, Maryland	6993	2331	—
U.S. 6, Denver, Colorado	6885	2295	—
Calif. 17, San Jose, California	6786	2262	—
I-5, Portland, Oregon	6474	2158	—
I-15, Salt Lake City, Utah	6357	2119	—
Southern State Pkwy, New York	5610	1870	—
I-35W, Minneapolis, Minnesota	6909	2303	—
I-290, Hillside, Illinois	6149	2047	—
URBAN 8-LANE FREEWAYS			
I-5, Seattle, Washington	—	—	2553
I-70, Columbus, Ohio	6198	1550	2298
I-405, Los Angeles California	8360	2090	—
I-71, Columbus, Ohio	6682	1670	2088
I-25, Denver, Colorado	8340	2085	—
U.S. 50, Sacramento, California	8284	2071	—
U.S. 59, Houston, Texas	8268	2067	—
U.S. 101, San Francisco, California	8180	2045	—
I-35W, Minneapolis, Minnesota	8168	2042	—

SOURCE: HCQS Survey; Ref. *1*

TABLE 2-2. MAXIMUM OBSERVED HOURLY VOLUMES ON TWO-LANE RURAL HIGHWAYS

LOCATION	TOTAL VOLUME (VPH)	PEAK DIR. VOLUME (VPH)	OFF-PEAK DIR. VOLUME (VPH)
2-LANE HIGHWAYS			
U.S. 50, Lake Tahoe, California	1690	1140	550
U.S. 77, Fremont, Nebraska	862	—	—
Hwy. 1, Banff, Alberta, Canada	2000	—	—
Hwy. 1, Banff, Alberta, Canada	2450	—	—
Hwy. 35, Kirby, Ontario, Canada	2050	—	—
Hwy. 35/115, Kirby, Ontario, Canada	2250	—	—
Calif. 84, Fremont, California	2364	1825	539
Trans-Canada Hwy., Alberta, Canada	1391	1150	241
Wasatch Blvd., Salt Lake City, Utah	2198	1504	694
2-LANE BRIDGES/TUNNELS			
Sagamore Bridge, Hudson, New Hampshire	2180	—	—
Underwood Bridge, Hampton, New Hampshire	1932	1061	871
Stanley Viaduct, Decatur, Illinois	1919	971	948

SOURCE: HCQS Survey; Refs. *2* and *3*

TABLE 2-3. MAXIMUM OBSERVED ONE-WAY HOURLY VOLUMES FOR MULTILANE HIGHWAYS

LOCATION	TOTAL VOLUME (VPH)	AVG. VOL. PER LANE (VPHPL)
4-LANE HIGHWAYS		
Utah 201, Salt Lake City, Utah	3670	1835
Utah 201, Salt Lake City, Utah	3632	1816
4-LANE TUNNELS		
I-279, Fort Pitt Tunnel, Pittsburgh, Pennsylvania	4278	2139
I-376, Squirrel Hill Tunnel, Pittsburgh, Pennsylvania	3922	1961

SOURCE: HCQS Survey

which suggests that a maximum capacity of 2,800 pcph be adopted, total in both directions under ideal conditions (*5*). These ideal conditions include a 50/50 directional distribution of traffic. Capacity on two-lane rural highways varies with directional distribution, and reduces as the split moves away from 50/50 to a minimum value of 2,000 pcph when the split is 100/0. This latter value is consistent with the standard for multilane flow in a single direction.

MULTILANE HIGHWAYS

The observation of multilane rural highways operating under capacity conditions is also difficult, because such operations rarely occur. Table 2-3, however, contains some data for four-lane multilane highways in suburban settings operating under uninterrupted flow conditions, as well as data for several four-lane tunnels. The tunnels in Pennsylvania are on freeway facilities, but it is viewed that tunnel operations are not significantly influenced by the type of facility forming the approaches to it. The procedures of this manual assume that the capacity of a surface multilane facility is the same as that for freeways for uninterrupted flow segments—2,000 pcphpl.

URBAN ARTERIALS

The interpretation of high-volume observations on urban arterials is not as straightforward as for uninterrupted flow facilities. Signal timing plays a major role in the capacity of such facilities, limiting the portion of time that is available for movement along the arterial at critical intersection locations. The volumes reported in Table 2-4 are shown with the green time-to-cycle length (g/C) ratios in effect for the subject segments. Flow rates in vehicles per hour of green time are estimated by taking the reported volumes and dividing by the reported g/C

TABLE 2-4. MAXIMUM OBSERVED ONE-WAY HOURLY VOLUMES ON URBAN ARTERIALS

LOCATION	TOTAL VOLUME (VPH)	AVG. VOL. PER LANE (VPHPL)	g/C RATIO	TOTAL FLOW RATE (VPHG)	AVG. FLOW RATE PER LANE (VPHGPL)
4-LANE ARTERIALS					
U.S. 50, Cambridge, Maryland	3164	1582	0.60	5273	2637
Ohio 161, Columbus, Ohio	2454	1227	0.48[a]	5112	2556
Woodway EB, Houston, Texas	2336	1168	0.44	5309	2655
Antoine, Houston, Texas	2310	1155	0.65	3553	1777
Woodway, WB, Houston, Texas	2156	1078	0.76	2836	1418
5-LANE ARTERIALS (WITH 2-WAY LEFT-TURN LANES)					
North Shepard NB, PM, Houston, Texas	2100	1050	0.60	3500	1750
6-LANE ARTERIALS					
Almaden Expressway, San Jose, California	3960	1320	0.66	6000	2000
Southwest Trafficway, Kansas City, Missouri	3492	1164	0.60	5820	1940
Ward Parkway, Kansas City, Missouri	3477	1159	0.61	5700	1900
Col. 2, Denver, Colorado	3435	1145	0.50	6870	2290
Seward Highway, Anchorage, Alaska	3105	1035	0.70	4436	1479
8-LANE ARTERIALS					
FM 1093, Houston, Texas	4500[b]	1125	0.70	6429	1607
FM 1093, Houston, Texas	4268[b]	1067	0.70	6097	1524

[a] Plus a left-turn phase.
[b] 9-ft lanes.
SOURCE: HCQS Survey

ratio. These estimates produce a set of flow observations on a comparable basis. It is significant to note also that the prevailing conditions on urban arterials may vary greatly and that such factors as curb parking, transit buses, and lane widths, among others, may substantially affect operations and observed volumes.

In addition, the comparison of maximum flow rates in vehicles per hour of green per lane varies widely for the various size arterials. These observations did not include such factors as left- and right-turn lanes at intersections, which may enhance the capacity of the intersection approach, nor were other prevailing conditions cited. The procedures of Chapter 11 for arterials focus on the issue of level of service. Capacity of the arterial is generally limited by the capacity of signalized intersections, with segment characteristics rarely playing a major role in the determination of capacity.

III. VOLUME CHARACTERISTICS

Traffic volumes vary in both space and time. These variations are critical determinants of the way highway facilities are used, and they control many of the planning and design requirements for adequately serving traffic demand.

Because traffic volume is not evenly distributed throughout the day, facilities are often designed to meet peak demands occurring for periods as short as 15 min or an hour. During other time periods, highways are often underutilized. Similarly, traffic does not distribute equally over available lanes or directions on a given facility.

TEMPORAL VARIATIONS

Traffic demand varies by month of year, by day of the week, by hour of the day, and by subhourly intervals within the hour. These variations in traffic demand are important if highways are to effectively serve peak demands without breakdown.

As discussed in Chapter 6, breakdowns into level-of-service F operation may occur because of the inability to process demand for periods as short as 15 min. The effects of a breakdown, however, may extend far beyond the time during which demand exceeds capacity, and may take up to several hours to dissipate. Thus, highways minimally adequate to handle a peak-hour demand may be subject to breakdown if flow rates within the peak hour exceed the capacity.

Seasonal peaks in traffic demand are also of great importance, particularly for primarily recreational facilities. Highways serving beach resort areas may be virtually unused during much of the year, only to be subject to regular congestion during peak summer periods.

The following sections present observed patterns of time variation in traffic demand for various types of facilities in North America.

Seasonal and Monthly Variations

Seasonal fluctuations in traffic demand reflect the social and economic activity of the area being served by the highway. Figure 2-1 shows monthly variation patterns observed in Illinois and Minnesota, and illustrates several significant characteristics:

1. Monthly variations are more severe on rural routes than on urban routes.
2. Monthly variations are more severe on rural routes serving primarily recreational traffic than on rural routes serving primarily business routes.
3. Daily traffic patterns vary by month of year most severely for recreational routes.

These observations lead to the conclusion that commuter and business-oriented travel occurs in more uniform patterns, and that recreational traffic is subject to the greatest variation among trip-purpose components of the traffic stream.

The data for Figure 2-1(b) were collected on the same Interstate route. One segment is within one mile of the central busi-

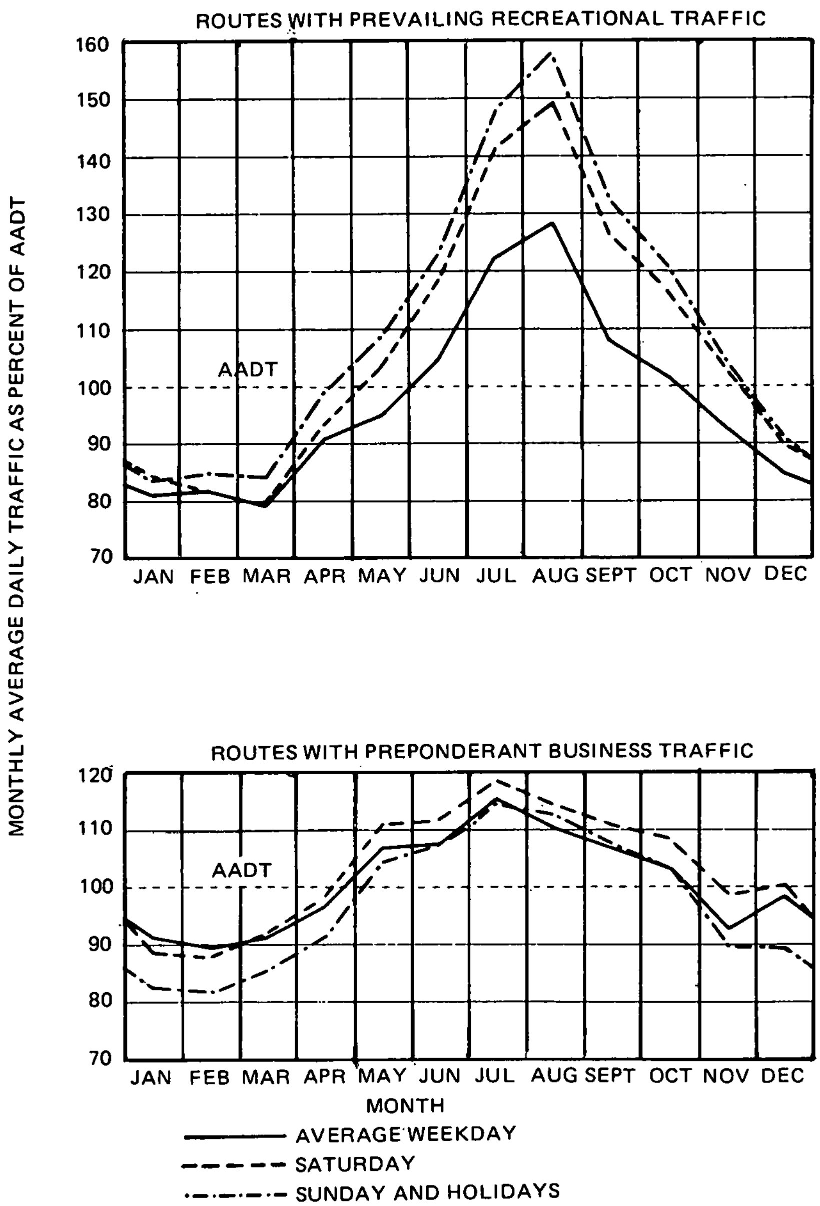


Figure 2-1(a). Examples of monthly traffic volume variations showing relative traffic volume trends by route type on rural roads in Lake County, Illinois. (Source: Ref. 6)

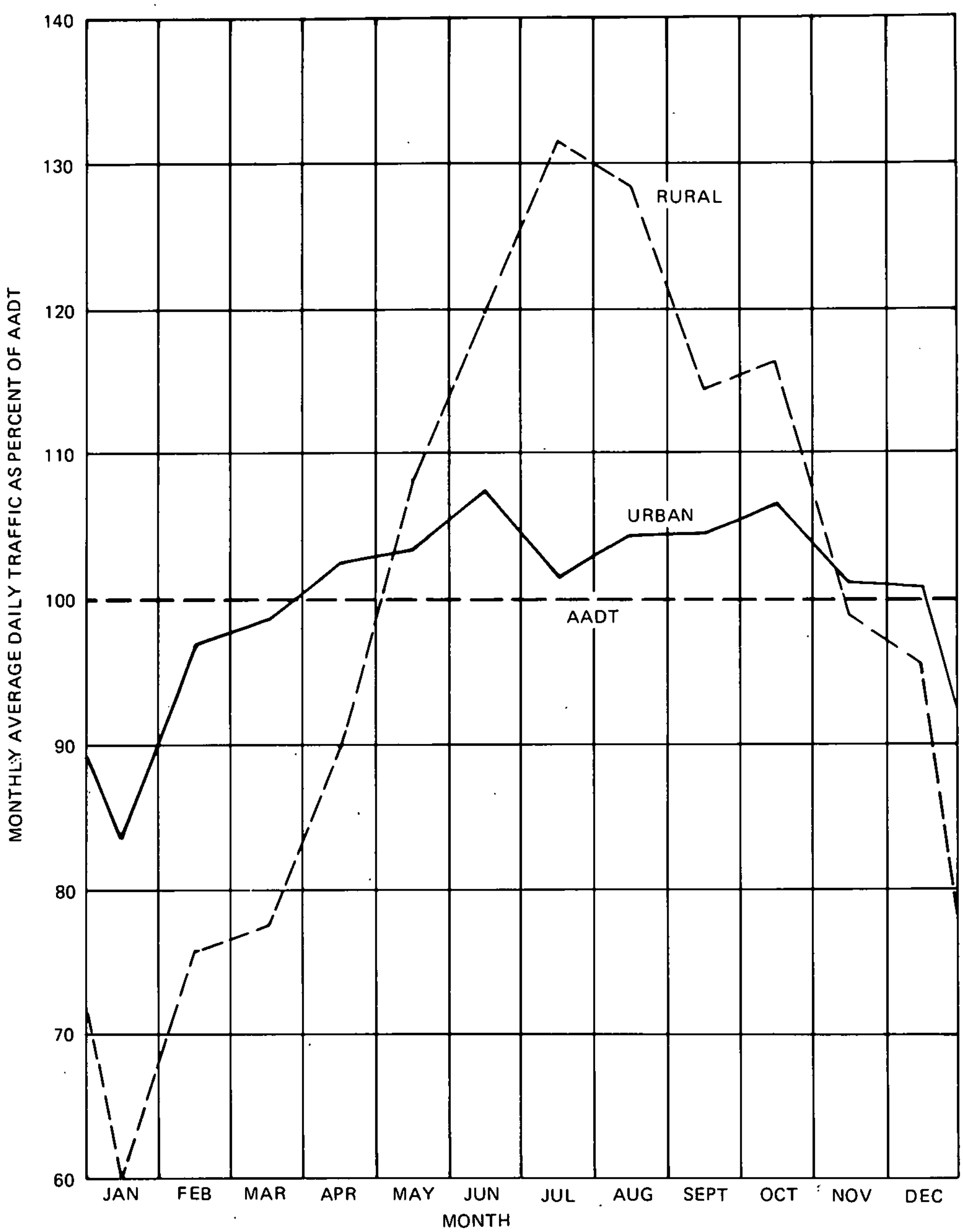


Figure 2-1(b). Examples of monthly traffic volume variations showing monthly variations in traffic for a freeway in Minnesota. (Source: Minnesota Department of Transportation, 1980–1982)

ness district of a large metropolitan area. The other segment is within 50 mi of the first, but serves a combination of recreational and intercity business travel. The wide difference in seasonal variation patterns for the two segments underscores the effect of trip purpose, and it may also reflect capacity restrictions on the urban section.

Daily Variations

Volume variations by day of the week are also related to the type of highway on which observations are made. Figure 2-2 shows that weekend volumes are lower than weekday volumes for highways serving predominantly business travel, such as urban freeways. In comparison, peak traffic occurs on weekends on main rural and recreational access facilities. Further, the magnitude of daily variation is highest for recreational access routes and least for urban commuter routes.

Figure 2-3 shows the variation in traffic by vehicle type for the shoulder lane of an urban freeway. From this figure, it is obvious that truck traffic is the most severely reduced on weekends.

The extent of daily volume variation decreases as volume increases, often reflecting the effect of capacity restrictions on demand.

Although the values shown in Figures 2-2 and 2-3 are illustrative of typical patterns that may be observed, they are not meant to be a substitute for local studies and analysis. The

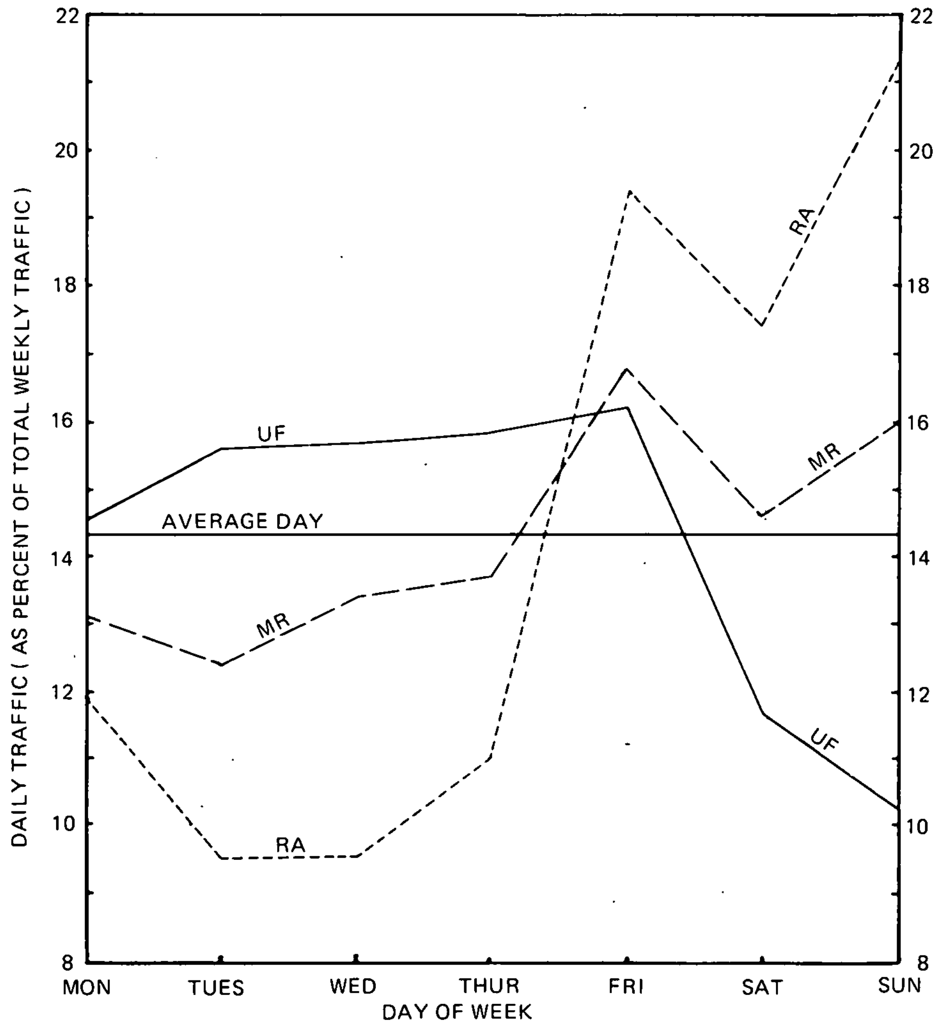


Figure 2-2. Examples of daily traffic variation by type of route. Legend: MR *curve represents main rural route I-35, Southern Minnesota, AADT 10,823, 4-lanes, 1980;* RA *curve represents recreational access route MN 169, North-Central Lake Region, AADT 3,863, 2-lanes, 1981;* UF *curve represents urban freeway, four freeways in Minneapolis-St. Paul, AADT's 75,000–130,000, 6–8 lanes, 1982.* (Source: Minnesota Department of Transportation, 1980–1982)

average daily traffic, averaged over a full year, and referred to as the *annual average daily traffic,* or AADT, is a value often used in forecasting and planning.

Hourly Variations

Typical hourly variation patterns, as related to highway type and day of the week, are shown in Figure 2-4. The typical morning and evening peak hours are evident for urban commuter routes on weekdays. The evening peak is generally somewhat more intense than the morning peak. On weekends, urban routes show a peak that is less intense and more "spread out," occurring in the early to midafternoon period.

Recreational routes also have single daily peaks. Saturday peaks on such routes tend to occur in the late morning or early afternoon (as travelers go *to* their recreational destination) and in late afternoon or early evening on Sundays (as they *return* home).

On intercity routes serving a mix of traffic, late afternoon peaks are evident, and there is less difference between the variation patterns for weekdays and weekends.

The repeatability of hourly variations is of great importance. The stability of peak-hour demands affects the viability of using such values in design and operational analysis of highways and other transportation facilities. Figure 2-5 shows data obtained over a 77-day period in metropolitan Toronto. The shaded area indicates the range within which one can expect 95 percent of the observations to fall. Although the variations by hour of the day are typical for urban areas, the relatively narrow and parallel fluctuations among the 77 days indicate the repeatability of the basic pattern. The observations depicted were obtained from detectors measuring one-way traffic only, as evidenced by the single peak hour shown for either morning or afternoon.

It is again noted that the data of Figures 2-4 and 2-5 are typical of observations that can be made. The patterns illustrated, however, do vary in response to local travel habits and environments, and these examples should not be used as a substitute for locally obtained data.

The Peak Hour

Capacity and other traffic analyses focus on the peak hour of traffic volume, because it represents the most critical period for operations and has the highest capacity requirements. The peak-hour volume, however, is not a constant value from day-to-day or from season-to-season.

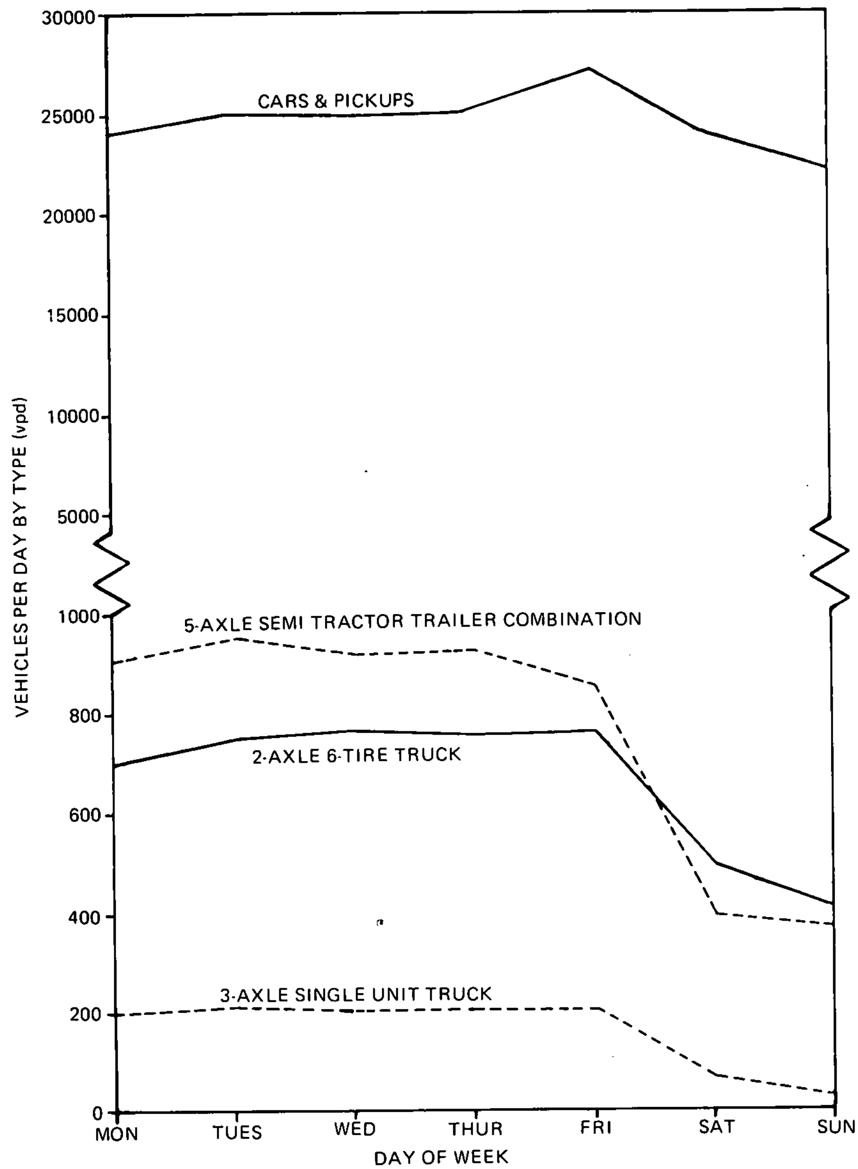


Figure 2-3. Daily variation in traffic by vehicle type. Data for this figure were collected on I-494, 4-lanes, in Minneapolis-St. Paul. (Source: Minnesota Department of Transportation, 1981–1982)

If the highest hourly volumes for a given location were listed in descending order, a large variation in the data would be observed, depending on the type of route and facility under study.

Rural and recreational routes often show a wide variation in peak-hour volumes, with several extremely high volumes occurring on a few selected weekends or other peak periods, and with traffic during the rest of the year at much lower volumes, even during the peak hour. This occurs because the traffic stream consists of few daily or frequent users, with the major component of traffic generated by seasonal recreational activities and special events.

Urban routes, on the other hand, show very little variation in peak-hour traffic. The majority of users are daily commuters or frequent users, with occasional and special event traffic at a minimum. Further, many urban routes are filled to capacity during each peak hour, and variation is therefore severely constrained. In many urban areas, both the AM and PM peak periods extend for more than one hour.

Figure 2-6 shows hourly volume relationships measured on a variety of highway types of Minnesota. Recreational facilities show the widest variation in peak-hour traffic, with values ranging from 30 percent of the AADT occurring in the highest hour of the year, to about 15.3 percent of AADT occurring in the 200th highest hour of the year and 8.3 percent in the 1,000th highest hour of the year. Main rural facilities also display a wide variation, with the highest hour subjected to 17.9 percent of the AADT, decreasing to 10 percent of the AADT in the 100th hour and 6.9 percent of the AADT in the 1,000th hour. Urban radial and circumferential facilities show far less variation, with the range in percent of AADT covering a narrow band, from approximately 11.5 percent for the highest hour to 7–8 percent for the 1,000th highest hour. It should be noted that Figure 2-6 includes all hours, not just peak hours of each day.

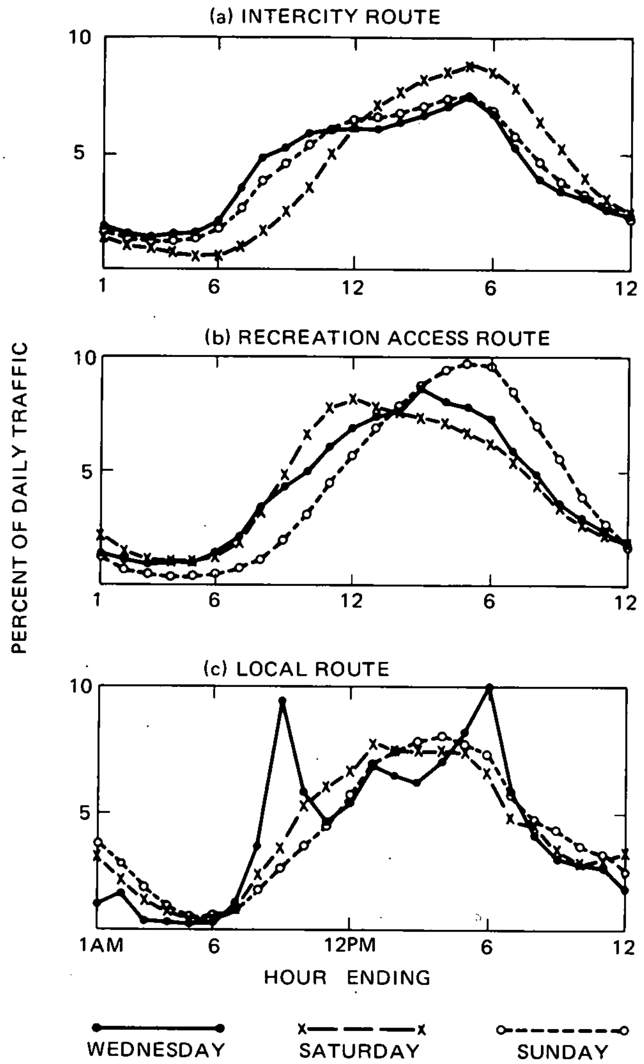


Figure 2-4. Examples of hourly traffic variations for rural routes in New York State. (Source: Ref. *7*)

It is apparent from these characteristics that traffic engineers are faced with the need for substantial judgments. Provision of a recreational facility adequate to handle the highest peak-hour volume of the year results in gross underutilization of capacity during all but a few hours of the year. On the other hand, providing sufficient capacity for the 30th, 100th, 500th, or other hour would guarantee the occurrence of substantial congestion and delay during those special event or recreational peak hours occurring less frequently.

The selection of an appropriate hour for design purposes is a compromise between providing an adequate level of service for every (or almost every) hour of the year and economic efficiency. Customary practice in the United States is to base design on an hour between the 10th and 50th highest hour of the year. This range generally encompasses the "knee" of the curve—the area in which the slope of the curve changes from sharp to flat. For rural highways, the knee has often been assumed to occur at the 30th highest hour, which is often used as the basis for estimates of design-hour volume. For urban highways, a design hour in the range of the 10th to 20th highest hour is common.

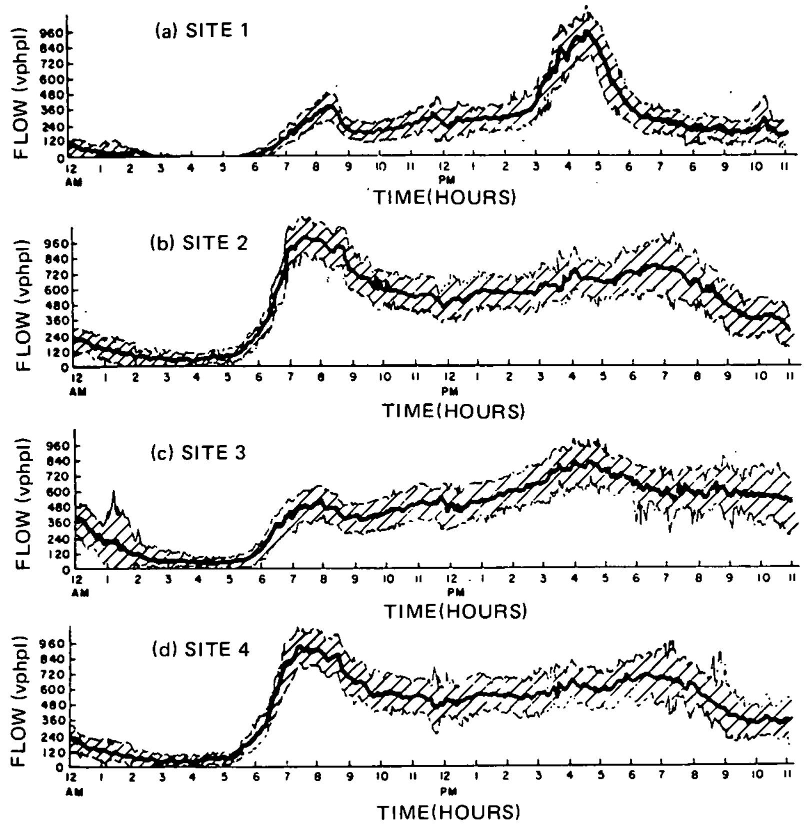
Recent studies (*9, 10*) have emphasized the difficulty in locating a distinct "knee" on hourly volume curves. Figure 2-7 shows hourly volumes for all hours of the year at a Kentucky counting station. The first and third curves illustrate the continuous nature of the relationship, with no distinct breaks or "knee" in the decreasing hourly volume pattern. The second curve shows a rather spreadout "knee" which could easily be located anywhere within the first 100 hours. These curves illustrate the point that arbitrary selection of a design hour between the 10th and 50th highest hours is not a rigid criterion, and points out the need for local data on which to make informed judgments.

The selection of a design hour must consider the impact of the selection on those higher volume hours that are not accommodated. The recreational access route curve of Figure 2-6 shows that the highest hours of the year have more than twice the volume of the 100th hour, while highest hours of an urban radial route are only about 15 percent higher than the volume in the 100th hour. Use of a design criterion set at the 100th hour would create substantial congestion on a recreational access route during the highest volume hours, but would have less of an impact on an urban facility, where the variation in peak-hour volumes is less. Another consideration is the level-of-service objective. A route designed to operate at LOS B can absorb larger amounts of additional traffic than a route designed to operate at LOS D during those extreme hours of the year with higher volumes than the design hour.

The proportion of AADT occurring in the design hour is often referred to as K. It is expressed as a decimal, and varies based on the hour selected for design and the characteristics of the subject route and its development environment. Where the K-factor is based on the 30th highest hour of the year, several general characteristics can be noted:

1. The K-factor generally decreases as the AADT on a highway increases.
2. High K-factors decrease faster than for lower values.
3. The K-factor decreases as development density increases.
4. The highest K-factors generally occur on recreational facilities, followed by rural, suburban, and urban facilities in descending order.

The various chapters of this manual address specific values of the K-factor typical to each facility type.



NOTE: (1) SITES 2 and 4 ARE ONE BLOCK APART ON SAME STREET, IN SAME DIRECTION
(2) ALL SITES ARE TWO MOVING LANES

Figure 2-5. Repeatability of hourly traffic variations for four 2-lane arterials in Toronto, Ontario, Canada. (Source: Ref. *8*)

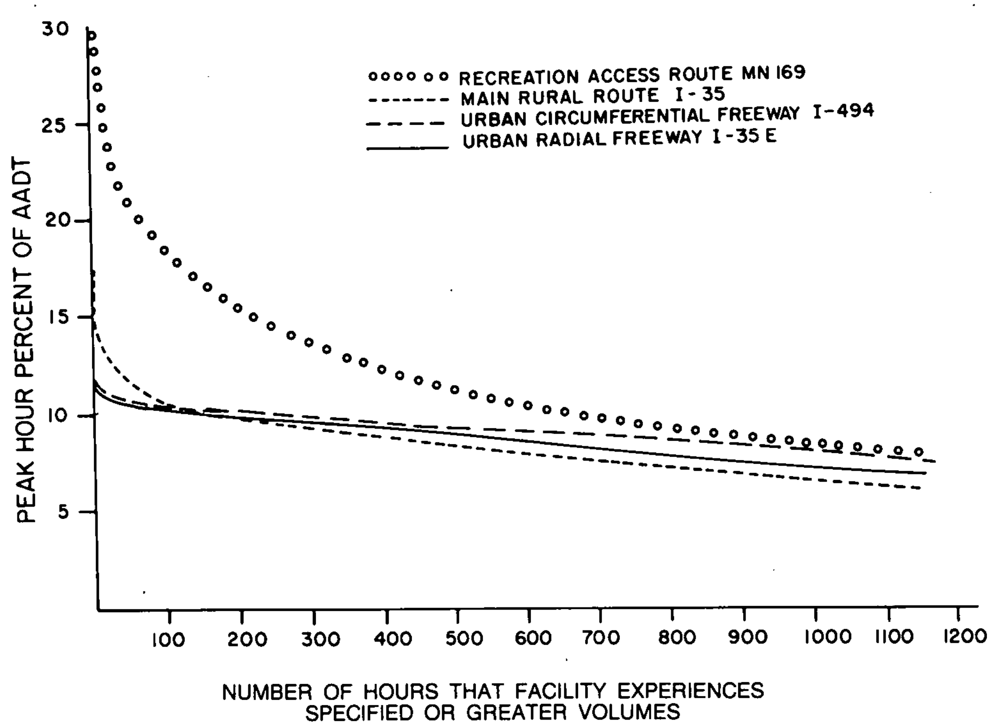


Figure 2-6. Ranked hourly volumes on Minnesota highways. (Source: Minnesota Department of Transportation, 1980–1982)

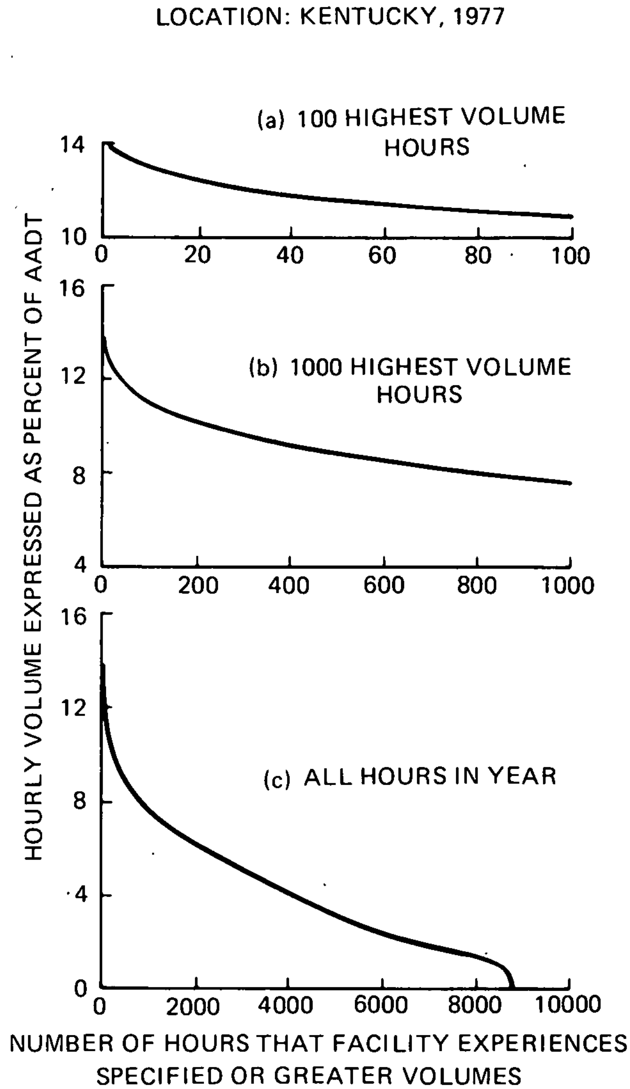


Figure 2-7. Ranked hourly volume distribution showing indistinct knee for Kentucky location in 1977. (Source: Ref. *9*)

Subhourly Variations in Flow

While volume forecasts for long-range planning studies are frequently expressed in units of AADT (vehicles per day), subsequently reduced to hourly volumes, the analysis of level of service is based on peak rates of flow occurring *within* the peak hour. Most of the procedures in this manual are based on peak 15-min rates of flow. Figure 2-8 illustrates the substantial short-term fluctuation in flow rate that can occur during an hour.

It can be seen from Figure 2-8 that the maximum 5-min rate of flow is 2,232 vph, and the maximum rate of flow for a 15-min period is 1,980 vph. The full-hour volume is only 1,622 vph. A design for a peak 5-min flow rate would result in substantial excess capacity during the rest of the peak hour, while a design for the peak-hour volume would result in congestion for a substantial portion of the hour. Note that Figure 2-8 treats discrete 15-min periods for clarity. In practice, the peak 15 min may occur during any 15-min interval within the hour.

Consideration of these peaks is important, because congestion due to inadequate capacity occurring for only a few minutes could take substantial time to dissipate because of the dynamics of breakdown flow, which are explained in greater detail in Chapter 6. Fifteen-minute flow rates have been selected as the basis for most procedures of this manual to incorporate these peak flows. Five-minute flow rates have been avoided, inasmuch as research has shown them to be statistically unstable. The operational effects of a 5-min flow surge are virtually impossible to predict with any certainty.

The relationship between the peak 15-min flow rate and the full hourly volume in the hour is given by the peak-hour factor, defined in Chapter 1 as:

$$\text{PHF} = \frac{\text{Hourly volume}}{4 \times (\text{Peak 15-min vol})} \tag{2-1}$$

Peak-hour factors in urban areas generally range between 0.80 and 0.98, with lower values signifying greater variability of flow within the subject hour, and higher values signifying little flow variation. Peak-hour factors over 0.95 are often indicative of capacity constraints on flow during the peak hour.

SPATIAL DISTRIBUTIONS

While traffic volume varies in time, it also varies in space. The two critical spatial characteristics of interest in capacity analysis are directional distribution and lane distribution. Volume may also vary longitudinally along various segments of a facility, but this does not explicitly impact capacity analysis computations. Each facility segment serving different traffic demands must be analyzed separately.

Directional Distribution

During any particular hour, traffic volume may be greater in one direction than in the other. An urban radial route, serving strong directional demands into the city in the morning and out of it at night, may display as much as a 2:1 imbalance in directional flows. Recreational and rural routes may also be subject to significant directional imbalances, which must be considered in the design process. Table 2-5 illustrates the directional distribution on various highway types in Minnesota between 1980 and 1982.

Directional distribution is an important factor in highway capacity analysis. This is particularly true for two-lane rural highways. Capacity and level of service vary substantially based on directional distribution because of the interactive nature of directional flows on such facilities. Procedures for two-lane highways include explicit consideration of directional distribution.

Although there is no explicit consideration of directional distribution in the analysis of multilane facilities, the distribution has a dramatic impact on both design and level of service. As indicated in Table 2-5, urban radial routes have been observed to have up to two-thirds of their peak-hour traffic in a single direction. Unfortunately, this peak occurs in one direction during the morning and in the other in the evening. Thus, both directions of the facility must be adequate for the peak directional flow. This characteristic has led to the use of reversible lanes on some urban freeways and arterials.

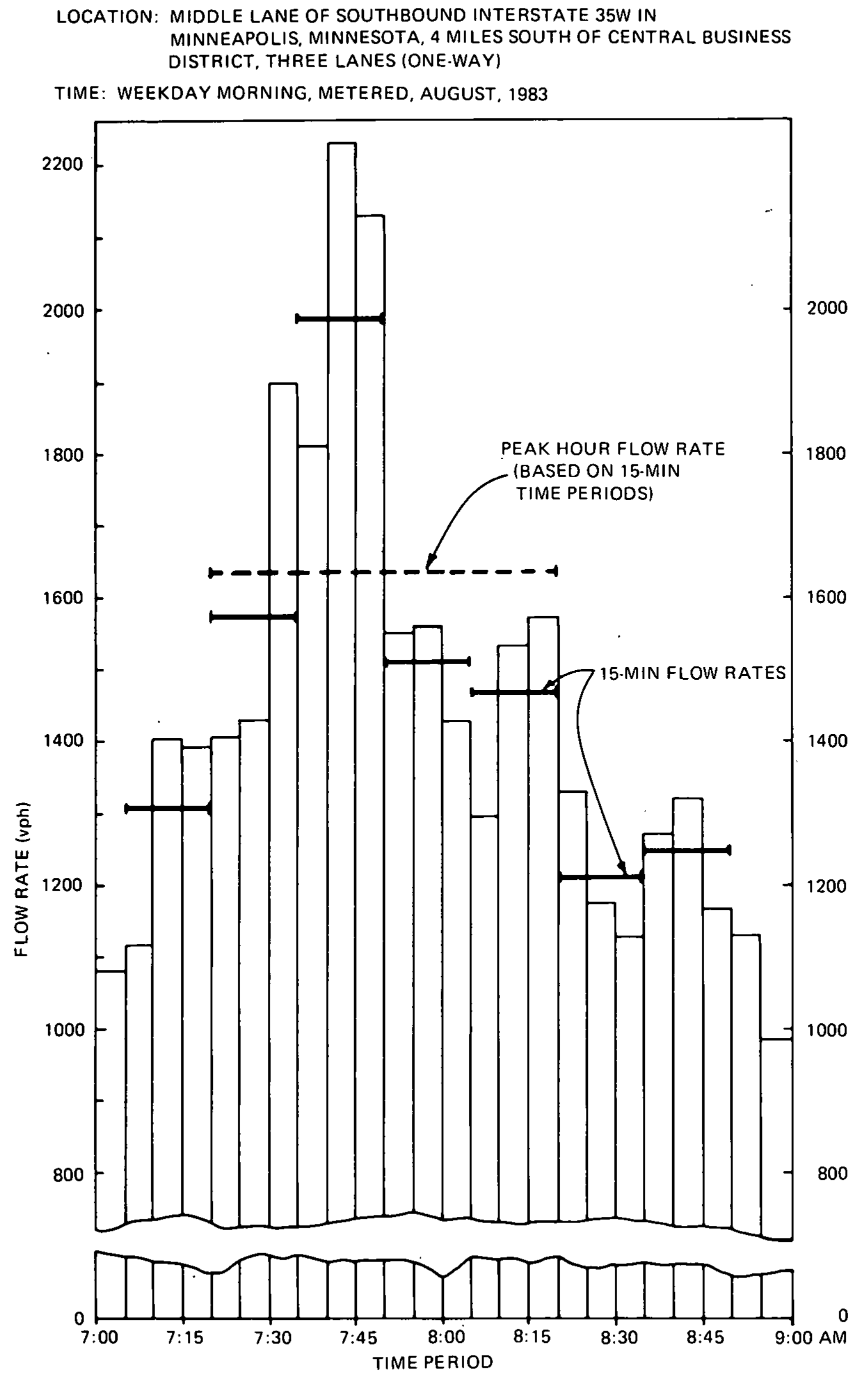


Figure 2-8. Relationship between short-term and hourly flows. (Source: Minnesota Department of Transportation, 1983)

TABLE 2-5. DIRECTIONAL DISTRIBUTION CHARACTERISTICS

HIGHEST HOUR OF THE YEAR	PERCENT TRAFFIC IN PEAK DIRECTIONS		
	TYPE OF FACILITY		
	URBAN CIRC	URBAN RADIAL	RURAL
1st	53	66	57
10th	53	66	53
50th	53	65	55
100th	50	65	52

SOURCE: Minnesota Department of Transportation, 1980–1982

It should also be noted that directional distribution is not a static characteristic in time. It changes by hours of the day, day of the week, season, and from year-to-year. Development in the vicinity of highway facilities often induces traffic growth that changes the existing directional distribution.

The proportion of traffic occurring in the peak direction of travel during peak hours is often denoted as D. The K-factor, the proportion of AADT occurring in the peak hour, was discussed previously. These factors are used to estimate the peak-hour traffic volume in the peak direction using the equation:

$$DDHV = AADT \times K \times D \tag{2-2}$$

Where:

DDHV = the directional design hour volume, in vph;
AADT = the average annual daily traffic, in vpd;
K = proportion of AADT occurring in the peak direction; and
D = proportion of peak-hour traffic in the peak direction.

The product of the factors K and D is tabulated for a number of facilities in Table 2-6. The product gives the proportion of AADT occurring in the maximum direction of the peak hour.

Lane Distribution

When two or more lanes are available for traffic in a single direction, the distribution in lane use will vary widely. The lane

TABLE 2-6. PEAK DIRECTIONAL VOLUMES AS A PERCENT OF ADT ($K \times D \times 100$) ON FREEWAYS AND EXPRESSWAYS

CITY AND 1970 URBANIZED AREA POPULATION	FACILITY	NUMBER OF LANES	YEAR	AVERAGE DAILY TRAFFIC	PEAK DIRECTIONAL VOLUMES	
					VEHICLES	PERCENT OF ADT
Atlanta, Ga. 1,172,778	I-20 E. of CBD at Moreland Ae.	6	1975	105,100	5,980	5.7
	I-75 S. of CBD at University Ave.	6	1975	110,800	6,200	5.6
	I-20 W. of CBD at Mozley Drive	6	1975	78,600	4,450	5.7
	I-75 N. of CBD (N. of I-85)	6	1975	72,800	4,500	6.2
	I-85 N. of I-75 at Monroe Drive	6	1975	90,100	5,500	6.1
Boston, Mass. 2,652,575	I-93 at Stoneham Town Line	6–8	1975	80,300	6,270	7.8
	S.E. Expwy at Southampton	6–8	1975	129,000	7,060	5.4
	Rt. 128 at Burlington Town Line	8	1975	86,400	5,660	6.6
Chicago, Ill. 6,714,578	Lake Shore Drive at 49th Street	6–8	1975	61,100	4,120	6.8
	Lake Shore Drive at Aldine	8	1975	117,000	9,380	8.0
Denver, Col. 1,047,311	I-25 between 38th Ave. and I-70	6	1974	145,000	7,500	5.2
	I-225 between I-25 and Washington St.	6	1974	105,000	5,400	5.1
Detroit, Mich. 3,970,584	Ford Fwy. (I-94) at Chrysler Fwy.	6	1975	161,500	5,570	3.4
	Jeffers Fwy. (I-96) at Warren		1974	72,100	4,850	6.7
	Southfield Fwy (M39) at Plymouth	6	1973	142,100	6,210	4.4
	Lodge (M10) at Pallister	6	1972	173,000	5,310	3.1
	Fisher Fwy. at Lodge	6–8	1972	118,100	5,310	4.5
Houston, Tex. 1,677,863	I-45—Gulf at Woodbridge	6	1976	106,600	4,910	4.6
	US 59—S.W. at Montrose	10	1976	145,900	8,470	5.8
	US 59 S.W. at Rice Ave.	8	1976	162,700	6,730	4.1
Houston, Tex.	I-45—North side of North Loop	8	1976	121,900	7,420	6.0
	I-10—East W. of Waco St.	8	1976	117,600	7,090	6.0
	I-610 West at Buffalo Bayou	8–10	1976	174,400	9,520	5.4
	I-10 North E. at North Main	8	1976	125,300	6,640	5.3
	I-610—Katy E. of Taylor St.	10	1976	109,500	7,600	6.9
	I-610—South West of Main	8	1976	100,300	6,700	6.7
Milwaukee, Wis. 1,252,457	N-S Fwy. at Wisconsin		1975	90,310	5,260	5.8
	N-S Fwy. at Greenfield		1975	96,770	5,780	6.0
	E-W Fwy. at 26th St.		1975	93,280	5,000	5.4
	Airport Fwy. at 68th		1975	62,300	3,520	5.7
New York City, N.Y. 16,206,841	Long Island Expwy.	6	1973	165,000	5,300	3.2
	FDR Drive	6	1974	117,000	4,400	3.8
	Holland Tunnel	4	1974	61,400	2,400	3.9
	Lincoln Tunnel	6	1974	97,300	4,900	5.0
	Brooklyn-Battery Tunnel	4	1974	46,700	3,400	7.3
San Francisco, Calif. 2,987,850	Oakland-Bay Bridge (I-80)	10	1973	184,000	8,120	4.4
	Southern Fwy. (I-280)	8	1969–73	114,000	6,150	5.4
	Golden Gate Bridge (U.S. 101)	6	1969–73	92,000	5,720	6.2
Washington, D.C.-Md.-Va. 2,481,489	Shirley Hwy. (N. of 4 Mile River)	8	1975	136,000	8,010	5.9
	Center Leg Fwy.	8	1975	68,000	3,410	5.0
	I-95 Bridge (over Potomac)	8	1975	142,700	6,260	4.4
	Balt.-Wash. Pkwy. (District Line)	6	1975	101,300	4,930	4.9
	Woodrow Wilson Bridge	6	1975	97,800	4,620	4.7

SOURCE: Ref. *30*

distribution will depend on traffic regulations, traffic composition, speed and volume, number and location of access points, origin-destination patterns of drivers, development environment, and local driver habits.

Because of these factors, there are no "typical" lane distributions. The procedures of this manual assume an *average* capacity of multilane uninterrupted flow facilities of 2,000 pcphpl, recognizing that flow in some individual lanes will be higher and in others lower. Recent data collected as part of the Highway Capacity and Quality of Service Committee survey of high-volume facilities indicate no consistency in lane distribution. For example, the peak lane on a six-lane freeway may be the shoulder, middle, or median lane, depending on local conditions. Table 2-7 gives lane distribution data for various vehicle types on selected freeways. However, these data are illustrative, and are not intended to represent "typical" values.

It is of interest to note that the trend indicated in Table 2-7 is reasonably consistent throughout North America. Heavier vehicles tend towards the right-hand lanes, partially because they tend to operate at lower speeds than other vehicles, and partially because of regulations prohibiting them from using leftmost lanes.

Lane distribution is a critical factor in the analysis of freeway ramp junctions, inasmuch as the traffic in the shoulder lane forms the merge or diverge volume in conjunction with the ramp vehicles. Procedures for their analysis in Chapter 5 focus on estimating traffic in the shoulder lane, as well as on truck presence in the lane.

TRAFFIC COMPOSITION

The fraction of trucks, RV's, and buses in the traffic stream is also required to apply the procedures of this manual. Adjustments for these three categories of vehicles, especially as they relate to grade-climbing capabilities, are given for each of the procedures in following chapters.

In response to fuel shortages and subsequent federal law, the mix of automobiles on U.S. highways is changing. Lighter weight vehicles with smaller engines dominate the new-car market. Figures 2-9 and 2-10 show trends in passenger car power characteristics since 1967, with projections to 1995. Although the trend is clearly towards less powerful vehicles (as indicated by the ratio of horsepower-to-weight in Figure 2-9), the 1995 average vehicle will have about 85 percent of the hp/lb of a 1978 average vehicle. The impact of these changes on capacity and operations is expected to be minimal.

Figures 2-11 and 2-12 show the distribution of truck weight-to-horsepower ratios on multilane and two-lane rural highways. These figures compare the results of two studies, and were prepared as part of the effort to develop the two-lane rural highway methodology of Chapter 8. Median weight-to-horsepower ratios of 150 to 175 lb/hp prevail on four-lane rural highways, while two-lane highways have median ratios in the 110 to 130 lb/hp range.

The change in traffic composition on rural trunk highways between 1960 and 1980 in one midwestern state is shown in Table 2-8. The percentage of trucks and buses has increased from 12.1 percent to 15.7 percent in this 20-year period. Recreational vehicles were not separately observed in this study, but were categorized by the number of axles with trucks and buses. Between 1969 and 1978, the percentage of buses and trucks on main rural roads increased even more dramatically from 11.7 percent to 18.0 percent. Buses and motorcycles represented only 0.6 percent and 0.9 percent of total traffic in the two study years.

These characteristics emphasize a growing problem for highway designers. Cars are becoming smaller, lighter, and less powerful, while trucks are becoming larger and more powerful. Further, there is a growing proportion of the traffic stream made up of trucks. The growing disparity among vehicle types presents a number of safety and design issues that, although beyond the scope of this manual, have an impact on highway systems.

IMPACT OF WEATHER ON MAXIMUM VOLUMES

There have been relatively few efforts to quantify the effects of adverse weather on capacity. Some measure of the impact can be gained from studies conducted on two freeways with automated data collection systems—the Gulf Freeway (I-45) in Houston (*12*) and I-35W in Minneapolis (*13*). For both freeways, observations were made on three-lane segments influenced by bottlenecks such that a history of "capacity volumes" was available. For the Gulf Freeway, it was reported that rain significantly reduces capacity by from 14 percent to 19 percent as compared to clear-weather values (with a 95 percent statistical confidence).

Results from the I-35W study suggest that even a "trace" amount of precipitation reduces capacity by 8 percent. Each 0.01 in. per hour increase in rainfall results in a further decrease

TABLE 2-7. LANE DISTRIBUTION BY VEHICLE TYPE

		PERCENT DISTRIBUTION BY LANE		
HIGHWAY	VEHICLE TYPE	LANE 1[b]	LANE 2	LANE 3
Lodge Freeway Detroit	Light[a]	29.2	38.4	32.4
	SU Trucks	30.8	61.5	7.7
	Combinations	88.5	2.9	8.6
	ALL VEH	30.9	37.8	31.3
Connecticut Turnpike	Light[a]	34.6	40.9	24.5
	ALL VEH	37.1	40.4	22.5

[a] Passenger cars, panel trucks, and pickup trucks.
[b] Lane 1 = shoulder lane, lanes numbered from shoulder to median.
SOURCE: Ref. *16*

of 0.6 percent in capacity. When precipitation falls as snow, the impact is even greater: an additional 2.8 percent decrease in capacity for each 0.01 in./hr of snow (water equivalent) beyond the initial "trace" decrease of 8 percent.

The procedures of this manual do not specifically account for inclement weather conditions. However, in areas where such conditions are prevalent, analysts may wish to modify results to account for these impacts.

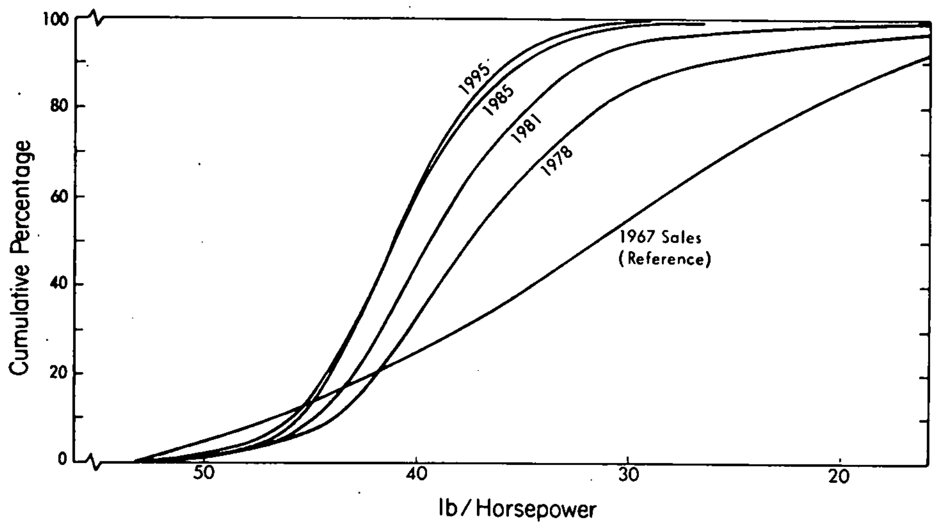


Figure 2-9. Distribution of power-to-mass ratios of passenger cars. (Source: Ref. *11*)

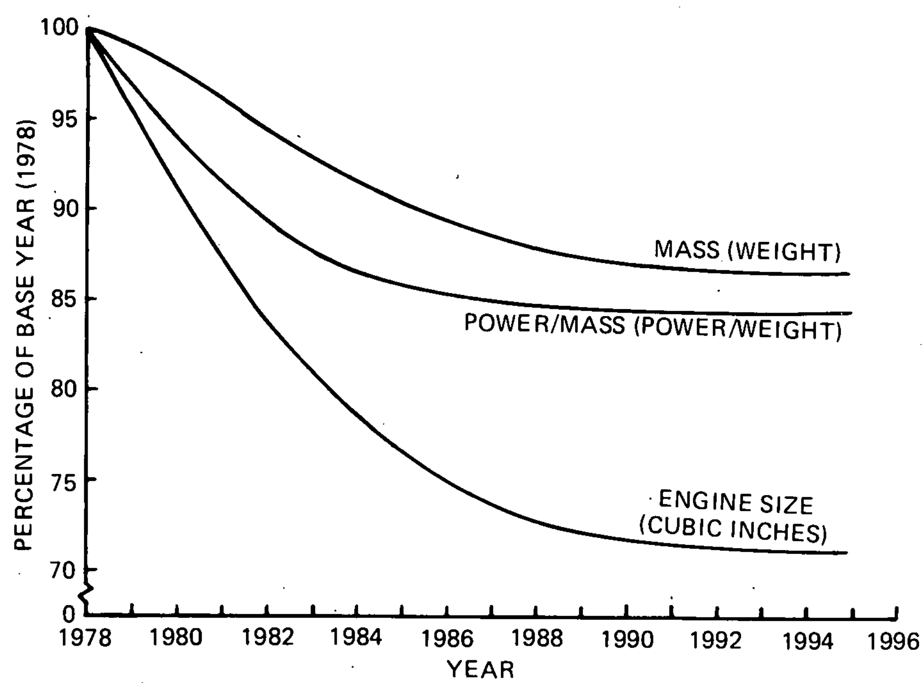


Figure 2-10. On-highway passenger car characteristics. (Source: Ref. *14, Fig. 2-13*)

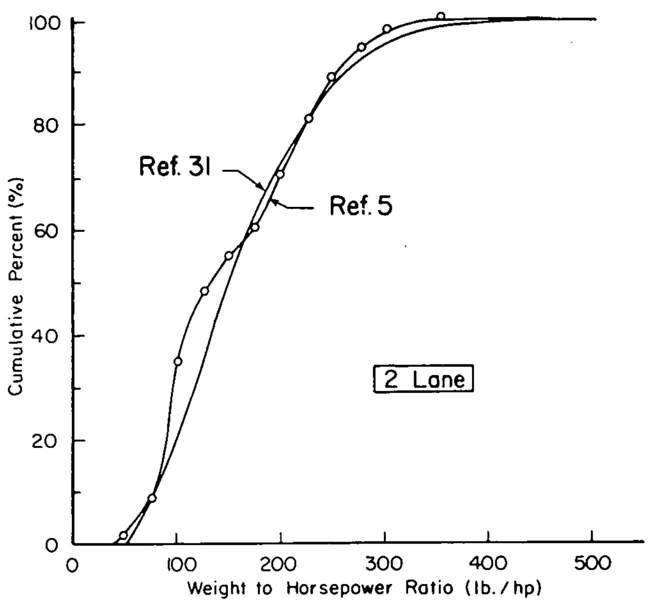


Figure 2-11. Distribution of truck weight-to-horsepower ratios on two-lane rural highways from two studies. (Source: Ref. *5, p. A-20*)

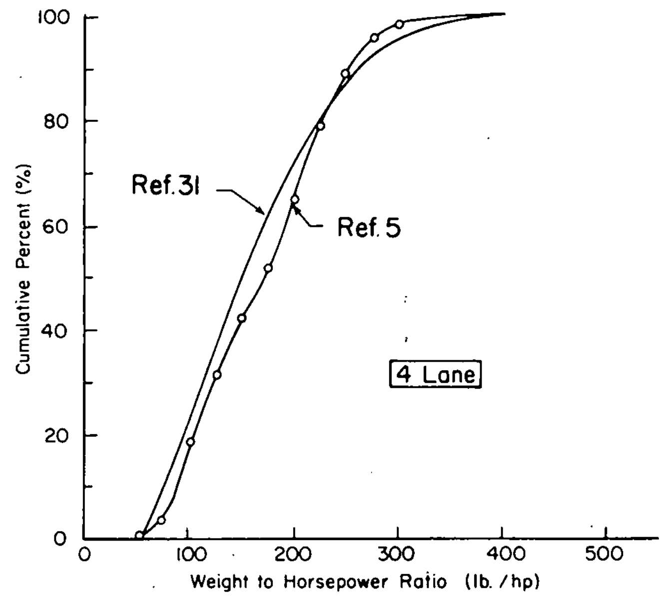


Figure 2-12. Distribution of truck weight-to-horsepower ratios on four-lane rural highways from two studies. (Source: Ref. *5, p. A-21*)

TABLE 2-8. TRAFFIC COMPOSITION ON RURAL HIGHWAYS IN 1960 AND 1980

VEHICLE TYPE	1960 (%)	1980 (%)
4-Tire Vehicles	87.9	84.3
2-Axle, Dual-Tire	4.1	2.4
3-Axle, Single-Unit	1.2	1.4
3-Axle, Combination	1.1	0.2
4-Axle, Combination	2.4	0.4
5-Axle, Combination	2.5	10.7
Buses, Miscellaneous	0.8	0.6
	100.0	100.0

SOURCE: Ref. *7*

IV. SPEED CHARACTERISTICS

Traffic volumes provide a method of quantifying capacity values. Speed (or its reciprocal—travel time) is an important measure of the quality of traffic service provided to the motorist. It is used as one of the more important measures of effectiveness defining levels of service for many types of facilities, such as rural two-lane highways, arterials, freeway weaving sections, and others.

When used as a measure of effectiveness, speed criteria must recognize driver expectations and roadway function. Thus, a driver expects a higher speed on a freeway than on an urban arterial. Lower speeds will be tolerated on a roadway with more severe horizontal and vertical alignment, because drivers will not be comfortable driving at extremely high speeds. Level-of-service criteria are predicated on these and other influencing factors. Table 2-9 summarizes some of the speed criteria for the levels of service discussed in subsequent chapters of this manual.

TABLE 2-9. LEVEL-OF-SERVICE C VS. SPEED CRITERIA ESTABLISHED IN THIS MANUAL

TYPE OF FACILITY	MINIMUM SPEED (MPH)
Basic Freeway (70-mph design speed)	54
Basic Freeway (60-mph design speed)	47
Basic Freeway (50-mph design speed)	43
Multilane Hwy. (70-mph design speed)	50
Multilane Hwy. (60-mph design speed)	44
Multilane Hwy. (50-mph design speed)	39
Arterials (30 to 35 mph free flow speed)	18

Level-of-service C is often associated with minimally desired operations. The range in speeds given in Table 2-9 reflects driver expectations and roadway function. The lower speed for arterials includes the impact of delays, while the speeds for multilane highways compared to freeways reflect increased side and median frictions that exist on such facilities.

TYPES OF SPEED MEASURES

There are several different speed parameters that can be applied to a traffic stream. These include:

1. *Average running speed*—This is also called "space mean speed" in the literature. It is a traffic stream measurement based on the observation of vehicle travel times traversing a section of highway of known length. It is defined as the length of the segment divided by the average running time of vehicles to traverse the segment. "Running time" includes only time which vehicles spend in motion, and does *not* include stopped delays.
2. *Average travel speed*—This is also a traffic stream measure based on travel time observations over a known length of highway. It is defined as the length of the segment divided by the average travel time of vehicles traversing the segment, including all stopped delay times. It is also a "space mean speed," as the use of average travel times effectively weights the average according to the length of time a vehicle occupies the defined roadway segment or "space."
3. *Time mean speed*—This is the arithemetic average of vehicle speeds observed passing a point on a highway, and it is also referred to as the "average spot speed." Individual speeds are recorded passing a point, and are arithmetically averaged.

Most of the procedures using speed as a measure of effectiveness in this manual use "average travel speed" as the defining parameter. For uninterrupted flow facilities not operating at LOS F, the average travel speed is equal to the average running speed.

Figure 2-13 shows a typical relationship between time mean and space mean speeds. Space mean speed is always slower than time mean speed, with the difference decreasing as the absolute value of speed decreases. This relationship is based on statistical analysis of observed data, and is useful because time mean speeds are often easier to measure in the field than space mean speeds.

NATIONAL SPEED TRENDS

Nationwide speed trends through 1975 are shown in Figure 2-14(a) for various vehicle types, and in Figure 2-14(b) speed trends are shown for all vehicles on Interstate rural highways (through 1981).

Figure 2-14(a), for main rural highways, points out an increasing speed trend from 1942 through the middle of 1972. This reflects the better design of both highways and vehicles throughout this period. In 1973, in response to a severe fuel shortage, the 55-mph national speed limit was introduced, and a sharp decline in speeds is observed. The figure also shows that buses and passenger cars travel at similar speeds on rural highways, while trucks travel at somewhat lower speeds. To 1973, the difference between average truck and passenger car speed is about 7 to 8 mph. After 1973, this difference was reduced considerably, to about 2 mph, because of the lower overall speeds being observed.

A similar trend can be observed in Figure 2-14(b), but the increase in speeds is more gradual since 1942, and the general speed level is higher on Interstate facilities than on other rural highways. In 1973, a sharp reduction in speeds is again observed followed by a more-or-less level trend.

The more recent data, however, given in Table 2-10, show that speeds have been gradually increasing on U.S. highways, despite the 55-mph national speed limit. It should be noted that all of the highways referenced in Table 2-10 had a 55-mph speed limit in effect. Thus, "urban arterials" are high-design urban multilane facilities with the 55-mph speed limit.

These speed trends have become an important safety issue as the fuel crises have waned. Many studies have related reductions in fatality and accident rates on U.S. highways to the 55-mph speed limit. As fuel problems decline, however, the observed trends clearly point to driver desires for higher speeds.

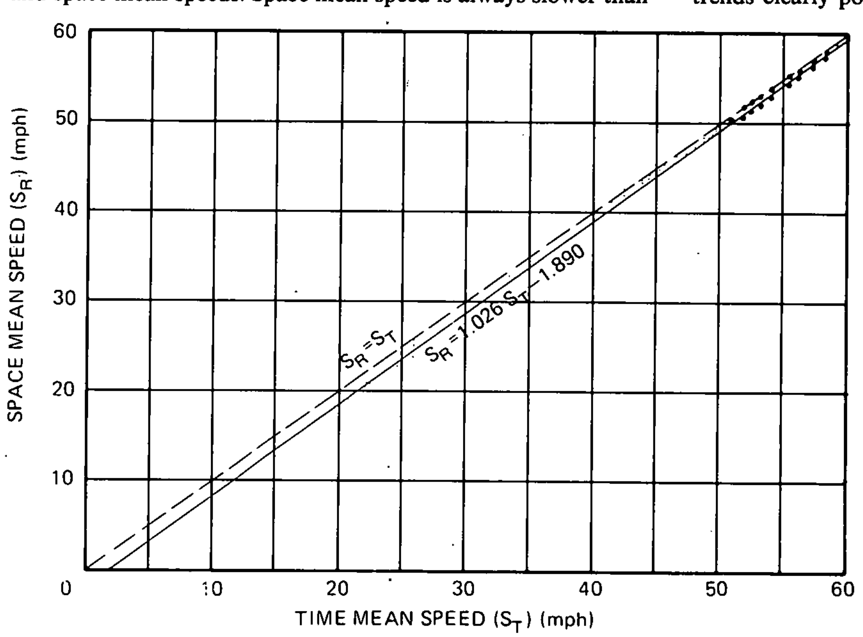


Figure 2-13. Typical relationship between time mean and space mean speed. (Source: Ref. *20*)

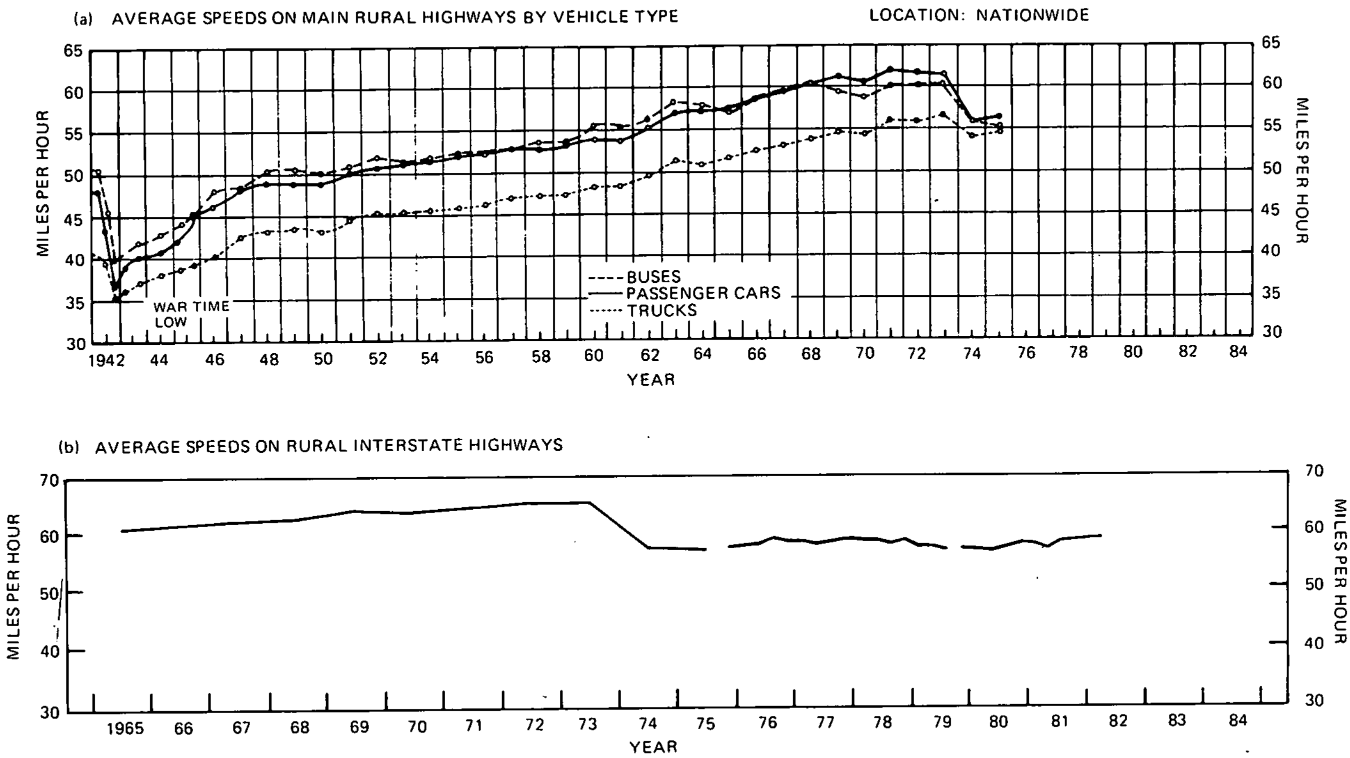


Figure 2-14. Nationwide speed trends through 1975 and 1981. (Source: Ref. *21*)

TABLE 2-10. RECENT NATIONAL SPEED TRENDS

FISCAL YEAR	AVERAGE SPEED (MPH)	85TH PERCENTILE SPEED (MPH)	PERCENT > 55 MPH
	INTERSTATE URBAN HIGHWAYS		
1980	55.4	60.1	51.2
1981	55.5	60.9	—
1982	56.3	62.7	58.4
1983	56.8	63.1	60.5
	INTERSTATE RURAL HIGHWAYS		
1980	57.5	62.1	65.9
1981	57.9	63.0	—
1982	59.0	65.1	73.1
1983	59.1	65.2	73.6
	RURAL ARTERIALS		
1981	54.1	59.9	—
1982	54.3	61.1	46.2
1983	54.6	61.3	47.2
	URBAN ARTERIALS		
1981	51.8	57.9	—
1982	51.5	58.6	32.8
1983	52.4	59.2	34.4

SOURCE: Federal Highway Administration, 1984

Aside from the general interest in the speed limit issue, these speed trends have an impact on the procedures presented in this manual. Uninterrupted flow procedures incorporate national average speed-flow and speed-density trends. The exact shape of these curves and the calibration of speeds (especially at the free-flow end of the relationships) reflect current trends. Curves used in this manual allow for average speeds up to 60 mph, 5 mph over the national speed limit in response to the observed increase in driver-selected speeds under free-flow conditions.

SPEED VARIATION BY TIME OF DAY

Figures 2-15 and 2-16 illustrate variations of speed with time of day, along with hourly volume variations, over a 24-hour period for I-35W in Minneapolis. Figure 2-15 shows a weekday variation pattern, and Figure 2-16 shows a similar distribution for a Saturday.

In these exhibits, note that speed remains relatively constant despite significant changes in volume. In Figure 2-15, speed shows a marked response to volume increases only when the volume exceeds approximately 1,600 vphpl. This trend is illustrated later, and is an important characteristic in all of the procedures of this manual. If speed does not vary with rate of flow over a broad range of flows, it becomes difficult to use speed as the sole measure of effectiveness defining level of service. This important characteristic is the major reason that such measures as density and percent time delay have been introduced

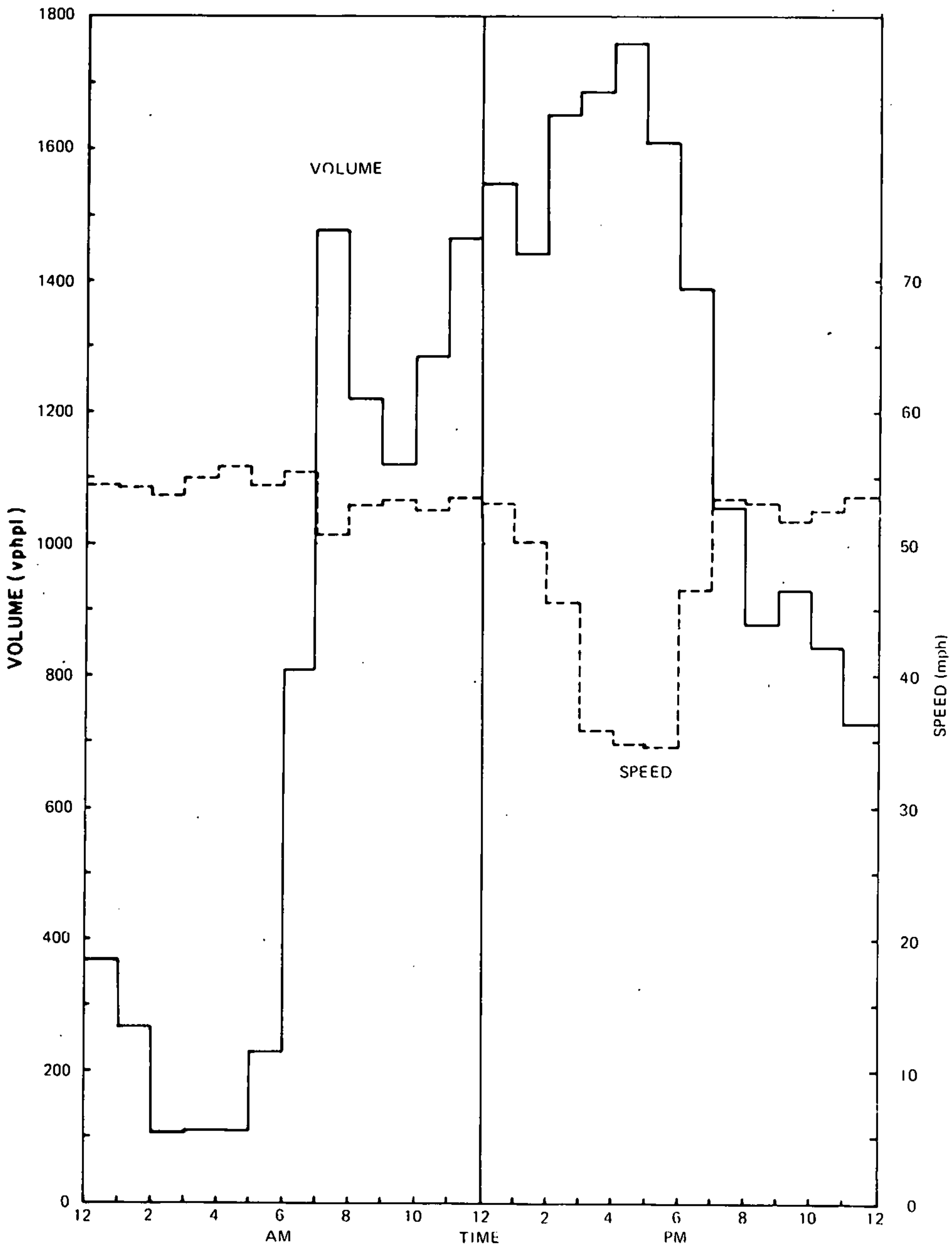


Figure 2-15. Speed variation by hour of day for I-35W in Minneapolis, week days. (Source: Minnesota Department of Transportation, 1983)

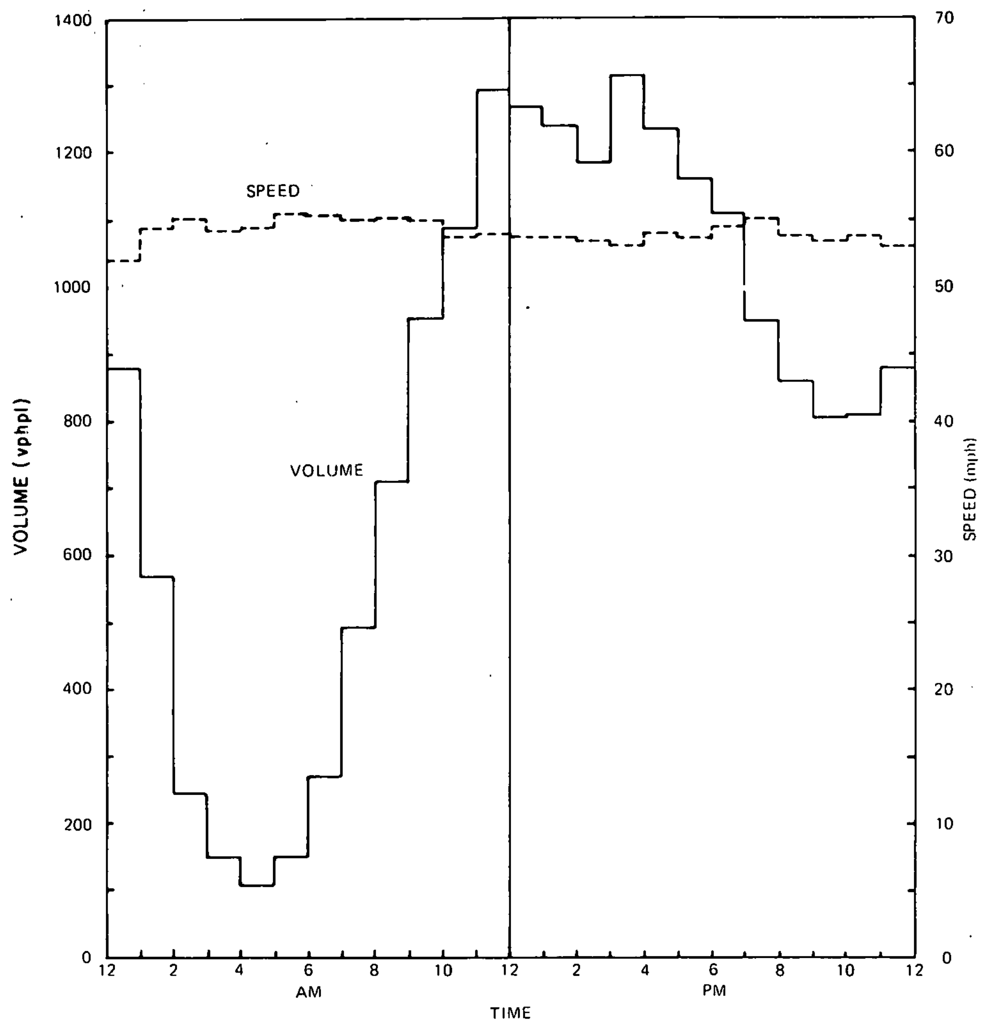


Figure 2-16. Speed variation by hour of day for I-35W in Minneapolis, Saturdays. (Source: Minnesota Department of Transportation, 1983)

as primary measures of effectiveness for uninterrupted flow facilities, with speed playing a secondary role.

The speeds in Figures 2-15 and 2-16 are also virtually the same, despite significantly lower volumes on weekends. This is a reflection of driver populations and trip purpose impacts. Saturday drivers may be less familiar with the facility, or if familiar, do not drive with the same sense of urgency devoted to the daily commute to work. Procedures of this manual also take this into account by introducing adjustments for driver population types in several of the chapters.

SPEED VARIATION BY LANE AND DAY VS. NIGHT

Table 2-11 shows a comparison of speeds by day vs. night conditions on the Connecticut Turnpike near Bridgeport. The table indicates that day/night variations are slight, in the order of 1 mph. Variations by lane are considerably greater, a factor also illustrated in Table 2-12 for a number of other facilities.

Level-of-service speed criteria in the manual refer to average values across all lanes of the facility, or of one direction of the facility. The data illustrate that drivers in general are using the lanes of multilane facilities as intended—slower drivers to the right, with faster drivers using middle and median lanes.

TABLE 2-11. AVERAGE SPEED BY DAY VS. NIGHT AND LANE IN MPH

	LANE 1[a]		LANE 2		LANE 3	
VEHICLE TYPE	DAY	NIGHT	DAY	NIGHT	DAY	NIGHT
Pass cars	49.5	48.8	57.7	57.4	64.1	61.6
Trucks	47.5	46.4	54.3	54.6	59.4	58.1
Percent trucks in lane	(15.0)	(17.3)	(7.5)	(13.0)	(0.7)	(5.4)

[a] Lane 1 = shoulder lane, lanes numbered from shoulder to median.
SOURCE: Ref. *16*

TABLE 2-12. AVERAGE SPEEDS BY LANE IN MPH

LOCATION	LANE 1[a]	LANE 2	LANE 3	LANE 4	AVG. FLOW RATE PER LANE
N.J. Tpke	46	55	60	—	1120
Conn. Tpke	49	57	64	—	692
L.I. Expwy. N.Y.	52	56	57	—	1460
I-8, San Diego	49	51	58	62	1503
	44	48	53	55	2386
SR 94, San Diego	50	53	57	56	1282
	47	49	52	49	2168

[a] Lane 1 = shoulder lane, lanes numbered from shoulder to median.
SOURCES: Refs. *16* and *17*, California Department of Transportation, 1984.

V. SPEED, FLOW, AND DENSITY RELATIONSHIPS FOR UNINTERRUPTED FLOW

Chapter 1 introduced the basic form of the relationships among speed, flow rate, and density for uninterrupted flow facilities. Rarely is it possible to observe these characteristics, especially at flow rates approaching capacity, under ideal conditions. Practically all data collected for the calibration of such relationships are subject to the influences of changing environmental conditions, nonhomogeneity of vehicles in the traffic stream, and (particularly for urban facilities) lack of complete isolation from ramps and interchanges.

The shape and calibration of such relationships are important, because they provide the basis for the selection of measures of effectiveness and the definition of level-of-service ranges for uninterrupted flow facilities. Such relationships also serve to estimate the capacity of uninterrupted flow facilities and the operating conditions under which the capacity occurs. The latter requires clear identification of the "peak" or maximum volume point on a speed-flow or speed-density curve, a process fraught with uncertainty due to the indistinct range of data generally observed in the vicinity of capacity on most facilities.

In recognition of such difficulties, many researchers have developed analytic "models" describing these relationships, from which it is possible to extrapolate the capacity of a highway or the service flow rates associated with the various levels of service. A brief discussion of these points provides the analyst with a starting point from which local calibrations may be applied to capacity and level-of-service analysis.

SPEED-DENSITY RELATIONSHIPS

As the number of cars occupying a roadway increases (density), there is an associated decrease in speed. Because of this phenomenon, the earliest investigations of stream flow were concerned with the speed-density relationship. Greenshields (*18*) postulated a linear relationship between speed and density in his 1934 study of capacity, a model which has the advantage of simplicity, and which provides a good fit to observed data in many cases.

The mathematical expression of Greenshield's model is:

$$S = S_f(1 - D/D_j) \qquad (2\text{-}3)$$

where

S = speed, in mph;
D = density, in vphpl;
S_f = free-flow speed, in mph; and
D_j = jam density, in vphpl.

The linear model is simple and useful. Other researchers, however, have used approaches based on physical phenomena to model traffic stream flow. Greenberg developed a model based on a "one-dimensional" fluid state, resulting in the following form:

$$S = S_c \times \ln(D_j/D) \qquad (2\text{-}4)$$

where S_c equals the critical speed (at capacity), in mph, and other variables are as previously defined.

The model is especially useful in describing congested flow, but it breaks down at low densities, as the theoretical speed at zero density approaches infinity.

References *19* and *20* contain discussions and compare calibrations of these and five other speed-density models, illustrating the variety of analytic approaches that may be taken in describing observed data mathematically.

DENSITY-FLOW RELATIONSHIPS

As flow rate, speed, and density are related by the formula $v = S \times D$, it follows that determination of a speed-density relationship also fixes the relationships between density and flow and speed and flow.

If there is zero density, there can be no flow. If the roadway is at jam density (where speed is zero), there is likewise no flow. Since space mean speed is equal to flow divided by density, v/D, the slope of a calibrated density-flow curve leaving the origin is the free-flow (zero-density) speed.

Since there are observable flows between zero density and jam density, there must be one or more points of maximum flow between these two points. Some researchers have fit continuous curves through density-flow data, yielding a single maximum flow rate. Others have projected discontinuous curves through data, with one curve treating stable flow points, and another unstable or forced flow points. In these cases two maxima are achieved, one for each curve. All such models indicate that the maximum flow rate for the stable flow curve is considerably higher than that for the unstable flow curve, perhaps as much as 200 vph higher. This is an interesting feature that projects a discontinuity in flow near capacity, the point at which flow breaks down. It also explains the difficulty in recovering from a breakdown, as the maximum flow that can be achieved from an unstable flow condition is less than that for stable flow.

Reference *20* contains several sample calibrations and illustrations of density-flow data.

SPEED-FLOW RELATIONSHIPS

Speed-flow relationships also follow directly from a curve fitted to either speed-density or density-flow. As speed and flow are the most readily measured traffic stream parameters, and since speed has historically been a major measure of effectiveness in level-of-service analysis, the speed-flow curve is the most often calibrated from field data.

Figure 2-17 shows the results of a study of flow on four-, six-, and eight-lane freeways. The data were collected on New York-area parkways under ideal conditions (no heavy vehicles, adequate geometrics). For both the four-lane and eight-lane parkways, there is little variation in speed up to flows of 1,500 pcphpl, while data for the six-lane parkway are too sparse at flows below this level to draw firm conclusions.

A study of flow on a six-lane freeway near Toronto, Canada, shows similar characteristics. As illustrated in Figure 2-18, speed

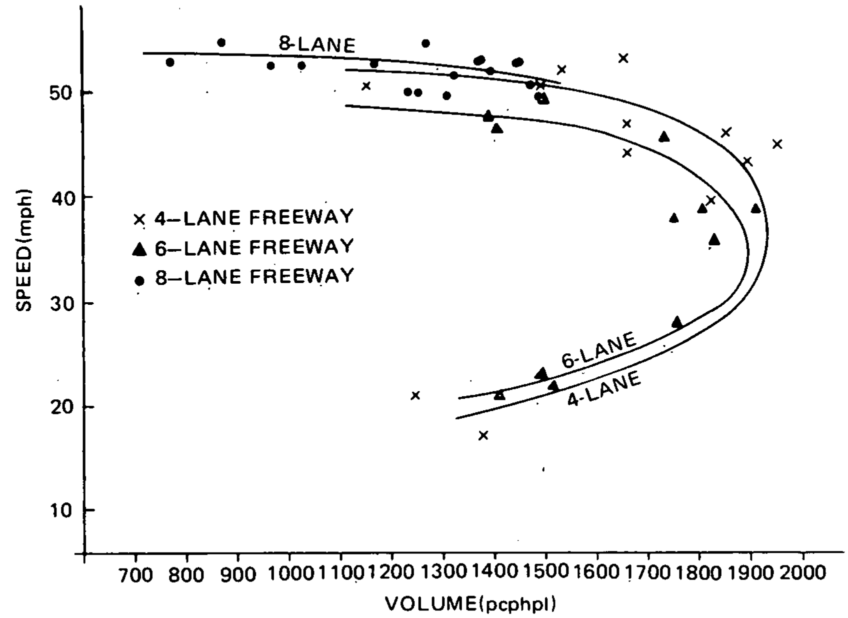


Figure 2-17. Speed-flow relationship for New York parkways in 1977. (Source: Ref. *1*)

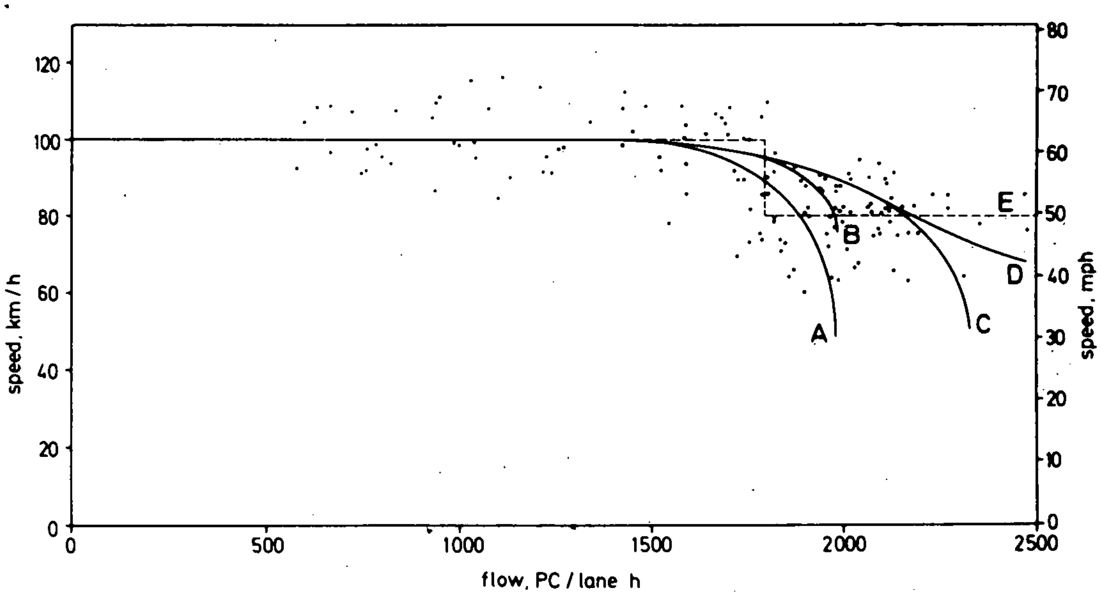


Figure 2-18. Speed-flow relationship for a six-lane freeway near Toronto in 1982. (Source: Ref. *21, Fig. 13*)

is constant to a flow of about 1,525 vphpl. The figure illustrates, however, the difficulty in firmly fixing the shape and location of the curve beyond this range. Data points are scattered considerably, and any one of several curves (labeled A through E in Figure 2-18) could be fit through the data—each with markedly differing characteristics. The researchers, Hurdle and Datta, advance the theory that perhaps speed is constant virtually to the point of capacity, after which breakdown occurs and unstable flow ensues. The data, collected at a bottleneck location, may suggest that some drivers forced to wait in a queue approaching the section simply do not accelerate to ambient speed, while those who do not have to wait in the queue traverse the bottleneck at the free-flow speed, no matter what level of flow exists.

A similar relationship between speed and flow on two-lane rural highways in Alberta, Canada, is reported by Krummins (*22*), and is illustrated in Figure 2-19. The curve shows a virtually constant speed for two-way flows up to 2,400 pcph, and the entire speed range is only 59 mph to 50 mph for the full range of flows. This in itself is interesting, because most of the speed-flow data for multilane flow suggest that capacity occurs at a critical speed in the vicinity of 30 mph, while speeds of 50 mph are suggested to exist at capacity for two-lane highways. It should be remembered, however, that capacity of a two-lane highway occurs at a *total* flow of between 2,000 and 2,800 pcph (depending on directional distribution)—only 1,000 to 1,400 pcphpl; while for multilane highways, the flow at capacity is 2,000 pcphpl. Speeds on multilane highways for similar per lane flows (1,000 to 1,400 pcphpl) are well over 50 mph. The capacity of two-lane highways is influenced more by interactions between directional flows than by roadway space availability.

As a result of observations indicating little sensitivity of speed to flow over a substantial range of stable flow rates, Roess, McShane, and Pignataro (*1*) have proposed that density be used as the primary parameter defining multilane level of service. Messer (*5*) has proposed that percent time delay be used as the principal level-of-service parameter for two-lane highways. Further discussion of these variables is given in the relevant chapters treating these facilities.

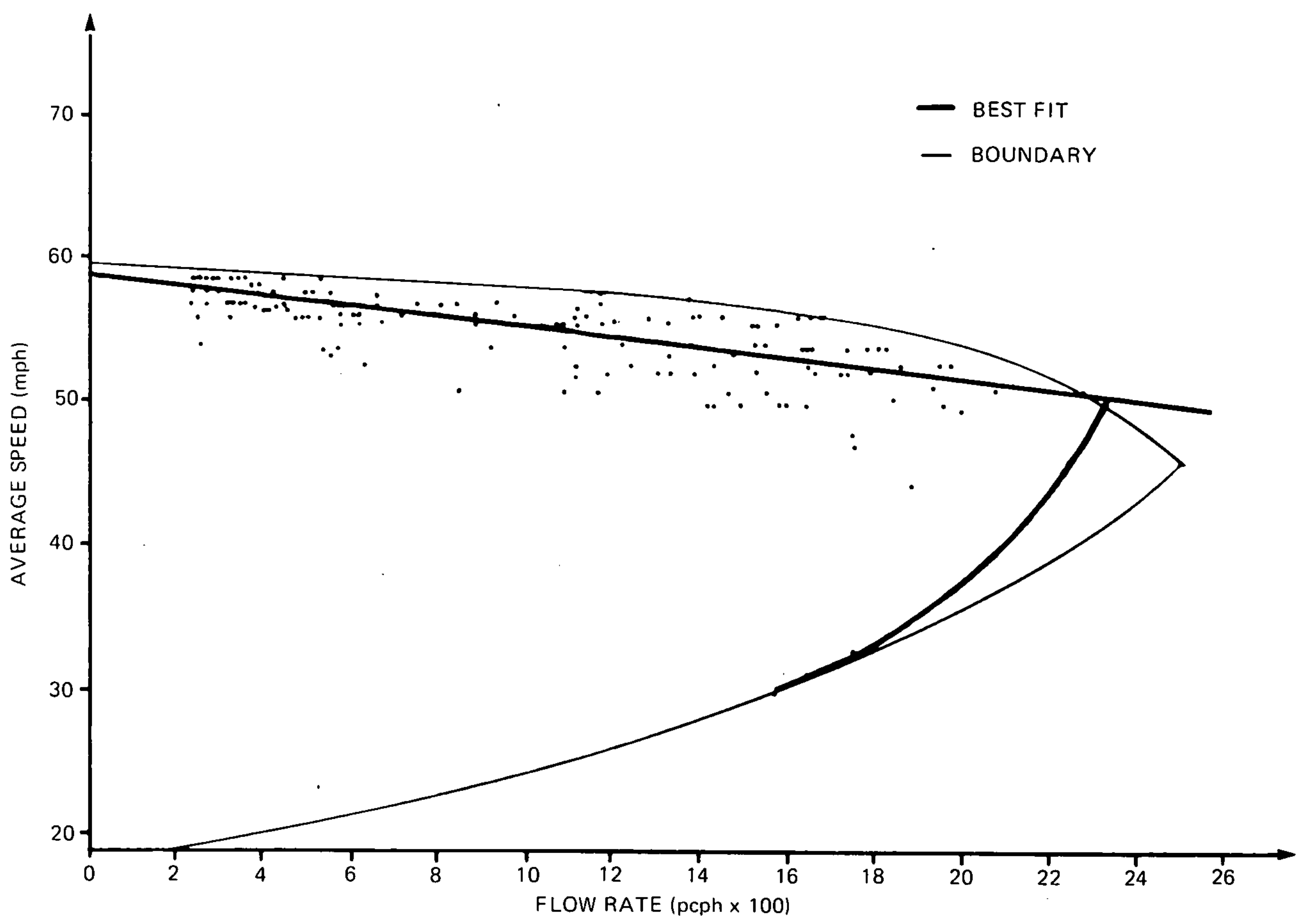


Figure 2-19. Speed-flow relationship for two-lane rural highways. (Source: Ref. *5*)

VI. SPACING AND HEADWAY CHARACTERISTICS

Spacing is defined as the distance between successive vehicles in a traffic stream, as measured from front bumper to front bumper. Headway is the time between successive vehicles as they pass a point on a lane or roadway, also measured from front bumper to front bumper. These characteristics are considered to be "microscopic," because they relate to individual pairs of vehicles within the traffic stream. Within any traffic stream, both spacing and headway of individual vehicles are distributed over a range of values that are generally related to the speed of the traffic stream and prevailing conditions. In the aggregate, these "microscopic" parameters are related to the "macroscopic" flow parameters density and rate of flow.

Headways are directly used as part of the Chapter 8 methodology to estimate percent time delay in a two-lane rural highway traffic stream. Defined as the percent of total time vehicles are delayed in an involuntary queue on a two-lane highway, "percent time delay" is estimated as the percent of vehicle headways less than or equal to 5 sec.

MATHEMATICAL RELATIONSHIPS

Spacing is a distance measure, in feet, and can be measured directly at a single point in time by measuring the distance between common points on successive vehicles. This generally requires complex aerial photographic techniques, so that spacing is usually derived from other direct measurements. Headway, on the other hand, can be measured more easily using stopwatch observations as vehicles pass a point on the roadway.

The average vehicle spacing in a traffic stream is directly related to the density of the traffic stream:

$$\text{Density (veh/mi/ln)} = \frac{\text{5,280 ft/mi}}{\text{Spacing (ft/veh)}} \qquad (2\text{-}5)$$

The relationship between average spacing and average headway in a traffic stream is dependent on speed:

$$\text{Headway (sec/veh)} = \frac{\text{Spacing (ft/veh)}}{\text{Speed (ft/sec)}} \qquad (2\text{-}6)$$

This relationship also holds for individual headways and spacings between pairs of vehicles. The speed would be that of the second vehicles in an individual pair of vehicles.

Flow rate is related to the average headway of the traffic stream:

$$\text{Flow rate (vph)} = \frac{\text{3,600 sec/hr}}{\text{Headway (sec/veh)}} \qquad (2\text{-}7)$$

HEADWAY DISTRIBUTIONS AND RANDOM FLOW

At any given lane flow rate, the mean or average headway is the reciprocal of flow rate. Thus, at a flow of 1,200 vphpl, the average headway is 3,600/1,200, or 3 sec. Vehicles do not, however, travel at constant headways. Vehicles tend to travel in groups, or platoons, with varying headways between successive vehicles. An example of the distribution of headways observed on the Long Island Expressway is shown in Figure 2-20. Lane 3 is seen to have the most uniform headway distribution, as evidenced by the range of values and the high frequency of the modal value—the peak of the distribution curve. The distribution in Lane 2 is similar to that of Lane 3, with slightly greater scatter (range from ½ sec to 9.0 sec). Lane 1 shows a much different pattern: it is far more dispersed, with headways ranging from ½ to 12.0 sec, and the frequency of the modal value is only about one-third of that for the other lanes. This reflects the lower flow usually occurring in Lane 1 (shoulder lane), and the driver desires of Lane 1 users.

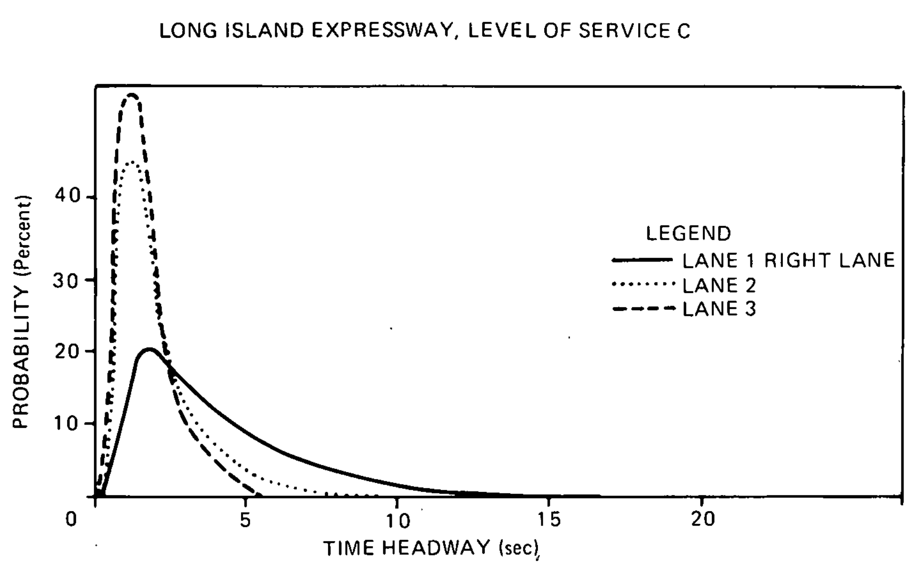


Figure 2-20. Time headway distribution for Long Island Expressway. (Source: Ref. *23*)

An examination of Figure 2-20 shows relatively few headways less than 1.0 sec. A vehicle traveling at 60 mph (88 fps) would have a spacing of 88 ft with a 1.0-sec headway, and only 44 ft with a ½-sec headway. This effectively reduces the space *between* vehicles (rear bumper to front bumper) to only 25 to 30 ft and would be extremely difficult to maintain and would allow little margin for driver error.

Drivers react to this inter-vehicle spacing, which they directly perceive, rather than to the traditional front bumper-to-front bumper measures used by traffic engineers. The latter includes the length of the vehicle, which is becoming smaller for passenger cars in the vehicle mix of the 1980's. If drivers maintain essentially the same inter-vehicle spacing, and car lengths continue to get shorter, some increases in capacity could conceivably result.

If traffic flow were truly random, small headways (less than 1.0 sec) would occur quite frequently. Several mathematical models have been developed that recognize the absence of small headways in most traffic streams. These models have been useful in developing simulation models of traffic flow, thereby extending research on flow characteristics beyond those conditions that can be observed and monitored in the field. Traffic flow is rarely purely random. Traffic signals and other controls regulate flows, and the trip generation characteristics of adjacent land generally produce trips in a nonrandom fashion.

VII. SATURATION HEADWAYS AND LOST TIMES UNDER INTERRUPTED FLOW

Chapter 1 introduced the basic concepts of saturation headway and saturation flow rate, and of start-up and change interval lost times. Figure 2-21 shows vehicle headway by position in the queue resulting from the studies of several different researchers. The studies referenced span over 30 years, from Greenshields in 1946 to KLD in 1975. Despite the advances in vehicle design and driver sophistication over this period, the results are remarkably consistent. Saturation headway ranges from a low of 2.1 sec to a high of 2.4 sec for these data—corresponding to a range of saturation flow rate of 1,714 vphgpl to 1,500 vphgpl. For all studies, the saturation headway does not become established until the 5th or 6th vehicle in the queue, indicating that the first 4 or 5 vehicles experience some start-up lost time. In discussing the results of Figure 2-21, Berry (*23*) noted that the variation in discharge headways of the first several vehicles depended on the choice of a screenline for measuring headways rather than any real difference in the observed headways.

Most studies of intersection discharge headways have focused on the observation of the first 10 to 12 vehicles. There is some indication that the saturation headway may increase somewhat when green time becomes quite long (approximately 60 sec or more). Although not well-documented, this effect implies that extremely long green phases may not be proportionally as efficient as those with green phases in the normal length range.

It should also be noted that prevailing conditions for the subject data varied considerably with respect to number and width of lanes, parking conditions, and mix of vehicle types.

Data from a single two-lane intersection approach in Evanston, Illinois, are given in Table 2-13. The data include *approach* headways across both lanes rather than discharge headways. Such headways are about one-half the discharge headway, which would make the data closely comparable to that of Figure 2-21.

A comprehensive study of saturation flow rates and start-up lost times in Kentucky (*27, 28*) also resulted in values in concert

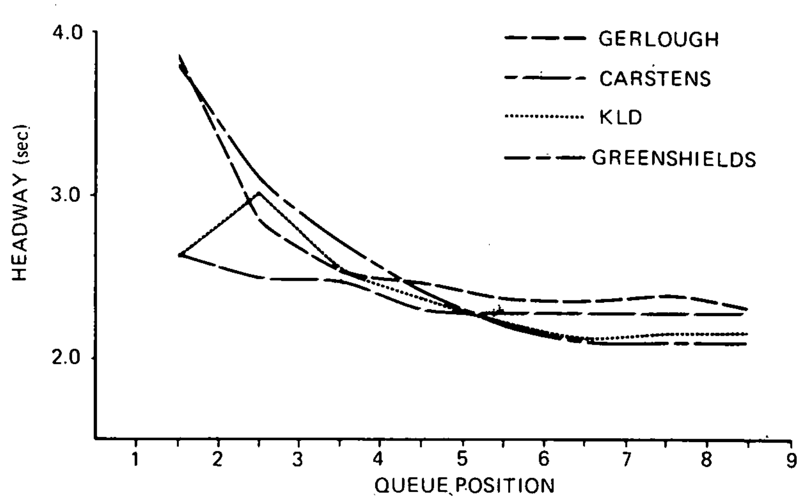


Figure 2-21. Comparison of various research results on queue discharge headways. (Source: Ref. *26*)

TABLE 2-13. ARRIVAL HEADWAYS AND LOST TIMES AT AN INTERSECTION IN EVANSTON, ILLINOIS

WEATHER	LIGHTING	START-UP LOST TIME FIRST VEHICLE (SEC)	AVERAGE ARRIVAL HEADWAY (SEC)
Dry	Day	2.48	1.09
Dry	Night	2.48	1.18
Wet	Night	2.72	1.29
Snow	Day	2.69	1.27
Snow	Night	2.64	1.28

SOURCE: Ref. *23*

with Figure 2-21. Saturation flow rates were measured for a variety of conditions of location within city, pedestrian activity, and other factors. The average of more than 18,000 measured saturation headways over all conditions was 2.19 sec, corresponding to a saturation flow rate of 1,646 vphgpl.

Start-up lost times were also measured for a variety of conditions, including city size (population), location within the city, signal timing, speed limit, and other factors. Values observed ranged from 1.01 sec to 1.95 sec. Corresponding values of change-interval lost time ranged from 1.21 to 2.80 sec, with the length of the change interval (yellow + all red) having a significant impact on the value observed—longer change intervals yielding longer lost times.

It should be pointed out that data on saturation headways and lost times depend heavily on prevailing conditions and on the definitions used for each term. The Kentucky study included the first 3 vehicles in queue as part of the start-up lost times, with subsequent vehicles representing saturation flow, whereas the data of Figure 2-21 suggest that up to the first 5 vehicles may experience start-up losses. Prevailing conditions of lane width, parking, transit interference, pedestrian interference, turning movements, and other factors, all influence these values. Observed values have varied widely where observed because of these factors.

For ideal conditions, including 12-ft lanes, all through vehicles, all passenger cars, no parking, no transit interference, and low pedestrian volumes, the procedures of Chapter 9 recommend a saturation flow rate of 1,800 pcphgpl, corresponding to a headway of 2.0 sec. Although this value is lower than most of the headways observed in the field, it should be remembered that few of the observed headways occurred under ideal conditions, and therefore reflect prevailing conditions that would increase the headway values. Start-up and change interval lost times are taken to be 2.0 sec apiece for ideal conditions.

The variation in the data presented here, however, suggests that local data collection to determine these values may be of some interest, and can lead to more accurate computations. Chapter 9 contains an appendix describing a data collection technique to make such measurements.

Signalized intersection procedures of this manual rely heavily on saturation headway and lost time calibrations as a means of describing the use of available green time.

Consider the following illustration. If there is a 60-sec cycle at an intersection, with a subject approach having 30 sec of green plus yellow time, the critical lane of that approach would have a capacity (under ideal conditions) determined as follows:

- Total time available for approach: 3,600 sec × (30/60) = 1,800 sec
- Total start-up lost time per hour: 2.0 sec/cycle × (3,600/60) cycles/hr = 120 sec
- Total change interval lost time per hour: 2.0 sec/cycle × (3,600/60) cycles/hr = 120 sec
- Total time available for movement at saturation headway: 1,800 − 120 − 120 = 1,560 sec
- Capacity of critical lane: 1,560 sec/2.0 sec/veh = 780 vph

Procedures essentially allow for the "book-keeping" of available time, with all lost times subtracted and all remaining time used at a rate of one vehicle per saturation headway. The procedures in Chapter 9 also contain numerous adjustments to reflect prevailing conditions other than the ideal values noted here and in the chapter.

VIII. SUMMARY

This chapter addresses the range and use of important highway traffic characteristics in capacity analysis. It emphasizes that these characteristics are not uniform nor are they constant throughout North America, and variations due to local driving habits and environments are to be expected. Direct measurement of such characteristics may be used to "fine tune" or improve the results of the analysis procedures of this manual, which are based on observed national averages.

Transit characteristics are treated in Chapter 12 and pedestrian characteristics are treated in Chapter 13.

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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.

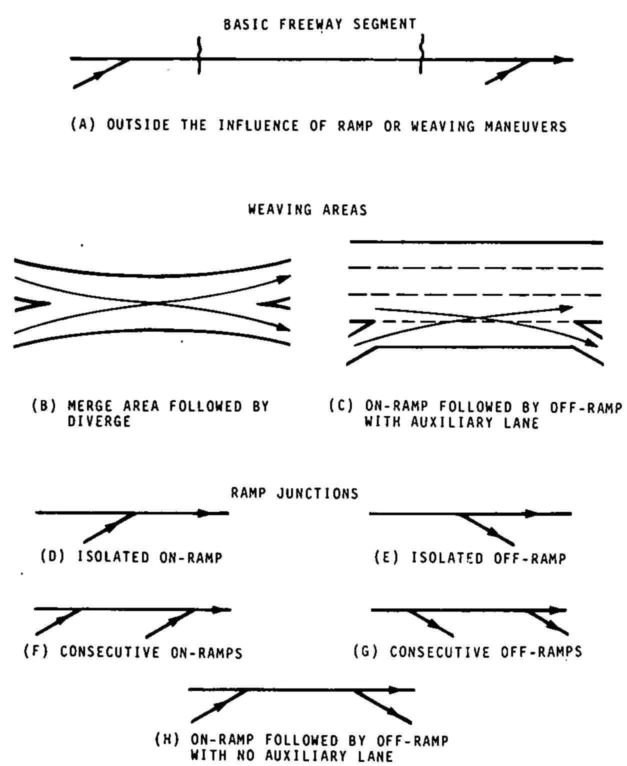


Figure 3-1. Freeway components.

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

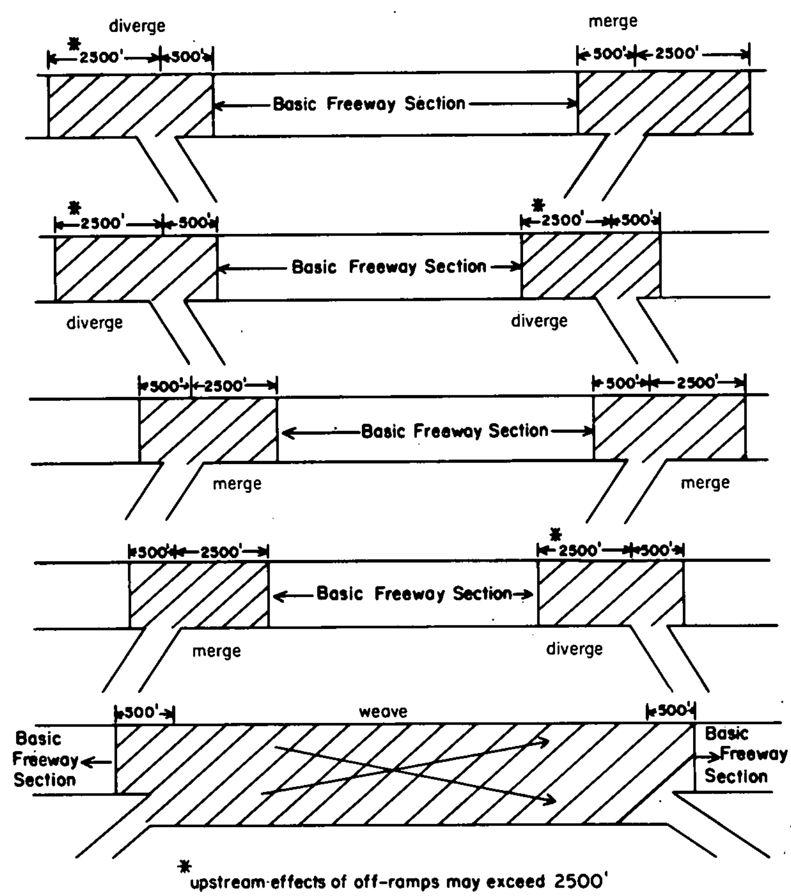


Figure 3-2. Influence areas of freeway components.

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.

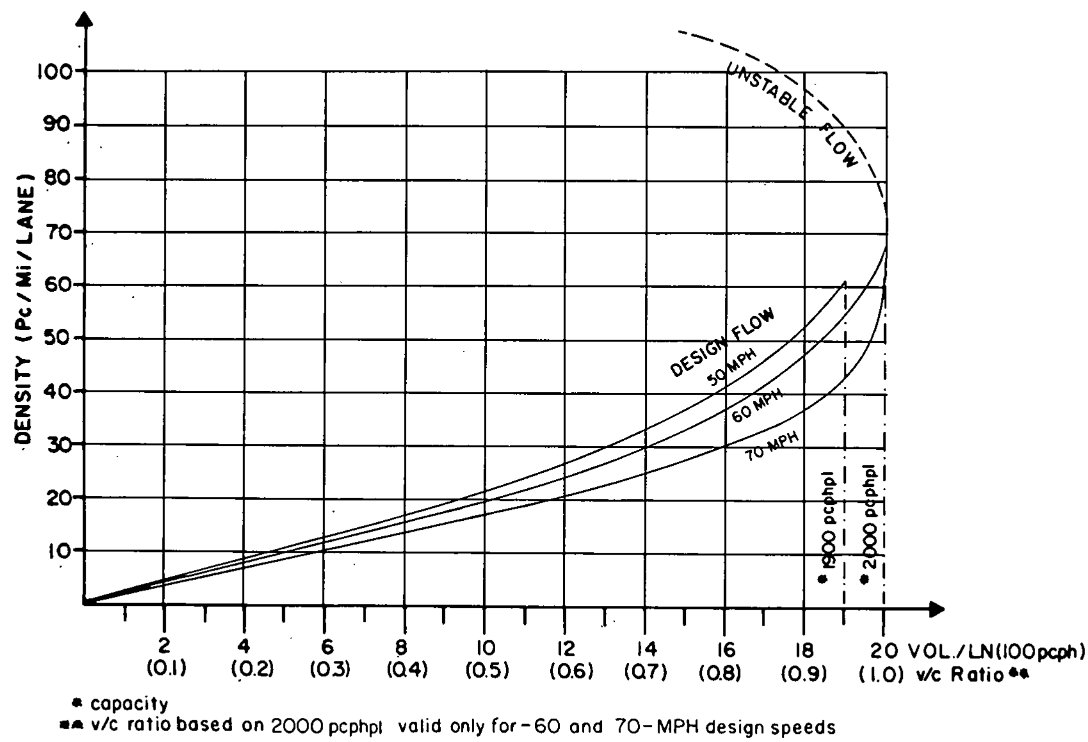


Figure 3-3. Density-flow relationships under ideal conditions.

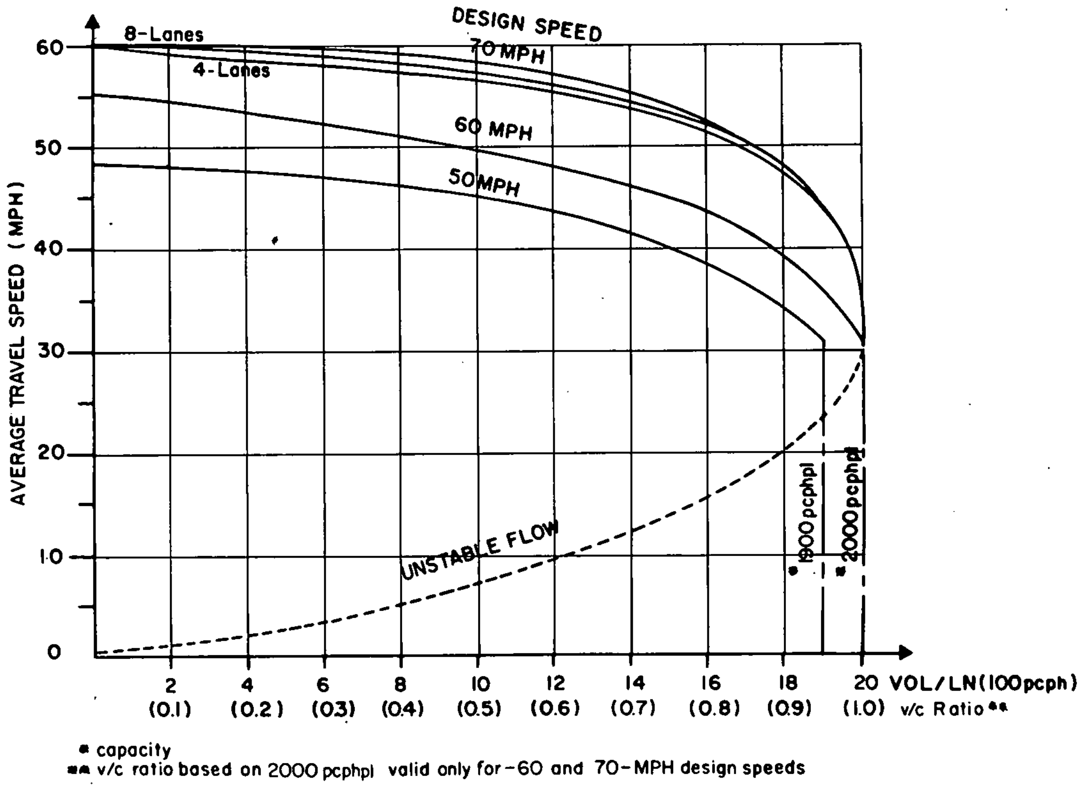


Figure 3-4. Speed-flow relationships under ideal conditions.

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 from 10 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 major concern of drivers with respect to service quality, freedom to maneuver and proximity to other vehicles are equally 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 non-ideal 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

		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	≤ 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 pcph.

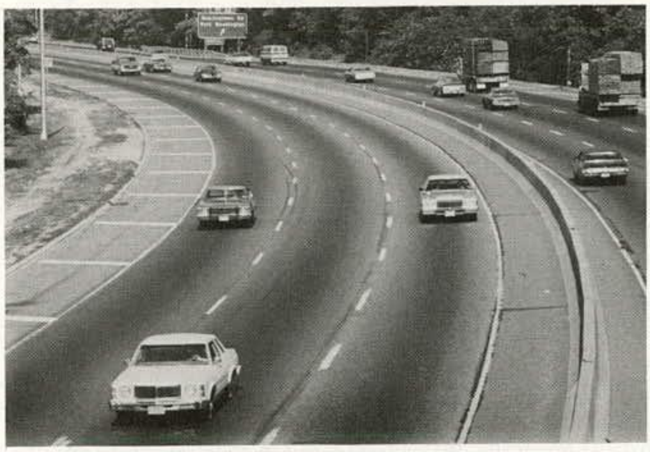


Illustration 3-5. Level-of-service A.

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

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

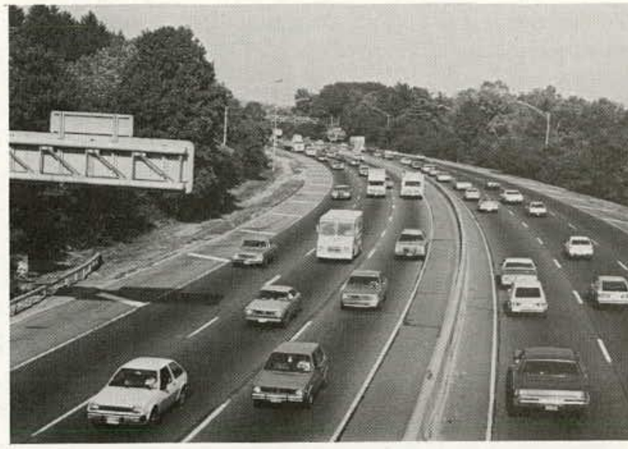


Illustration 3-8. Level-of-service D.

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 boundary between LOS D and 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

Maximum Service Flow Rate Per Lane

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.

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

where:

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

$(v/c)_i$ = 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 pcphpl.

Service Flow Rate

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 = service flow rate for LOS i under prevailing roadway and traffic conditions for N lanes in one direction, in vph;
N = 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 FLOW RATE

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 traffic 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."

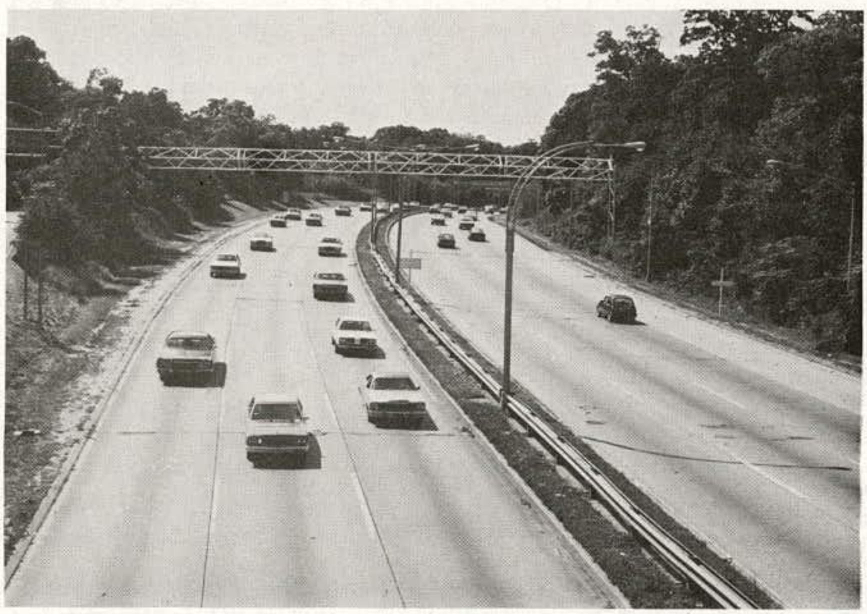


Illustration 3-12. The freeway section shown here is also ideal with respect to lane width and lateral clearances. The W-beam median barrier is another type of barrier which generally does not cause vehicles to shift their lateral lane placement, and also would not be considered an "obstruction" in most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

Total Rise = 2,640 × 0.03 + 5,280 × 0.04 = 290.4 ft
Average Grade = 290.4/7,920 = 0.037 or 3.7 percent
Note: 2,640 ft = ½ 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}														
	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 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 as low as 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 and/or local data must be used 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 maximum service flow rate, *MSF*, or the 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:

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

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 *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 *MSF* or the 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 *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.

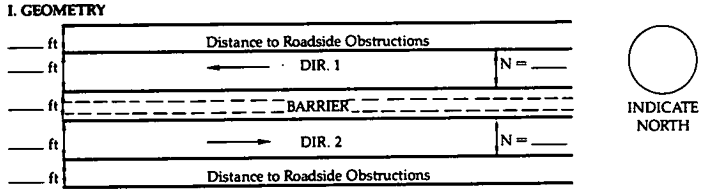
For example, if a 70-mph freeway were found to have an *MSF* of 1,685 pcphpl, Table 3-1 would be used to find the level of service. Because 1,685 pcphpl is less than 1,850 pcphpl (the

OPERATIONAL ANALYSIS WORKSHEET

Facility Section: ______

Date: ______ Time: ______ (of analysis data)

I. GEOMETRY



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

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

$f_{HV} =$

	v/c =	SF	/[c_j	× N	× f_w	× f_p	× 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
			Table 3-1		Table 3-2	Table 3-10				
Dir. 1										
Dir. 2										

	c	v/c	LOS (Table 3-1)	Speed (Fig. 3-4)	Density (Fig. 3-3)
Dir. 1					
Dir. 2					

Name: ______ Date: ______

Checked by: ______

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 51 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.

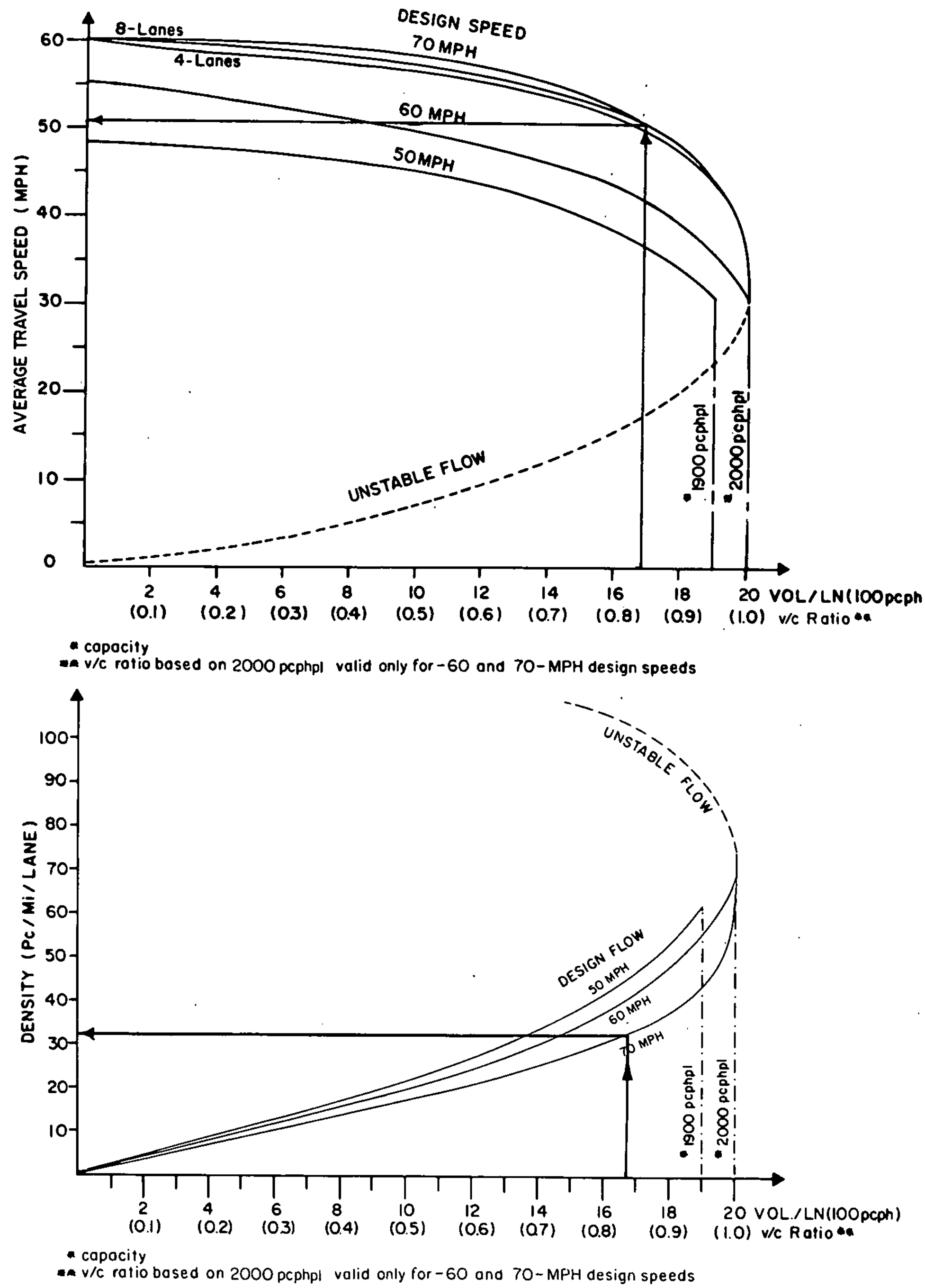


Figure 3-6. Example solutions for approximate density and speed of a freeway traffic stream.

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*:

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

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 Table 3-11	Design Speed (mph)	Lane Width (ft)	Lateral Clearance (ft) Roadside	Lateral Clearance (ft) Median	Terrain or Grade (L, R, or M)	Terrain or 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 Table 3-1	× (v/c)	× f_w Table 3-2	× f_p Table 3-10	× f_{HV}] Table 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:

$$DDHV = AADT \times K \times D \tag{3-6}$$

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	0.09 – 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:

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

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 50 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:

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

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, MP25*

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*	-	-	☒ Commuter ☐ Other
Dir. 2	*2,100*	*0.95*	*2,211*	*6*	-	-	☒ Commuter ☐ Other

III. ANALYSIS

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

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

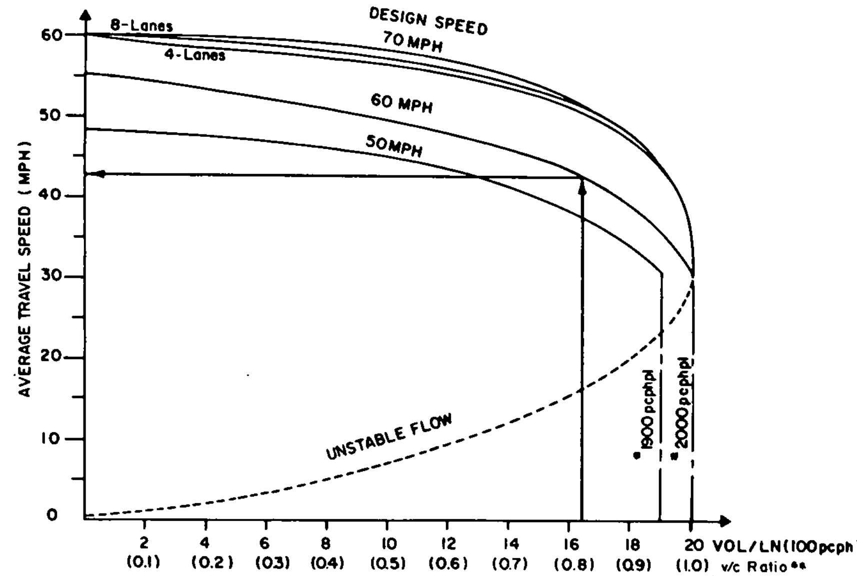
$f_{HV} =$

	v/c =	SF	$/[c_j$	$\times N$	$\times f_w$	$\times f_p$	$\times f_{HV}]$	$1/[1 + P_T(E_T - 1) + P_B(E_B - 1) + P_R(E_R - 1)]$		
			Table 3-1		Table 3-2	Table 3-10		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*	-	-

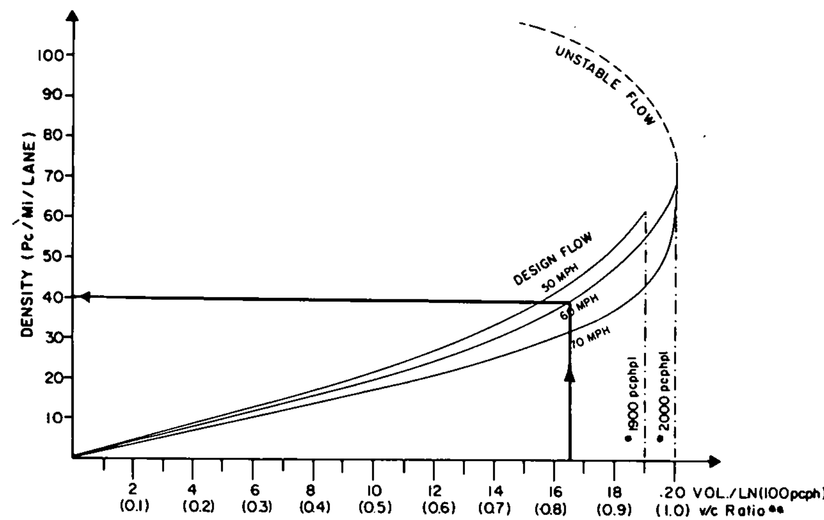
	c	v/c	LOS (Table 3-1)	Speed (Fig. 3-4)	Density (Fig. 3-3)
Dir. 1	*2,686*	*0.82*	*D*	*43 mph*	*40 pc/mi/ln*
Dir. 2	*2,686*	*0.82*	*D*	*43 mph*	*40 pc/mi/ln*

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

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:

$$\begin{aligned} \text{Capacity} &= 2{,}686 \text{ vph} \\ \text{Actual flow rate} &= \underline{2{,}211 \text{ vph}} \\ & \quad\; 475 \text{ vph} \end{aligned}$$

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 in one direction, 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

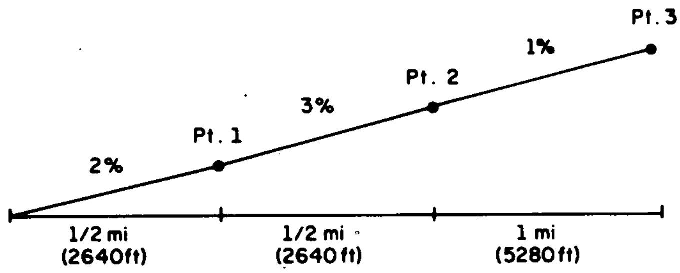


Figure 3-9. Composite grade for Calculation 2.

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, 6 lanes);
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) = $4{,}118/[2{,}000 \times 3 \times 0.97 \times 0.97 \times 1.00] = 0.73$

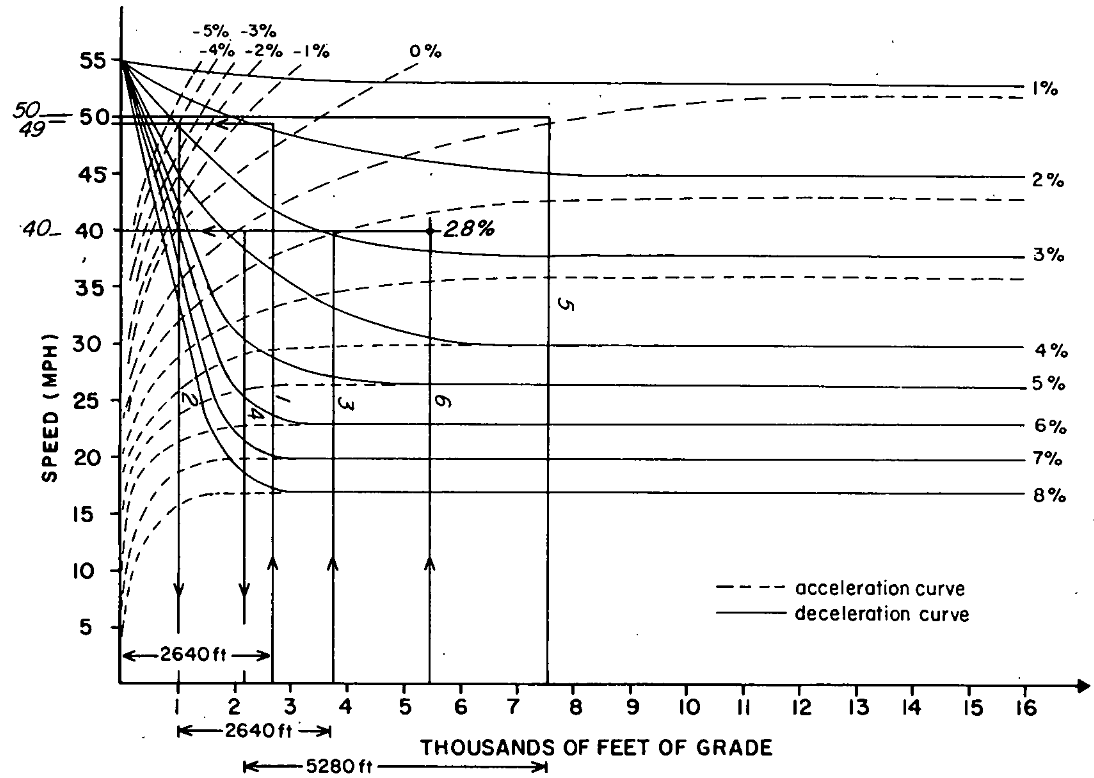


Figure 3-10. Solution of composite grade for Problem 2.

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 PM (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	-	-	☒ Commuter ☐ Other
Dir. 2	3,500	0.85	4,118	5	-	-	☒ 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	$/[c_j \times$	$N \times$	$f_w \times$	$f_p \times$	$f_{HV}]$	$1/[1 + P_T(E_T - 1) + P_B(E_B - 1) + P_R(E_R - 1)]$ E_T	E_B	E_R
			Table 3-1		Table 3-2	Table 3-10				
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 (Table 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:

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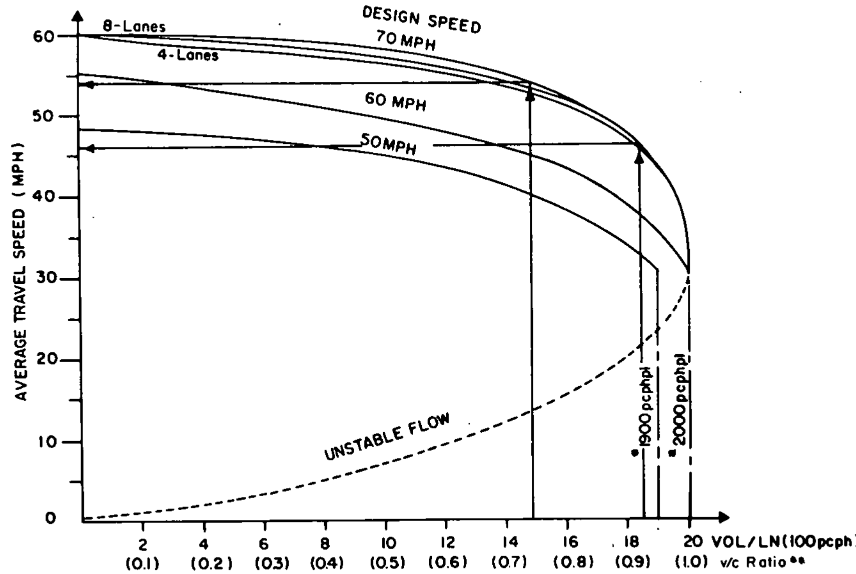
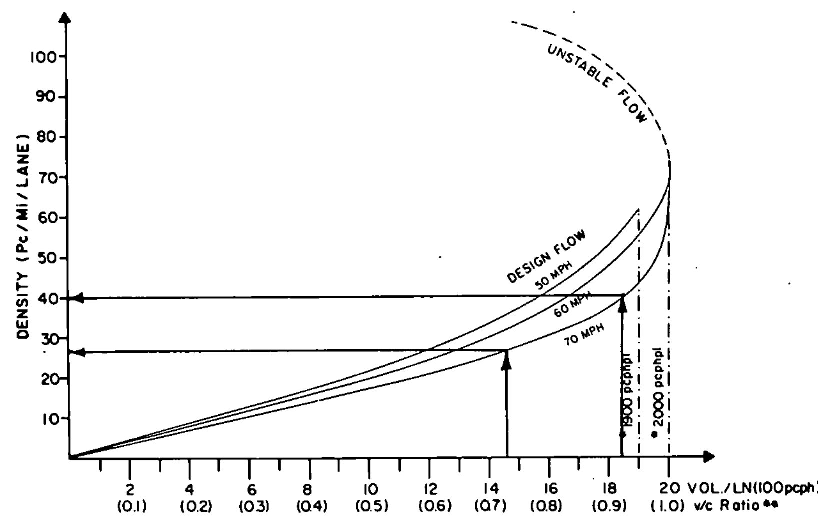


* capacity
** v/c ratio based on 2000 pcphpl valid only for 60 and 70-MPH design speeds



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

Figure 3-11. 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 Table 3-11	Design Speed (mph)	Lane Width (ft)	Lateral Clearance (ft) Roadside	Median	Terrain or Grade (L, R, or M)	(%)	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

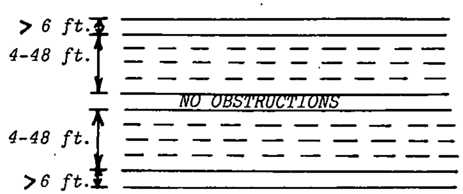
	DDHV (vph)	PHF	SF=(DDHV/PHF)	% Trucks	% Buses	% RV's	Driver Population
Dir. 1	4500	0.90	5,000	12	-	-	☒ Commuter ☐ Other
Dir. 2	4500	0.90	5,000	12	-	-	☒ Commuter ☐ Other

III. DESIGN ANALYSIS

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

	N	= SF	$/[c_j$	$\times (v/c) \times$	$f_w \times$	f_p	$\times f_{HV}]$	$f_{HV} = 1/[1 + P_T(E_T - 1) + P_B(E_B - 1) + P_R(E_R - 1)]$ E_T	E_B	E_R
			Table 3-1		Table 3-2	Table 3-10	Table 3-9			
Dir. 1	3.5	5,000	2,000	0.77	1.00	1.00	0.92	1.7	-	-
Dir. 2	3.5	5,000	2,000	0.77	1.00	1.00	0.92	1.7	-	-

IV. SKETCH DESIGN



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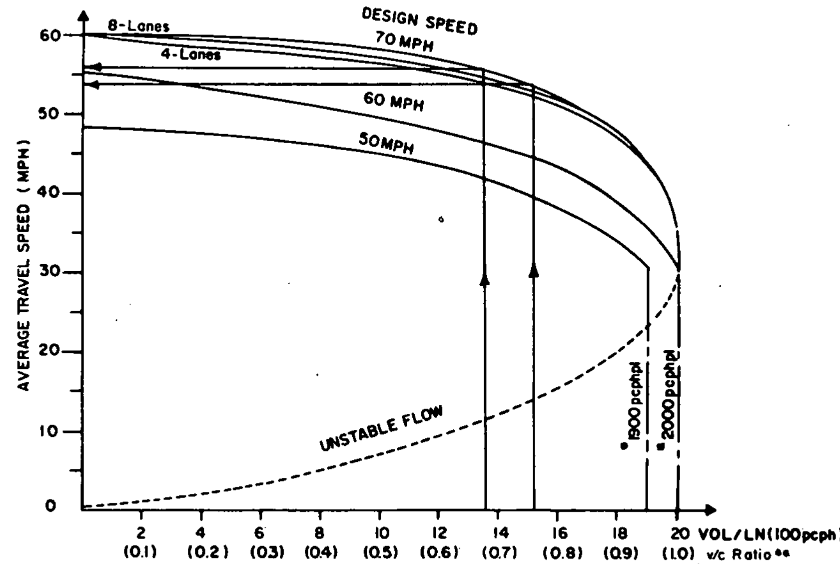
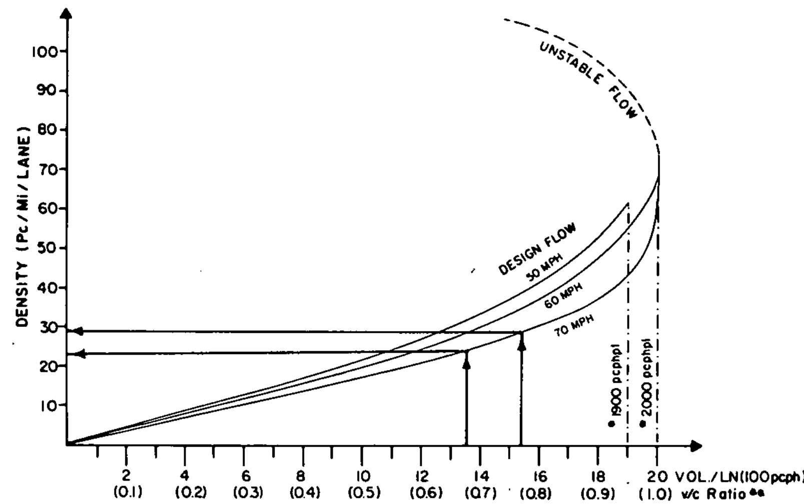


• capacity
** v/c ratio based on 2000 pcphpl valid only for -60 and 70-MPH design speeds



• 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:

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

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

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 Table 3-11	Design Speed (mph)	Lane Width (ft)	Lateral Clearance (ft) Roadside	Lateral Clearance (ft) Median	Terrain or Grade (L, R, or M)	Terrain or Grade (%)	Length (mi)
Dir. 1	C	0.60	70	12	≥ 6	≥ 6		+5	1 1/2
Dir. 2									

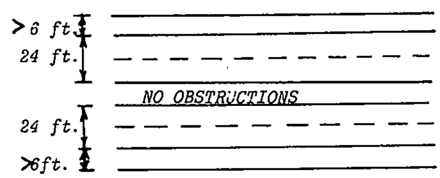
II. TRAFFIC FORECASTS

	DDHV (vph)	PHF	SF=(DDHV/PHF)	% Trucks	% Buses	% RV's	Driver Population
Dir. 1	1,000	0.95	1,052	-	5	20	☐ Commuter ☒ Other
Dir. 2							☐ Commuter ☐ Other

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

	N	= SF	/[c_j × Table 3-1	× (v/c)	× f_w Table 3-2	× f_p Table 3-10	× f_{HV}] Table 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., POLUS, A., and GRINBERG, I., "A Revised Method for the Determination of Passenger Car Equivalencies." *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.

FHWA-RD-74-42, Midwest Research Institute, Kansas City, Mo. (1975).
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.

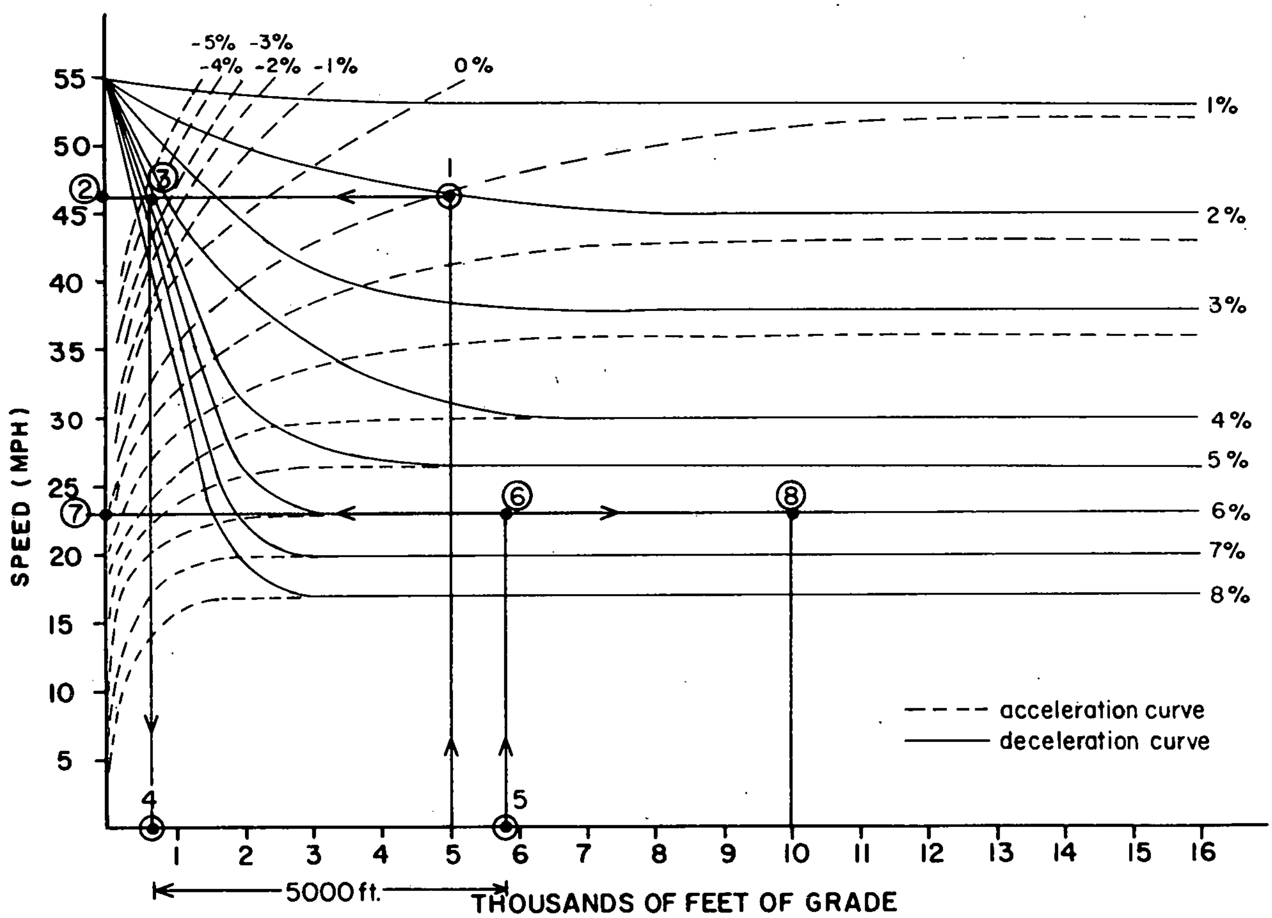


Figure I.3-1. Sample solution for equivalent grade using 200-lb/hp performance curves.

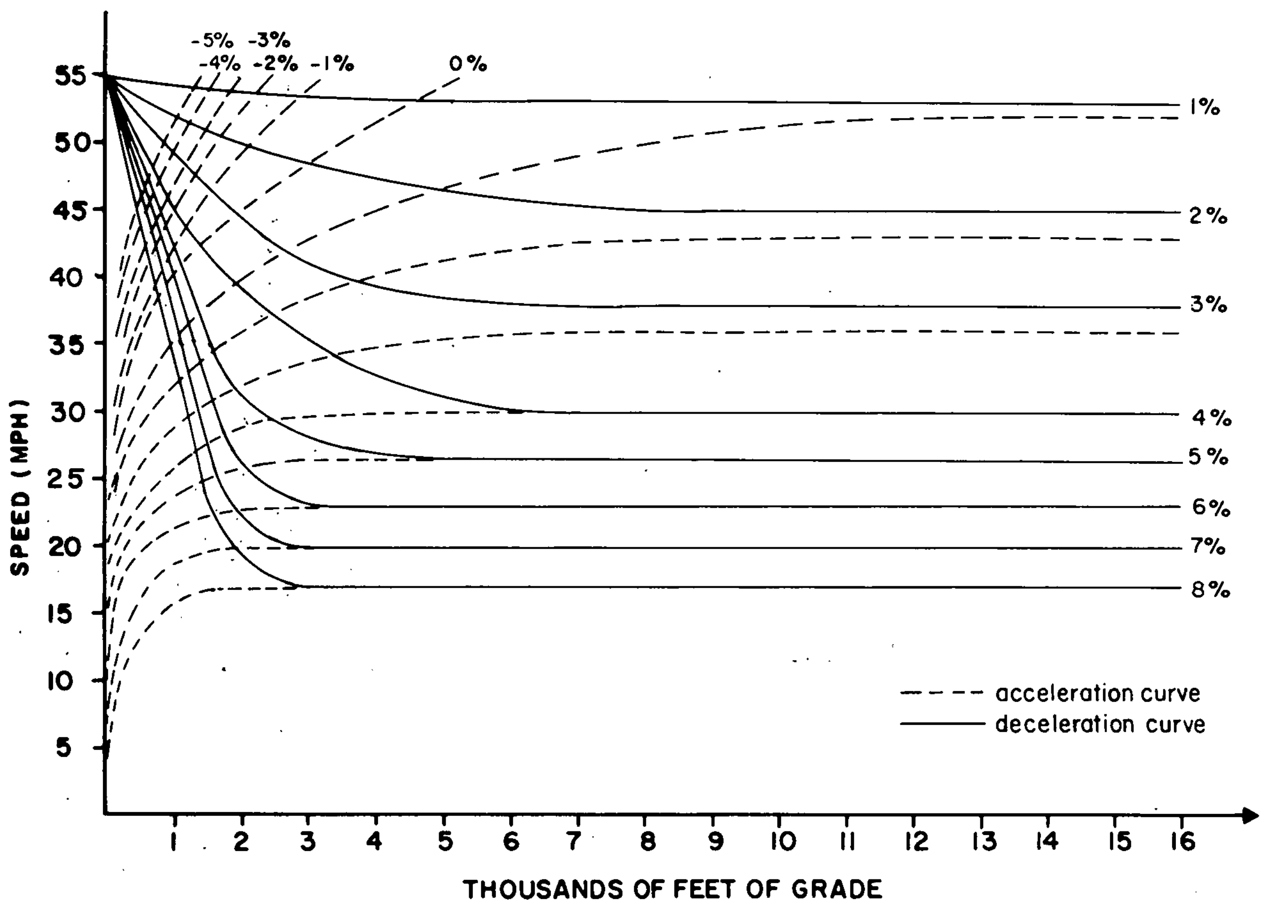


Figure I.3-2. Performance curves for a standard truck (200 lb/hp).

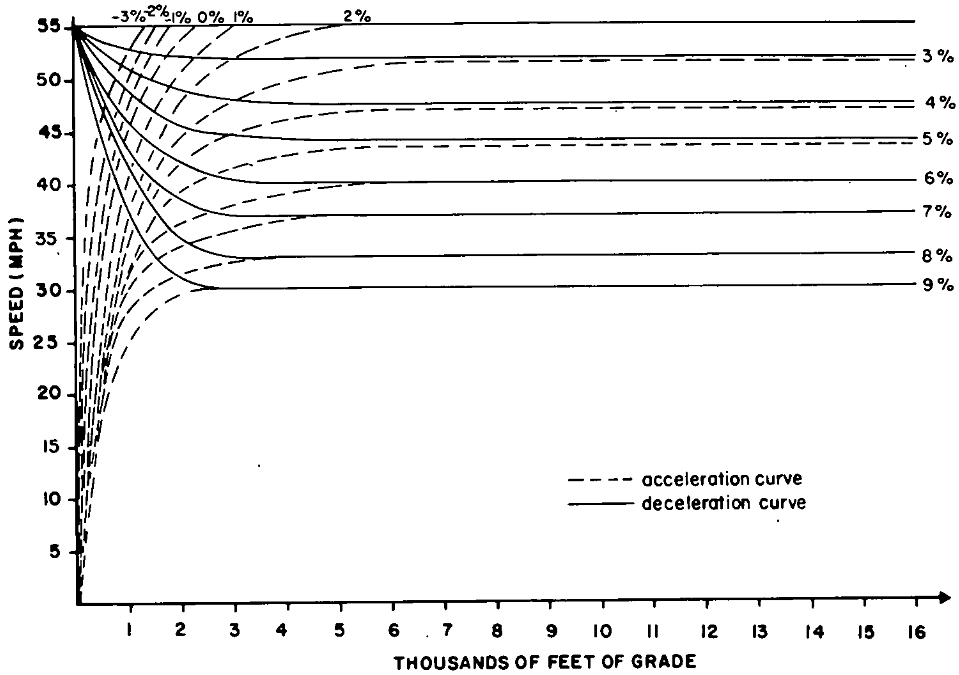


Figure I.3-3. Performance curves for light trucks (100 lb/hp).

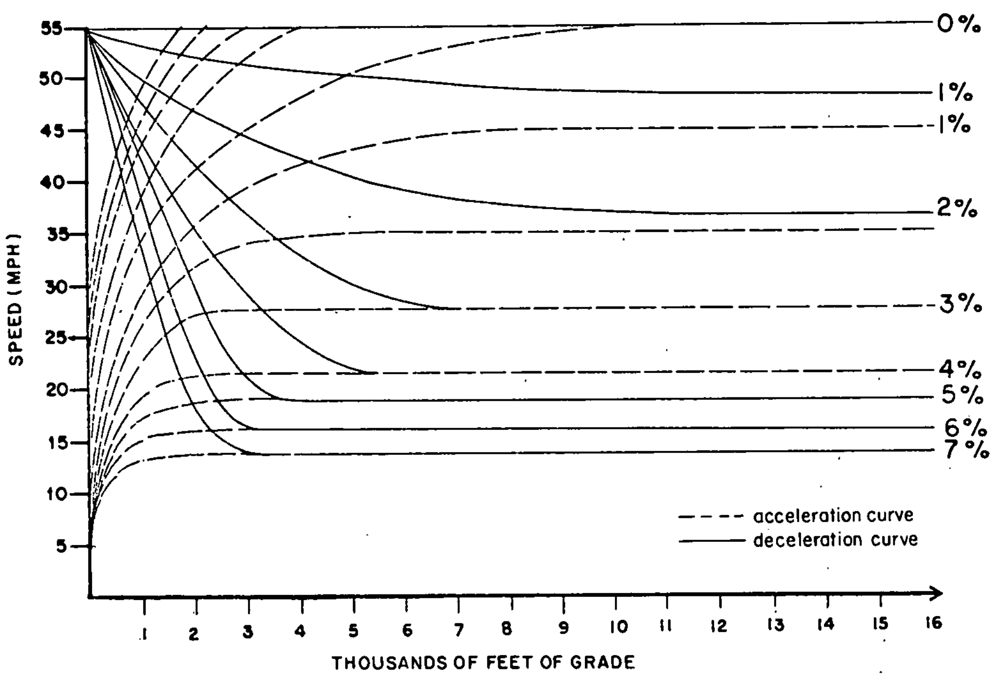


Figure I.3-4. Performance curves for heavy trucks (300 lb/hp).

APPENDIX II

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

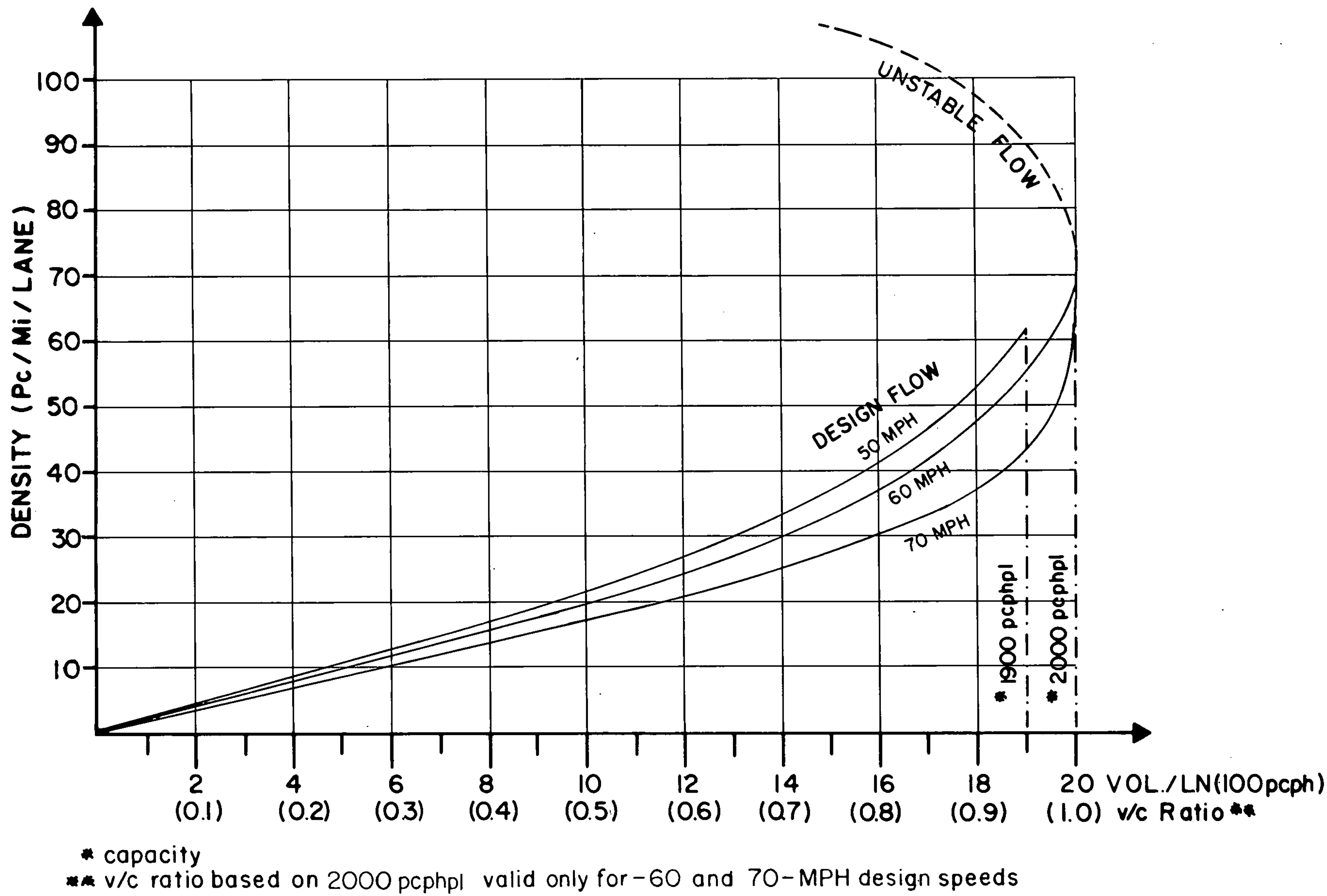
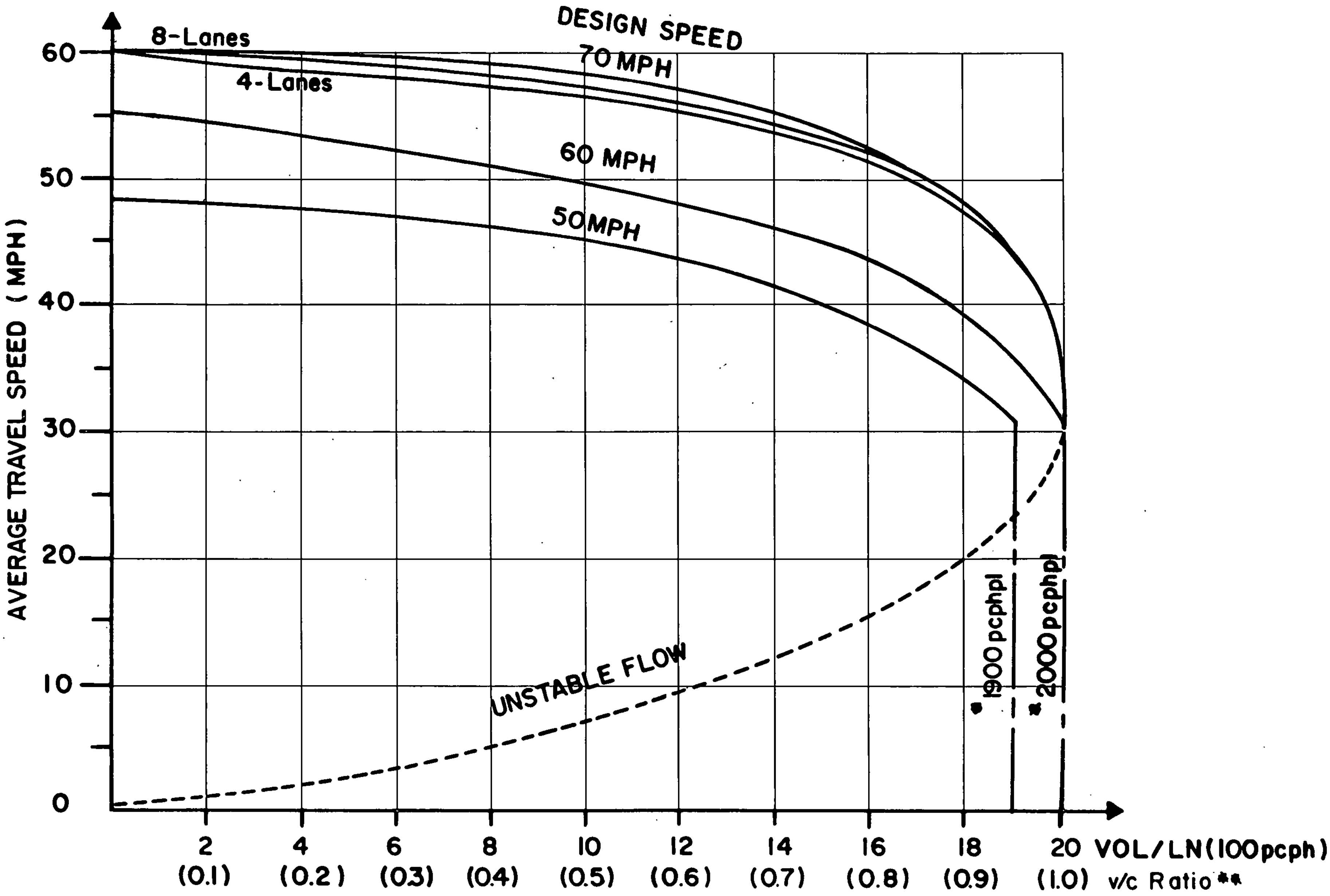


Figure 3-3. Density-flow relationships under ideal conditions.



* 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.

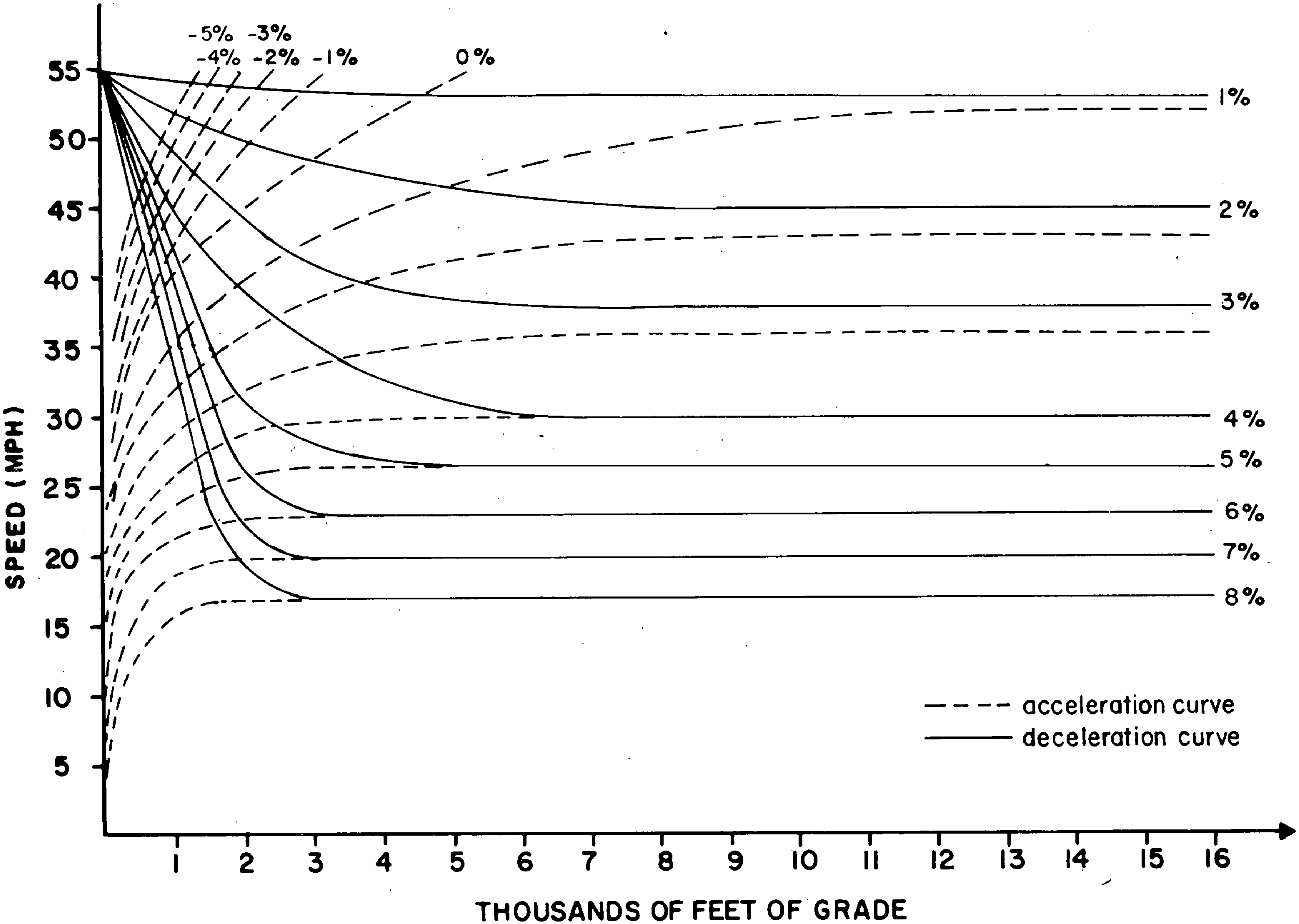


Figure I.3-2. Performance curves for a standard truck (200 lb/hp).

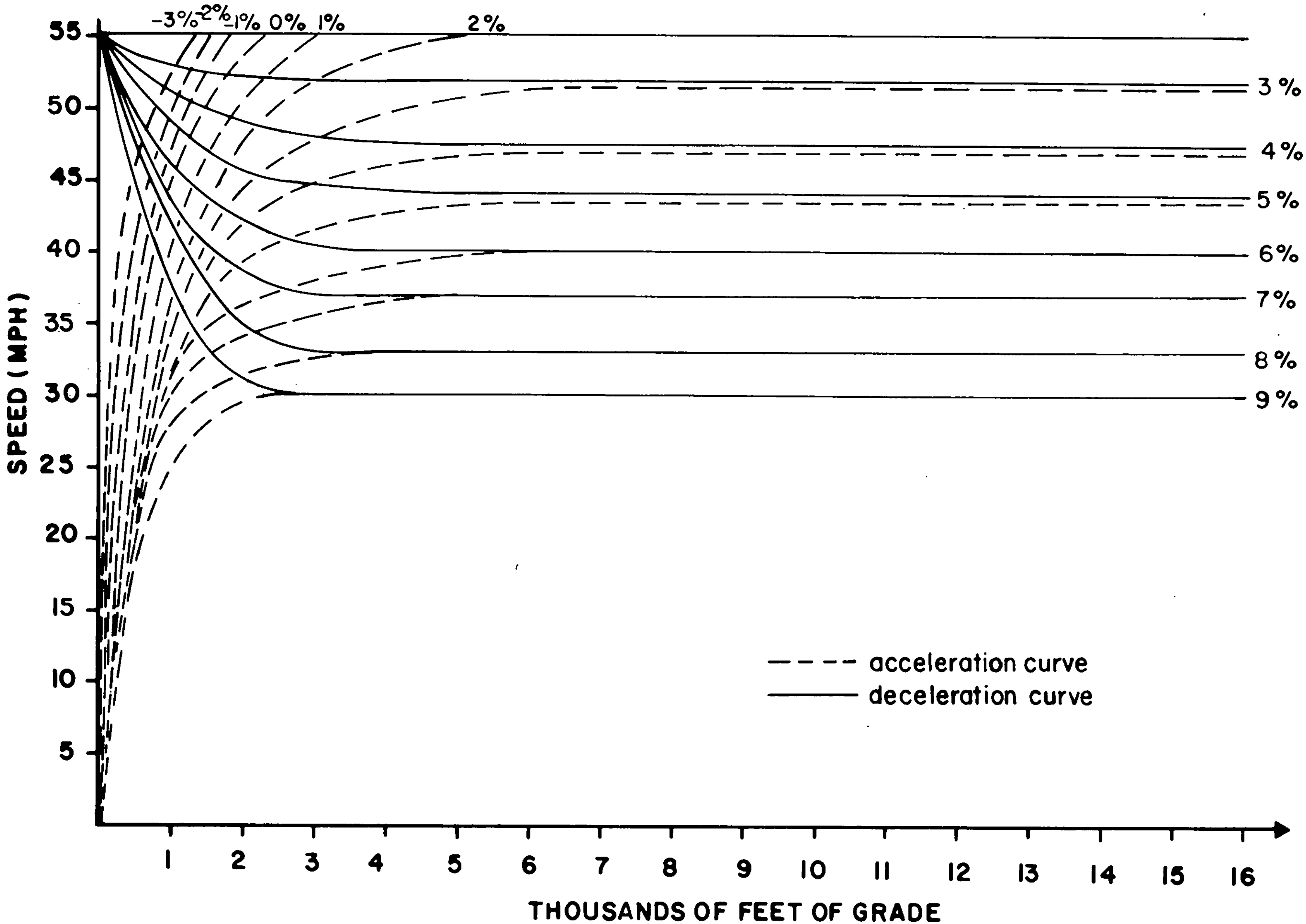


Figure I.3-3. Performance curves for light trucks (100 lb/hp).

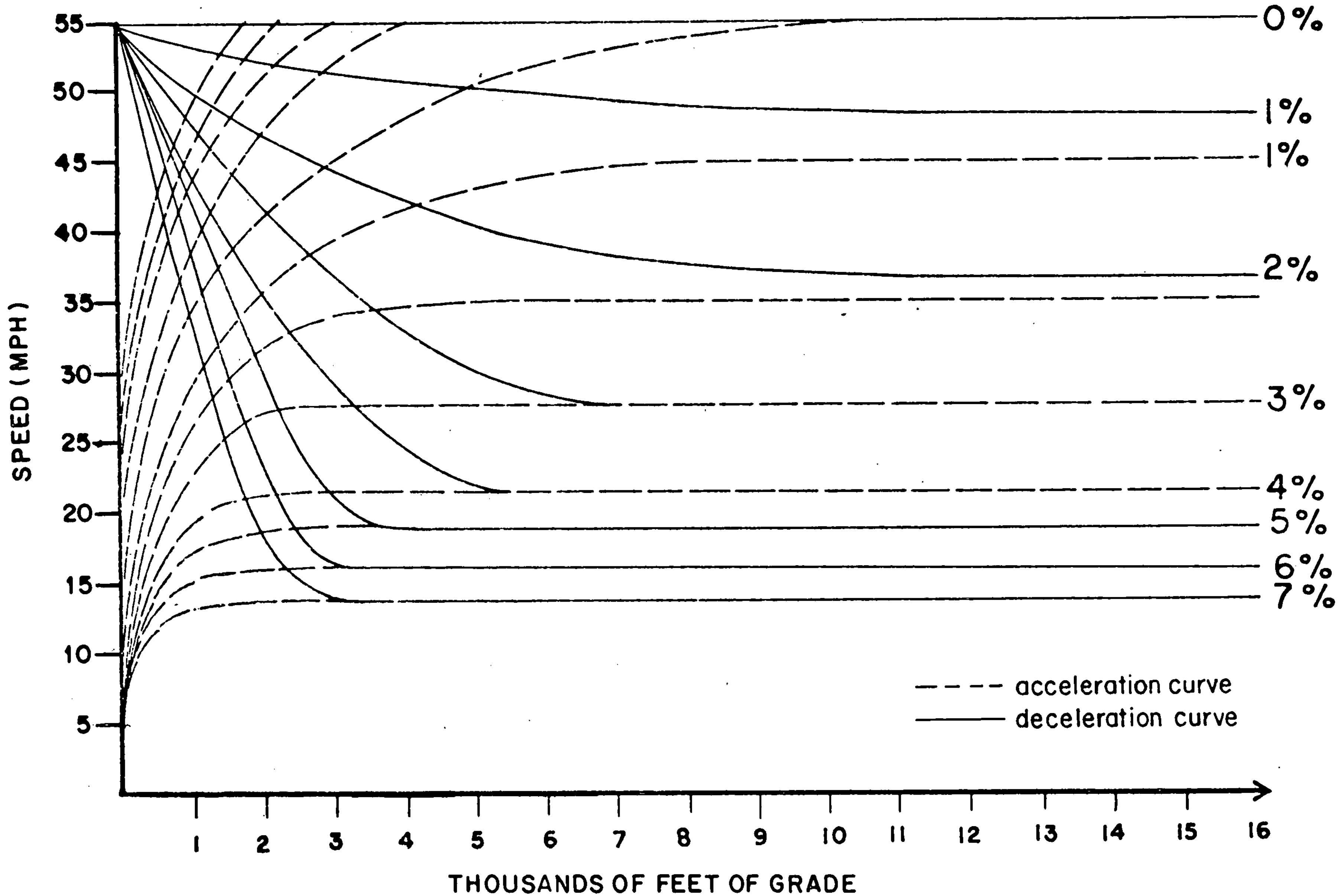


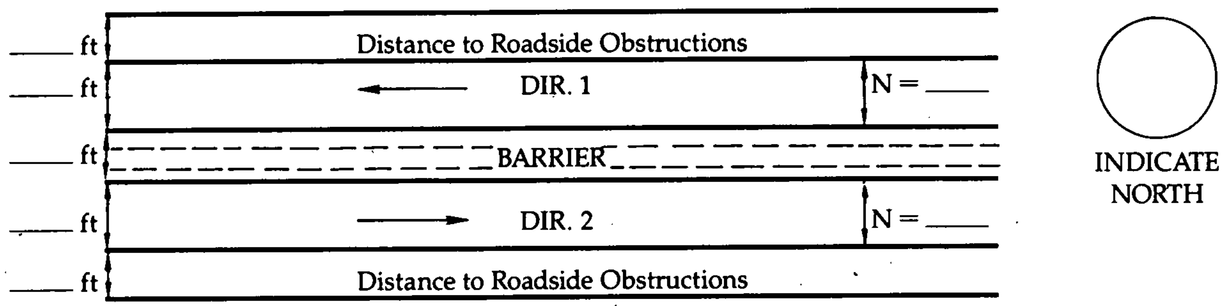
Figure I.3-4. Performance curves for heavy trucks (300 lb/hp).

OPERATIONAL ANALYSIS WORKSHEET

Facility Section: ____________________

Date: ____________________ Time: ____________________ (of analysis data)

I. GEOMETRY



	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

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

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

$f_{HV} =$

	$v/c =$	SF	$/[c_j$	$\times$ N	$\times$ f_w	$\times$ f_p	$\times f_{HV}]$	$1/[1+P_T(E_T-1)+P_B(E_B-1)+P_R(E_R-1)]$ E_T	E_B	E_R
			Table 3-1		Table 3-2	Table 3-10				
Dir. 1										
Dir. 2										

	c	v/c	LOS (Table 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 Table 3-11	Design Speed (mph)	Lane Width (ft)	Lateral Clearance (ft) Roadside	Lateral Clearance (ft) Median	Terrain or Grade (L, R, or M)	Terrain or 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	$= SF$	$/[c_j$ Table 3-1	$\times (v/c)$	$\times f_w$ Table 3-2	$\times f_p$ Table 3-10	$\times f_{HV}]$ Table 3-9	E_T	E_B	E_R
Dir. 1										
Dir. 2										

IV. SKETCH DESIGN

Name: ______________________ Date: ______________________

Checked by: ______________________________

CHAPTER 4

WEAVING AREAS

CONTENTS

I. INTRODUCTION

Weaving is defined as the crossing of two or more traffic streams traveling in the same general direction along a significant length of highway, without the aid of traffic control devices. Weaving areas are formed when a merge area is closely followed by a diverge area, or when an on-ramp is closely followed by an off-ramp and the two are joined by an auxiliary lane.

Weaving areas require intense lane-changing maneuvers as drivers must access lanes appropriate to their desired exit point. Thus, traffic in a weaving area is subject to turbulence in excess of that normally present on basic highway sections. This turbulence presents special operational problems and design requirements that are addressed by the procedures of this chapter.

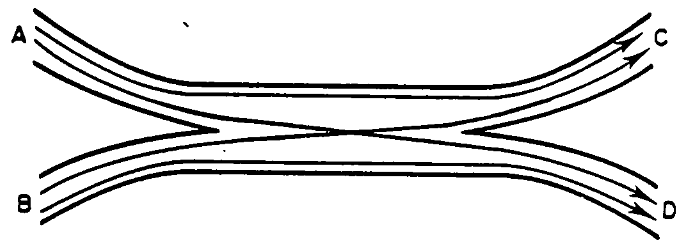


Figure 4-1. Formation of a weaving section.

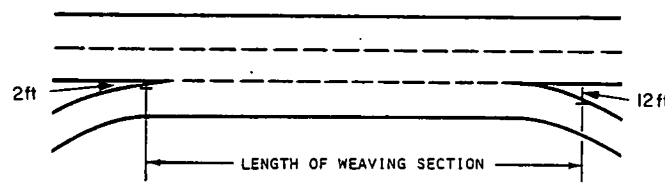


Figure 4-2. Measuring length of a weaving section.

Figure 4-1 shows the formation of a weaving area. If entry and exit roadways are referred to as "legs," vehicles traveling from leg A to leg D must cross the path of vehicles traveling from leg B to leg C. Flows A-D and B-C are, therefore, referred to as *weaving flows.* Flows A-C and B-D may also exist in the section, but these need not cross the path of other flows, and are referred to as *nonweaving flows.* Figure 4-1 shows a *simple weaving area,* formed by a single merge point followed by a single diverge point. *Multiple weaving areas,* formed by one merge followed by two diverges or two merges followed by a single diverge, are discussed later in this chapter.

Weaving areas may exist on any type of highway: freeways, multilane highways, two-lane highways, or arterials. They are most prevalent, however, as part of freeway systems, and most recent research has focused on freeway weaving areas. The procedures of this chapter apply to freeway weaving areas, but may be applied as an approximation to other types of facilities. In such applications, differences in predicted operating characteristics for alternative situations may be reasonably approximated using the procedures of this chapter. The absolute values of speeds and other parameters predicted, however, would be less accurate.

It should also be noted that the procedures herein involve a series of equations that require the use of a calculator having exponential functions for easy implementation.

WEAVING LENGTH

The requirement that drivers execute lane changes to complete many weaving movements introduces a new geometric parameter for consideration—*weaving length.* The length of the weaving section constrains the time and space in which the driver must make all required lane changes. Thus, as the length of a weaving area decreases (all other factors being constant), the intensity of lane-changing, and the resulting level of turbulence, increases. The measurement of weaving area length is shown in Figure 4-2. Length is measured from the merge gore area at a point where the right edge of the freeway shoulder lane and the left edge of the merging lane(s) are 2 ft apart to a point at the diverge gore area where the two edges are 12 ft apart.

Procedures of this chapter generally apply to weaving sections of up to 2,500 ft in length. Weaving may exist in longer sections, but merging and diverging movements often segregate, with lane-changing tending to concentrate near merge and diverge gore areas. For longer sections, merge and diverge areas may be separately analyzed using the procedures of Chapter 5. Weaving turbulence may exist throughout a long section to some degree, but operations are approximately the same as for a basic freeway section.

CONFIGURATION

Because lane-changing is the critical operational feature of weaving areas, another critical geometric characteristic can drastically affect performance: configuration. *Configuration* refers to the relative placement and number of entry lanes and exit lanes for the section, and it can have a major impact on how much lane-changing must take place in the section.

The procedures of this chapter deal with three primary types of weaving configuration. These are referred to as Type A, Type B, and Type C sections, and are shown in Figures 4-3, 4-4, and 4-5, respectively. The types are defined in terms of the minimum number of lane changes which *must* be made by weaving vehicles as they travel through the section.

Type A Weaving Areas

Type A weaving areas require that each weaving vehicle make one lane change in order to execute the desired movement. Figure 4-3 shows two examples of Type A weaving areas. In Figure 4-3(a), an on-ramp is followed by an off-ramp, with a continuous auxiliary lane between the ramps. All on-ramp vehicles must make a lane change out of the auxiliary lane into the shoulder lane of the freeway, and all off-ramp vehicles must make a lane change from the shoulder lane of the freeway to the auxiliary lane. Lane changes to and from the outer lanes of the freeway may also take place within the section, but these are not mandated or required by the weaving movement.

Sections formed by on-ramp/off-ramp sequences joined by continuous auxiliary lanes are often referred to as *ramp-weave sections.* They may also be referred to as *one-sided weaving sections,* because all weaving movements take place on one side of the roadway. It should be noted that on-ramps followed by off-ramps that are *not* joined by a continuous auxiliary lane are not considered to be weaving areas. They are treated as separate merge and diverge areas and analyzed using the procedures of Chapter 5.

Figure 4-3(b) illustrates a *major weaving section.* Major weaving sections are characterized by three or more entry and exit roadways having multiple lanes. In Figure 4-3(b), two two-lane sections join to form a four-lane roadway, only to separate into two two-lane sections again at the diverge point. Note that all weaving vehicles must make at least one lane change, regardless of the direction in which they are weaving.

Figures 4-3(a) and 4-3(b) are similar in that each has a *crown line,* that is, a lane line that connects the nose of the entrance gore area to the nose of the exit gore area. The lane change that each weaving vehicle must make is across this crown line.

The two sections illustrated differ primarily in the impact of ramp geometrics on speed. For many ramp-weave sections, the design speed of ramps is significantly lower than that of the freeway. Thus, on- or off-ramp vehicles must accelerate or decelerate as they traverse the weaving section. For major weaving sections, the design of multilane entry and exit legs is more compatible with the design of the freeway mainline, and the impact of acceleration and deceleration in the section is minimal. It should be noted, however, that this difference is not reflected in the procedures of this chapter because of the relative scarcity of major weave sites with crown lines and the lack of data concerning operations in such sites.

Because weaving vehicles in a Type A weaving area must cross the crown line, weaving vehicles are usually confined to occupying the two lanes adjacent to the crown line while in the weaving section. Normally, some nonweaving vehicles will also remain in lanes adjacent to the crown line. Lanes adjacent to the crown line are, therefore, generally shared by weaving and nonweaving vehicles. One of the most significant effects of configuration on operations is to limit the maximum number of lanes which weaving vehicles may occupy while traversing the section.

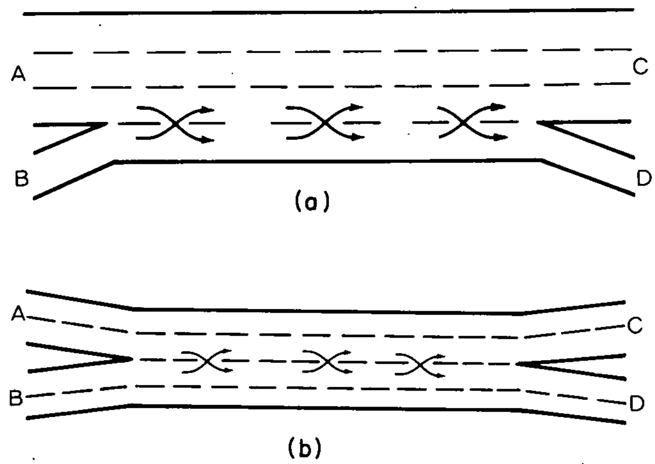


Figure 4-3. Type A weaving areas: (a) ramp-weave/one-sided weave, and (b) major weave with crown line.

Figures 4-4(a) and (b) show two such weaving areas. In both illustrations, movement B-C can be made without executing any lane changes, while movement A-D requires only one lane change. In Figure 4-4(a), this is accomplished by providing a diverging lane at the exit gore. From this lane, a vehicle may proceed on either exit leg without making a lane change. This type of design is also referred to as *lane balanced,* i.e., the number of lanes leaving the diverge is one greater than the number of lanes approaching it. In Figure 4-4(b), a lane from leg A is merged with a lane from leg B at the entrance gore area.

Type B Weaving Areas

All weaving areas classified as Type B may also be referred to as major weaving sections, because all involve multilane entry and/or exit legs. Two critical characteristics distinguish Type B weaving areas from all others:

1. One weaving movement may be accomplished without making any lane changes.
2. The other weaving movement requires at most one lane change.

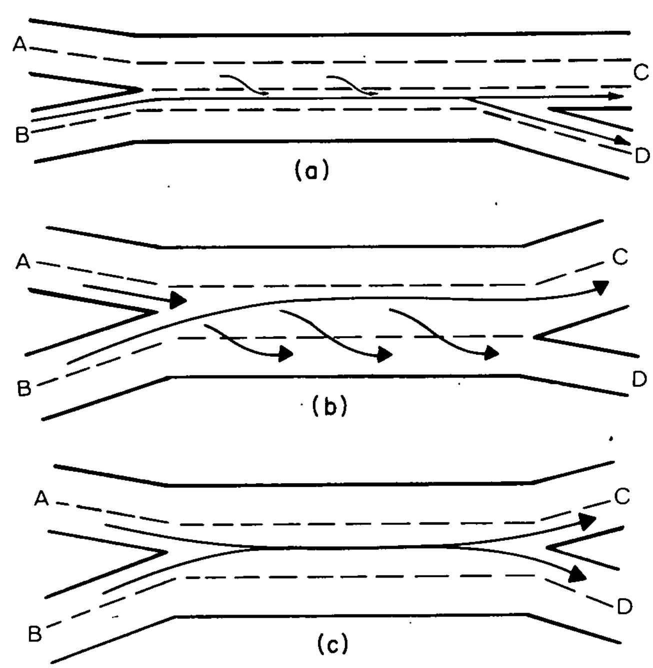


Figure 4-4. Type B weaving areas: (a) major weave with lane balance at exit gore, (b) major weave with merging at entrance gore, and (c) major weave with merging at entrance gore and lane balance at exit gore.

Type B weaving areas are extremely efficient in carrying large weaving volumes, primarily because of the provision of a "through lane" for one of the weaving movements. Weaving maneuvers can be accomplished with a single lane change from the lane or lanes adjacent to this "through lane." Thus, weaving vehicles can occupy a substantial number of lanes in the weaving section, and are not as restricted in this regard as in Type A sections.

Figure 4-4(c) shows an unusual configuration in which both a merge of two lanes at the entrance gore and lane balance at the exit gore are provided. In this case, *both* weaving movements can be made without a lane change. Again, weaving movements can be made with a single lane change from the two lanes adjacent to the "through lane." Such configurations are usually found on collector-distributor roadways. While some weaving movements are accomplished as a merge followed by a diverge, lane changes to and from lanes adjacent to the "through lane" yield real weaving activity, and these sections are analyzed as weaving areas.

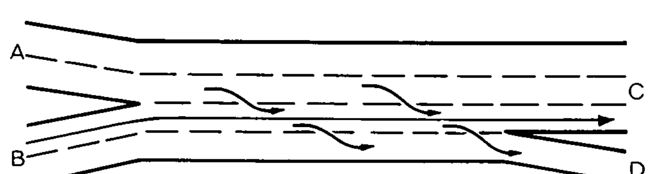
Type C Weaving Areas

Type C weaving areas are similar to Type B sections in that one or more "through lanes" are provided for one of the weaving movements. The distinguishing feature between Type B and Type C sections is the number of lane changes required for the other weaving movement. A Type C weaving area is characterized by:

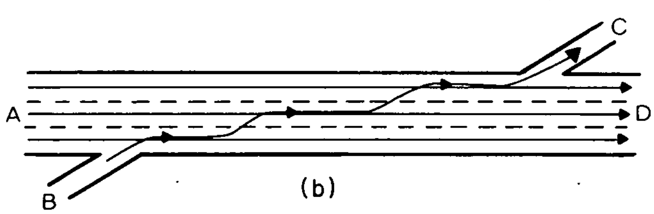
1. One weaving movement may be accomplished without making a lane change.
2. The other weaving movement requires two or more lane changes.

Figure 4-5 shows two Type C weaving areas. In Figure 4-5(a), movement B-C does not require lane-changing, while movement A-D requires *two* lane changes. This type of section is formed when there is neither a merging of lanes at the entrance gore nor lane balance at the exit gore and no crown line exists. While such a section is relatively efficient for weaving movements in the direction of the "through lane," it cannot efficiently handle large weaving volumes in the other direction.

Figure 4-5(b) shows a *two-sided weaving area.* It is formed when a right-hand on-ramp is followed by a left-hand off-ramp or vice-versa. In such cases, the through volume on the freeway is functionally a weaving movement. Ramp-to-ramp vehicles must cross all lanes of the freeway to execute their desired maneuver. Freeway lanes are, in effect, through weaving lanes. Ramp-to-ramp drivers must execute three lane changes in Figure 4-5(b). Although technically a Type C configuration, there is little information concerning the operation of such sections, and the methodology of this chapter is only a rough approximation of their characteristics. They should generally be avoided in cases where there is any significant ramp-to-ramp volume.



(a)



(b)

Figure 4-5. Type C weaving areas: (a) major weave without lane balance or merging, and (b) two-sided weave.

Determining Configuration Type

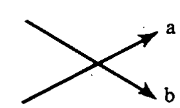
Figures 4-3, 4-4, and 4-5 show the three basic types of weaving area configuration. Weaving configuration is determined on the basis of the number of required lane changes that must be performed by the two weaving flows in the section. This determination ignores lane changes that are not necessary to the completion of a particular weaving movement. Table 4-1 identifies the configuration type based on lane-changing characteristics.

WEAVING WIDTH AND TYPE OF OPERATION

The third geometric characteristic with a significant impact on weaving area operations is the width of the weaving area, measured as the number of lanes in the section. It is, however, not only the total number of lanes that impacts weaving area operations, but the proportional use of those lanes by weaving and nonweaving vehicles.

The nature of weaving movements creates traffic stream turbulence, and results in a weaving vehicle consuming more of the available roadway space than a nonweaving vehicle. The

TABLE 4-1. CONFIGURATION TYPE VS. NUMBER OF REQUIRED LANE CHANGES



NUMBER OF REQ'D LANE CHANGES FOR WEAVING MVT. a	NUMBER OF REQ'D LANE CHANGES FOR WEAVING MVT. b: 0	1	≥ 2
0	Type B	Type B	Type C
1	Type B	Type A	—
≥ 2	Type C	—	—

exact nature of relative space use depends on the relative weaving and nonweaving volumes using the weaving area and the number of lane changes weaving vehicles must make. The latter is, as discussed, dependent on the configuration of the weaving section. Thus, the proportional use of space is dependent not only on relative volumes, but on the configuration of the weaving area.

Configuration has a further impact on proportional use of available lanes. The configuration can limit the ability of weaving vehicles to use outer lanes in the section. This limitation is most severe in Type A sections, in which all weaving vehicles must cross a crown line, and is least severe in Type B sections.

In general, vehicles in a weaving area will make use of available lanes in such a way that all component flows achieve approximately the same average running speed, with weaving flows somewhat slower than nonweaving flows. Occasionally, the configuration limits the ability of weaving vehicles to occupy the proportion of available lanes required to achieve this equivalent or balanced operation. In such cases, weaving vehicles occupy a smaller proportion of the available lanes than desired, while nonweaving vehicles occupy a larger proportion of lanes than for balanced operation. When this occurs, the operation of the weaving area is classified as *constrained* by the configuration. The result of constrained operation is that nonweaving vehicles will operate at significantly higher speeds than weaving vehicles.

Where configuration does not restrain weaving vehicles from occupying a balanced proportion of available lanes, the operation is classified as *unconstrained.* Average running speeds of weaving and nonweaving vehicles generally differ by less than 5 mph, except in short Type A sections, where acceleration and deceleration of ramp vehicles limit their average speed regardless of the use of available lanes.

A major component of the procedure presented in this chapter is the determination of whether operations in a given section are constrained or unconstrained. This is discussed in the "Methodology" section.

WEAVING AREA PARAMETERS

The introductory portions of this chapter have discussed a number of parameters that may affect the operation of weaving areas. For convenience, Table 4-2 presents these measures and defines the symbols that will be used to depict them.

TABLE 4-2. PARAMETERS AFFECTING WEAVING AREA OPERATION

SYMBOL	DEFINITION
L	Length of weaving area, in ft.
L_H	Length of weaving area, in hundreds of ft.
N	Total number of lanes in the weaving area.
N_w	Number of lanes used by weaving vehicles in the weaving area.
N_{nw}	Number of lanes used by nonweaving vehicles in the weaving area.
v	Total flow rate in the weaving area, in passenger car equivalents, in pcph.
v_w	Total weaving flow rate in the weaving area, in passenger car equivalents, in pcph.
v_{w1}	Weaving flow rate for the larger of the two weaving flows, in passenger car equivalents, in pcph.
v_{w2}	Weaving flow rate for the smaller of the two weaving flows, in passenger car equivalents, in pcph.
v_{nw}	Total nonweaving flow rate in the weaving area, in passenger car equivalents, in pcph.
VR	Volume ratio v_w/v.
R	Weaving ratio v_{w2}/v_w.
S_w	Average running speed of weaving vehicles in the weaving area, in mph.
S_{nw}	Average running speed of nonweaving vehicles in the weaving area, in mph.

II. METHODOLOGY

The methodology presented in this chapter has four distinct components:

1. Equations predicting the average running speed of nonweaving and weaving vehicles in a weaving area based on known roadway and traffic conditions. Equations are specified for each configuration type, and for unconstrained and constrained operations.
2. Equations describing the proportional use of available lanes by weaving and nonweaving vehicles, used to determine whether operations are constrained or unconstrained.
3. Definitions of limiting values of key parameters for each type of weaving configuration, beyond which equations do not apply.
4. Definition of level-of-service criteria based on average running speeds of weaving and nonweaving vehicles.

These components are discussed in the sections that follow.

PREDICTION OF WEAVING AND NONWEAVING SPEEDS

A series of 12 equations is used to predict weaving and nonweaving speeds in a weaving section. For each type of configuration (A, B, or C) and type of operation (constrained, unconstrained), equations for S_w and S_{nw} have been calibrated. These equations predict the average running speed of weaving and nonweaving vehicles during a 15-min interval based on the weaving and nonweaving flow rates for that period, expressed in passenger car equivalents for ideal conditions.

The equations are of a common format, as follows:

$$S_w \text{ or } S_{nw} = 15 + \frac{50}{1 + a(1 + VR)^b (v/N)^c / L^d} \qquad (4\text{-}1)$$

where:

a, b, c, d = constants;
S_w = average running speed of weaving vehicles, in mph;
S_{nw} = average running speed of nonweaving vehicles, in mph;
VR = volume ratio;
v = total flow rate in the weaving area, in pcph;
N = total number of lanes in the weaving area; and
L = length of the weaving area, in ft.

The general form of the equation is asymptotic at 15 mph and 65 mph, and keeps all speed predictions to a reasonable range. As length increases, speed increases, because the intensity of lane-changing declines. Both nonweaving and weaving speeds decline as the proportion of weaving vehicles in total flow, VR, increases, a reflection of the increased turbulence present with larger proportions of weaving vehicles in the section. As the average total flow rate per lane, v/N, increases, speeds of both weaving and nonweaving vehicles decrease.

Table 4-3 lists the constants used to develop each of the 12 speed prediction equations.

The sensitivities displayed by the equations resulting from the application of the constants in Table 4-3 follow logical and expected patterns. Some key aspects of these sensitivities follow:

1. Constrained operations have lower weaving speeds and higher nonweaving speeds than equivalent unconstrained operation. The difference between nonweaving and weaving speeds becomes significant in constrained operations.
2. Type B sections are the most efficient for handling high weaving flow rates. For high flow rates, weaving speeds are higher than for equivalent Type A or Type C sections.
3. Because of the multiple lane changes required by some weaving vehicles in a Type C section, weaving and nonweaving speeds are low when heavy weaving flows are present.
4. The sensitivity of weaving speed to increasing VR is great-

TABLE 4-3. CONSTANTS FOR PREDICTION OF WEAVING AND NONWEAVING SPEEDS IN WEAVING AREAS

GENERAL FORM:

$$S_w \text{ or } S_{nw} = 15 + \frac{50}{1 + a\,(1 + VR)^b\,(v/N)^c / L^d}$$

TYPE OF CONFIGURATION	CONSTANTS FOR WEAVING SPEED, S_w a	b	c	d	CONSTANTS FOR NONWEAVING SPEED, S_{nw} a	b	c	d
TYPE A								
Unconstrained	0.226	2.2	1.00	0.90	0.020	4.0	1.30	1.00
Constrained	0.280	2.2	1.00	0.90	0.020	4.0	0.88	0.60
TYPE B								
Unconstrained	0.100	1.2	0.77	0.50	0.020	2.0	1.42	0.95
Constrained	0.160	1.2	0.77	0.50	0.015	2.0	1.30	0.90
TYPE C								
Unconstrained	0.100	1.8	0.80	0.50	0.015	1.8	1.10	0.50
Constrained	0.100	2.0	0.85	0.50	0.013	1.6	1.00	0.50

est for Type A configurations and least for Type B configurations. This illustrates the greater efficiency of Type B sections in handling large proportions of weaving flows in the traffic stream. It also suggests that Type A sections are most effective where the proportion of weaving vehicles in the traffic stream is low.

5. The sensitivity of weaving speed to increasing length is greatest for Type A sections, as vehicles are often accelerating or decelerating through the section for this configuration. The sensitivity of weaving speed to length is less for Type B and C configurations, where at least one weaving movement does not require lane-changing.

It is also important to note that Type A configurations are quite different from Type B and Type C configurations. As all weaving vehicles must cross a crown line in Type A configurations, weaving and nonweaving flows tend to segregate in such sections, with weaving vehicles concentrating in lanes adjacent to the crown line, and nonweaving vehicles gravitating to the outer lanes. In Type B and C configurations, there is substantial mixing of weaving and nonweaving vehicles across a number of lanes.

This difference makes Type A sections behave somewhat differently from sections of either Type B or Type C. Speeds tend to be higher in a Type A section than for a Type B or Type C section with the same flows, length, and number of lanes. This does not suggest that Type A sections are always superior, however, as there are restrictions on the types of flows that they can accommodate which are more severe than for other types, as is discussed in a subsequent section.

DETERMINATION OF TYPE OF OPERATION

The determination of whether a particular section is operating in a constrained or unconstrained state is based on the comparison of two variables:

N_w = the number of lanes that must be used by weaving vehicles in order to achieve balanced or unconstrained operation; and

$N_w(\max)$ = the maximum number of lanes that may be used by weaving vehicles for a given configuration.

Fractional values for lane requirements of weaving vehicles may occur because lanes are shared with nonweaving vehicles.

Cases for which $N_w \leq N_w(\max)$ will be unconstrained, because there are no impediments to weaving vehicles using the required number of lanes. Where $N_w > N_w(\max)$, the configuration constrains weaving vehicles to a smaller number of lanes than required for balanced operation. Such cases are constrained, and will result in average nonweaving vehicle speeds significantly higher than average weaving vehicle speeds.

Table 4-4 contains equations for the computation of N_w, and values for $N_w(\max)$, both of which vary with the type of configuration.

The equations for N_w are based on weaving and nonweaving speeds for unconstrained operation. Computed values are compared to the maximum values shown in the third column of Table 4-4 to determine whether operations are constrained or unconstrained. Values of $N_w(\max)$ in Table 4-4 reflect observations in the data bases reported in Refs. *1*, *2*, and *4*.

Type A sections are the most restrictive in terms of the maximum number of lanes that can be used by weaving vehicles. As noted previously, weaving vehicles must, in general, confine themselves to the two lanes adjacent to the crown line in order to execute their desired maneuvers. However, nonweaving vehicles will also remain in these lanes, and full use of them by weaving vehicles is not a reasonable expectation. For Type A sections, weaving vehicles generally use at most 1.4 lanes, regardless of the total number of lanes available.

Type B sections do not greatly restrict weaving vehicles in their use of available lanes. Weaving vehicles may occupy up to 3.5 lanes in a Type B section. This is based on the full use of "through" weaving lanes and lanes immediately adjacent to the through lane, as well as partial use of outer lanes. Such configurations are most efficient when weaving flows comprise substantial portions of the traffic stream. Because weaving vehicles may filter through most of the lanes in the segment, nonweaving vehicles tend to share lanes, and are generally unable to segregate themselves from weaving flows.

Type C sections are similar to Type B sections in the provision of a "through" weaving lane. The multiple lane-changing required of one weaving movement, however, restricts the ability of weaving vehicles to use outer lanes of the sections. Thus, in Type C sections, weaving vehicles can use no more than 3.0 lanes. One exception to this rule is a two-sided weaving area (see Fig. 4-5(b)). For two-sided configurations, all freeway lanes are "through" weaving lanes, and weaving vehicles may therefore use all lanes without restriction.

The proportional use of available lanes by weaving vehicles is again quite different for Type A sections as compared to

TABLE 4-4. CRITERIA FOR UNCONSTRAINED VS. CONSTRAINED OPERATION OF WEAVING AREAS[a]

TYPE OF CONFIGURATION	NO. OF LANES REQ'D FOR UNCONSTRAINED OPERATION, N_w	MAX. NO. OF WEAVING LANES, N_w (max)
Type A	$2.19\ N\ VR^{0.571}\ L_H^{0.234}/S_w^{0.438}$	1.4
Type B	$N\{0.085 + 0.703\ VR + (234.8/L) - 0.018(S_{nw} - S_w)\}$	3.5
Type C	$N\{0.761 - 0.011\ L_H - 0.005(S_{nw} - S_w) + 0.047\ VR\}$	3.0 [b]

[a] All variables are as defined in Table 4-2.
[b] For 2-sided weaving areas, *all* freeway lanes may be used as weaving lanes.
NOTE: When $N_w \leq N_w(\max)$, operation is unconstrained.
When $N_w > N_w(\max)$, operation is constrained.

TABLE 4-5. LIMITATIONS ON WEAVING AREA EQUATIONS

TYPE OF CONFIGURATION	WEAVING CAPACITY, MAXIMUM v_w	MAXIMUM v/N	MAXIMUM VOL. RATIO, VR	MAXIMUM WEAVING RATIO, R	MAXIMUM WEAVING LENGTH, L
Type A	1,800 pcph	1,900 pcphpl	N — VR 2 — 1.00 3 — 0.45 4 — 0.35 5 — 0.22	0.50	2,000 ft
Type B	3,000 pcph	1,900 pcphpl	0.80	0.50	2,500 ft
Type C	3,000 pcph	1,900 pcphpl	0.50	0.40	2,500 ft

NOTE: Type C limitations do not apply to two-sided weaving areas.

Types B and C sections. In Type A sections, more lanes are required by weaving vehicles for balanced operation as length increases. This is primarily due to the substantial segregation of weaving and nonweaving flows in such sections, and the higher speeds of weaving vehicles which result. As length increases, weaving speeds become quite high, and more space is required by weaving vehicles to maintain these speeds. This characteristic produces, however, an interesting result. For any given set of flows and number of lanes, it is more likely for a Type A section to operate in the constrained mode as length is increased.

Type B and Type C sections show an opposite trend. Increasing length has a much smaller impact on weaving speed than for Type A sections, primarily because of the mixing of weaving and nonweaving flows. As length increases, the proportion of lanes required by weaving vehicles for balanced operation decreases, and it is less likely that constrained operation will occur.

LIMITS ON WEAVING AREA OPERATIONS

Table 4-5 gives a number of limitations on the application of this methodology which may not be obvious from either the speed or lane use equations described previously. These include maximum weaving flow rates, total flow rate per lane, volume ratios, and weaving ratios at which the various configuration types generally operate, as well as length limits beyond which merge and diverge areas may operate independently.

The interpretation of each of these limitations varies. In the case of limitations on weaving flow rate, v_w, and total flow rate per lane, v/N, acceptable operations are unlikely beyond these values. They are, therefore, maximum values which may be accommodated in a weaving section, within the maximum lengths shown in Table 4-5. Limits on volume ratio, VR, and weaving ratio, R, represent values beyond which weaving operations are rarely observed. Higher values may occur, but these fall outside the prediction range of the methodology, and results should be taken as approximate. Length limitations, L, represent the range of the calibration data base. As noted previously, weaving may occur in longer sections. In such cases it is generally considered that merging and diverging maneuvers tend to segregate, and that the procedures of Chapter 5 may be applied. Speeds in longer sections tend to approach those which would be achieved in a basic freeway section, even where some weaving turbulence exists.

The weaving capacity of a Type A section is limited to a flow rate of 1,800 pcph. This is because all weaving vehicles must cross a single crown line, restricting the number of vehicles that may cross from one side of the section to the other. Type B and Type C sections can accommodate weaving flow rates up to 3,000 pcph, due to the flexibility in the use of lanes by weaving vehicles provided by such configurations. It is critical to note that weaving flow rates higher than these values cannot normally be accommodated in a weaving section within the length ranges of Table 4-5. As the length is increased beyond the range shown, maximum weaving flow rates are difficult to define. When the length increases to a point where weaving lane-changing is no greater than the lane-changing occurring on a basic freeway segment, weaving flow rates are unrestricted. As the length needed to achieve this, however, cannot be defined precisely, analysts and designers should view with caution any weaving flow rates in excess of the Table 4-5 values. Changes in the basic design of the freeway system, including provision of grade separations, may be considered to accommodate higher flows.

A maximum limitation on v/N of 1,900 pcphpl should also be observed in weaving areas within the length limits of Table 4-5. The additional turbulence due to the presence of weaving movements makes attainment of average lane flows in excess of this value unlikely.

Limitations on volume ratio, VR, reflect the character of each configuration type. Type A sections are intended to handle small weaving flows comprising a minority of the traffic stream. As weaving vehicles do not normally use more than 1.4 lanes in such sections, the limiting VR depends on the total number of lanes available, and decreases as N increases. Freeway weaving areas with Type A configurations generally should not be used where weaving traffic comprises a proportion of total flow larger than that shown in Table 4-5.

Type C configurations are more generous in handling larger proportions of weaving traffic, but are still not efficient where weaving flows dominate total flow. Only Type B configurations effectively handle situations in which $VR > 0.50$ and $N > 2$.

The weaving ratio, R, is the ratio of the smaller weaving flow to the total weaving flow. Its maximum value is 0.50, which occurs when the two weaving flows are equal. Neither Type A nor Type B configurations have any practical limitation on R, as both can accommodate equal weaving flows without operational problems. Type C configurations, however, are most efficient where weaving flows are unequal. This is because one weaving movement requires no lane-changing, while the other requires two or more lane changes. Such sections generally do not operate efficiently when the weaving ratio exceeds 0.40, with the larger flow in the direction requiring no lane changes.

LEVEL-OF-SERVICE CRITERIA

Levels of service in weaving areas are directly related to the average running speeds of weaving and nonweaving vehicles. A level of service is separately assigned to weaving and nonweaving vehicles to reflect cases in which significant differences in the speed of component flows exist, as well as those in which balanced operation occurs. The criteria are given in Table 4-6.

Unlike basic freeway sections, in which speed is insensitive to flow rates up to approximately 1,600 pcphpl, speed in weaving areas is sensitive to flow rates throughout the range of stable flows. This is due to the additional turbulence caused by weaving vehicles and their lane-changing maneuvers.

In general, speed of weaving vehicles is expected to be somewhat lower than that of nonweaving vehicles even when balanced or unconstrained operation occurs. This difference tends to get smaller as speeds get lower. This is reflected in the criteria defined in Table 4-6.

Level-of-service F is defined as any speed below 35 mph for either weaving or nonweaving vehicles when computed speeds are used. This is a result of the speed prediction equations used in this chapter. The equations tend to somewhat overpredict low speeds, and predictions of lower than 30 mph are difficult to obtain, even where the average flow rate per lane is in excess of 1,900 pcphpl. The use of 35 mph as the boundary for level-of-service F adjusts for this characteristic of the equations, and results in the more accurate identification of cases in which breakdowns will occur. When LOS criteria are to be compared to measured speeds, a 30-mph value is used.

The speed criteria for any given level of service are generally several mph lower than similar criteria for a basic freeway section with a 70-mph design speed. This allows for reasonable consistency with the levels of service defined in Chapter 3. It is possible, however, that a given weaving section will operate at a better LOS than a basic freeway section with equal flows and the same number of lanes because of the lower speed criteria for weaving sections. This is an unusual result, and is consistent with the LOS definitions established in Chapter 3 and herein.

TABLE 4-6. LEVEL-OF-SERVICE CRITERIA FOR WEAVING SECTIONS

LEVEL OF SERVICE	MIN. AVG. WEAVING SPEED, S_w (MPH)	MIN. AVG. NONWEAVING SPEED, S_{nw} (MPH)
A	55	60
B	50	54
C	45	48
D	40	42
E	35/30 [a]	35/30 [a]
F	< 35/30 [a]	< 35/30 [a]

[a] The 35-mph boundary for LOS E/F is used when comparing to computed speeds using the equations of Table 4-3. The 30-mph boundary is used for comparison to field-measured speeds.

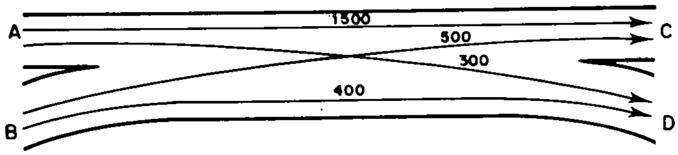
III. PROCEDURES FOR APPLICATION

SIMPLE WEAVING AREAS

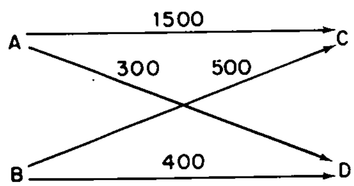
Procedural steps for the analysis of simple weaving areas are given below. Computations are performed in the *operational analysis* mode, i.e., a known or projected situation is analyzed for the probable level of service. All roadway and traffic conditions must be specified, including weaving length, type of configuration, number of lanes, lane widths, terrain or grade, weaving and nonweaving flow rates by movement, the peak-hour factor, and traffic composition.

Weaving analysis is made easier through the use of a *weaving diagram.* A weaving diagram is a schematic drawing showing weaving and nonweaving flows in a weaving area. Figure 4-6 shows the construction of such a diagram. Note that the weaving diagram depicts actual flows in a straight-line form. The relative placement of entry and exit points (A,B,C,D) in the diagram matches the actual site to ensure proper placement of weaving and nonweaving flows relative to each other. Flows on the weaving diagram should represent flow rates for the peak 15-min under ideal conditions, expressed in pcph. It is also convenient to use the weaving diagram as a guide in computing the parameters used during an analysis.

Evaluation of the level of service in an existing or projected weaving area is accomplished using the following computational steps.



SECTION AND FLOWS



WEAVING DIAGRAM

THEN:

v_{w1} = WEAVING FLOW WITH THE HIGHEST NUMERIC VALUE (500)

v_{w2} = WEAVING FLOW WITH THE SMALLEST NUMERIC VALUE (300)

v_w = TOTAL WEAVING FLOW (500 + 300 = 800)

v = TOTAL FLOW IN THE WEAVING AREA (500+300+1500+400 = 2700)

VR = VOLUME RATIO = v_w/v (800/2700 = 0.296)

R = WEAVING RATIO = v_{w2}/v_w (300/800 = 0.375)

L = WEAVING LENGTH = 1,200 FT

N = NUMBER OF LANES = 4

Figure 4-6. Construction and use of weaving diagrams.

Step 1—Establish Roadway and Traffic Conditions

All existing or projected roadway and traffic conditions must be specified. Roadway conditions include the length, number of lanes, and type of configuration for the weaving area under study. Table 4-1 should be consulted in assigning the type of configuration. Other roadway features of importance are lane widths and the general terrain or grade conditions for the section.

Traffic conditions include the distribution of vehicle types in the traffic stream, as well as the peak-hour factor or peak-hour factors where the component flows have differing peaking characteristics.

As the weaving area should be analyzed on the basis of peak flow rates for a 15-min interval within the hour of interest, hourly volumes must be adjusted by dividing by the peak-hour factor. Such a conversion, however, ignores the fact that the four component flows in a weaving area may not all peak during the same interval. Where possible, weaving flows should be observed and recorded for 15-min intervals, so that critical periods may be identified for analysis. Where hourly volumes are available or projected, it will be assumed that all component flows peak simultaneously—a conservative procedure. The predicted speeds of weaving and nonweaving vehicles will be lower than those actually occurring in such cases. It should also be noted that the component movements in a weaving area may not have the same peak-hour factor. Where possible, each flow and its peaking characteristics should be considered separately.

Step 2—Convert all Traffic Volumes to Peak Flow Rates Under Ideal Conditions

As all of the speed and lane-use algorithms presented earlier are based on peak flow rates under ideal conditions, expressed in pcph, all component flows must be converted to this basis:

$$v = \frac{V}{\text{PHF} \times f_{HV} \times f_w \times f_p} \tag{4-2}$$

where:

v = flow rate for peak 15 min, in pcph under ideal conditions;
V = hourly volume, in vph, under prevailing conditions;
PHF = peak-hour factor;
f_{HV} = heavy vehicle adjustment factor, determined using the procedures of Chapter 3;
f_w = lane width and lateral clearance adjustment factor, determined using the procedures of Chapter 3; and
f_p = driver population adjustment factor, determined using the procedures of Chapter 3.

Step 3—Construct Weaving Diagram

A weaving diagram of the type illustrated in Figure 4-6 is now constructed, with all flows indicated as peak flow rates under ideal conditions, in pcph. Critical analysis variables are identified and computed as shown in Figure 4-6.

Step 4—Compute Unconstrained Weaving and Nonweaving Speeds

Using the unconstrained equations for the appropriate configuration from Table 4-3, compute the predicted values of average running speed for weaving vehicles, S_w, and nonweaving vehicles, S_{nw}.

Step 5—Check for Constrained Operation

Using the speeds computed in Step 4, estimate the number of lanes needed by weaving vehicles to achieve unconstrained operation using the equations in Table 4-4. Compare the computed value of N_w to the tabulated value of N_w (max) to determine whether operation is constrained or unconstrained.

If $N_w \leq N_w$ (max), the operation is *unconstrained,* and the speeds computed in Step 4 are accurate. If $N_w > N_w$ (max), the operation is *constrained.* Values of S_w and S_{nw} must be recomputed using the constrained equations for the appropriate configuration given in Table 4-3.

Step 6—Check Weaving Area Limitations

Table 4-5 should be consulted to ensure that none of the limitations specified for speed predictions are exceeded. Where one or more of these limits are exceeded, consult the "Methodology" section of this chapter for the appropriate interpretation.

Step 7—Determine the Level of Service

The estimated values of S_w and S_{nw} are compared to the LOS criteria in Table 4-6 to determine the prevailing level of service.

Care should be taken in applying the limiting values given in Table 4-5. Where the weaving capacity is exceeded, it is likely that breakdowns will occur and that level-of-service F will prevail, at least for weaving vehicles. Where limitations on VR or R are exceeded, breakdowns need not occur, but speeds would be lower than those anticipated by the equations of Table 4-3. Maximum lengths reflect the limits of the predictive equations. Lengths beyond the values shown may be analyzed as separate merge and diverge areas using the procedures of Chapter 5. It would not be expected that speeds within the section would be significantly lower than those for a basic freeway section serving the same volume.

MULTIPLE WEAVING AREAS

Multiple weaving areas are formed when one merge point is followed closely by two diverge points, or where two merge points are closely followed by a single diverge point. In such cases, several sets of weaving movements take place over the same segments of freeway, and lane-changing turbulence may be higher than that found in simple weaving areas.

Drivers will carefully select where to execute their required lane changes in a manner that minimizes interference with other weaving movements. Figures 4-7 and 4-8 show the two types of multiple weaving areas, and where weaving movements are most likely to take place. This results in the formation of weaving diagrams for each subsegment of the weaving area, each of which can be analyzed as a simple weaving area using the procedures specified earlier.

Figure 4-7 depicts a single merge area followed by two diverge areas. The weaving of movement 5 with movements 3 and 4 must take place in the first segment, as vehicles in movement 5 leave at the first diverge point. The weaving of movement 2 with movement 3 may take place anywhere in either segment of the section. However, to avoid the turbulence of weaving that must take place in the first segment, these latter weaving movements will tend to concentrate in the second segment of the section.

Figure 4-8 depicts two merge areas followed by a single diverge area. In this case, the weaving of movements 3 and 4 with movement 5 must take place in the second segment of the section, as movement 5 enters at the second merge. While the weaving of movements 2 and 3 could take place anywhere in the section, it will tend to concentrate in the first segment, as drivers seek to avoid the turbulence of other weaving movements in the second segment.

Thus, the analysis of multiple weaving areas involves the construction of appropriate weaving diagrams for each subsegment of the area using Figures 4-7 and 4-8. Once these diagrams are established, each subsegment may be analyzed as a simple weaving area, according to the procedures of this chapter. Limits established in Table 4-5 would apply to the individual subsegments.

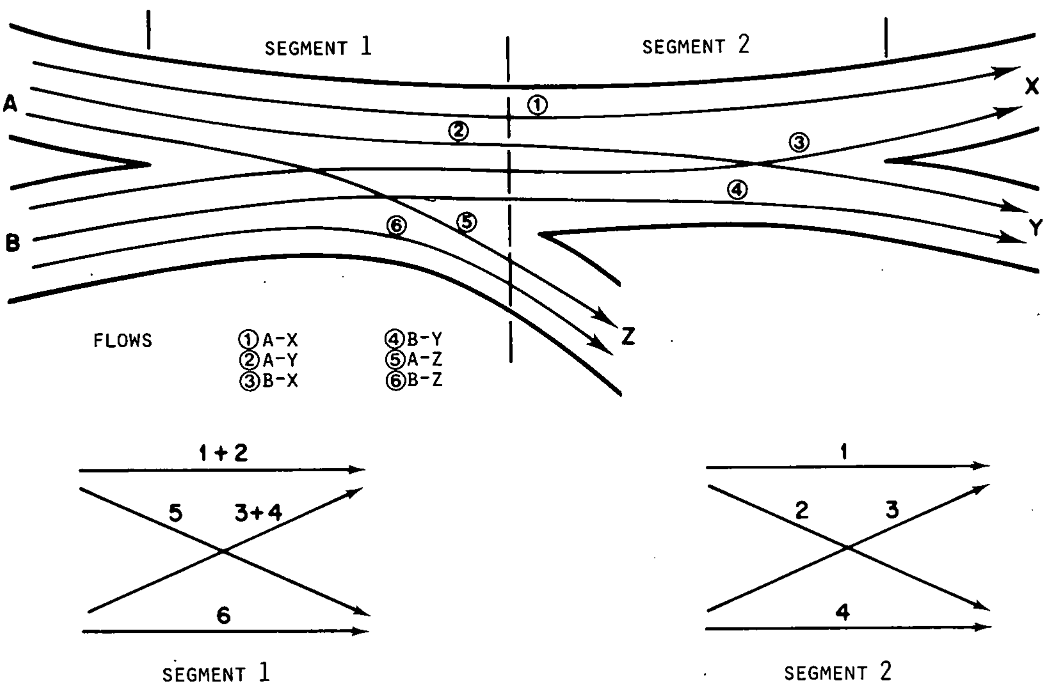


Figure 4-7. Weaving flows in a multiple weave formed by a single merge followed by two diverges.

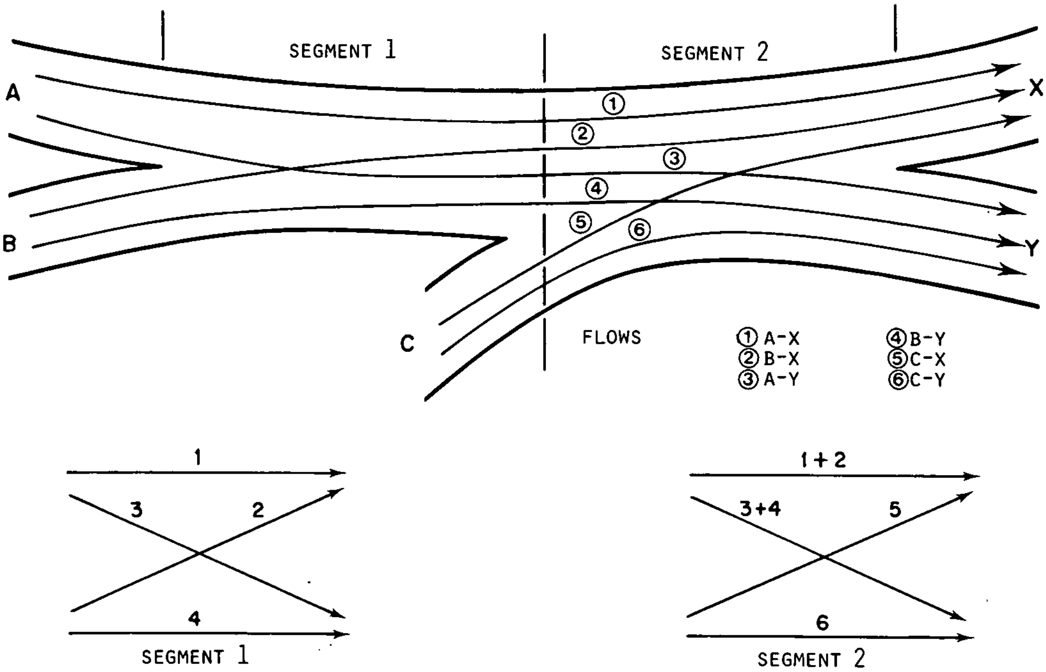


Figure 4-8. Weaving flows in a multiple weave formed by two merge points followed by a single diverge.

IV. SAMPLE CALCULATIONS

The following sample calculations illustrate the application and interpretation of the methodology presented in this chapter.

CALCULATION 1—ANALYSIS OF A MAJOR WEAVING AREA

1. *Description*—The weaving area illustrated in Figure 4-9 serves the following traffic volumes: A-C = 1,815 vph; A-D = 692 vph; B-C = 1,037 vph; B-D = 1,297 vph. Traffic volumes include 7 percent trucks, and the PHF is 0.91. The section is located in generally level terrain, and lane widths are 12 ft. There are no lateral obstructions. The driver population is composed primarily of commuters. At what LOS will the section operate?

2. *Solution*—The calculation is conducted according to the steps outlined in the "Procedures for Application" section of this chapter.

a. The existing geometrics and traffic volumes are stated in the description. Note that the section is a Type B configuration (see Table 4-1). Weaving movement B-C may be made without a lane change, while movement A-D can be made with a single lane change.

b. All volumes must be converted to peak flow rates under ideal conditions, expressed in passenger cars per hour.

$$v = \frac{V}{\text{PHF} \times f_{HV} \times f_w \times f_p}$$

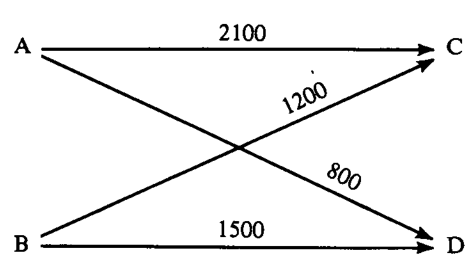
where:

PHF = 0.91 (Given);
E_T = 1.7 (Table 3-3);
f_{HV} = 0.95 (Computed as $1/[1 + 0.07(1.7 - 1)]$);
f_w = 1.00 (Table 3-2); and
f_p = 1.00 (Table 3-10).

Then:

A-C = 1,815/(0.91 × 0.95 × 1.00 × 1.00) = 2,100 pcph
A-D = 692/(0.91 × 0.95 × 1.00 × 1.00) = 800 pcph
B-C = 1,037/(0.91 × 0.95 × 1.00 × 1.00) = 1,200 pcph
B-D = 1,297/(0.91 × 0.95 × 1.00 × 1.00) = 1,500 pcph

c. A weaving diagram for the calculation is now constructed, using the converted flow rates of step b:



Critical ratios may also be computed for use in analysis:

$$v_w = 1{,}200 + 800 = 2{,}000 \text{ pcph}$$
$$v = 2{,}000 + 2{,}100 + 1{,}500 = 5{,}600 \text{ pcph}$$
$$R = 800/2{,}000 = 0.400$$
$$VR = 2{,}000/5{,}600 = 0.357$$

d. The *unconstrained* speeds of weaving and nonweaving vehicles in the section may be estimated by using the equations for Type B configurations from Table 4-3.

$$S_w = 15 + \frac{50}{1 + 0.10(1 + VR)^{1.2}(v/N)^{0.77}/L^{0.5}}$$

$$S_{nw} = 15 + \frac{50}{1 + 0.02(1 + VR)^{2.0}(v/N)^{1.42}/L^{0.95}}$$

where:

$VR = 0.357$;
$v = 5{,}600$ pcph;
$N = 4$; and
$L = 1{,}500$ ft.

Then:

$$S_w = 15 + \frac{50}{1 + 0.10(1 + 0.357)^{1.2}(5{,}600/4)^{0.77}/1{,}500^{0.5}}$$

$S_w = 40.2$ mph *or* Say 40 mph

$$S_{nw} = 15 + \frac{50}{1 + 0.02(1 + 0.357)^{2.0}(5{,}600/4)^{1.42}/1{,}500^{0.95}}$$

$S_{nw} = 39.5$ mph *or* Say 40 mph

e. Using the constrained estimates of weaving and nonweaving speeds, the number of weaving lanes required to achieve such operation is computed and compared with the maximum value of 3.5 lanes for a Type B configuration. The equation is obtained from Table 4-4.

$$N_w = N\{0.085 + 0.703\, VR + (234.8/L) - 0.018(S_{nw} - S_w)\}$$

$$N_w = 4\{0.085 + 0.703(0.357) + (234.8/1{,}500) - 0.018(39.5 - 40.2)\}$$

$N_w = 2.02$ lanes $<$ 3.50 lanes

Therefore, the section will operate in the *unconstrained mode*, and the speeds computed in step d are the final solution.

f. All values for the calculation are below the limits established in Table 4-5, and the operation is expected to be as indicated in the computations in previous steps.

g. Comparing to the criteria of Table 4-6 shows that the level of service for weaving vehicles is D, while the level of service for nonweaving vehicles is E.

CALCULATION 2—ANALYSIS OF A RAMP-WEAVE SECTION

1. *Description*—The weaving section shown in Figure 4-10 serves the traffic flows indicated. Lane widths are 12 ft and the section is located in level terrain. There are no lateral obstructions. For convenience, all traffic flows are given in terms of peak flow rates for ideal conditions, expressed in passenger cars per hour. At what LOS will the section operate?

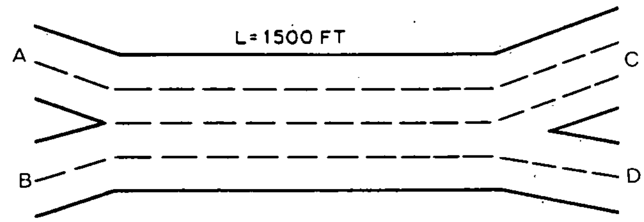


Figure 4-9. Weaving area for Calculation 1.

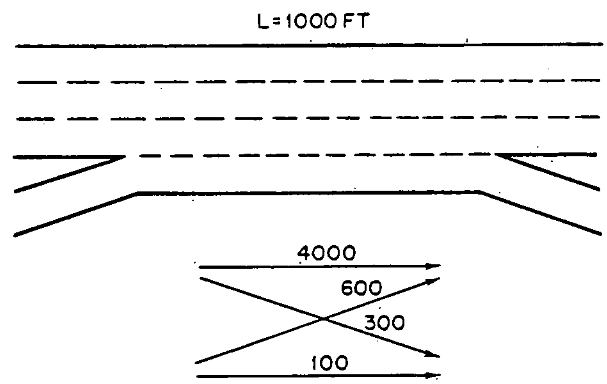


Figure 4-10. Weaving area and flows for Calculation 2.

2. *Solution*—

a. All prevailing traffic and roadway conditions are specified in the calculation description and in Figure 4-10. Note that this is a Type A configuration, because both weaving movements are required to make one lane change.

b. No conversions of stated traffic demands are required, because they are given in terms of peak flow rates under ideal conditions, expressed in passenger cars per hour.

c. The weaving diagram is shown in Figure 4-10. Critical ratios may be computed as:

$$v_w = 600 + 300 = 900 \text{ pcph}$$
$$v = 900 + 4{,}000 + 100 = 5{,}000 \text{ pcph}$$
$$VR = 900/5{,}000 = 0.18$$
$$R = 300/900 = 0.33$$

d. Equations for weaving and nonweaving speed are selected from Table 4-3 for Type A unconstrained operations:

$$S_w = 15 + \frac{50}{1 + 0.226(1 + VR)^{2.2}(v/N)^{1.0}/L^{0.9}}$$

$$S_{nw} = 15 + \frac{50}{1 + 0.020(1 + VR)^{4.0}(v/N)^{1.3}/L^{1.0}}$$

where:

$VR = 0.18$;
$v = 5{,}000$ pcph;
$N = 4$; and
$L = 1{,}000$ ft.

Then:

$$S_w = 15 + \frac{50}{1 + 0.226(1 + 0.18)^{2.2}(5{,}000/4)^{1.0}/1{,}000^{0.9}}$$

$S_w = 42.6$ mph *or* Say 43 mph

$$S_{nw} = 15 + \frac{50}{1 + 0.020(1 + 0.18)^{4.0}(5{,}000/4)^{1.3}/1{,}000^{1.0}}$$

$S_{nw} = 50.4$ mph *or* Say 50 mph

e. The required number of weaving lanes is computed and compared to the maximum value of 1.4 lanes for Type A configurations by using the unconstrained speed estimates of step d. The equation for N_w and the maximum value of 1.4 lanes are taken from Table 4-4.

$$N_w = 2.19\ N\ VR^{0.571}\ L_H^{0.234}/S_w^{0.438}$$

$$N_w = 2.19(4)(0.18)^{0.571}(10)^{0.234}/42.6^{0.438}$$

$$N_w = 1.09 \text{ lanes} < 1.40 \text{ lanes}$$

Therefore, the section will operate in the unconstrained mode, and the speed estimates of step d are correct.

f. None of the limiting values given in Table 4-5 is exceeded by the section under study, and the calculations of previous steps would, therefore, be considered to be appropriate.

g. Comparing the estimated weaving and nonweaving speeds to the criteria of Table 4-6, it is found that weaving vehicles experience level-of-service D, while nonweaving vehicles experience level-of-service C. The disparity in levels of service is due to the 7-mph difference in nonweaving and weaving speeds. This difference is primarily due to the length of the section, which limits the ability of ramp vehicles to accelerate or decelerate as they pass through the section, and is not a reflection of "constrained" operation.

CALCULATION 3—A CONSTRAINED OPERATION

1. *Description*—The ramp-weave section shown in Figure 4-11 serves the following demand volumes: A-C = 975 vph; A-D = 650 vph; B-C = 520 vph; B-D = 0 vph. Traffic includes 10 percent trucks, is composed of daily commuters, and the PHF is 0.85. Twelve-ft lanes are provided with no lateral obstructions, and the section is located in generally rolling terrain. What is the expected LOS for the section?

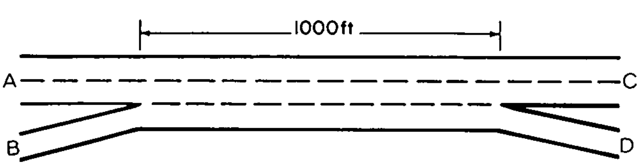


Figure 4-11. Weaving area for Calculation 3.

2. *Solution*—

a. All roadway and traffic conditions are specified in the calculation description and Figure 4-11. Note that this is a Type A configuration, as both movements A-D and B-C require one lane change.

b. The given demand volumes must be converted to peak flow rates under ideal conditions, expressed in passenger cars per hour:

$$v = \frac{V}{\text{PHF} \times f_{HV} \times f_w \times f_p}$$

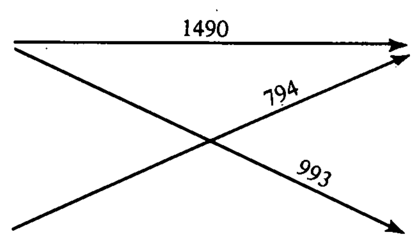
where:

PHF = 0.85 (Given);
$E_T = 4$ (Table 3-3);
$f_{HV} = 0.77 = 1/[1 + 0.10(4 - 1)]$;
$f_w = 1.00$ (Table 3-2); and
$f_p = 1.00$ (Table 3-10).

Then:

A-C = 975/(0.85 × 0.77 × 1.00 × 1.00) = 1,490 pcph
A-D = 650/(0.85 × 0.77 × 1.00 × 1.00) = 993 pcph
B-C = 520/(0.85 × 0.77 × 1.00 × 1.00) = 794 pcph
B-D = 0 pcph

c. A weaving diagram for the calculation is now constructed using the converted flow rates of step b:



Critical ratios may be computed as follows:

$$v_w = 993 + 794 = 1{,}787 \text{ pcph}$$

$$v = 1{,}787 + 1{,}490 = 3{,}277 \text{ pcph}$$

$$VR = 1{,}787/3{,}277 = 0.55$$

$$R = 794/1{,}787 = 0.44$$

d. The speed of weaving and nonweaving vehicles is estimated by using equations from Table 4-3 for unconstrained operation of Type A configurations.

$$S_w = 15 + \frac{50}{1 + 0.226(1 + VR)^{2.2}(v/N)^{1.0}/L^{0.9}}$$

$$S_{nw} = 15 + \frac{50}{1 + 0.020(1 + VR)^{4.0}(v/N)^{1.3}/L^{1.0}}$$

where:

$VR = 0.55$;
$v = 3{,}277$ pcph;
$N = 3$; and
$L = 1{,}000$ ft.

Then:

$$S_w = 15 + \frac{50}{1 + 0.226(1 + 0.55)^{2.2}(3{,}277/3)^{1.0}/1{,}000^{0.9}}$$

$S_w = 36.8$ mph *or* Say 37 mph

$$S_{nw} = 15 + \frac{50}{1 + 0.020(1 + 0.55)^{4.0}(3{,}277/3)^{1.3}/1{,}000^{1.0}}$$

$S_{nw} = 39.6$ mph *or* Say 40 mph

e. The required number of weaving lanes is computed and compared to the maximum value of 1.4 lanes given in Table 4-4 for Type A configurations by using the unconstrained estimates of weaving and nonweaving speed. The equation for N_w is also selected from Table 4-4.

$$N_w = 2.19\ N\ VR^{0.571}\ L_H^{0.234}/S_w^{0.438}$$

$$N_w = 2.19(3)(0.55)^{0.571}(10)^{0.234}/36.8^{0.438}$$

$$N_w = 1.64 \text{ lanes} > 1.40 \text{ lanes}$$

The section will therefore operate in the *constrained* mode, as the number of lanes required by weaving vehicles for unconstrained operation cannot be achieved. Speeds must now be recomputed using the constrained equations for Type A configurations from Table 4-3:

$$S_w = 15 + \frac{50}{1 + 0.28(1 + VR)^{2.2}(v/N)^{1.0}/L^{0.9}}$$

$$S_w = 15 + \frac{50}{1 + 0.28(1 + 0.55)^{2.2}(3{,}277/3)^{1.0}/1{,}000^{0.9}}$$

$S_w = 34.5$ mph *or* Say 35 mph

and

$$S_{nw} = 15 + \frac{50}{1 + 0.02(1 + VR)^{4.0}(v/N)^{0.88}/L^{0.6}}$$

$$S_{nw} = 15 + \frac{50}{1 + 0.02(1 + 0.55)^{4.0}(3{,}277/3)^{0.88}/1{,}000^{0.6}}$$

$S_{nw} = 42.0$ mph

f. In consulting the limitations on weaving areas depicted in Table 4-5, it is seen that the Type A weaving capacity of 1,800 pcph is only slightly higher than the existing flow rate for the section under study. Further, the VR of 0.55 exceeds the maximum recommended value of 0.45 for a three-lane Type A section. Thus, it might be expected that operations will be somewhat worse than those indicated by speed predictions.

g. If the speeds computed in step e are taken to be appropriate, the level of service for nonweaving vehicles would be D, while the level of service for weaving vehicles would be E, if the rounded value were used. Clearly, however, weaving vehicles are at the point of breakdown, as the speed is just at the LOS E/F boundary. Levels of service are determined by comparing predicted speeds to the criteria in Table 4-6. Given that operations are likely to be somewhat worse than these speeds indicate, it is probable that breakdowns will occur in the section.

It is noted that under constrained operation, weaving speeds decrease and nonweaving speeds increase compared to unconstrained operation. This is the result of weaving vehicles occupying less of the roadway space than required for balanced operation, with nonweaving vehicles occupying correspondingly more.

The operation of this section is clearly not acceptable. Given the fact that VR exceeds recommended limits for Type A configurations, and that the demand is close to the weaving capacity of such sections, provision of a Type B or Type C configuration should be considered to improve existing operations.

CALCULATION 4—A DESIGN APPLICATION

1. *Description*—A weaving area is being considered as a major junction between two urban freeways. The configuration of entry and exit roadways is expected to be as shown in Figure 4-12, which also shows the expected demand flow rates, expressed as peak flow rates under ideal conditions in passenger cars per hour. Design constraints limit the section length to a maximum of 1,500 ft. A level-of-service C design is desired for the section.

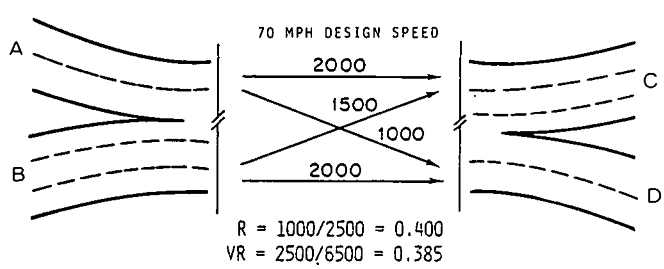
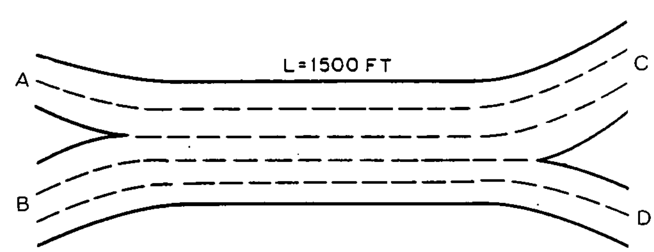


Figure 4-12. Weaving area for Calculation 4.

2. *Solution*—Design of weaving areas is best achieved by trial-and-error analysis of likely design scenarios. Because the length of the section is limited to 1,500, trial designs will start with this assumed length. Given the anticipated design of entry and exit roadways, the most obvious design would be a five-lane section as shown below:



This results from simply connecting each of the 5-entry lanes with the 5-exit lanes. Note that the resulting configuration is Type C, because movement B-C may be made without lane-changing, while movement A-D requires a minimum of two lane changes. The resulting section is now analyzed for the anticipated level of service.

a. All required roadway and traffic conditions are specified in the description.

b. No conversions are required because all demands are stated as peak flow rates under ideal conditions, in passenger cars per hour.

c. A weaving diagram and critical ratios are shown in Figure 4-12.

d. The unconstrained weaving and nonweaving speeds are estimated for the Type C configuration by using equations from Table 4-3:

$$S_w = 15 + \frac{50}{1 + 0.100(1 + VR)^{1.8}\,(v/N)^{0.8}/L^{0.5}}$$

$$S_{nw} = 15 + \frac{50}{1 + 0.015(1 + VR)^{1.8}(v/N)^{1.1}/L^{0.5}}$$

where:

$VR = 0.385$;
$v = 6{,}500$ pcph;
$N = 5$; and
$L = 1{,}500$ ft.

Then:

$$S_w = 15 + \frac{50}{1 + 0.100(1 + 0.385)^{1.8}\,(6{,}500/5)^{0.8}/1{,}500^{0.5}}$$

$S_w = 35.5$ mph *or* Say 36 mph

$$S_{nw} = 15 + \frac{50}{1 + 0.015(1 + 0.385)^{1.8}(6{,}500/5)^{1.1}/1{,}500^{0.5}}$$

$S_{nw} = 32.5$ mph *or* Say 33 mph

e. Using these estimates, the type of operation is checked using equations and values given in Table 4-4:

$$N_w = N\,\{0.761 - 0.011\,L_H - 0.005(S_{nw} - S_w) + 0.047\,VR\}$$

$$N_w = 5\,\{0.761 - 0.011(15) - 0.005(32.5 - 35.5) + 0.047(0.385)\}$$

$N_w = 3.15$ lanes $>$ 3.00 lanes

As the number of lanes required by weaving vehicles for unconstrained operation is greater than the maximum number of lanes that can be achieved in a Type C configuration, the operation will be *constrained,* and the speeds must be recomputed.

$$S_w = 15 + \frac{50}{1 + 0.100(1 + VR)^{2.0}(v/N)^{0.85}/L^{0.5}}$$

$$S_w = 15 + \frac{50}{1 + 0.100(1 + 0.385)^{2.0}(6{,}500/5)^{0.85}/1{,}500^{0.5}}$$

$S_w = 30.7$ mph *or* Say 31 mph

and

$$S_{nw} = 15 + \frac{50}{1 + 0.013(1 + VR)^{1.6}(v/N)^{1.00}/L^{0.5}}$$

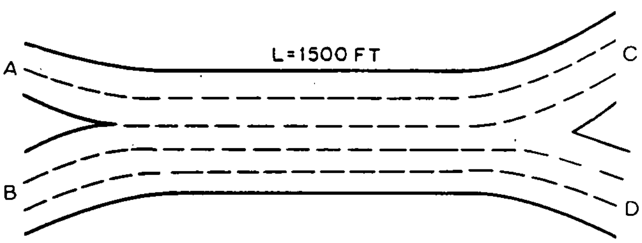
$$S_{nw} = 15 + \frac{50}{1 + 0.013(1 + 0.385)^{1.6}(6{,}500/5)^{1.00}/1{,}500^{0.5}}$$

$S_{nw} = 43.8$ mph *or* Say 44 mph

f. None of the limits specified in Table 4-5 is exceeded.

g. If the constrained speeds of step e are taken to be correct, the level of service for nonweaving vehicles would be D and the level of service for weaving vehicles would be F. These are obtained by comparing the predicted speeds with the criteria of Table 4-6.

The section obviously does not operate acceptably. Weaving vehicles are subject to breakdown conditions, and the disparity between weaving and nonweaving vehicles is a concern. Because the maximum length is already provided, and it would be difficult to provide more than five lanes in the section, only a change in configuration would be a practical alternative design. If one lane is added to leg D, a Type B configuration may be formed, as shown below:



This revised trial design may now be analyzed using the procedures of this chapter.

a. All roadway and traffic conditions have been stated.

b. All flows are expressed in peak flow rates under ideal conditions, in passenger cars per hour.

c. Figure 4-12 includes a weaving diagram.

d. Speed equations are now selected from Table 4-3 for unconstrained operation on a Type B configuration:

$$S_w = 15 + \frac{50}{1 + 0.10(1 + VR)^{1.2}(v/N)^{0.77}/L^{0.5}}$$

$$S_w = 15 + \frac{50}{1 + 0.10(1 + 0.385)^{1.2}(6{,}500/5)^{0.77}/1{,}500^{0.5}}$$

$S_w = 40.6$ mph *or* Say 41 mph

and

$$S_{nw} = 15 + \frac{50}{1 + 0.02(1 + VR)^{2.0}(v/N)^{1.42}/L^{0.95}}$$

$$S_{nw} = 15 + \frac{50}{1 + 0.02(1 + 0.385)^{2.0}(6{,}500/5)^{1.42}/1{,}500^{0.95}}$$

$S_{nw} = 40.3$ mph *or* Say 40 mph

e. The type of operation is now checked using equations and values from Table 4-4:

$N_w = N\{0.085 + 0.703\ VR + (234.8/L) - 0.018(S_{nw} - S_w)\}$

$N_w = 5\{0.085 + 0.703(0.385) + (234.8/1{,}500) - 0.018(40.3 - 40.6)\}$

$N_w = 2.58$ lanes < 3.50 lanes

The operation is, therefore, unconstrained.

f. None of the limiting values of Table 4-5 is exceeded by the trial design.

g. Comparing predicted speeds to the criteria of Table 4-6, it is seen that the level of service of nonweaving vehicles is E, while the level of service for weaving vehicles is D.

Note that the Type B configuration represents a significantly better design than the initial Type C trial. The operation is now unconstrained, as opposed to constrained with the Type C design, and both weaving and nonweaving speeds are improved. This illustrates the advantages of Type B configurations over Type C configurations for handling large weaving volumes. The resulting level of service (D for weaving vehicles, E for nonweaving vehicles), however, is still not the LOS C desired. The design engineer would have to determine whether to accept somewhat poorer operations than desired, or to investigate a full interchange as an alternative. Note that neither the length nor the width of the trial designs could be practically expanded, so that the final trial design would have to be accepted, or the idea of a weaving area abandoned in favor of a full interchange at this location. The final decision would have to consider many factors, incuding economic and environmental aspects, as well as the confidence in projected flow rates used in the design analysis.

CALCULATION 5—A MULTIPLE WEAVING AREA

1. *Description*—Figure 4-13 shows a multiple weaving area. Peak flow rates in passenger cars per hour for the sections are:

a. A-X = 900 pcph
b. B-X = 400 pcph
c. A-Y = 1,000 pcph
d. B-Y = 200 pcph
e. C-X = 300 pcph
f. C-Y = 100 pcph

All geometric conditions are ideal, and the terrain is generally level. At what level of service would the section operate?

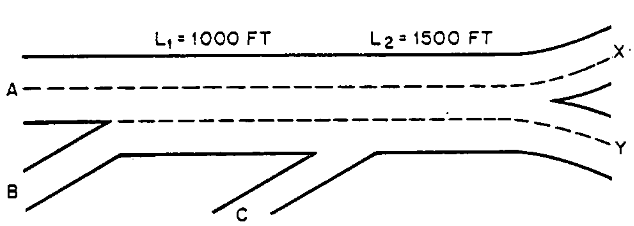
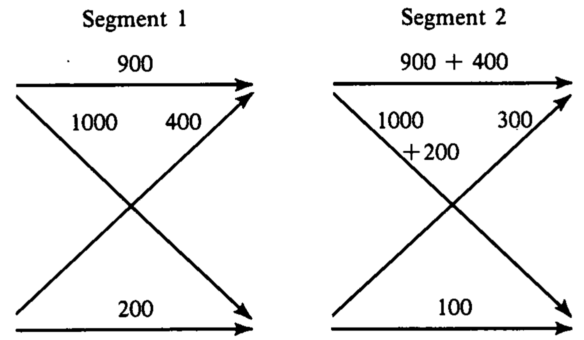


Figure 4-13. Weaving area for Calculation 5.

2. *Solution*—A multiple weaving section is analyzed as two separate simple weaving areas. The initial step in the analysis is to construct weaving diagrams for the two subsegments of the multiple weaving area. Because all demands are stated in peak flow rates under ideal conditions and no conversion computations are required, this is done immediately. The weaving area under study is of the type illustrated in Figure 4-8, i.e., two merge areas followed closely by a diverge area. Weaving diagrams are constructed in accordance with Figure 4-8, as follows:



$R = 400/1{,}400 = 0.286$ $R = 300/1{,}500 = 0.200$
$VR = 1{,}400/2{,}500 = 0.560$ $VR = 1{,}500/2{,}900 = 0.517$

Note that both segments of the weaving area are Type B configurations. In segment 1, movement A-Y may be made with no lane changes while movement B-X requires one lane change. In segment 2, movements A-Y and B-Y may be made with no lane changes, while movement C-X requires a single lane change.

Computations for speed are now done for each segment. Note that the first three steps of the procedure have been completed in the establishment of weaving diagrams for the two segments.

• Segment 1

a. Unconstrained speed equations for Type B configurations are selected from Table 4-3:

$$S_w = 15 + \frac{50}{1 + 0.10(1 + VR)^{1.2}(v/N)^{0.77}/L^{0.5}}$$

$$S_w = 15 + \frac{50}{1 + 0.10(1 + 0.56)^{1.2}(2{,}500/3)^{0.77}/1{,}000^{0.5}}$$

$S_w = 40.5$ mph *or* Say 41 mph

and

$$S_{nw} = 15 + \frac{50}{1 + 0.02(1 + VR)^{2.0}(v/N)^{1.42}/L^{0.95}}$$

$$S_{nw} = 15 + \frac{50}{1 + 0.02(1 + 0.56)^{2.0}(2{,}500/3)^{1.42}/1{,}000^{0.95}}$$

$S_{nw} = 40.4$ mph *or* Say 40 mph

b. The number of lanes required by weaving vehicles for unconstrained operation is computed using the appropriate equation from Table 4-4, and compared to the maximum value of 3.5 lanes, also obtained from Table 4-4, for Type B configurations:

$N_w = N\{0.085 + 0.703\ VR + (234.8/L) - 0.018(S_{nw} - S_w)\}$

$N_w = 3\ \{0.085 + 0.703(0.56) + (234.8/1{,}000) - 0.018(40.4 - 40.5)\}$

$N_w = 2.15$ lanes < 3.50 lanes

The section is therefore unconstrained.

c. None of the limitations of Table 4-5 is violated. From Table 4-6, the nonweaving LOS is E, and the weaving LOS is D.

- Segment 2

a. Using the same equations as for segment 1, because both are Type B configurations:

$$S_w = 15 + \frac{50}{1 + 0.10(1 + 0.517)^{1.2}(2{,}900/3)^{0.77}/1{,}500^{0.5}}$$

$S_w = 42.1$ mph *or* Say 42 mph

$$S_{nw} = 15 + \frac{50}{1 + 0.02(1 + 0.517)^{2.0}(2{,}900/3)^{1.42}/1{,}500^{0.95}}$$

$S_{nw} = 43.3$ mph *or* Say 43 mph

b. The number of lanes required by weaving vehicles is:

$N_w = 3\ \{0.085 + 0.703(0.517) + (234.8/1{,}500) - 0.018(43.3 - 42.1)\}$

$N_w = 1.75$ lanes < 3.50 lanes

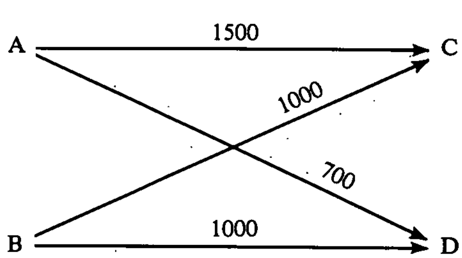
Operation is unconstrained.

c. None of the limitations of Table 4-5 is violated. From Table 4-6, the level of service is D for both nonweaving and weaving vehicles.

The analysis indicates that the entire weaving area will operate in the range of 40 to 43 mph, a range which stradles the boundaries between levels-of-service D and E.

CALCULATION 6—A SENSITIVITY ANALYSIS WITH DESIGN APPLICATION

1. *Description*—A major interchange is to be built to join two major freeways in a suburban area. The issue of handling some of the interchanging movements in a weaving section is to be investigated. The flows in question are shown below, and are given in terms of flow rates in passenger cars per hour under ideal conditions.



Because the interchange joins two future facilities, there is substantial flexibility in both the length and width that may be considered for the section. Level-of-service C operation is desired.

2. *Solution*—Since the length, width, and configuration to be used are open to question, as is the issue of whether or not to use a weaving section, many trial computations must be made. Speeds can be computed for weaving and nonweaving vehicles for a range of conditions covering 3, 4, or 5 lanes, lengths from 500 to 2,500 ft, and all three types of configuration. Although this is a time-consuming process, it is easily set up on a programmable calculator, microcomputer spreadsheet, or any type of computer. The results of such computations are tabulated:

		Type A		Type B		Type C	
		Weaving and Nonweaving Speeds					
No. of Lanes	Length (ft)	S_w (mph)	S_{nw} (mph)	S_w (mph)	S_{nw} (mph)	S_w (mph)	S_{nw} (mph)
3	500	29	32	33	27	28	26
	750	33	37	35	31	31	28
	1,000	36	40	37	34	32	29
	1,500	23[a]	47[a]	40	39	34	31
	2,000	27[a]	49[a]	41	42	36	33
	2,500	—	—	43	45	38	34
4	500	15[a]	42[a]	30[a]	41[a]	31	29
	750	19[a]	45[a]	38	36	33	31
	1,000	22[a]	47[a]	40	39	35	33
	1,500	27[a]	50[a]	42	44	37	35
	2,000	30[a]	51[a]	44	47	39	37
	2,500	—	—	46[b]	49[b]	40	38
5	500	17[a]	44[a]	32[a]	44[a]	29[a]	42[a]
	750	22[a]	47[a]	40	40	31[a]	45[a]
	1,000	25[a]	49[a]	42	43	32[a]	46[a]
	1,500	29[a]	52[a]	44	47	35[a]	49[a]
	2,000	33[a]	53[a]	46[b]	50[b]	41	40
	2,500	—	—	47[b]	53[b]	43	41

[a] Constrained operation.
[b] Speeds indicate cases meeting LOS C

This analysis illustrates a number of interesting characteristics of weaving area operations.

a. As discussed previously, drivers in a Type A configuration behave differently from other types. Note that as length increases, the Type A configuration is more likely to be constrained. Adding length or width to the Type A section benefits primarily the nonweaving vehicles. More length and/or width does not necessarily benefit weaving vehicles, and in some cases is detrimental, due to the complex interactions of component flows attempting to segregate while operating at high speed. Longer and wider Type A configurations do not add to the ability to effectively serve large weaving flows. This configuration is more appropriate for smaller weaving flows, with nonweaving flows dominant. Note also that for four-lane and five-lane Type A configurations, the *VR* of 0.40 exceeds the normal maximum value noted in Table 4-4.

b. Type B configurations are consistently better than similar Type C sections, because the smaller weaving flow needs to make only one lane change as opposed to two in the Type C arrangement. Both Type B and Type C sections are less likely to be constrained as length is increased.

c. No Type A or Type C section tested meets the criteria for level-of-service C, which requires 45 mph for weaving vehicles and 48 mph for nonweaving vehicles, as indicated in Table 4-6.

d. Three type B configurations do meet the criteria for LOS C. These are the four-lane section of 2,500 ft, the five-lane section of 2,000 ft, and the five-lane section of 2,500 ft. Any of these would provide the desired level of service. A review of the entry and exit flows, however, indicates that no more than two lanes would normally be provided on any leg, using a level-of-service C service flow rate of 1,550 pcphpl (Table 3-1, LOS C, 70-mph design speed):

Leg	*Flow Rate*	*Lanes Req'd*
A	2,200 pcph	2
B	2,000 pcph	2
C	2,500 pcph	2
D	1,700 pcph	2

Thus, it would be impractical to provide five lanes in the weaving section. The four-lane, 2,500-ft weaving section could be implemented, but a lane would have to be added to one of the exit legs to provide for a Type B configuration. It would be added to leg C, with the higher exit flow rate, and might have to be dropped at some downstream point if additional downstream flow did not justify its continuance. The lane should be carried for approximately 2,000 ft to be effective in the weaving section.

If the lane addition were impractical, a grade separation would have to be provided to accommodate the expected flows.

V. REFERENCES

The methodology presented in this chapter is the combined result of several research efforts sponsored by the National Cooperative Highway Research Program (NCHRP) and the Federal Highway Administration (FHWA). The methodology incorporates the concepts of configuration developed as part of the NCHRP Weaving Area Operations Study (*1*), later modified as part of the FHWA Freeway Capacity Analysis Procedures Study (*2*, *3*). The form of the predictive speed equations was developed as part of the FHWA Weaving Analysis Procedures for the *HCM* study (*4*). Additional background concepts and materials were adapted from research originally developed independently by Jack Leisch and Associates, and later documented and updated with FHWA sponsorship (*5*). The effect of configuration on weaving area operations is discussed and illustrated in Ref. *6*.

1. PIGNATARO, L., MCSHANE, W., ROESS, R., CROWLEY, K., AND LEE, B., "Weaving Areas—Design and Analysis." *NCHRP Report 159*, (1975) 119 pp.
2. 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).
3. ROESS, R., MCSHANE, W., LINZER, E., and PIGNATARO, L., *Freeway Capacity Procedures.* Procedural Manual, Project No. DOT-FH-11-9336, Polytechnic Institute of New York, Brooklyn N.Y. (May 1978).
4. REILLY, W., KELL, J., and JOHNSON, P., *Weaving Analysis Procedures for the New Highway Capacity Manual.* Technical Report, JHK & Associates, Tucson, Ariz. (Aug. 1984).
5. LEISCH, J., *Completion of Procedures for Analysis and Design of Traffic Weaving Sections.* Vol. 1-Final Report, Vol. 2-User's Guide, Federal Highway Administration, Washington, D.C. (1983).
6. ROESS, R., MCSHANE, W., and PIGNATARO, L., "Configuration and the Design and Analysis of Weaving Sections." *Transportation Research Record 489*, Transportation Research Board, Washington, D.C. (1974).

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)*.

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 follow 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 that 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.

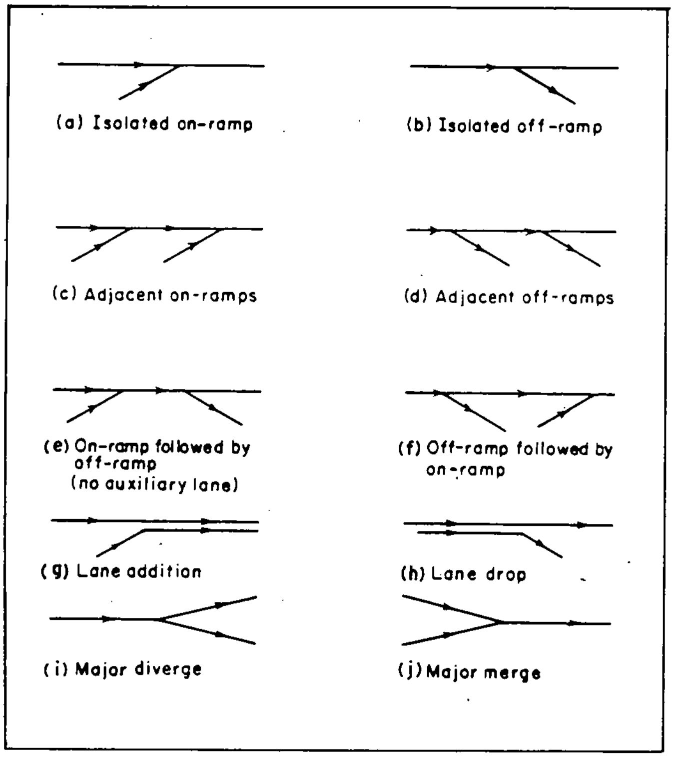


Figure 5-1. Ramp configurations covered by procedures.

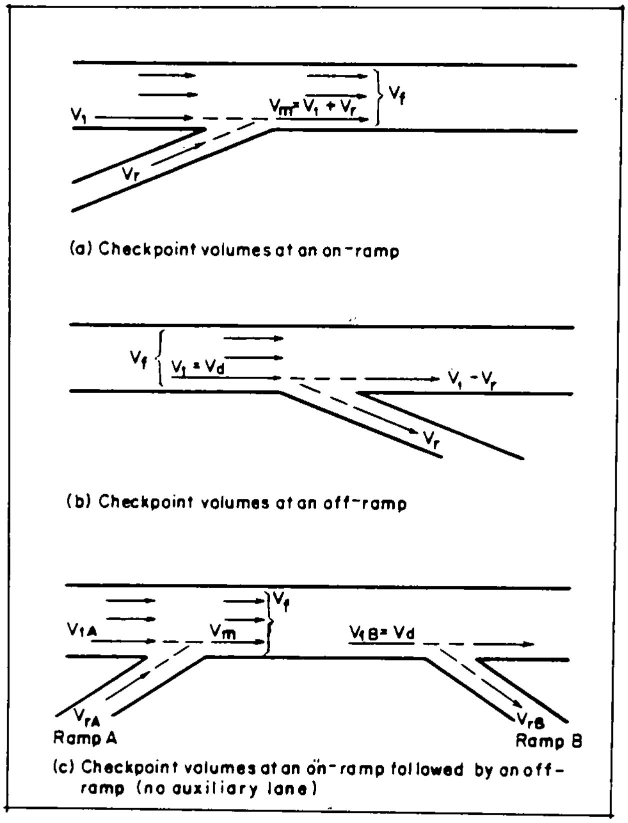


Figure 5-2. Checkpoint volumes for ramp-freeway terminals.

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

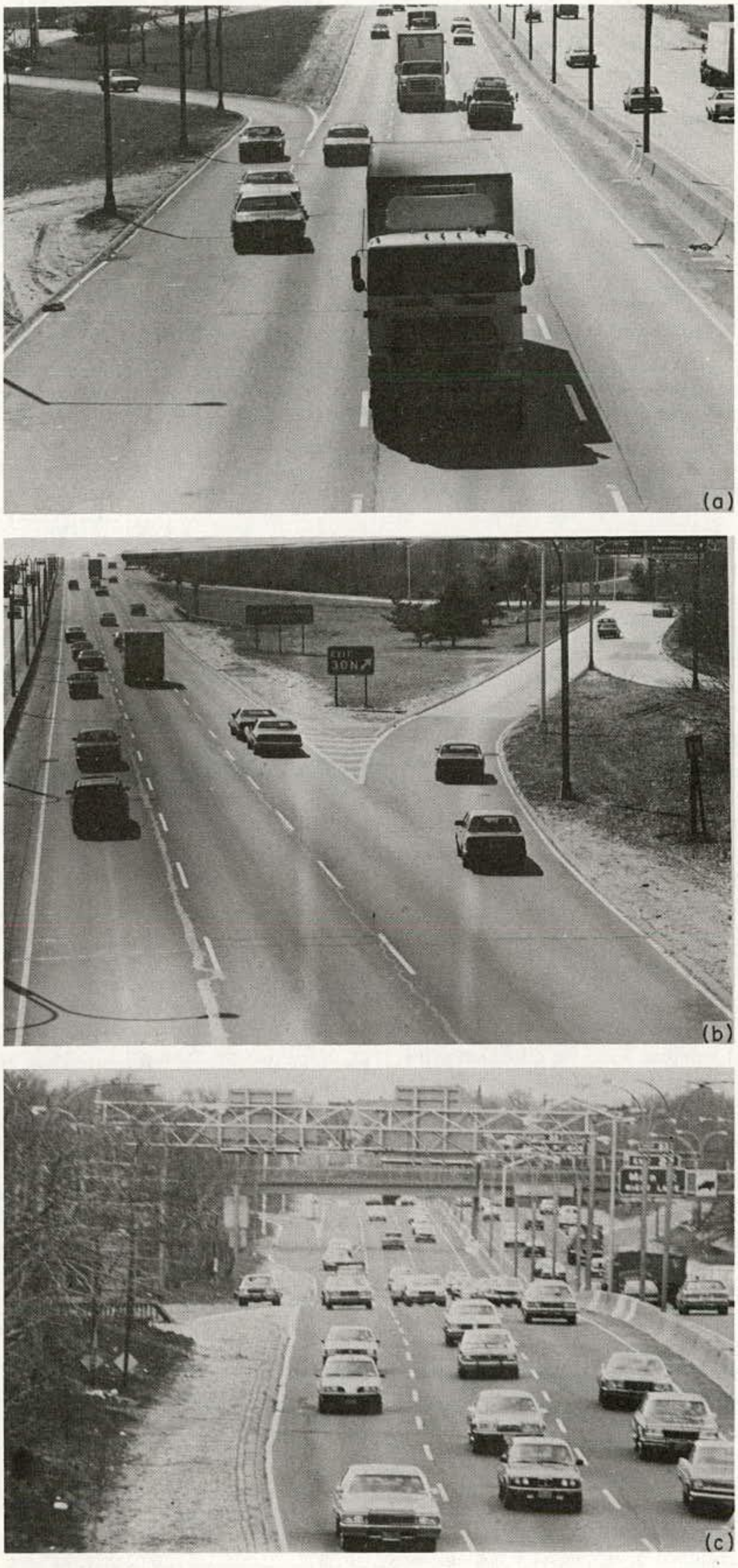


Illustration 5-1. Typical ramp configurations include (a) an isolated on-ramp, (b) an isolated off-ramp, (c) an on-ramp off-ramp sequence.

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.

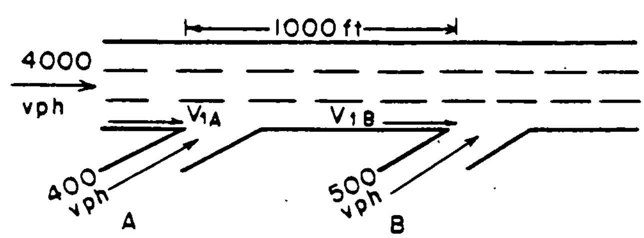
Table 5-2 contains an index to various equations and associated nomographs that are used in the computation of lane 1 volumes. Appendix I to this chapter contains these nomographs (with equations), which cover the various ramp configurations enumerated earlier. Because of the numerous ramp configurations 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 configurations 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

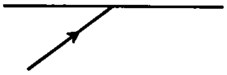
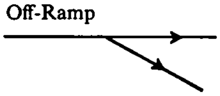
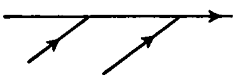
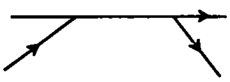
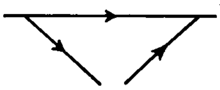
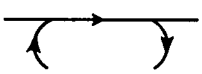
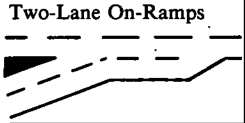
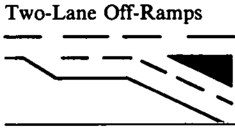
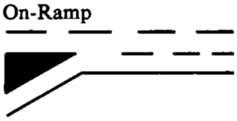
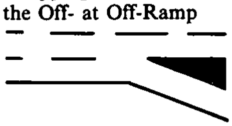
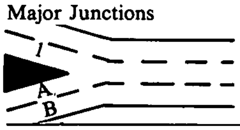
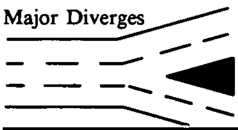
	4-LANE FREEWAY (2 LANES EACH DIRECTION)		6-LANE FREEWAY (3 LANES EACH DIRECTION)		8-LANE FREEWAY (4 LANES EACH DIRECTION)	
CONFIGURATION	1st RAMP	2nd RAMP	1st RAMP	2nd RAMP	1st RAMP	2nd RAMP
Isolated, One Lane On-Ramp 	Fig. I.5-1	—	Fig. I.5-6	—	Fig. I.5-9	—
Isolated, One Lane Off-Ramp 	Fig. I.5-2	—	Fig. 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)		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
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:

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

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:

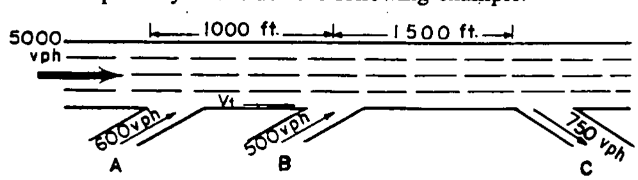
$$\begin{aligned} V_f &= 4{,}400 \text{ vph;} \\ V_r &= 500 \text{ vph;} \\ D_u &= 1{,}000 \text{ ft;} \\ V_u &= 400 \text{ vph; and} \\ V_{1B} &= 574 + 0.228(4{,}400) - 0.194(500) - 0.714(1{,}000) \\ &\quad + 0.274(400) \\ V_{1B} &= 876 \text{ vph.} \end{aligned}$$

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.

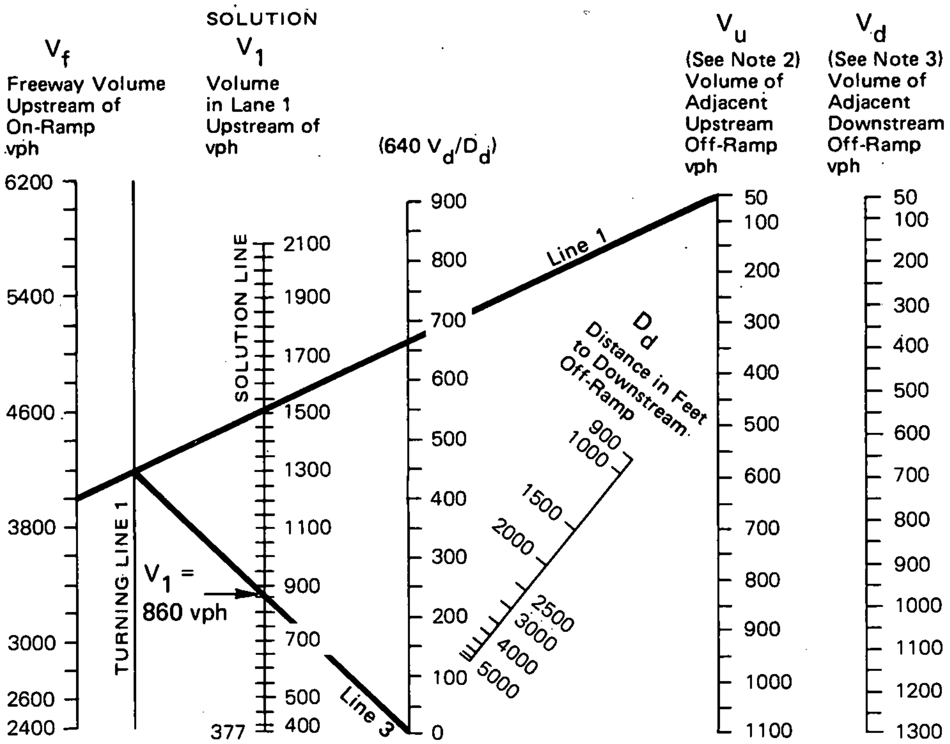
Approximation Procedure

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:

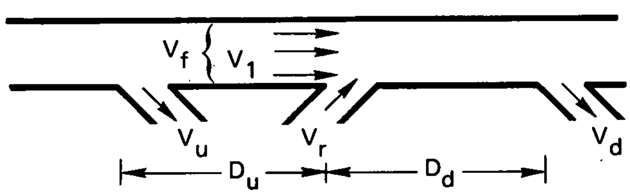


NOTE: There are no other ramps within 4,000 ft of this segment.



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

Diagram:



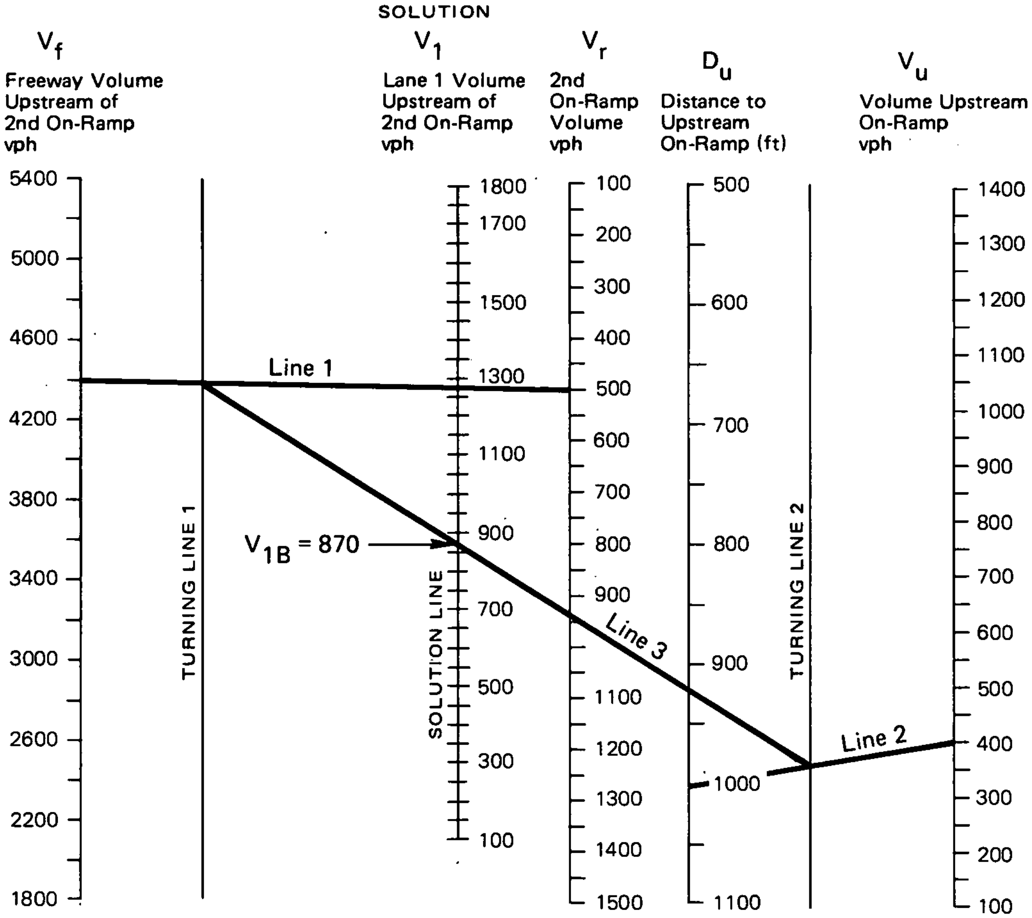
Conditions for Use:

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 5700 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:

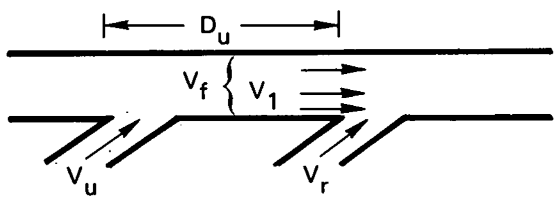
1. Draw a line from V_f value to V_u value, intersecting turning line 1.
2. Draw a line from V_d value to D_d value, intersecting 640 V_d/D_d line.
3. Draw a line from the step 1 intersection with turning line 1 to the 640 V_d/D_d value of step 2; read solution at intersection with V_1 line.

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.

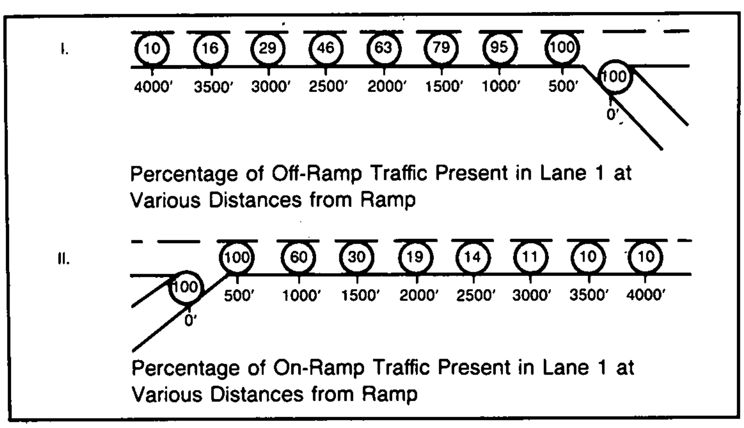
TRUCK PRESENCE IN LANE 1

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.



Percentage of Off-Ramp Traffic Present in Lane 1 at Various Distances from Ramp

Percentage of On-Ramp Traffic Present in Lane 1 at Various Distances from Ramp

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.

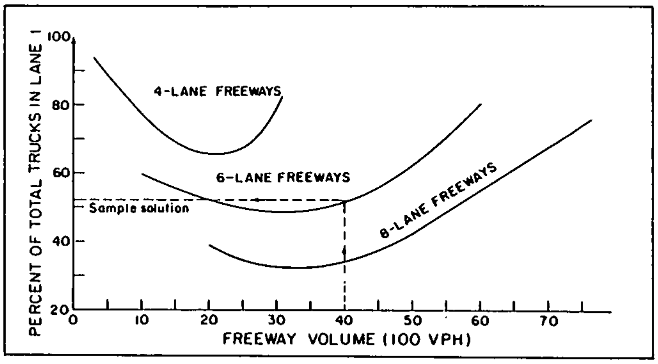


Figure 5-6. Truck presence in lane 1.

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 flow rates 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.

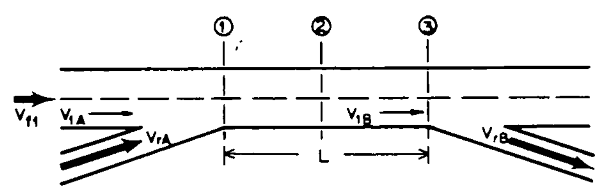
Before converting lane 1 volume to pcph, it is necessary to determine truck presence in this lane. Figure 5-6 or local data are used to estimate the percentage of total freeway trucks in lane 1, from which the proportion of trucks in the lane 1 volume may be computed.

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. *3* 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 *3* 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 (*6*) 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*), 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*), but these are not related to specific operational characteristics. Leisch (*3*) 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 *3* 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. *3*, 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.

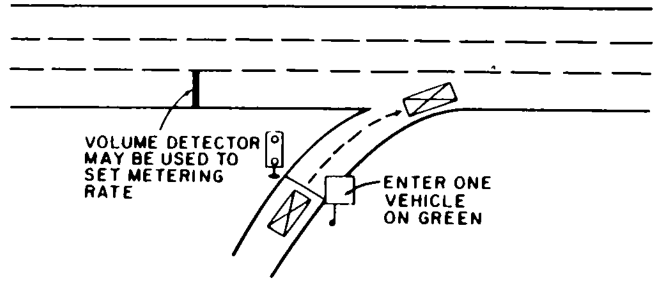


Figure 5-8. A typical ramp metering installation.

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).

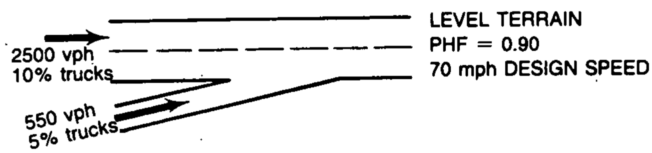
Of course, all values must be converted to passenger cars per hour and peak flow rates, as described elsewhere in this chapter. Sample Calculation 8 illustrates this process for determining an appropriate ramp metering rate.

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:

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

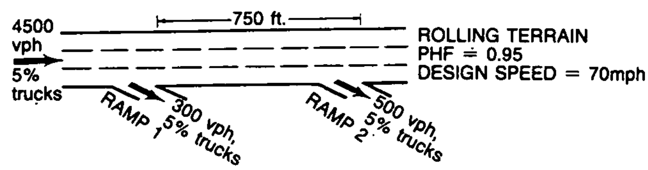
These values are now expanded to peak flow rates by dividing by the peak hour factor. The level of service is then found by comparing the merge and freeway checkpoint flow rates to the criteria of Table 5-1:

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

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:

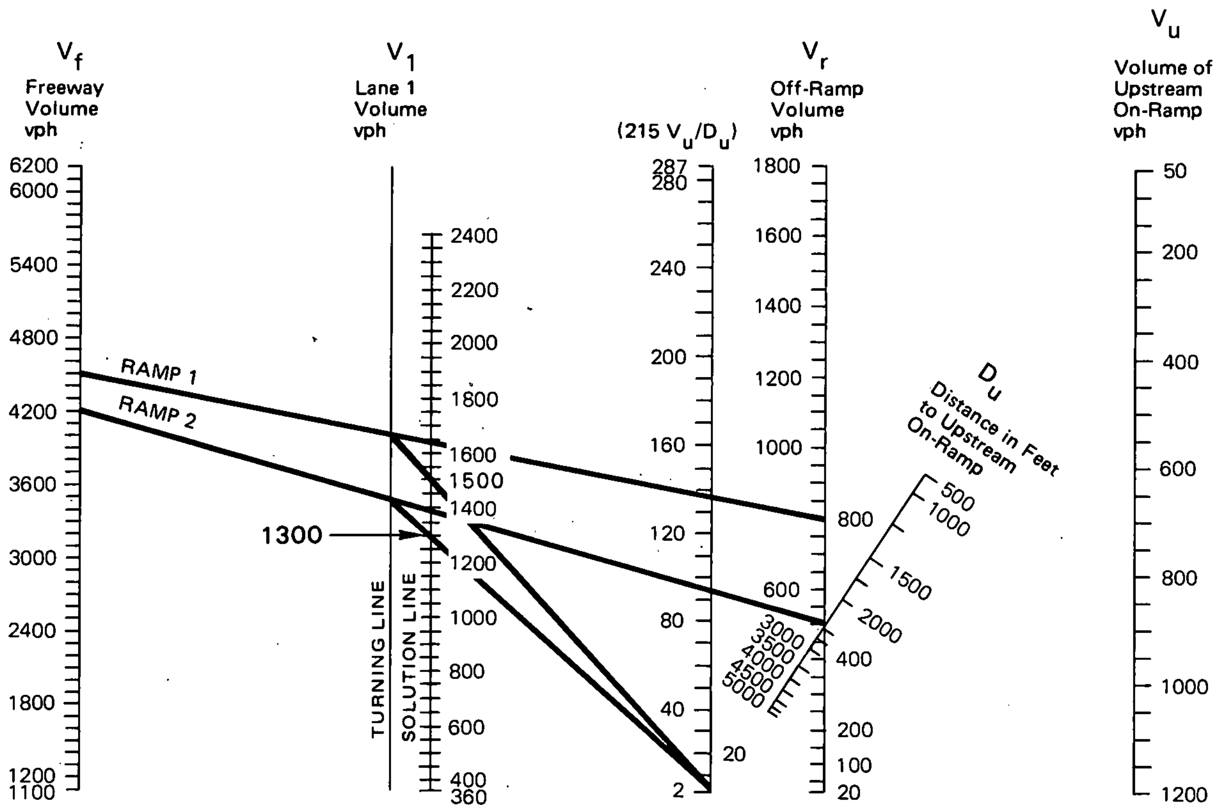
$$V_1 = 94 + 0.231(4{,}200) + 0.473(500) + 2$$
$$V_1 = 1{,}303 \text{ vph}$$

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:

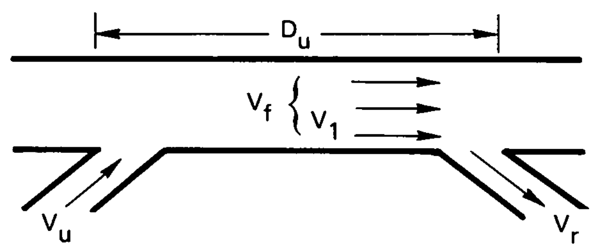
- Ramp 1

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

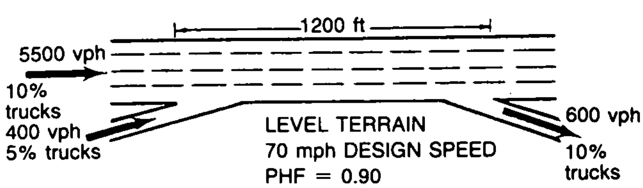
Three checkpoint volumes are of interest: (1) the freeway volume at the maximum point, *before* the two off-ramps, and (2) the diverge volumes before each of the off-ramps. Each checkpoint volume must be converted to a peak flow rate and compared with the criteria of Table 5-1.

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]	PHF	Flow Rate (pcph) = Vol. (vph)/f_{HV} × PHF
V_1 (On)	1,010	0.27	1.7	0.84	0.90	1,336
V_1 (Off)	1,233	0.25	1.7	0.85	0.90	1,612
V_f	5,900	0.10	1.7	0.93	0.90	7,049
V_r (On)	400	0.05	1.7	0.97	0.90	458
V_r (Off)	600	0.10	1.7	0.93	0.90	717

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

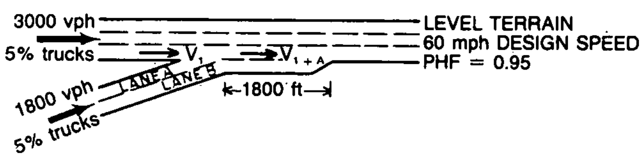
Critical checkpoint volumes may now be computed and compared with the criteria in Table 5-1.

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

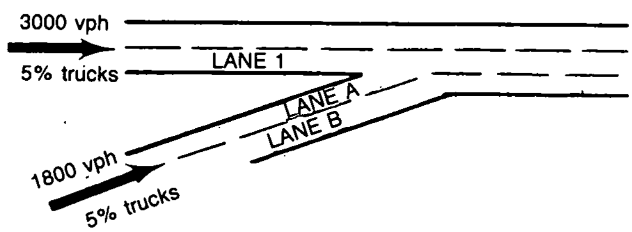
[a] Table 3-3
[b] Computed as $f_{HV} = 1/[1 + P_T(E_T - 1)]$

Checkpoint flow rates 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:

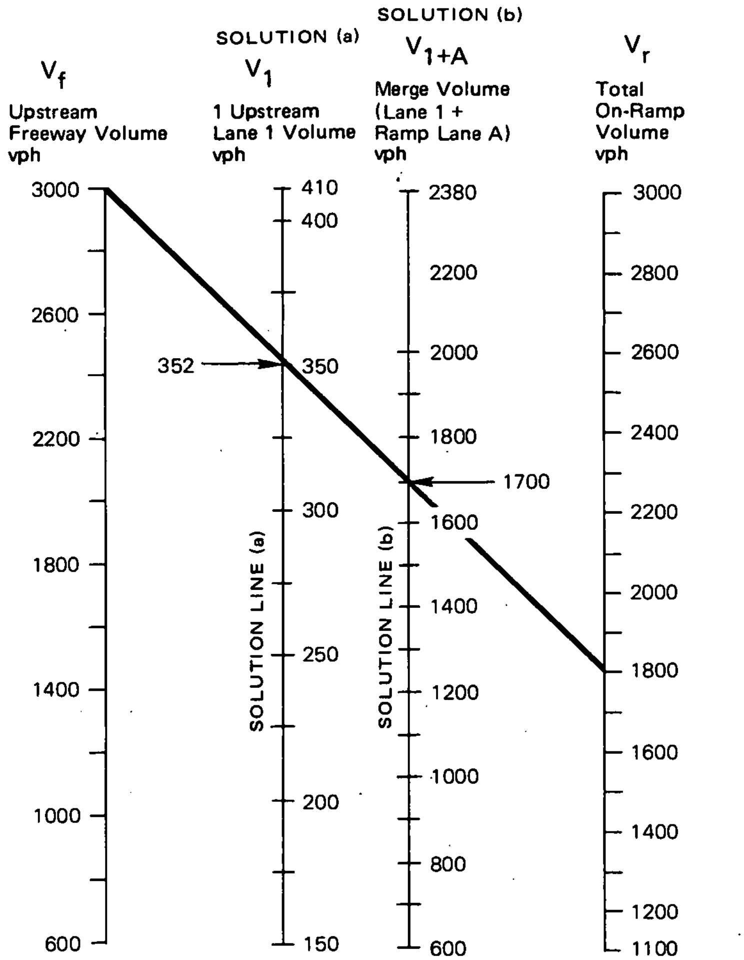


From Table 5-2, this alternative may be analyzed using a multistep trial-and-error process.

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.

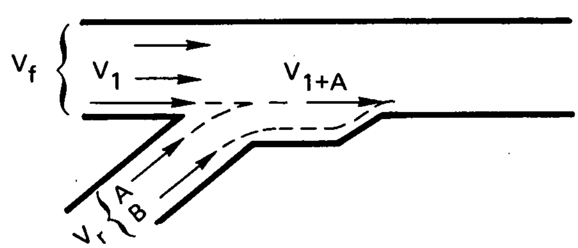
Table 5-2 indicates the use of Figure I.5-1 to compute V_1, but directs the use of only the lane A volume for V_r:

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



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 $V_A = V_{1+A} - V_1$; $V_B = V_r - V_A$.
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. Nomograph 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]	PHF	Flow Rate (pcph) = Vol. (vph)/f_{HV} × PHF
V_1	1,073	0.11	1.7	0.93	0.95	1,214
V_A	850	0.05	1.7	0.97	0.95	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]	PHF	Flow Rate (pcph) = Vol. (vph)/f_{HV} × PHF
V_1	1,155	0.10	1.7	0.93	0.95	1,307
V_A	138	0.05	1.7	0.97	0.95	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 0.71. 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 D operations, a more acceptable result.

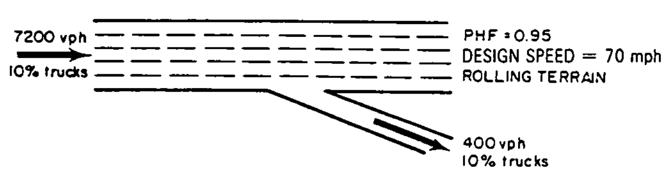
A 41- to 50-mph loop ramp, however, will create an extremely long loop, consuming a great deal of land in its wake. The designer is faced with several options:

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]	PHF	Flow Rate (pcph) = Vol. (vph)/f_{HV} × PHF
V_1	936	0.32	4	0.51	0.95	1,932
V_r	400	0.10	4	0.77	0.95	547
V_f	7,200	0.10	4	0.77	0.95	9,842

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

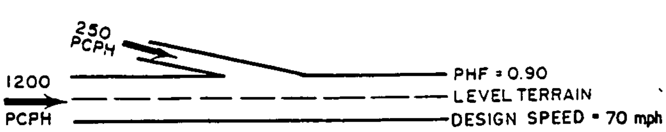
Computing the checkpoint flow rates:

$$v_d = V_1 = 1{,}932 \text{ pcph (LOS E, Table 5-1)}$$
$$v_f = 9{,}842/5 = 1{,}968 \text{ pcphpl (LOS E, Table 3-1)}$$

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_i can be approximated as $1.25 \times V_1$, 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:

$$V_1 = 520 \times 1.25 = 650 \text{ vph}$$

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_i.

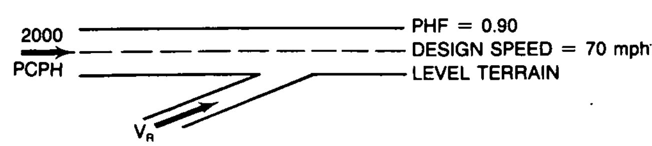
Computing checkpoint volumes and dividing by the PHF:

$$v_m = (650) + 250)/0.90 = 1{,}000 \text{ pcph (LOS B, Table 5-1)}$$
$$v_f = (1{,}200 + 250)/0.90 = 1{,}611 \text{ pcph (LOS B, Table 5-1)}$$

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 *(2)*. Some of these modifications appeared in a text by Pignataro *(4)*. Special applications for four-lane freeway segments and left-side ramps were adapted from a study by Leisch *(3)*, as were capacities for ramp roadways. References *5* and *6* provide a background in gap acceptance theory and its application to merge area analysis. AASHTO standards for geometric design of ramps are given in Ref. *1*.

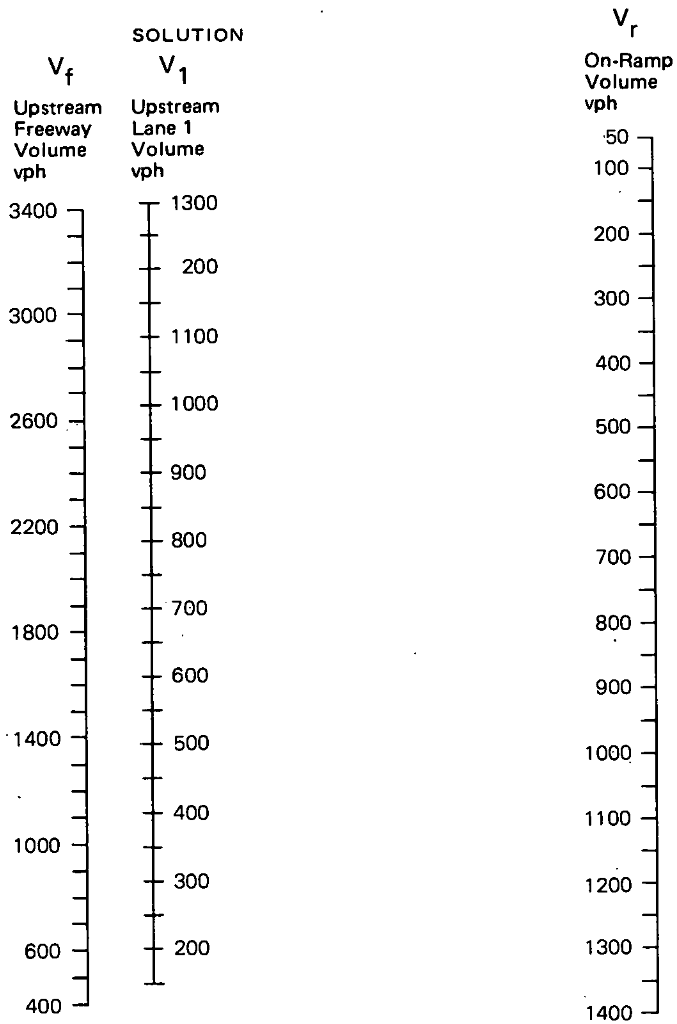
1. *A Policy on Geometric Design of Highways and Streets,* American Association of State Highway and Transportation Officials, Washington, D.C. (1984)
2. PIGNATARO, L. J., MCSHANE, W. R., ROESS, R. P., LEE, B., and CROWLEY, K. W., "Weaving Areas—Design and Analysis." *NCHRP Report 159* (1975) 119 pp.
3. 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).
4. PIGNATARO, L. J., *Traffic Engineering: Theory and Practice.* Prentice-Hall Inc., Englewood Cliffs, N.J. (1974).
5. 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).
6. DREW, D., *Traffic Flow Theory and Control.* McGraw-Hill Inc., New York, N.Y. (1968).
7. 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

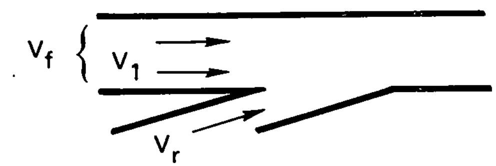
In using the nomographs of this appendix, note the following:

- *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:



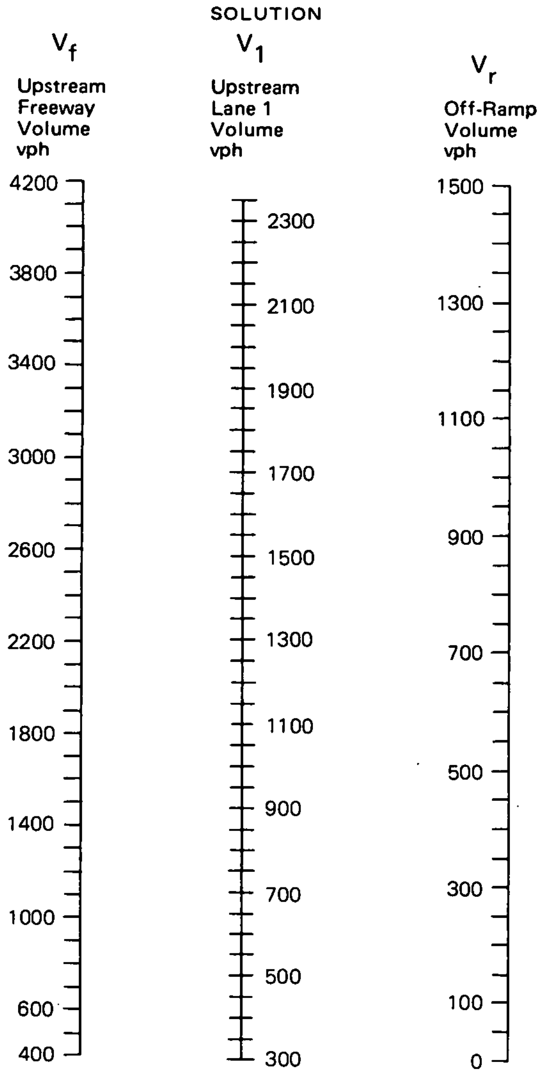
Conditions for Use:

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:

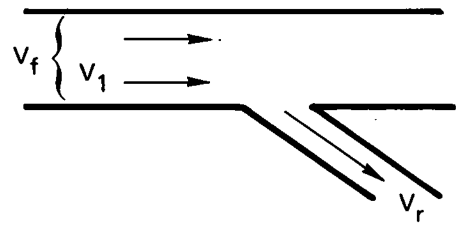
1. Draw line from V_f value to V_r value; read solution on V_1 line.

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:



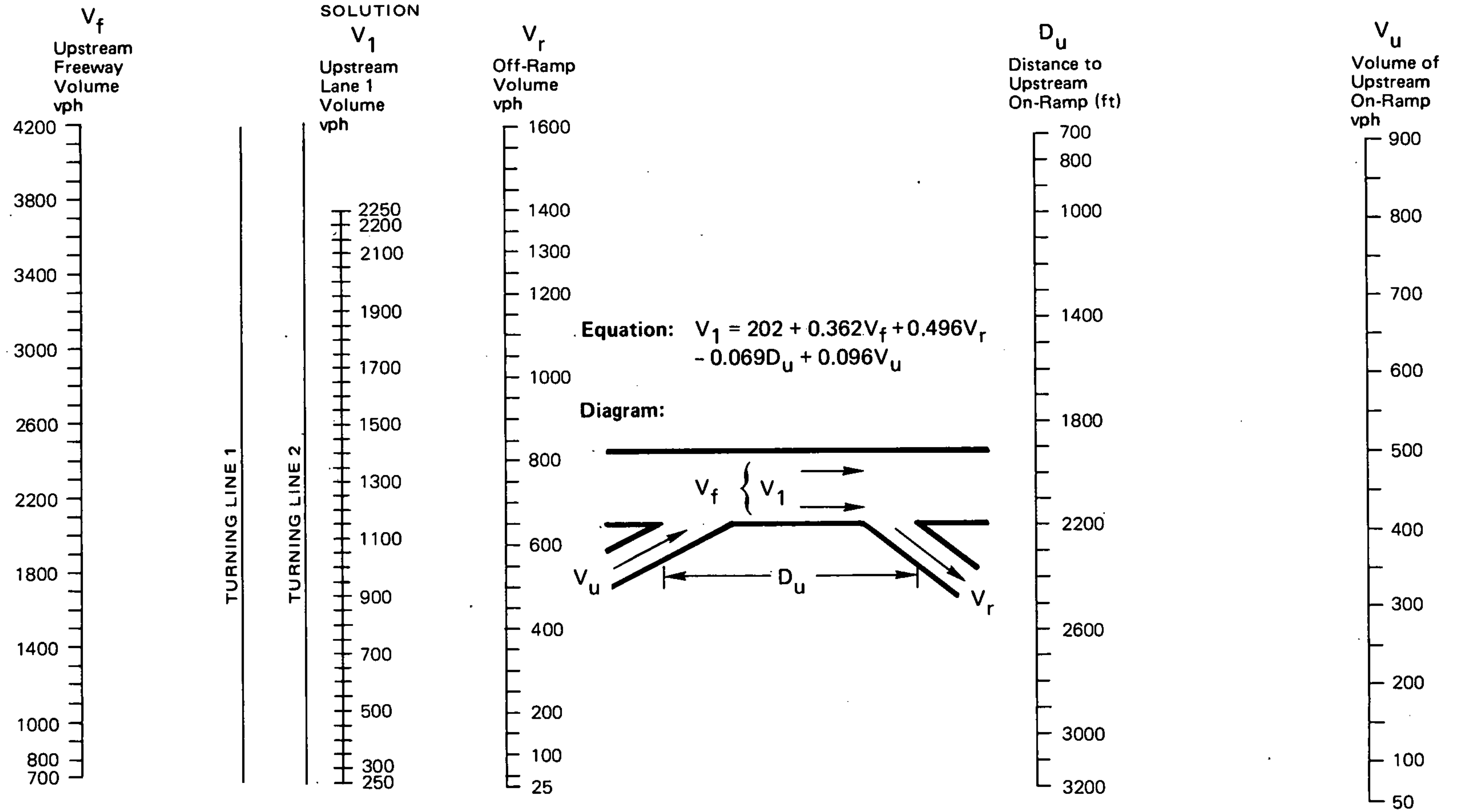
Conditions for Use:

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: $V_1 = 202 + 0.362V_f + 0.496V_r - 0.069D_u + 0.096V_u$

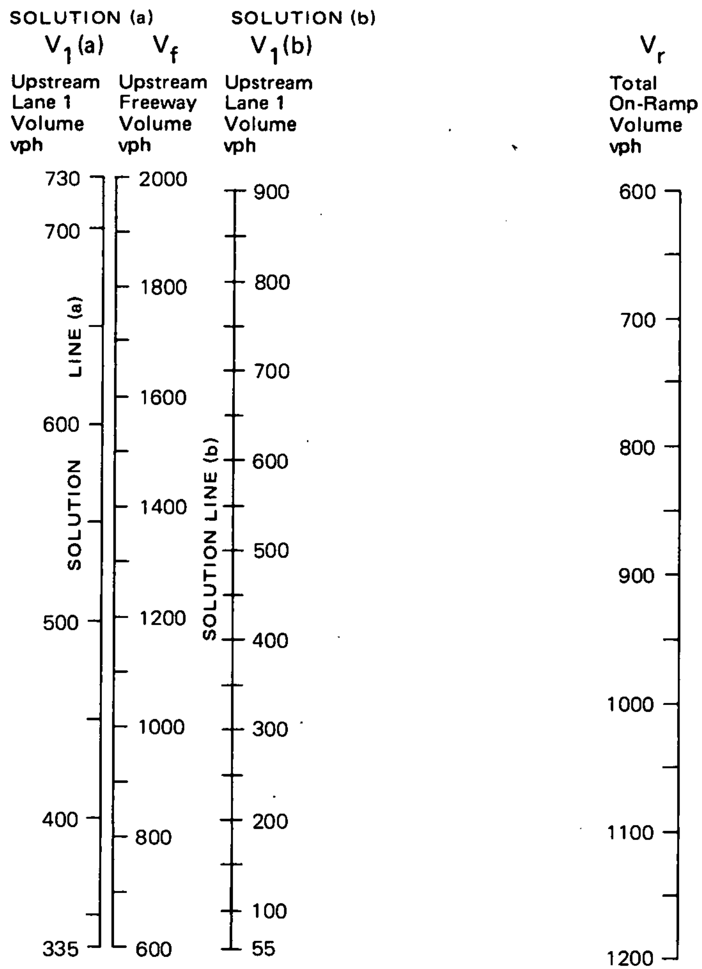
Conditions for Use:

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:

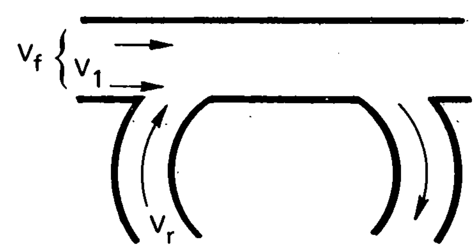
1. Draw line from V_f value to V_r value, intersecting turning line 1.
2. Draw a line from the point defined in step 1 to the D_u value, intersecting turning line 2.
3. Draw a line from the point defined in step 2 to the V_u value. Read the result from the V_1 line.

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:



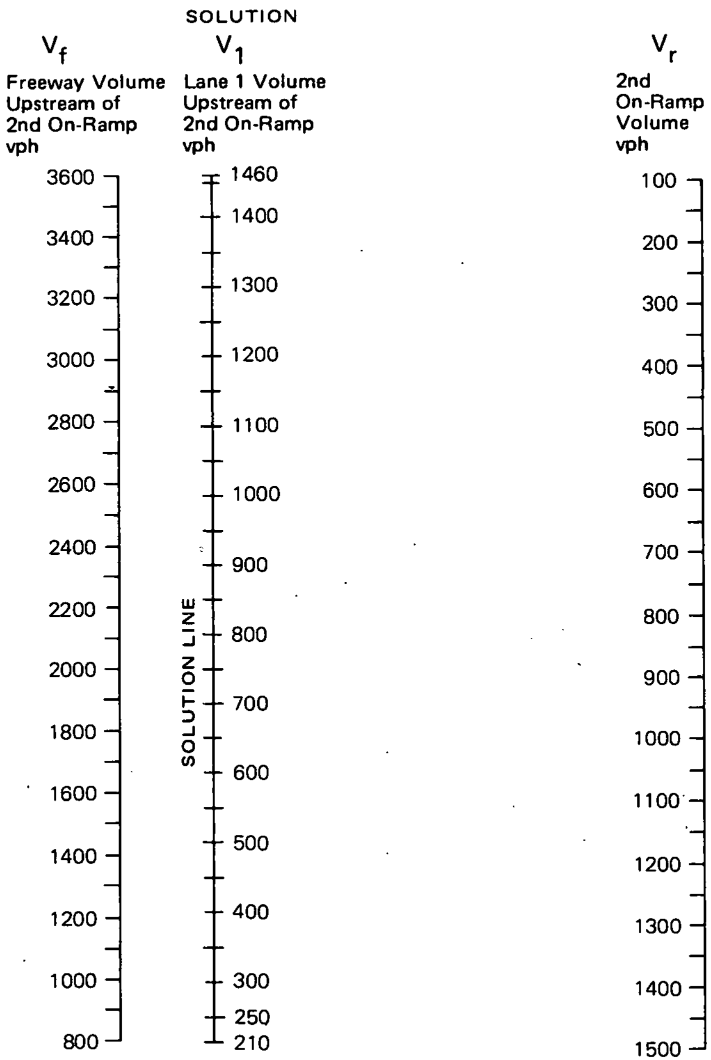
Conditions for Use:

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:

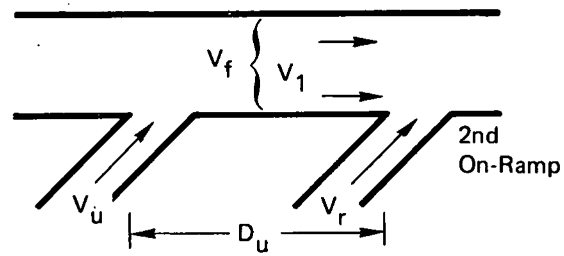
1. If $V_r < 600$ vph, read solution on solution line (a), horizontally to the left of V_f value.
2. If V_r is between 600 and 1200 vph, draw a line from V_f value to V_r value; read result on solution line (b).

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:



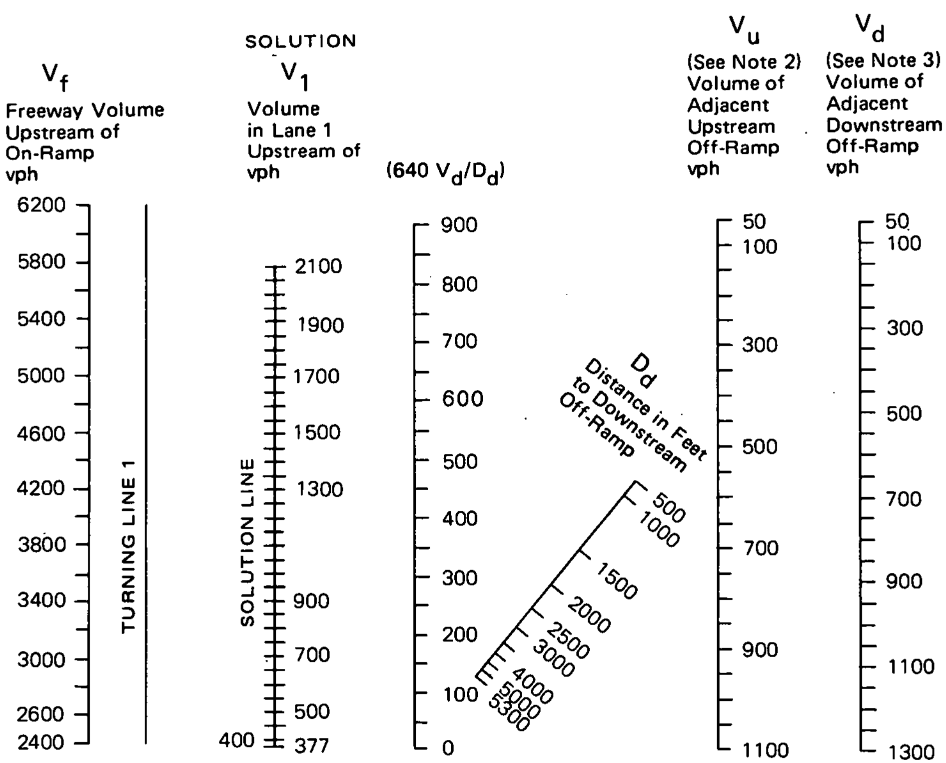
Conditions for Use:

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:

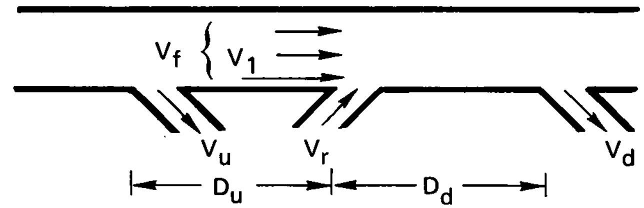
1. Draw a line from V_f value to V_r value; read solution on V_1 line.

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:



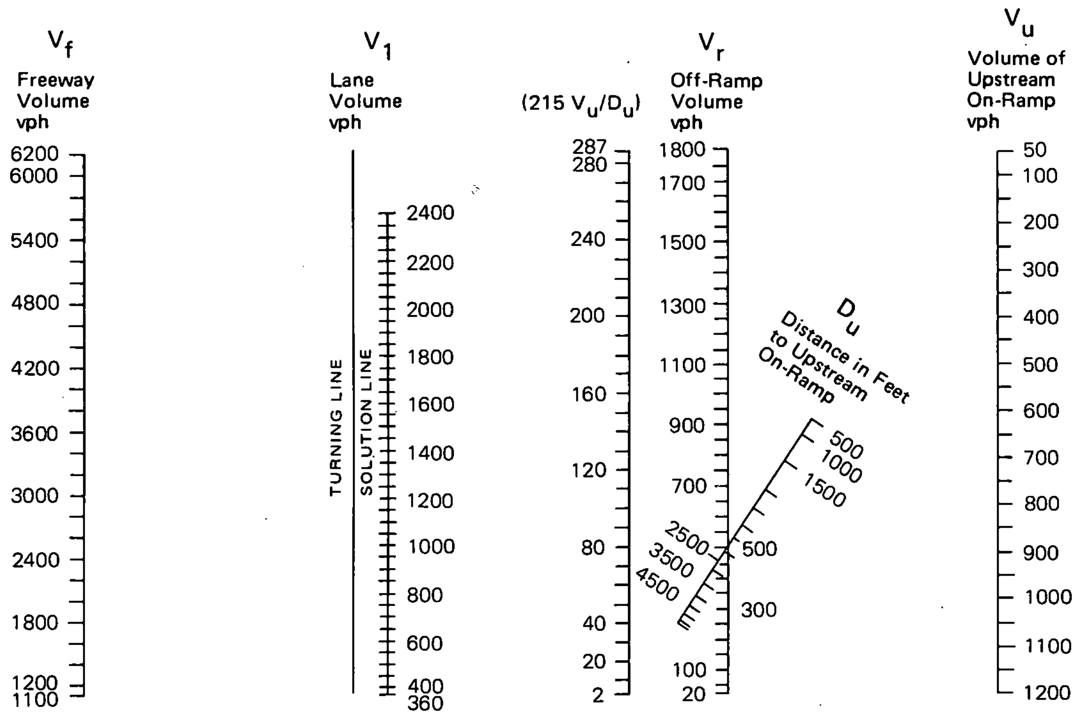
Conditions for Use:

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:

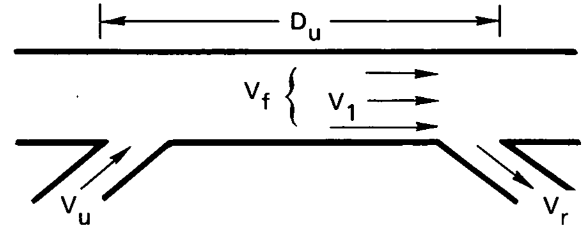
1. Draw a line from V_f value to V_u value, intersecting turning line 1.
2. Draw a line from V_d value to D_d value, intersecting 640 V_d/D_d line.
3. Draw a line from the step 1 intersection with turning line 1 to the 640 V_d/D_d value of step 2; read solution at intersection with V_1 line.

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:



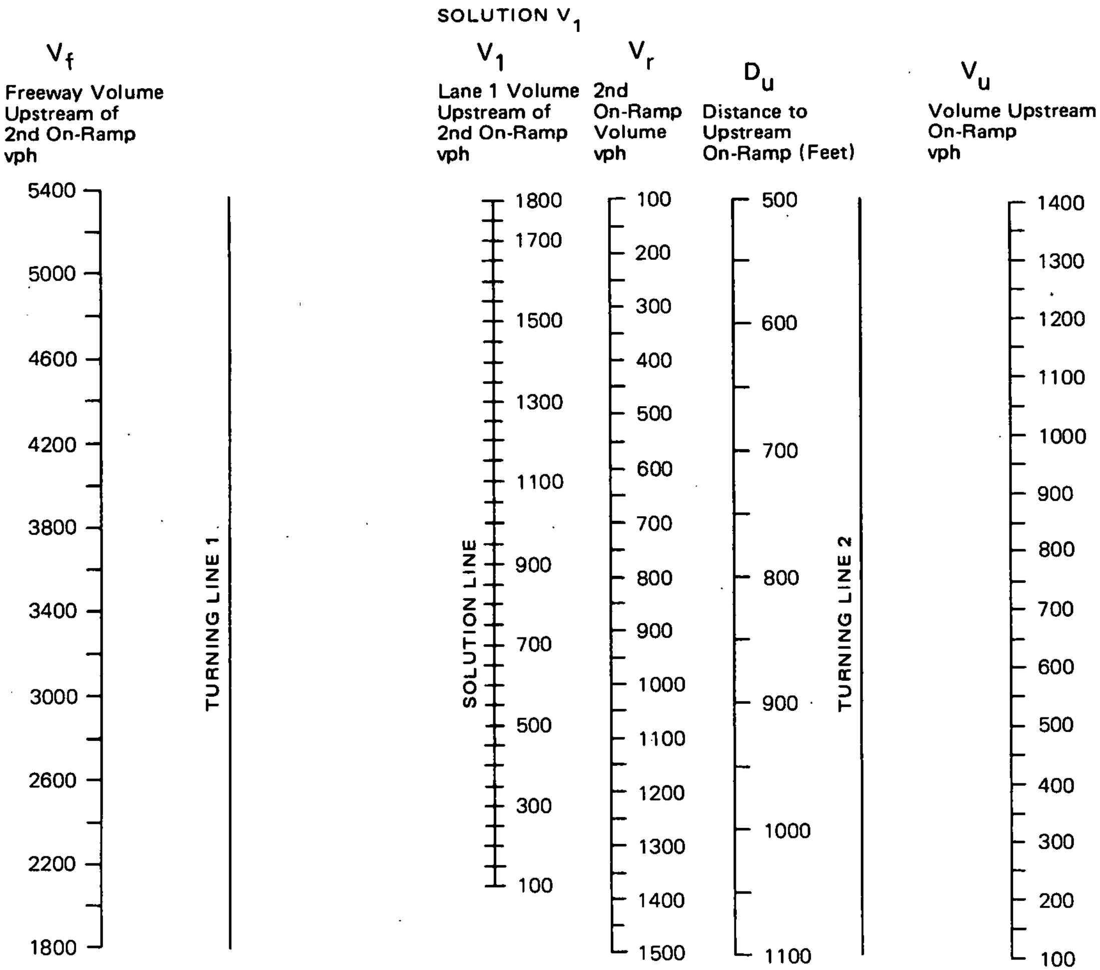
Conditions for Use:

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:

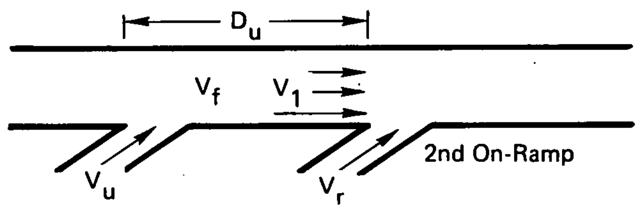
1. Draw a line from V_f value to V_r value, intersecting the turning line.
2. Draw a line from V_u value to D_u value, intersecting the $215\ V_u/D_u$ line.
3. Draw a line from intersection point on the turning line of step 1 to the value on the $215\ V_r/D_u$ line of step 2; read solution on V_1 line.

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:



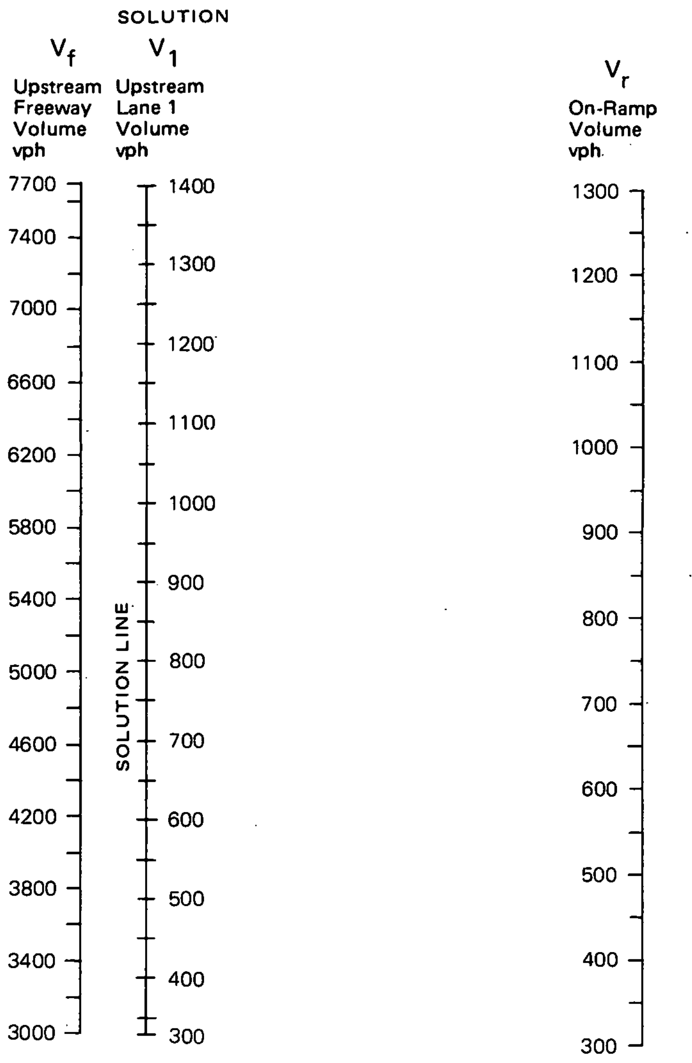
Conditions for Use:

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:

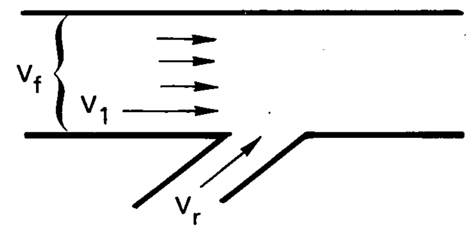
1. Draw a line from V_f value to V_r value, intersecting turning line 1.
2. Draw a line from V_u value to D_u value, intersecting turning line 2.
3. Draw a line from intersection on turning line 1 of step 1 to the intersection on turning line 2 of step 2; read solution on V_1 line.

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:



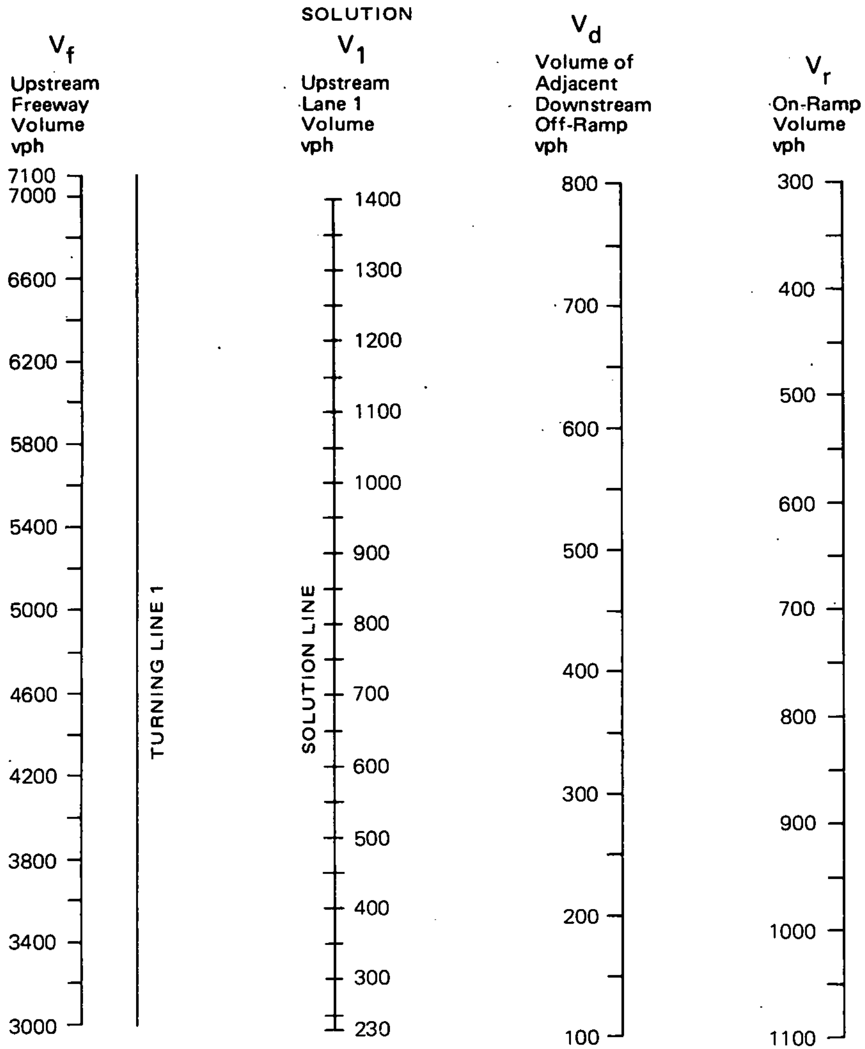
Conditions for Use:

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:

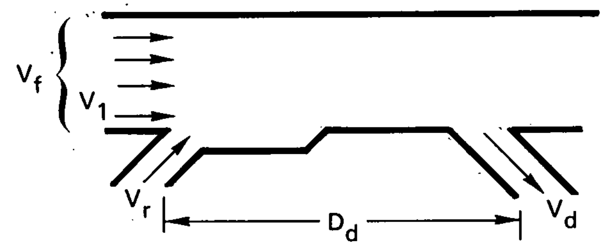
1. Draw a line from V_f value to V_r value; read solution on V_1 line.

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:



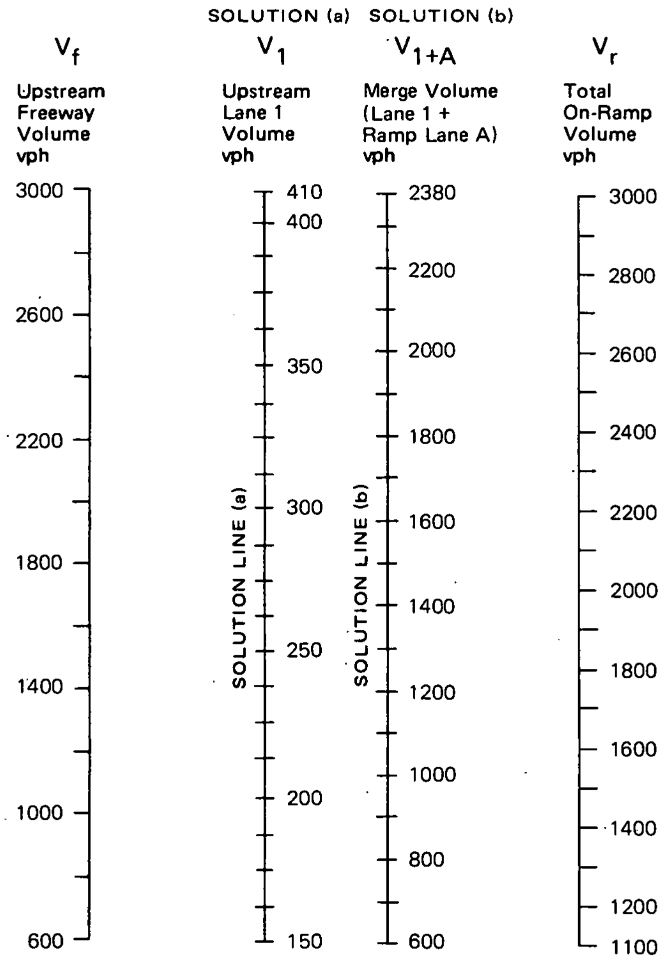
Conditions for Use:

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:

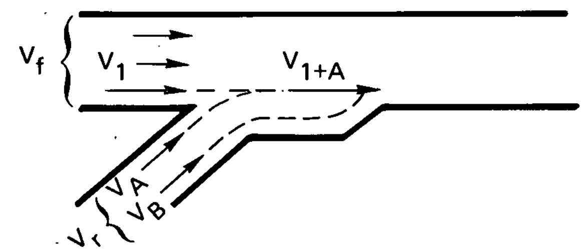
1. Draw a line from V_f value to V_r value; intersecting turning line.
2. Draw line from intersection of step 1 with turning line 1 to V_d value; read result on solution line.

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:



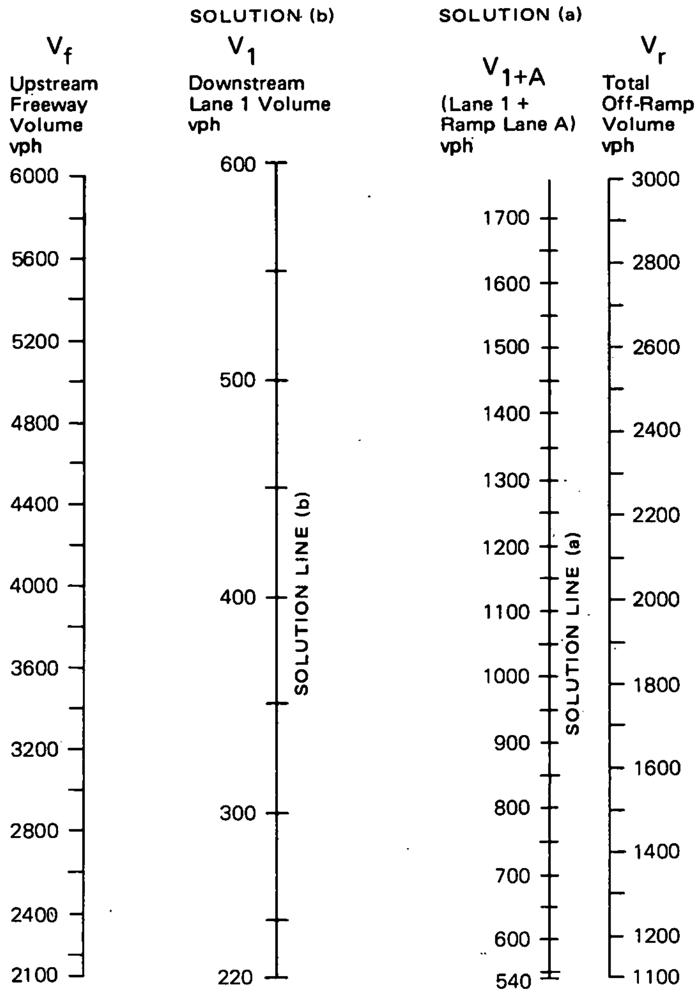
Conditions for Use:

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:

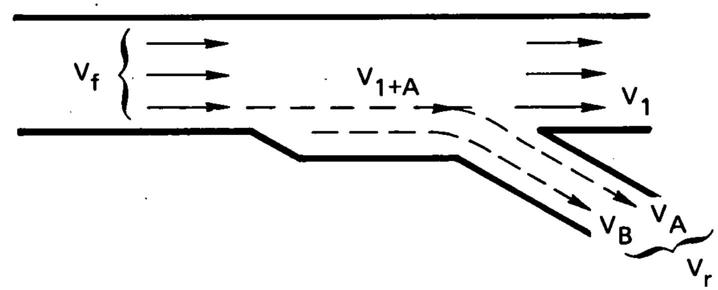
1. Draw line from V_f value to V_r value. Read V_1 on V_1 line, V_{1+A} on V_{1+A} line.
2. Compute $V_A = V_{1+A} - V_1$; $V_B = V_r - V_A$.
3. Check Level of Service for two merge points: $V_{m1} = V_1 + V_A$ and $V_{m2} = V_{1+A} + V_B$.

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:



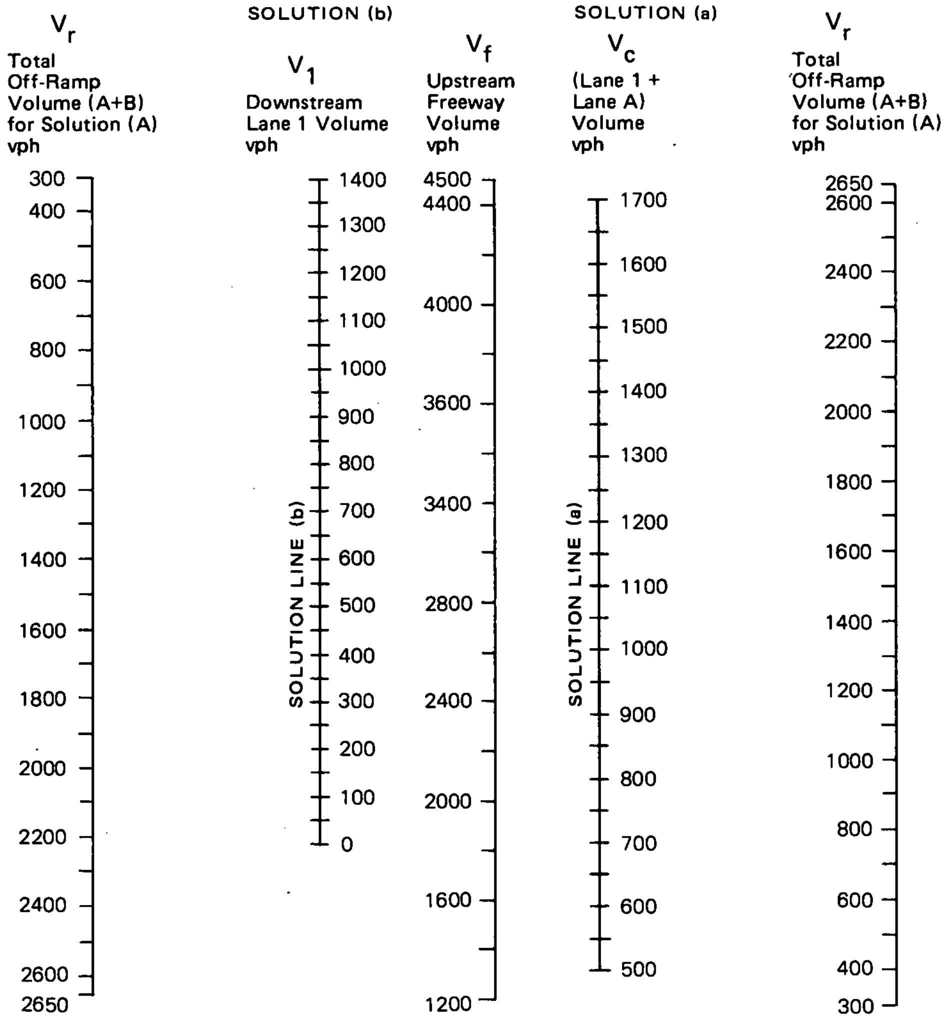
Conditions for Use:

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 $V_{d2} = V_B$.

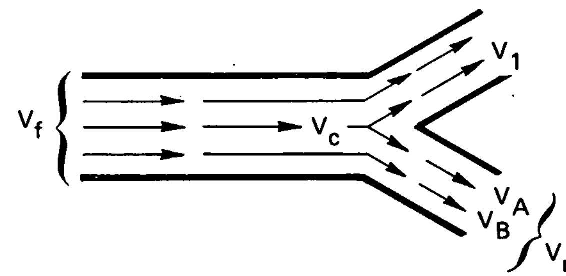
Figure I.5-12. Determination of lane 1 volume upstream of two-lane off-ramps on six-lane freeways (three lanes in each direction).



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

FIGURE FOR USE IN THE ANALYSIS OF RAMPS AND RAMP JUNCTIONS

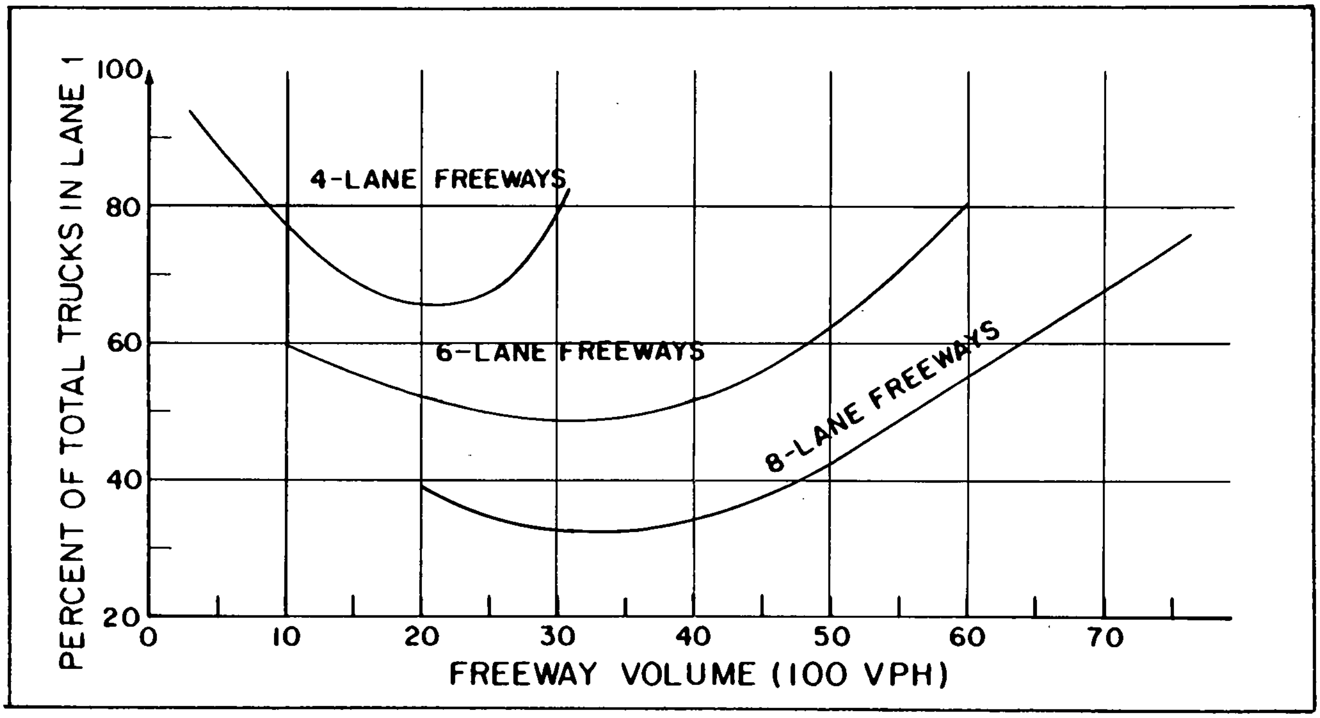


Figure 5-6. Truck presence in lane 1.

CHAPTER 6

FREEWAY SYSTEMS

CONTENTS

I. INTRODUCTION

Chapters 3, 4, and 5 of this manual have treated in detail the planning, design, and analysis of basic freeway segments, weaving areas, and ramp junctions, respectively. This chapter addresses how these elements may be combined into a complete freeway design or analysis, and a number of special features that may be present and significantly impact operations. Because of the many complexities of freeway system operations, these procedures tend to be more approximate and less precise than those applied to specific freeway subsections. They nevertheless provide a basis for insight and understanding of system effects.

II. COMBINED ANALYSIS OF FREEWAY SEGMENTS

DESIGN ANALYSIS

Procedures

When approaching the design use of procedures herein, it is necessary to consider the kinds of information that generally would be available, and what results are desired. Capacity analysis is only one of several inputs into the design process. Others include geometric standards, safety standards, standards for signing, and so on.

Capacity analysis procedures are used primarily in the design of cross-sectional elements (number of lanes, lane widths, shoulders) and in the selection of lane configurations for individual freeway elements. In general, the following information is required for a design analysis:

- Horizontal and vertical alignments.
- Approximate location of ramps and interchanges.
- Forecasted demand volumes.
- Forecasted demand characteristics, such as, to name a few, the percentages of trucks, buses, and recreational vehicles in the traffic stream, and PHF.

The principal problem in coordinating the design analysis of an overall freeway facility is the segmenting of the freeway into component parts for individual considertion via the methods of Chapters 3, 4, and 5. In general, the following guidelines may be used:

1. Each section of freeway between ramps or major junctions should be considered to be a separate "basic freeway segment."

2. Within these basic freeway segments, any grade of more than ¼ mi (for grades $\geq$ 3 percent) or ½ mi (for grades $<$ 3 percent) must be considered as a separate "basic freeway segment." Any sharp change in terrain, such as from level-to-rolling terrain, would also necessitate the division of a single segment into separate subsegments. Long basic segments with no single grade of significance may be considered as extended segments of level, rolling, or mountainous terrain, as defined in Chapter 3. Downgrade segments would normally be considered to be "level terrain" unless local data allow for more specific treatment (see Chapter 3).

3. Each ramp junction should be considered separately, in combination with the adjacent downstream ramp, and in conjunction with the adjacent upstream ramp. Ramps that are *clearly* part of a weaving section would *not* be analyzed using ramp procedures, but would be treated in step 4 below.

4. Potential weaving and multiple weaving areas should be investigated as such. "Potential" is used in that some segments may turn out to be either weaving areas or ramp combinations, depending on the final configuration adopted.

In application, these guidelines lead to fairly straightforward computations in the following sequence:

1. Establish design level of service, demand volume and traffic characteristics, horizontal and vertical alignments, and approximate ramp locations.

2. Determine the basic number of lanes required for each of the basic freeway segments identified as previously noted, using the procedures detailed in Chapter 3. The basic number of lanes for each ramp may also be determined using techniques described in Chapter 5.

3. The results of step 2 will suggest probable configurations for ramp junctions and potential weaving areas. Analyze each ramp junction from three points of view: (a) as an isolated ramp, (b) in combination with the adjacent downstream ramp, and (c) in combination with the adjacent upstream ramp using the procedures of Chapter 5. *Usually, one or two of these views will be invalidated by those procedures,* but in other cases, there will be more than one valid analysis. In such cases, the analysis indicating the poorest operations or level of service is taken as the controlling solution.

4. Weaving areas should be analyzed using the procedures of Chapter 4 to determine likely operating conditions. Note that in design, the case of an on-ramp followed by an off-ramp must be looked at both as a potential weaving section with an auxiliary lane and a ramp combination without an auxiliary.

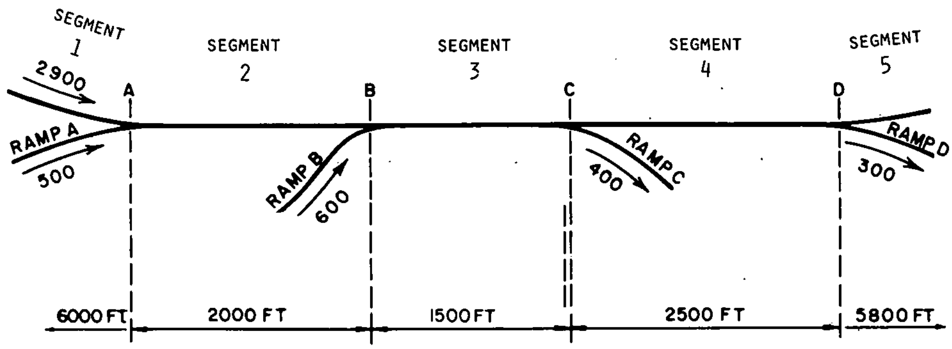
5. If the results of steps 3 and 4 are unsatisfactory, consideration may be given to:

- Altering the number and/or location of ramps (which may affect demand distribution).
- Changing the design of ramps and/or mainline segments determined in step 2 to create new configurations.
- Changing the design of major interchanges to achieve different configurations, reduce weaving, etc.

Steps 2 through 4 are then repeated.

Sample Calculation

The design problem indicated in Figure 6-1 illustrates the



Flow Rates on Mainline Segments:

segment	volume
1.	2900 pcph
2.	2900 + 500 = 3400 pcph
3.	3400 + 600 = 4000 pcph
4.	4000 - 400 = 3600 pcph
5.	3600 - 300 = 3300 pcph

Rolling Terrain

Design Objective :
Level of Service B, desirable
Level of Service C, minimum

Figure 6-1. Sample design problem.

foregoing procedures. *Note that the given demand volumes are already expressed as peak rates of flow in pcph.*

Step 1—Establish Demand, Alignment, Ramp Location

These are indicated in Figure 6-1.

Step 2—Determine Basic Number of Lanes for Open Freeway Segments and Ramps

The demand on each open freeway segment is shown in Figure 6-1. Using Table 3-1 criteria directly for level-of-service B, the number of lanes in each may be found. Note that 12-ft lanes, adequate lateral clearance, and 70-mph AHS are to be provided as the result of design decisions.

Segment	Flow Rate	No. of Lanes Req'd
1	2,900	3
2	3,400	3–4
3	4,000	4
4	3,600	3–4
5	3,300	3

Table 5-6 may be used to estimate the number of lanes required for each of the ramps. It will be assumed that all ramps will be designed for a minimum of 40-mph design speed. Using these criteria, all of the ramps of Figure 6-1 are *single-lane ramps.*

On the basis of these results, the configuration illustrated in Figure 6-2 is most likely to be appropriate. Note that in this configuration, because there is an auxiliary lane between ramps

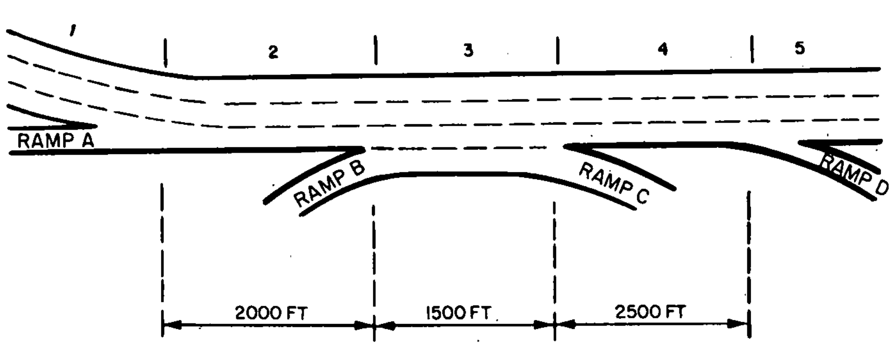


Figure 6-2. A likely design for sample problem.

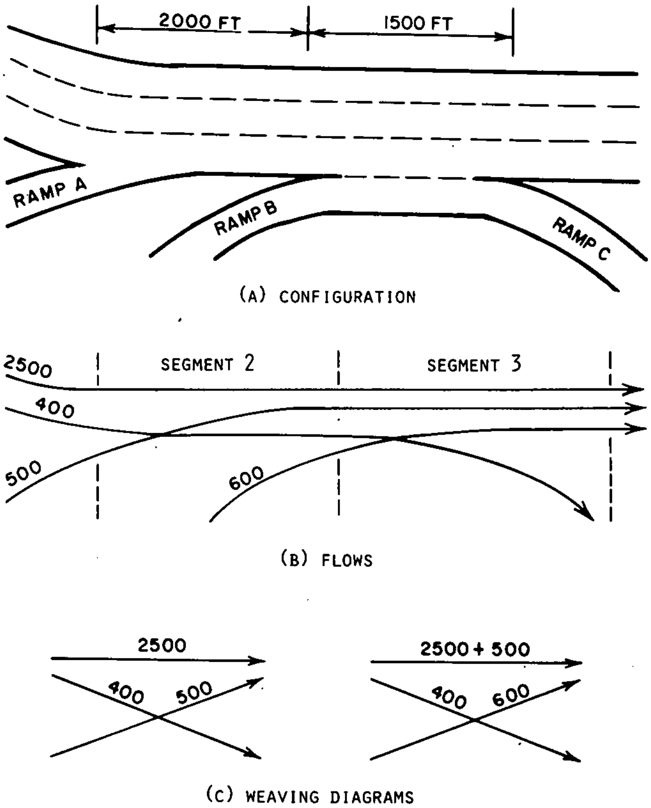


Figure 6-3. Consideration of multiple weave.

B and C, this is a weaving area. Segments 2, 3, and 4 together comprise a multiple weaving area.

Step 3—Analyze Ramp Junctions

Given that ramps B and C are definitely part of a weaving section for the trial design of Figure 6-2, the following ramp combinations remain to be analyzed using ramp procedures:

- Ramp A, Isolated
- Ramp D, Isolated

Ramp A and ramp D could conceivably be considered both as isolated ramps with a simple weaving section in segment 3, or as part of a multiple weaving configuration *with* segment 3. Both cases would be analyzed.

Ramp A. Isolated. From Table 5-2, the nomograph depicted in Figure I.5-6 is used. As the ramp is taken to be isolated, V_u is set at 50 (note 2, Fig. I.5-6) and 640 V_d/D_d at 5 (note 3, Fig. I.5-6).

V_1 = 600 (Fig. 1.5-6)

V_m = 600 + 500 = 1,100 Level-of-Service B (Table 5-1)

Ramp D. Isolated. From Table 5-2, the nomograph depicted in Figure I.5-7 is used. Thus, for:

$V_r = 300$, $V_f = 3{,}600$, 215 $V_u/D_u = 2$

("Conditions for Use," note 2):

V_1 = 1,050 (Fig. 1.5-7)

V_d = 1,050 Level-of-Service B (Table 5-1)

Ramps B and C should not be considered as a part of a ramp configuration, because the trial design of Figure 6-2 shows them to be in a *weaving* configuration, and, as such, they are analyzed in step 4.

Step 4—Analyze Potential Weaving Areas

Segments 2 and 3 should be considered as a multiple weave. For the purposes of this analysis, all off-ramp vehicles at C will be assumed to originate from the freeway mainline, a worst-case assumption. Figure 6-3 depicts the resulting flows and weaving diagrams.

Segment 2. Because one of the segment 2 weaving movements is made with no lane change and another with one lane change, this is a Type B section. For segment 2:

$$VR = 900/3{,}400 = 0.26$$

$$R = 400/900 = 0.44$$

From Table 4-3, the speed of weaving and nonweaving vehicles is computed using the equation:

$$S_w \text{ or } S_{nw} = 15 + \frac{50}{1 + a(1 + VR)^b (v/N)^c / L^d}$$

where, for unconstrained Type B sections:

	S_w Computation	S_{nw} Computation
a =	0.100	0.020
b =	1.2	2.0
c =	0.77	1.42
d =	0.50	0.95

and:

$$v = 3{,}400 \text{ pcph}$$
$$N = 3 \text{ lanes}$$
$$L = 2{,}000 \text{ ft}$$

This results in the following estimates of speed for unconstrained operation:

$$S_w = 45.0 \text{ mph}$$
$$S_{nw} = 48.2 \text{ mph}$$

The number of weaving lanes used is now computed, using the equation given in Table 4-4. This is done to check on whether operations are actually unconstrained:

$$N_w = N\{0.085 + 0.703\ VR + (234.8/L) - 0.018(S_{nw} - S_w)\}$$

where S_{nw} and S_w are as computed above. Substituting the appropriate values:

$$N_w = 1.00 \text{ lanes}$$

Because this is less than the maximum value of 3.50 lanes for Type B sections (Table 4-4), the section is unconstrained, and the original estimates of weaving and nonweaving speeds are taken to be correct. From Table 4-6, both weaving and nonweaving speeds are within the level-of-service C boundaries, and meet the minimum desired for the design.

Segment 3. This should be considered as a Type A weaving area, because it has an auxiliary lane, as shown in Figure 6-3, and all weaving vehicles make at least one lane change. Note that consideration of segment 3 of the multiple weave is the same as considering it as a simple weaving section. For segment 3:

$$VR = 1{,}000/4{,}000 = 0.25$$

$$R = 400/1{,}000 = 0.40$$

From Table 4-3, for unconstrained Type A weaving areas:

	S_w Computation	S_{nw} Computation
a =	0.226	0.020
b =	2.2	4.0
c =	1.00	1.30
d =	0.90	1.00

and:

$$v = 4{,}000 \text{ pcph}$$
$$N = 4 \text{ lanes}$$
$$L = 1{,}500 \text{ ft}$$

Then:

$$S_w = 48.1 \text{ mph}$$
$$S_{nw} = 54.7 \text{ mph}$$

From Table 4-4, the minimum number of weaving lanes needed to support unconstrained operation is:

$$N_w = 2.19\ NVR^{0.571} L_H^{0.234}/S_w^{0.438}$$

$$N_w = 1.37 \text{ lanes}$$

Because this is less than the maximum value of 1.4 lanes given in Table 4-4, the operation is unconstrained, and the computed speeds are correct. From Table 4-6, both weaving and nonweaving vehicles experience level-of-service B operation, which is within the desired range for the design under consideration.

Segments 3 and 4 should now be looked at as a multiple weaving area, as is shown in Figure 6-4. Again, it will be assumed that no on-ramp vehicles at B leave the freeway at C or D (a worst-case assumption).

Segment 3, in this case, remains the same as previously, so no additional analysis is required.

Segment 4, however, should be analyzed as a Type B weaving section, because one weaving movement is made with no lane change, and the other requires only one lane change. For segment 4:

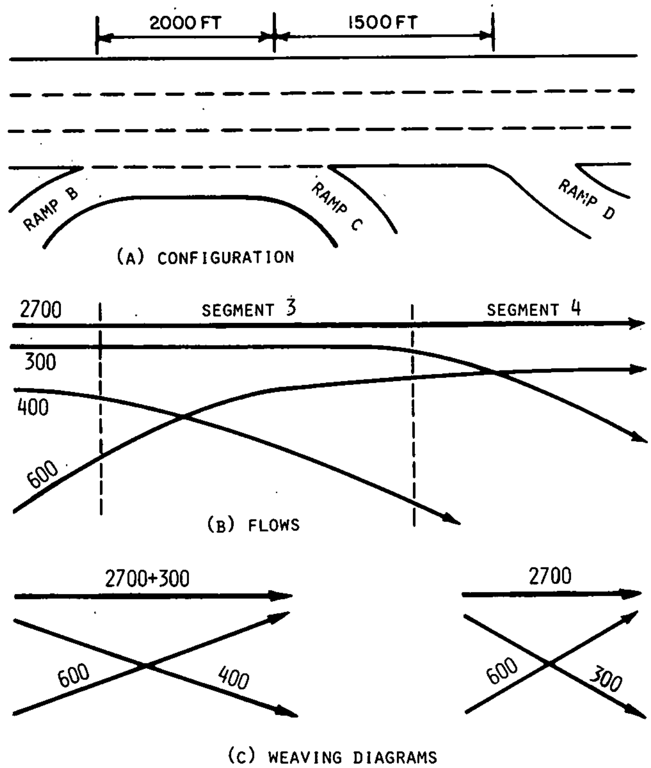


Figure 6-4. Consideration of multiple weave.

$$VR = 900/3{,}600 = 0.25$$

$$R = 300/900 = 0.33$$

From Table 4-3, for unconstrained Type B weaving sections:

	S_w Computation	S_{nw} Computation
a =	0.100	0.020
b =	1.2	2.0
c =	0.77	1.42
d =	0.50	0.95

and:

$$v = 3{,}600 \text{ pcph}$$
$$N = 3 \text{ lanes}$$
$$L = 2{,}500 \text{ ft}$$

Then:

$$S_w = 46.0 \text{ mph}$$
$$S_{nw} = 49.9 \text{ mph}$$

From Table 4-4, the number of weaving lanes required for unconstrained operation is:

$$N_w = N\{0.085 + 0.703\ VR + (234.7/L) - 0.018(S_{nw} - S_w)\}$$

$$N_w = 0.86 \text{ lanes}$$

Because this is less than the maximum allowable value of 3.50 lanes (Table 4-4), the operation is unconstrained, and the speeds computed are correct. From Table 4-6, the level of service for weaving vehicles is C, and for nonweaving vehicles it is C. These are both within the minimum criteria established for the design problem.

Given that all of the weaving areas and ramp junctions meet the minimum LOS criteria established for the design, the trial design of Figure 6-2 would appear to be acceptable for implementation.

OPERATIONAL ANALYSIS

The analysis approach for total freeway evaluation is quite similar to the design approach, but is simpler in that there are no alternates to consider. All volumes, geometrics, and traffic conditions are known, and the freeway may be segmented with certain knowledge of ramp locations, weaving configurations, and other features.

Once the freeway has been divided into uniform segments according to the guidelines previously noted, the following computational sequence may be followed:

1. Determine the level of service for each potential basic freeway segment using the procedures of Chapter 3.
2. Determine the level of service for each ramp junction, considering each ramp:

- As an isolated ramp.
- In conjunction with the adjacent downstream ramp.
- In conjunction with the adjacent upstream ramp.

These checks are made using the procedures of Chapter 5. Ramps that are *clearly* part of a *weaving configuration* would not be examined using Chapter 5 procedures.

3. Determine the level of service of each weaving and multiple weaving segment using the procedures of Chapter 4.

Where a given segment falls under several of these analyses, the analysis resulting in the worst level of service is the controlling solution.

Once the analysis of segments is complete, the overall interpretation of results is subject to the exercise of judgment. As was presented in Chapter 3, there are general guidelines on the extent of influence of weaving areas and ramp junctions. Other research has yielded varying results that tend to indicate that the extent of influence of any individual element can range from as little as several hundred feet to more than a mile. Inasmuch as it is not possible to exactly determine the extent of such impacts, weaving and ramp junction areas that operate at levels of service poorer than adjacent segments should be viewed with caution because they may affect the operation of upstream sections.

A graphic technique presented in Figure 6-5 is useful as a tool to get a pictorial overview of overall operations. The technique assumes standard areas of influence as follows:

On-ramps—500 ft upstream, 2,500 ft downstream
Off-ramp—2,500 ft upstream, 500 ft downstream
Weaving areas—500 ft upstream of on-ramp and 500 ft downstream of off-ramp.

Levels of service are plotted for each segment. The illustration shown clearly indicates that the "bottleneck" or limiting segment is the weaving area of segment 4. As long as the indicated operations hold, segment 4 will operate poorly, at level-of-service E, while other segments *could* operate at levels B and C if not prevented from doing so by spillback from segment 4. As noted previously, the effect of segment 4 on upstream segments cannot be determined with certainty. What can be said is that segment 4 should not have an extended effect as long as it does not break down, in other words slip to level-of-service F.

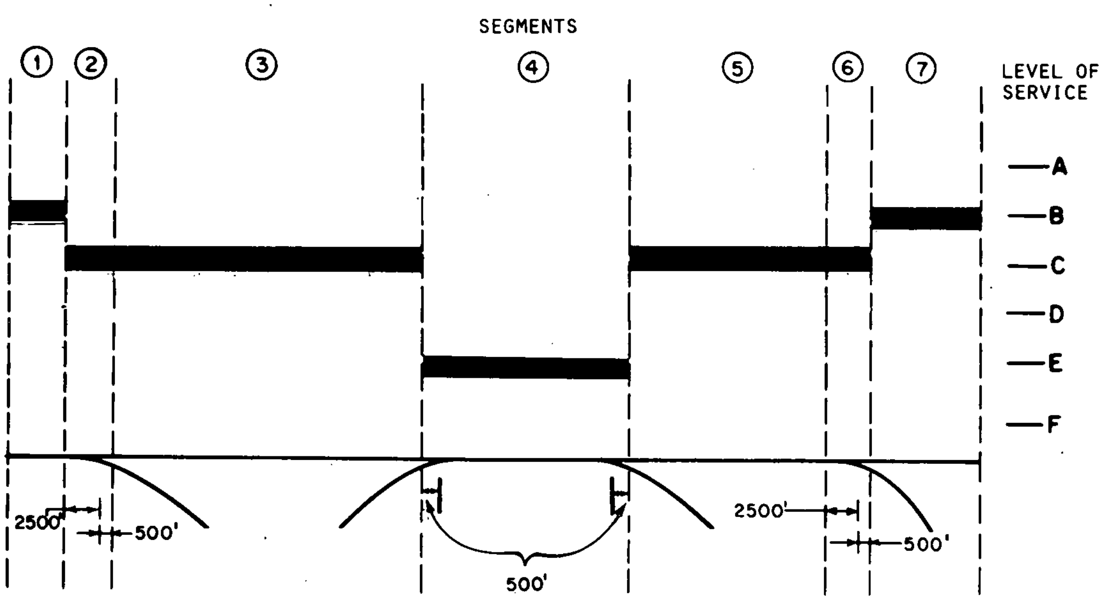


Figure 6-5. Graphic representation of overall level of service.

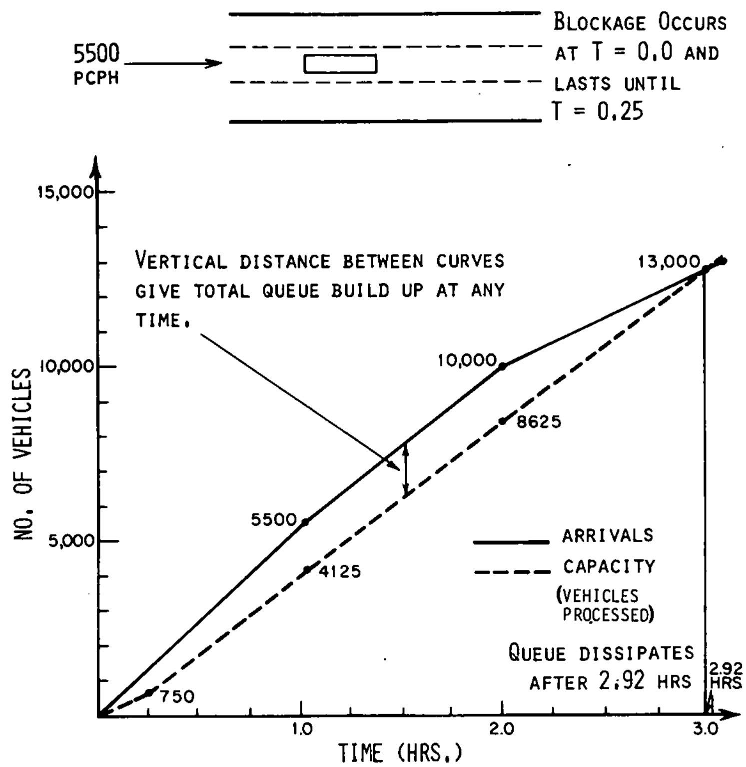


Figure 6-6. Effects of breakdown illustrated.

If more flow is added, segment 4 would be the first to break down—and segment 4 is the most susceptible to breakdowns caused by incidents, weather, or other extraneous factors. Once breakdown occurs here, the spatial and time extent of the breakdown can be estimated using techniques detailed in the next section.

ANALYSIS OF BREAKDOWN CONDITIONS

The behavior of traffic streams during and immediately after the occurrence of a breakdown is not well understood. A critical issue, however, is the rate at which vehicles can depart a standing queue in an uninterrupted traffic stream. In many cases, vehicles are unable to depart a standing queue at the normal capacity rate of 2,000 pcphpl. In their studies of uninterrupted flow characteristics, Edie and others (*30*) have noted that the relationships among speed, density, and flow may be discontinuous at the point of capacity, and that the maximum rate of flow of vehicles departing a queue may be less than capacity under stable flow. Various observations of freeway queue departure rates range from as low as 1,500 pcphpl to as high as 2,000 pcphpl. Local driving characteristics have a major influence on this effect, which ranges from a significant reduction in capacity (compared to 2,000 pcphpl) of up to 25 percent to cases in which there is virtually no reduction.

Where the information of standing queues due to incidents or permanent bottlenecks does cause a reduction in lane capacity, the impact of this on the extent of queuing and its dissipation can be major.

Consider the case illustrated in Figure 6-6: a three-lane freeway segment operating under ideal conditions with a demand of 5,500 pcph during a peak hour, 4,500 pcph during the hour after the peak, and 3,000 pcph thereafter. What will occur if an incident blocks one lane for 15 min at the beginning of the peak period? For illustration purposes, it is assumed that the formation of a standing queue reduces the lane capacity to 1,500 pcphpl.

The following operational effects should be anticipated:

1. When blockage occurs, capacity immediately drops from 6,000 pcph to 4,000 pcph or lower, which quickly creates stop-and-go queues due to the 5,500 pcph demand. This further deteriorates capacity to 3,000 pcph (assuming a drop to 1,500 pcphpl with two lanes open). Thus, during the first 15 min. $5{,}500/4 = 1{,}375$ pc arrive and only $3{,}000/4 = 750$ pc are processed, and a queue of 625 pc is formed behind the blockage.
2. After the blockage is removed, capacity improves to $1{,}500 \times 3 = 4{,}500$ pcph because standing queues still exist. Full capacity cannot be regained until all queues are dissipated. Thus, in the ensuing 45 min, $5{,}500 \times 3/4$ or 4,125 pc arrive and $4{,}500 \times 3/4$ or 3,375 pc are processed. The queue continues to build to $625 + 750 = 1{,}375$ pc.
3. During the second hour, 4,500 pc arrive, and exactly 4,500 pc are processed. The queue is stable, but it does not dissipate.
4. Thereafter, the queue will dissipate, as 3,000 pcph arrive, and 4,500 pcph may be processed. The 1,375 queued vehicles dissipate in $1{,}375/(4{,}500 - 3{,}000) = 0.92$ hours, and full capacity is restored, some 2.92 hours after the occurrence of a 15-min blockage. The queue length (assuming three lanes and 40 ft per vehicle) reached $(1{,}375/3) \times 40 = 18{,}333$ ft, or more than 3 mi at its peak, which lasted for one full hour.

Figure 6-6 illustrates this analysis in graphic form. The illustration here is extreme, using the assumed queue discharge rate of 1,500 pcphpl for computational simplicity. In many areas, this value will be exceeded. Nevertheless, the expanded time and spatial effects of a breakdown are clearly indicated, as is the need to consider potential incidents in the analysis of freeway system operation. The value of 40 ft per queued vehicle is approximate, and is based on the assumption of stop-and-go movement within the queue.

This technique is approximate, and does not account for many microscopic properties of unstable freeway flows. It is, however, useful in estimating the effect of a breakdown in one location on overall operations. However, as the queue discharge rate varies widely depending on local conditions, such an analysis should be coordinated with sample field measurements of an appropriate discharge rate.

III. FREEWAY SURVEILLANCE AND CONTROL

A complete treatment of this subject is beyond the current scope of this report, but there are excellent references on the subject. The interested reader is referred to a state-of-the-art report by FHWA (*2*) and to NCHRP Report 232 (*3*).

BACKGROUND

It is important to recognize that freeway surveillance and control is employed relatively commonly and that it has a number of potential advantages. Some of the key potential advantages are:

- Relief of congestion by virtue of exercising control over excessive entries.
- Decrease in delay, for the same reasons.
- Protection of level of service.
- Response to freeway incidents.

There is an interesting distinction between the first and third items: a freeway can be controlled with a single objective—to avoid breakdown (by restricting entries at appropriate locations), or it can be controlled so that some specified level of service is maintained. In the latter case, one may specify ramp metering rates in *anticipation* of future growth in demand. Thus, freeway management can be used at the planning stage, and not simply as an operational correction. It is rare to implement a control scheme which diverts vehicles from the freeway to maintain a level of service better than E.

A freeway management system may be planned, or it may be responsive to traffic variations. Further, it may or may not have explicit response to incidents.

CONTROL ELEMENTS

The principal elements that are added to the facility because of a surveillance and control/management effort are:

- Vehicular detectors.
- Ramp metering.
- Video and/or other observation.
- Control policies, implemented by central computer or other hardware.
- Static and perhaps variable message signing to inform motorists of alternate routes and/or conditions.

Of these elements, the ramp metering is the most essential, because it is the most positive control action exercised. Chapter 5 has addressed the lack of detailed knowledge on lane 1 flow effects of metering, but its known advantages in control are in "smoothing out" disruptive arrival platoons. It is useful to consider an illustration of the ramp and mainline effects of a metered ramp in order to make that discussion meaningful.

Consider the situation of Figure 6-7: an on-ramp has the demand depicted ranging from 250 to 575 vph (flow rate); the mainline has 3,500 vph already on it, with a capacity of 4,000 vph. Clearly, if the ramp demand is allowed to enter, a level-of-service F situation will occur upstream of the ramp. How may the ramp be metered to avoid this? What delay and queue will occur at the ramp because of this?

The ramp must be metered at 500 vph to avoid exceeding capacity on the mainline. This means 1 vehicle every (3,600/500) = 7.2 sec. With a green-red signal at the ramp, this would usually mean 2 sec of green followed by 5.2 sec of red. This may be implemented in a number of ways, including a conventional electromechanical controller, another local controller (possibly a microprocessor), or a command from a remote computer.

From Figure 6-7, ramp demand reaches the 500-vph level at approximately 5:09 PM, and does not decrease below that level again until 5:51 PM. In the interim, a queue will form and continue to enlarge, as illustrated in Figure 6-8.

Figure 6-8 is a plot of ramp vehicles vs. time. At any given time, the horizontal distance between the demand and vehicles serviced curves is the delay/vehicle, and the vertical distance between the curves is the queue length. From Figure 6-8, the maximum delay/vehicle would occur at 5:51 PM, and would be approximately 5 min. The queue length at this time would be about 50 vehicles.

It should be noted here, however, that many drivers will be unwilling to accept 5-min delays. (In Los Angeles, 1- or 2-min delays are the average usually observed.) Many of the queued vehicles might be expected to seek alternate routings to avoid the delay. Thus, a critical consideration on ramp metering is the availability of alternate routes and the impact of diverted traffic on those routes.

It shouild also be noted that some freeway management systems operate on nothing more than application of the above principle in a consecutive set of freeway segments: the section input is monitored; the segment capacity is known; the ramp input is not allowed to cause mainline flow to exceed capacity.

DETERMINATION OF PROBLEMS AND CONTROL

Freeway management is more frequently motivated by operational problems: one or more sections are "bottlenecks," with significant mainline congestion occurring. The problem is then to alleviate the congestion and to maintain a level of service better than F. In some cases, the project includes construction at some locations to provide additional capacity, or includes the incorporation of high-occupancy-vehicle lanes.

Although an entire treatment of freeway management is not appropriate in the present context, two problem areas deserve special mention: hidden bottlenecks, and origin-destination patterns.

Figure 6-9 depicts a hypothetical freeway with five sections, and with the input demands shown. Clearly, demand will exceed capacity in segment 3, and level-of-service F will result. Stop-and-go operation can occur in all upstream sections, depending on the duration over which demand exceeds capacity (i.e., over which the congestion has a chance to spread).

In practice, the capacities are not computed and one simply observes severe congestion in segment 2, caused by segment 3. The congestion may spread to segment 1 if the peak period is long enough or if segment 2 is short.

Assume that some physical reconstruction, perhaps coupled with decreasing the 5,300 pcph input via ramp metering further upstream, alleviates the problem. Lacking the capacity figures for *all* sections, one may overlook the fact that if segment 3

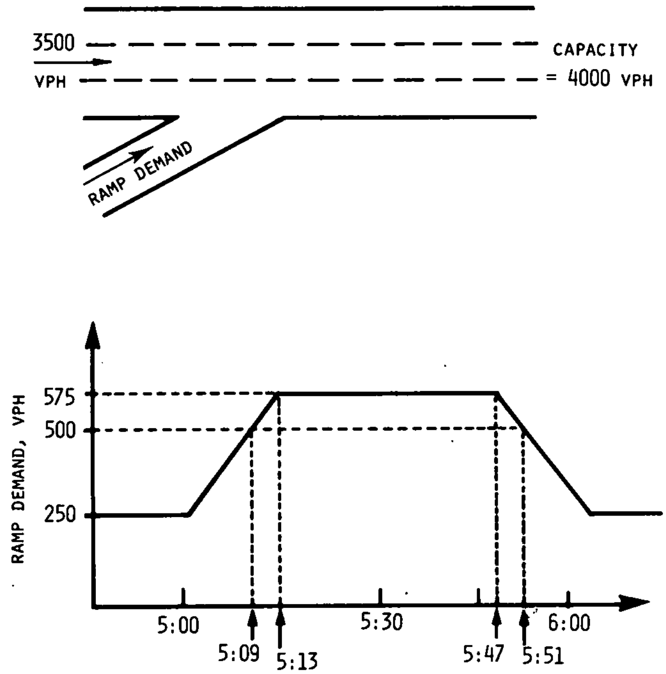


Figure 6-7. Illustration of a ramp-metering need.

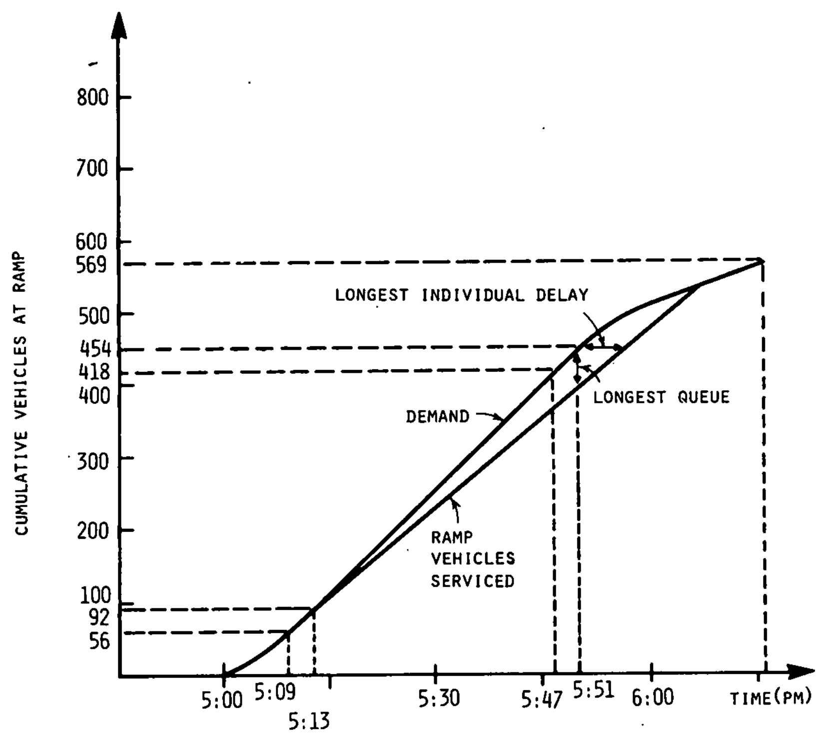


Figure 6-8. Plot of cumulative ramp demand and output.

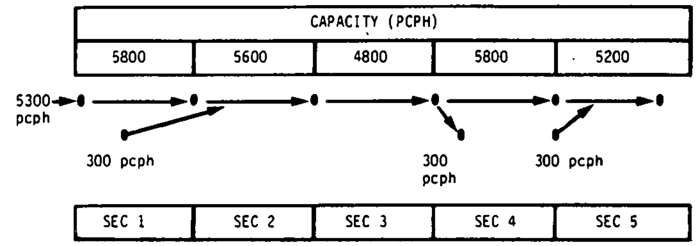


Figure 6-9. Potential for hidden bottlenecks.

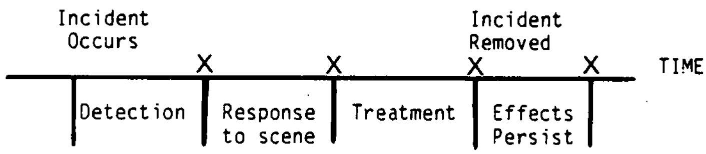


Figure 6-10. Phases of a traffic incident.

now outputs a flow rate higher than 5,200 pcph, a *bottleneck will appear at segment 5 for the first time.* It was always there, but only the solution of the segment 3 problem allowed the demand to attain levels necessary to exhibit it. That is, it was "hidden" by the upstream bottleneck in segment 3.

A complete capacity analysis of the facility should be conducted to avoid the "hidden bottleneck" problem. In doing so, changes in flow due to the improvements must be anticipated. For instance, is the off-ramp in segment 4 shown at a level of 300 pcph because it is the true demand or because it is the observed amount which could get past the original bottleneck? In addition, it must be recognized that the service flow rates in some sections (e.g., weaving sections) are functions of the traffic mix, which may change.

Because the flow pattern may be distorted, it is important to have some knowledge of the origin-destination pattern of traffic. Further, the origin-destination pattern influences what *can* be done and what *should* be done. Consider a freeway on which virtually all the outlying ramp entries stay on the facility until it terminates in the downtown area. Consider an identical physical facility, but with traffic using it for many short trips, with much outlying traffic exiting before another "layer" of traffic enters. The control opportunities and the equity of various control options vary radically between these two extremes.

INCIDENTS

Incidents occur relatively commonly on traffic facilities, although it is standard practice to design to a level of service for the nonincident condition. Clearly, incidents require attention because they:

- Disrupt the level of service being provided.
- Reduce the capacity radically.
- Present hazards to the motorists, particulary those directly involved.

Certainly incident response is desired in order to provide assistance to the motorists involved (tow, medical, police) as the need arises. Incident response can also be directed to minimizing the impact on other vehicles and to recovering use of the facility.

One study (*4*) showed that an incident removed to the shoulder on a three-lane facility still reduced capacity by one-third; a single-lane blockage reduced capacity by 50 percent; a two-lane blockage reduced capacity by 79 percent. In addition to the magnitude of the impact, the *duration* must also be considered. Refer to Figure 6-10, which identifies four critical phases of an incident history. Analogous to the ramp metering illustration under section heading, "Control Elements," the effect can persist long after the incident itself is removed because of the backups created. At one facility (*5*), it was estimated that peak-period incidents were responsible for more delay than recurrent peak period congestion at the location in question.

Incidents may be detected by video-observation, audio-reports (call-boxes, CB), or roadway sensors. Incidents may be responded to by some combination of required assistance, ramp restrictions or closure, and alternative route advisories. The control actions may be preplanned or dynamic decisions.

IV. CAPACITY OF FREEWAY WORK ZONES

One of the more frequently occurring disruptions to traffic flow on freeways is the required maintenance operations that must take place periodically, either as part of regular maintenance programs or to correct physical defects in the roadway, roadside, or supporting structures. An assessment of capacity is a necessary part of the planning of traffic control strategies during maintenance operations if severe disruptions and delays to traffic are to be avoided. This section details the results of several work zone capacity studies that provide considerable insight (*26, 27, 28*).

It should be noted that work zone capacities will vary depending on the exact nature of the work being done, the number and size of equipment at the site, and the exact location of equipment and crews with respect to moving lanes of traffic. Thus, the criteria and observations cited herein must be taken as averages subject to some variation.

CAPACITY WITH WORK CREW AT SITE

Figure 6-11 shows the range of capacities measured at several worksites in Texas, with an active work crew at the site. The observations are taken to be approximate capacities, as continuous queues of vehicles were present upstream of the sites included.

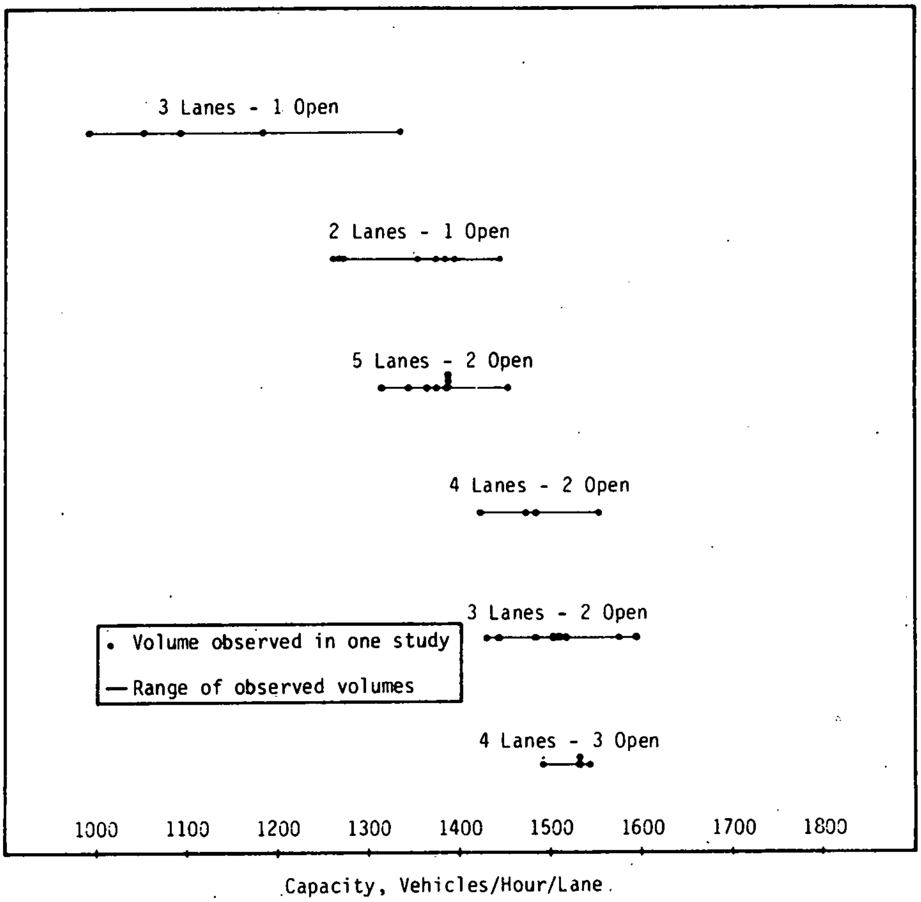


Figure 6-11. Range of observed work zone capacities—work crew at site. (Source: C.L. Dudek and S.H. Richards, "Traffic Capacity Through Urban Freeway Work Zones in Texas," *Transportation Research Record 869,* 1982)

The designation (A,B) is used to identify the various lane closure situations evaluated. "A" represents the normal number of lanes in one direction, while "B" represents the number of lanes open during maintenance operations. Table 6-1 gives the *average* capacity for each closure situation studied.

Average open-lane capacities for (4,2), (3,2), and (4,3) closures are approximately 1,500 vphpl. For (5,2) and (2,1) closures, the reductions are more severe, in the range of 1,350 vphpl. The capacities of (3,1) closures were the most damaging, averaging only 1,170 vphpl.

Figure 6-12 shows the cumulative distributions of the observed work zone capacities. The function of this illustration is to assist analysts in identifying the risks in using certain capacity values for given lane closures. For example, the 85th percentile capacity for a (3,1) closure is only 1,030 vphpl. The average capacity for this situation (1,170 vphpl) occurs at the 58th percentile. Thus, use of the average value in analysis leads to an overestimate in capacity (and consequently, an underestimate of queues and delays) in 42 percent of the cases to which it is applied, based on the observed range of values. Given the variation in observed capacities, analysts may wish to use 85th or higher percentile values, rather than averages, to reduce the risk of capacity overestimates.

Because of the limited amount of data available, it is not possible to statistically correlate capacity to the particular type of road work taking place. Table 6-2, however, tabulates individual observations vs. the type of maintenance operations for informational purposes. Note that flow through the work zone is also affected by presence of merging, diverging, or weaving movements, grades, alignment, truck presence, and other factors. The data in Table 6-2 reflect studies in both Texas and California. California observations represent peak flow rates, while the Texas data reflect full-hour capacities.

TABLE 6-1. MEASURED AVERAGE WORK ZONE CAPACITIES

NUMBER OF LANES A NORMAL	B OPEN	NUMBER OF STUDIES	AVERAGE CAPACITY (VPH)	(VPHPL)
3	1	7	1,170	1,170
2	1	8	1,340	1,340
5	2	8	2,740	1,370
4	2	4	2,960	1,480
3	2	9	2,980	1,490
4	3	4	4,560	1,520

SOURCE: Ref. *29*

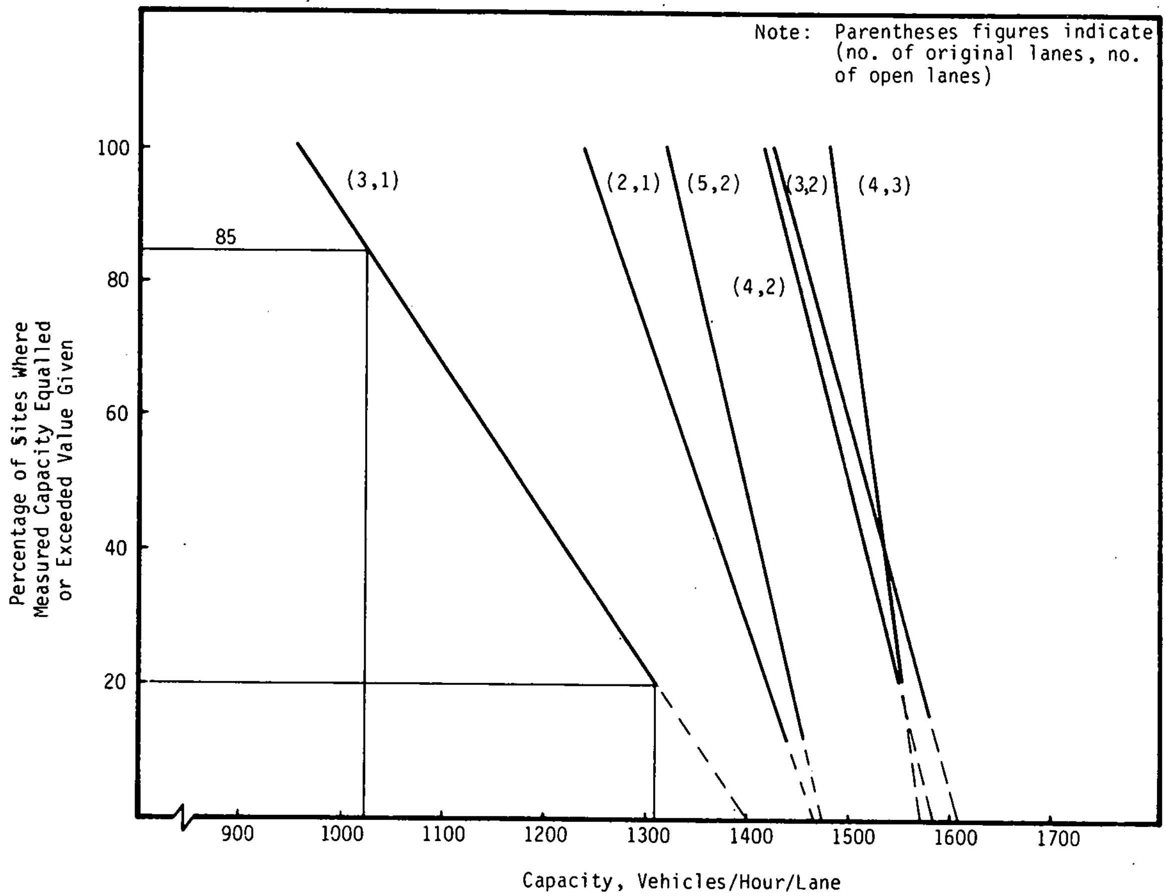


Figure 6-12. Cumulative distribution of observed work zone capacities. (Source: C.L. Dudek and S.H. Richards, "Traffic Capacity Through Urban Freeway Work Zones in Texas," *Transporation Research Record 869,* 1982)

TABLE 6-2. SUMMARY OF OBSERVED CAPACITIES FOR SOME TYPICAL OPERATIONS (VPH)*

NO. OF LANES IN ONE DIR.					
NORMAL OPER.	3	2	5	3 or 4	4
DURING WORK	1	1	2	2	3
TYPE OF WORK					
Median Barrier/Guardrail Installation/Repair	—	1,500	—	3,200 2,940[a]	4,800 4,570[a]
Pavement Repair	1,050[a]	1,400	—	3,000 2,900[a]	4,500
Resurfacing, Asphalt Removal	1,050[a]	1,200 1,300[a]	2,750[a]	2,600 2,900[a]	4,000
Striping, Slide Removal	—	1,200	—	2,600	4,000
Pavement Markers	—	1,100	—	2,400	3,600
Bridge Repair	1,350[a]	1,350[a]	—	2,200	3,400

[a] Texas data, full-hour capacities; all other data are from California, expressed as peak flow rates.
* Adapted from Ref. *31*.

TABLE 6-3. CAPACITY OF LONG-TERM CONSTRUCTION SITES WITH PORTABLE CONCRETE BARRIERS

NUMBER OF LANES NORMAL	OPEN	NUMBER OF STUDIES	CAPACITY RANGE (VPHPL)	AVG. CAPACITY VPH	VPHPL
3	2	7	1,780–2,060	3,720	1,860
2	1	3	—	1,550	1,550

SOURCE: Ref. *29*

LONG-TERM CONSTRUCTION SITES—WORK AREA SEPARATED FROM TRAFFIC BY PORTABLE CONCRETE BARRIERS

Table 6-3 illustrates the results of 10 studies of locations with long-term, more permanent types of construction operations in progress. Note that the capacities at such sites are higher than those for more temporary disruptions primarily because of the use of more permanent barriers and other controls, and the dissipation of "rubber-necking" as drivers become familiar with the site.

CAPACITY AT SHORT-TERM MAINTENANCE SITES WITH NO WORK ACTIVITY ADJACENT TO TRAFFIC

One study was conducted in Houston, Texas, where the right two lanes of a four-lane section were closed to traffic. No work activity, however, was in the lane immediately adjacent to moving traffic. In effect, the closure included one full buffer lane between traffic and maintenance operations. Although capacity operations were not observed, capacity of the location was estimated to be about 1,800 vphpl, considerably larger than a standard (4,2) closure with work activities taking place in the lane adjacent to moving traffic.

SHOULDER USE AND TRAFFIC SPLITTING ON THREE-LANE SEGMENTS

Generally, when work is required on the middle lane of a three-lane section, both the middle and one of the exterior lanes are closed. Table 6-1 indicates that the average capacity of the single open lane is 1,170 vphpl. Several studies have indicated that this capacity can be increased to 3,000 vph by using a traffic control approach called "shifting," in which drivers are encouraged to use the shoulder as an additional traffic lane, thereby leaving two effective lanes for traffic movement. "Shifting" is generally accomplished through the use of traffic cones directing drivers onto the shoulder and adjacent shoulder lane.

This same capacity could be achieved using the "splitting" approach, in which only the middle lane is closed, and traffic is permitted to move on both sides of the work activity. Since such an operation is often confusing to drivers, a control approach is recommended in which the left lane is closed as much as 1,000 to 1,500 ft upstream of the site. Thus, only two lanes approach the site. At the maintenance zone, cones are used to direct one lane to the left and one lane to the right of the closed middle lane.

LANE NARROWING

One study in Houston considered the effect of lane narrowing without closures due to maintenance or construction operations. The subject sites included lane-width reductions to 10 and 11 ft, with portable concrete barriers used to separate moving traffic from work operations. Capacities in the range of 1,800 vphpl were observed at these sites, which included both three- and four-lane segments.

ESTIMATING QUEUE LENGTH AND DELAY

Figure 6-6, presented earlier, illustrates a graphic technique for estimating queue buildup and delays for breakdown conditions. This same technique can be applied to work zones where arrival or demand flows exceed the capacity of the work zone for some period of time. In particular, the length of the queue may be estimated as:

$$L_t = \frac{Q_t \times \ell}{N} \tag{6-1}$$

where:

L_t = length of queue, in ft;
Q_t = number of vehicles in queue at time t;
N = number of open lanes upstream of the site; and
ℓ = average length of vehicle.

The value of Q_t would be found using the graphic technique illustrated in Figure 6-6.

SAMPLE CALCULATION

Consider the case of a maintenance operation requiring the closure of the median lane of a three-lane freeway segment. The work will require four hours to complete, including the installation and removal of traffic control devices. Data obtained from a nearby traffic counter during the previous two weeks were used to estimate the following demand pattern:

Time Period	Volume Anticipated (vph)
9 to 10 AM	2,920
10 to 11	3,120
11 to 12 Noon	3,200
12 to 1 PM	3,500
1 to 2	3,830
2 to 3	3,940
3 to 4	4,620
4 to 5	5,520

Referring to Table 6-1 and Figure 6-11, it is seen that the average capacity for a (3,2) work zone configuration is 1,500 vphpl or 3,000 vph. The 85th percentile capacity is 1,450 vphpl or 2,900 vph, and the 100th percentile capacity is 1,420 vphpl or 2,840 vph. Assuming these capacity values, Figure 6-13 illustrates the graphical depiction of queue build-up and delays.

In Figure 6-13, work is assumed to begin at 9 AM. The estimated queue length at 1 PM, four hours after the beginning of work, and the time work is assumed to stop, is 2.1 mi based on the average capacity of 3,000 vph. This, however, is a 58th percentile value. Thus, the queue would be longer than this value 42 percent of the time. If the 85th percentile capacity is used, the queue reaches 2.9 mi, but would be exceeded only 15 percent of the time. The 100th percentile queue length reaches 3.5 mi, which is not expected to be exceeded under most circumstances.

Clearly, such a back-up would be most undesirable, and other options would be explored in terms of the work zone operations, including:

1. Perform work on a Saturday or Sunday if volumes are lower during these periods.
2. Perform the work at night.
3. Reduce the work time, or split the work into two shifts.
4. Implement additional traffic control strategies.

Curves similar to those in Figure 6-13 could be developed for weekend or night volume conditions. A review of Figure 6-13 also indicates that queues could be greatly reduced if the work could be accomplished in 3 hours or less. At average capacity, the queue after 3 hours would be only 0.8 mi, considerably less than the 2.1-mi queue which develops after 4 hours. If the work could be divided into two 2-hr shifts on two separate days, the queue (at average capacity) would be limited to about 0.5 mi.

Other traffic control strategies might include closing of on-ramps upstream of the site to reduce demand, or directing vehicles to use the shoulder past the work zone. The latter would add up to 1,500 vph of additional capacity. The issue of ramp closures, however, would have to be carefully considered in terms of where diverted vehicles would go, and what their impact on traffic along diversion routes would be. Ramp closures would also have to be carefully signed to avoid driver confusion.

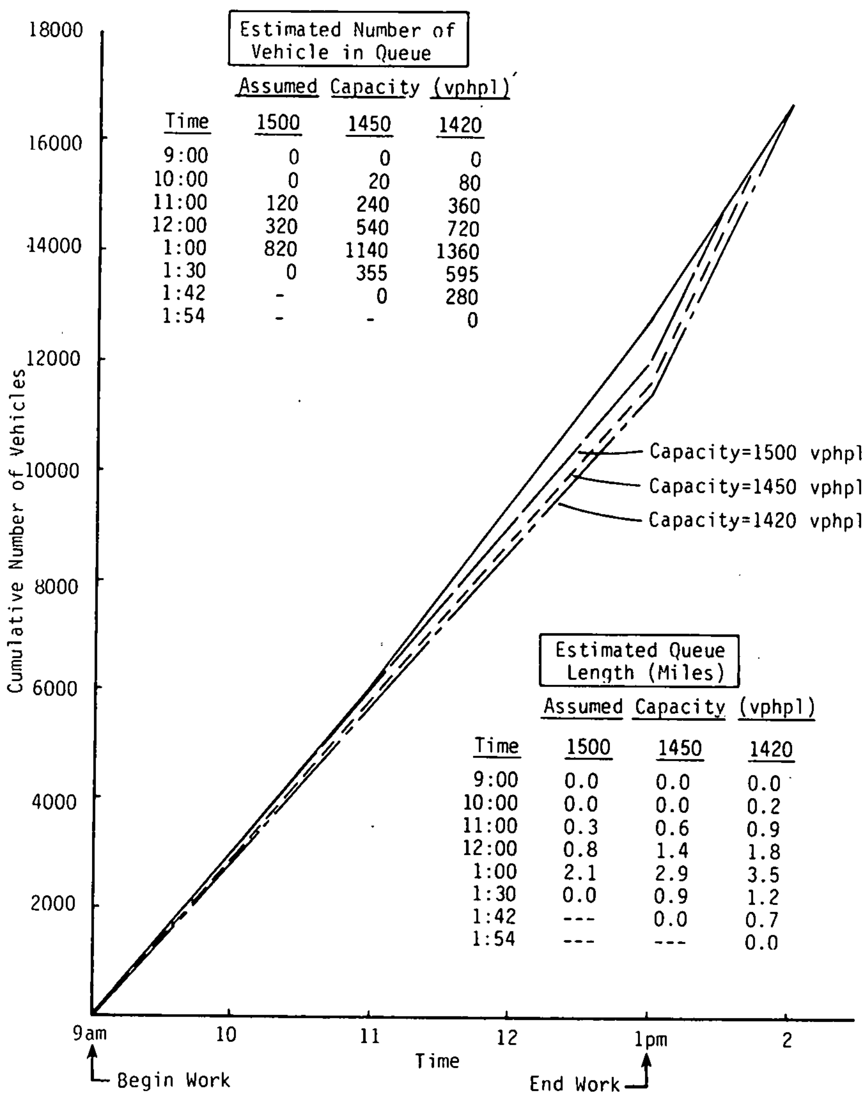


Figure 6-13. Sample calculation—queue analysis for a work zone. (Source: C.L. Dudek and S.H. Richards, "Traffic Capacity Through Urban Freeway Work Zones in Texas," *Transportation Research Record 869,* 1982)

V. WEATHER

The capacity of freeway systems is also affected by weather. The most extreme case is represented by heavy snowfalls that cause multiple lane closings. However, a variety of weather conditions—rain, snow, fog, glare, and others—affect capacity without such dramatic evidence of their existence.

Quantitative information is sparse, but some indications do exist: one study found that rain reduced capacity by 14 percent (*6, 7*). Another found a typical figure of 8 percent for rain (*8*), although much variation was observed. Indeed, the substantial variations due to the intensity of the weather condition and the specifics of the location are entirely rational. It is most important to recognize that 10 to 20 percent reductions are typical, and higher percentages are quite possible. These effects must be considered in facility design, particularly when adverse conditions are common.

VI. HIGH-OCCUPANCY VEHICLE LANES ON FREEWAYS (HOV LANES)

The existence of exclusive high-occupancy vehicle lanes on freeways raises two issues: (1) what is their capacity and what are the operating characteristics of such lanes, and (2) what effect does their presence have on the operation of the remainder of the freeway.

CAPACITY ANALYSIS FOR HOV LANES

This issue is quite complex. High-occupancy vehicle lanes come in many forms, including:

- Exclusive bus lanes.
- Exclusive bus/taxi lanes.
- Exclusive bus/car-pool lanes, with varying occupancy restrictions.
- Exclusive bus/taxi/car-pool lanes.

In addition, each type may be implemented as a contraflow lane, with the exclusive lane taken from the opposing freeway lanes, or as a concurrent flow lane, in which the lane is taken from freeway lanes in the same direction of flow. HOV lanes are adopted to provide for smooth and speedy flow of passengers in vehicles using the lanes, and they are used to circumvent freeway segments operating at or near breakdown conditions. The contrast of high-occupancy vehicles progressing smoothly while other vehicles are mired in heavy congestion is also intended to act as an inducement to motorists to abandon their car for a bus or car pool.

Thus, it is not practical for such a lane to operate at or near capacity, or at a poor level of service. To do so would defeat its function and purpose. The issue of the "capacity" of such lanes is therefore highly speculative, because few (if any) existing lanes approach this condition at any time. Chapter 12 provides guidelines and LOS criteria for HOV lanes, based primarily on the work of Levinson (*9,10,11*). This section attempts to provide a general framework for defining the impacts of such a lane on freeway operations. Numerous studies of existing operations (*12–22*) may also be used for general insight on the subject.

EFFECT OF HOV LANES ON FREEWAY OPERATIONS

The existence of a HOV lane on a freeway influences the operation of remaining freeway lanes in three ways:

1. A lane is removed from one direction of flow (occasionally two are removed, the second being used as a buffer lane).
2. Cones or other devices used to demark the lane (where used) pose lateral obstructions to flow in the adjacent lane, if a buffer lane is not provided.
3. The movement of vehicles into or out of the HOV lane may be disruptive to other traffic.

Unfortunately, there is no meaningful body of data which has quantified these effects. Estimates of the first two factors can, however, be made using techniques presented in Chapter 3.

The removal of a lane is simply handled by assuming that the eight-lane freeway becomes a six-lane freeway, and the six-lane freeway a four-lane, etc. The effect of cones or other dividers may be estimated by treating them as lateral obstructions at the roadside edge. Depending on their placement, they may also have the effect of narrowing the lane as well.

In contraflow lanes, this latter effect is marked, as vehicles shy away from the imposing opposite flow of large vehicles at relatively high speeds. In some instances, an entire adjacent lane is taken out of service to act as a buffer zone.

SAMPLE CALCULATION

Figure 6-14 illustrates a problem using this estimating technique. The problem is to analyze the impact of a proposed contraflow lane on level of service in the direction from which the lane is taken, and on the concurrent direction of flow.

- <u>Before HOV Lane Is Initiated</u>

Primary flow = $5{,}100 + (1.6 \times 300) = 5{,}580$ pcph in three lanes of 12 ft each, with no lateral obstructions. Level-of-service D, approximate speed 48 mph (Table 3-1, Fig. 3-4)

Contraflow = 2,800 pcph in three lanes of 12 ft, with no lateral obstructions. Level-of-service B, speed 58 mph (Table 3-1, Fig. 3-4)

- After HOV Lane Is Initiated

Primary flow = 5,100 pcph in three lanes of 12 ft, with no lateral obstructions. Level-of-service C, approximate speed 50 mph (Table 3-1, Fig. 3-4)

Contraflow = 2,800 pcph in two lanes, with a lateral obstruction at 0 ft on one side, assume 11-ft lanes due to divider placement—W = 0.87 (Table 3-2)—effective flow = 2,800/0.87 = 3,218 pcph. Level-of-service C, approximate speed 56 mph (Table 3-1, Fig. 3-4)

On the basis of this approximate analysis, the creation of the new lane improves flow in the concurrent direction by removing buses from the stream. Level of service improves from D to C, and average running speed increases from 48 mph to 50 mph, which saves each vehicle (5/48 − 5/50) 60 = 0.25 min or 15 sec.

Level of service on the freeway in the reverse direction decreases from B to C, and speed from 58 mph to 56 mph, causing each vehicle to lose (5/56 − 5/58) 60 = 0.18 min or 10.8 sec.

While not totally definitive, this approximate technique is useful in evaluating the gross effects of HOV lane implementation on remaining freeway flows. These impacts would have to be evaluated in light of the benefits and costs of the HOV lane itself and related issues.

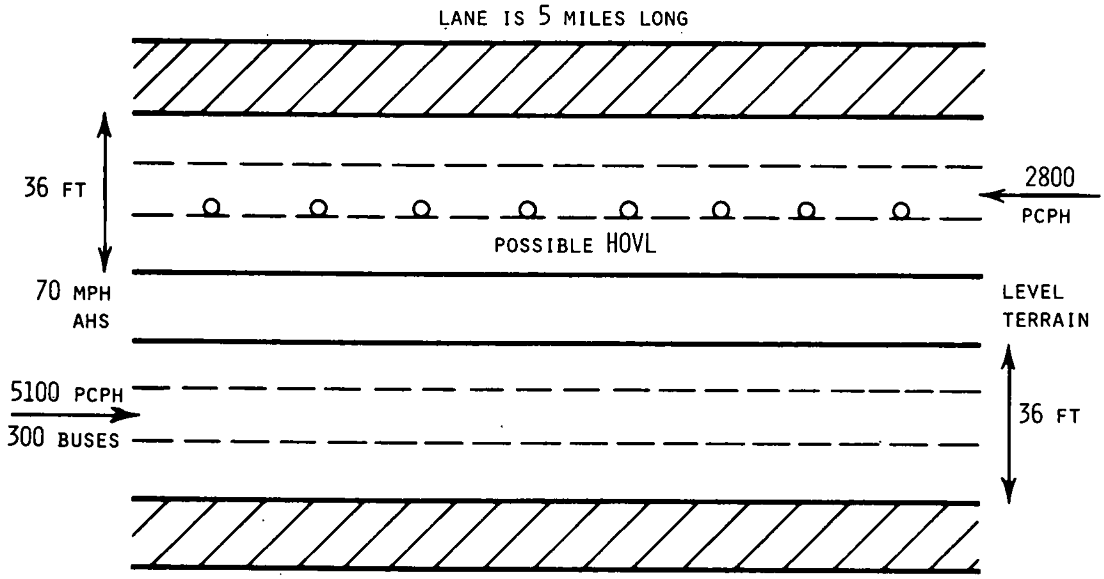


Figure 6-14. Example for analysis of HOV lane impact (1 bus = 1.6 passenger cars).

VII. SUMMARY

The freeway is a complex facility made up of many component segments and sections, each having a potential impact on operations in upstream and downstream segments. This chapter has attempted to identify these impacts, as well as various system operational components which may impact overall capacity and level of service. The techniques presented should be considered to be approximate, and serve primarily to indicate the relative magnitude of various operational impacts.

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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 median-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 2 mi or less, the procedures in Chapter 11, "Urban and Suburban Arterials," should be used.

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 median friction than a similar divided highway not having left-turn lanes, assuming similar flow levels. The number

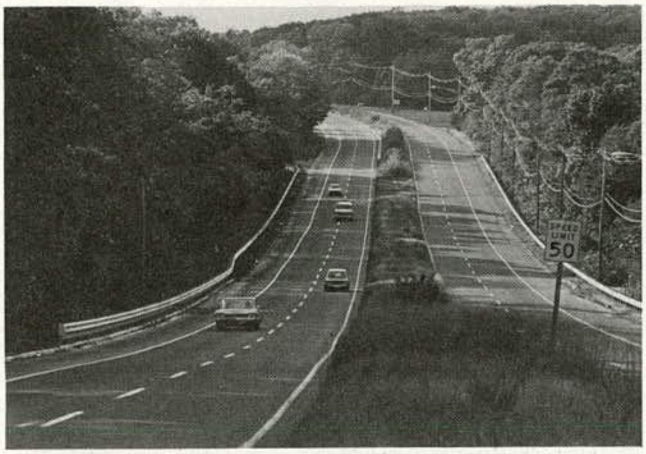


Illustration 7-1. A divided multilane highway in a rural environment.

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 there 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.

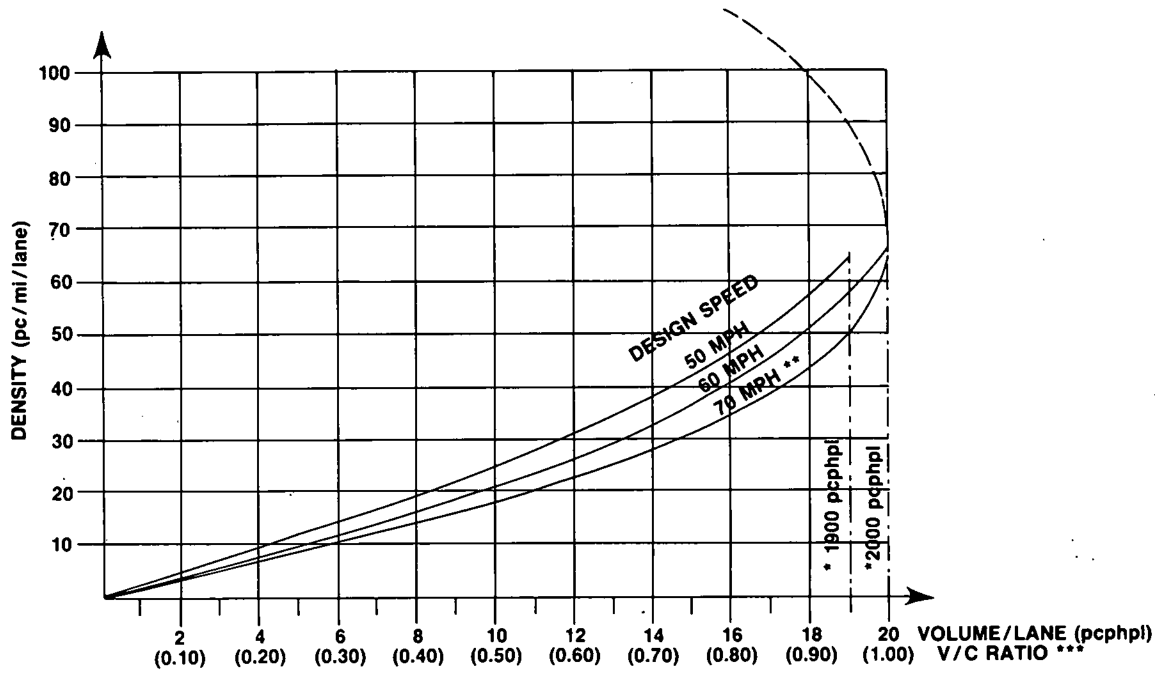
Heavy vehicles have a detrimental effect on traffic flow for two reasons: (1) they are larger than passenger cars, and therefore occupy more roadway space; and (2) their performance characteristics are generally inferior to passenger cars, leading to the formation of gaps in the traffic stream which 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 median-frictions that occur on other categories of multilane highways have a further adverse effect on traffic flow characteristics.

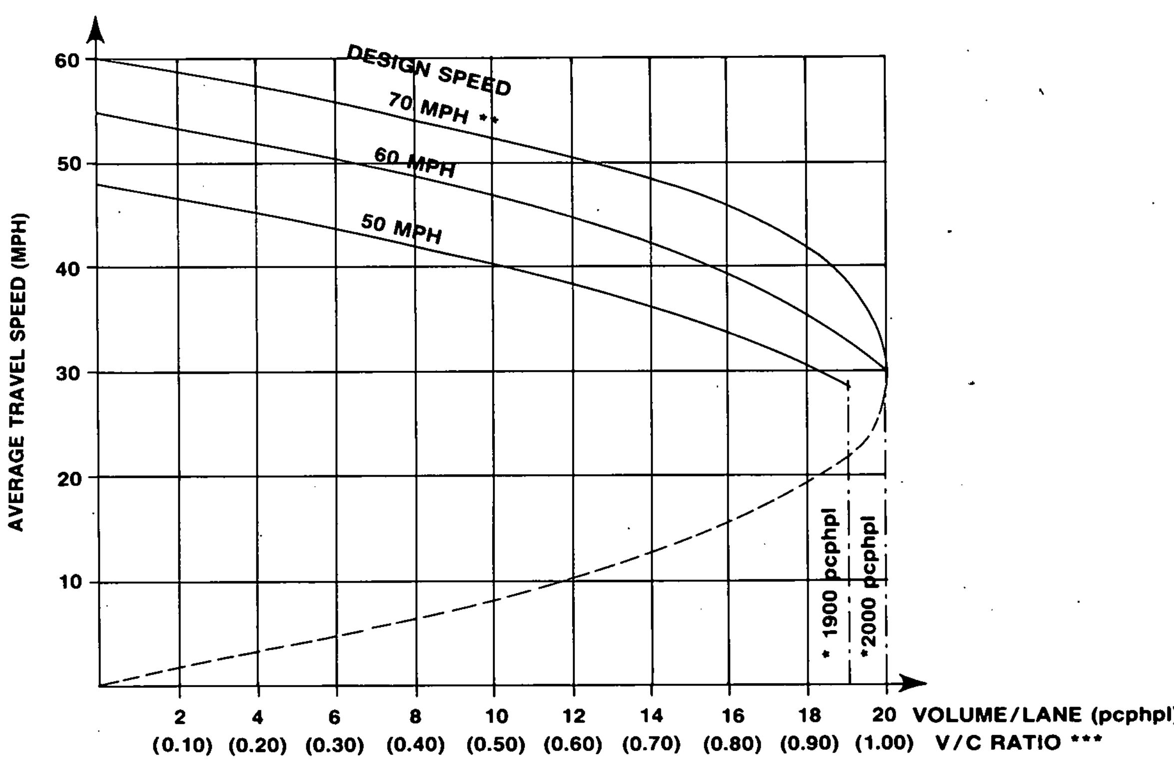
Driver Population

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.



* 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.

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	≥ 30	1.00	2,000	≥ 28	1.00	1,900
F	> 67	< 30	[c]	[c]	< 30	[c]	[c]	< 28	[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.

$$SF_i = MSF_i \times N \times f_w \times f_{HV} \times f_E \times f_p \qquad (7\text{-}1)$$

$$MSF_i = c_j \times (v/c)_i \qquad (7\text{-}2)$$

$$SF_i = c_j \times (v/c)_i \times N \times f_w \times f_{HV} \times f_E \times f_p \qquad (7\text{-}3)$$

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.

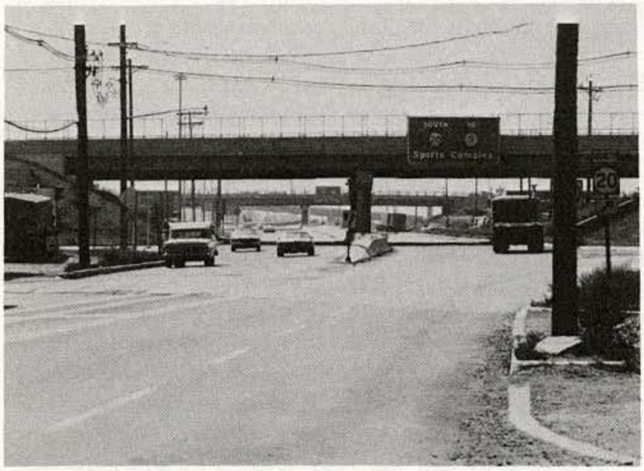


Illustration 7-5. Note the bridge pier located in the center of a normally undivided suburban multilane highway. Vehicles will tend to adjust their position in adjacent travel lanes to avoid traveling too closely to the abutment.

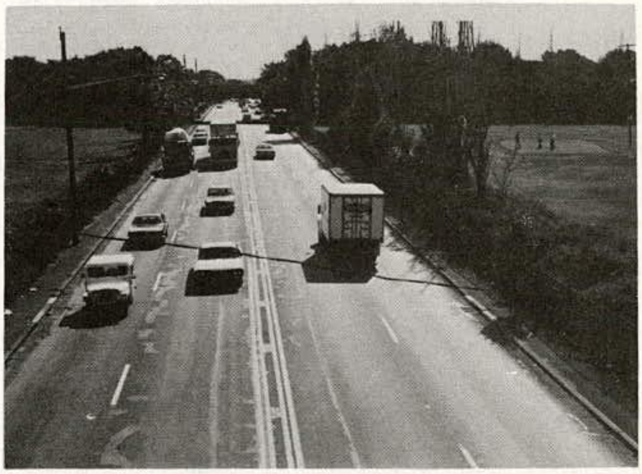


Illustration 7-6. The absence of a usable shoulder and the close proximity of obstructions to the edge of the traveled way on this highway will also influence driver behavior.

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 6 ft 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:

$$\begin{aligned} 1{,}000 \times 0.03 &= 30 \text{ ft} \\ 2{,}000 \times 0.04 &= 80 \text{ ft} \\ 1{,}000 \times 0.02 &= \underline{20 \text{ ft}} \\ & \quad 130 \text{ ft} \end{aligned}$$

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 4,000-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	PERCENTAGE OF TRUCKS, P_T; RV'S, P_R; OR BUSES, P_B														
E_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, and (f) 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, (d) percent buses, and (e) driver population.

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:

$$v/c = SF/[c_j \times N \times f_w \times f_{HV} \times f_E \times f_p] \quad (7\text{-}6)$$

Alternatively, Eq. 7-1 may be used to compute the effective maximum service flow rate, *MSF:*

$$MSF = SF/[N \times f_w \times f_{HV} \times f_E \times f_p] \quad (7\text{-}7)$$

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

OPERATIONAL ANALYSIS WORKSHEET

Facility Section: ____________________

Date: ____________ Time: ____________ (of analysis data)

I. GEOMETRY

___ ft
Distance to Roadside Obstructions
___ ft
← DIR. 1 N = ___
___ ft
___ ft
DIR. 2 → N = ___
___ ft
Distance to Roadside Obstructions

INDICATE NORTH

	Highway Classification D or U, S or R	Design 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 $\boxed{v/c = SF/[c_j \times N \times f_w \times f_{HV} \times f_E \times f_p]}$

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

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

	v/c = SF	/[c_j	× N	× f_w	× f_E	× f_p	× f_{HV}]	E_T	E_B	E_R
		Table 7-1		Table 7-2	Table 7-10	Table 7-11				
Dir. 1										
Dir. 2										

	c	v/c	LOS (Table 7-1)	Speed (Fig. 7-2)	Density (Fig. 7-1)
Dir. 1					
Dir. 2					

COMMENTS on back of page.

Name: ____________ Date: ____________

Checked by: ____________

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:

$$SF = DDHV/\mathrm{PHF} \tag{7-8}$$

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.

$$N = SF/[c_j \times (v/c) \times f_w \times f_{HV} \times f_E \times f_p] \tag{7-9}$$

or alternatively, using Eq. 7-2:

$$N = SF/[MSF \times f_w \times f_{HV} \times f_E \times f_p] \tag{7-10}$$

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 weighed 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 Table 7-12	Highway Classification D or U, S or R	Design Speed	Lane Width	Lateral Clearance (ft) Roadside	Median	Terrain or Grade Type L, R, or M	(%)	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 $\boxed{N = SF/[c_j \times v/c \times f_w \times f_{HV} \times f_E \times f_p]}$

$N = SF \quad /c_j \times (v/c) \times f_w \times f_E \times f_p \times {}^{*}f_{HV}$

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

			Table 7-1		Table 7-2	Table 7-10	Table 7-11		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:

$$DDHV = AADT \times K \times D \qquad (7\text{-}11)$$

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	0.09 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)

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 in one direction, 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);

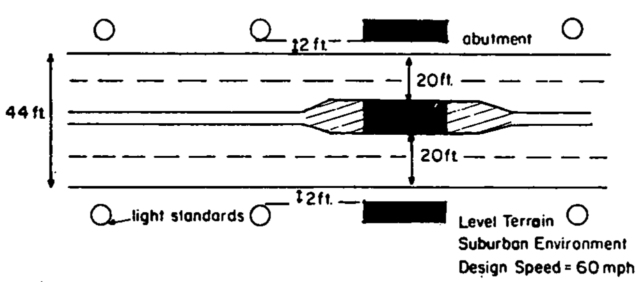


Figure 7-5. Multilane highway segment for Calculation 1.

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

	Highway Classification D or U, S or R	Design 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*	-	-	☒ Commuter ☐ Other
Dir. 2	*2,000*	*0.91*	*2,198*	*15*	-	-	☒ 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 Table 7-1	× N	× f_w Table 7-2	× f_E Table 7-10	× f_p Table 7-11	× 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 (Table 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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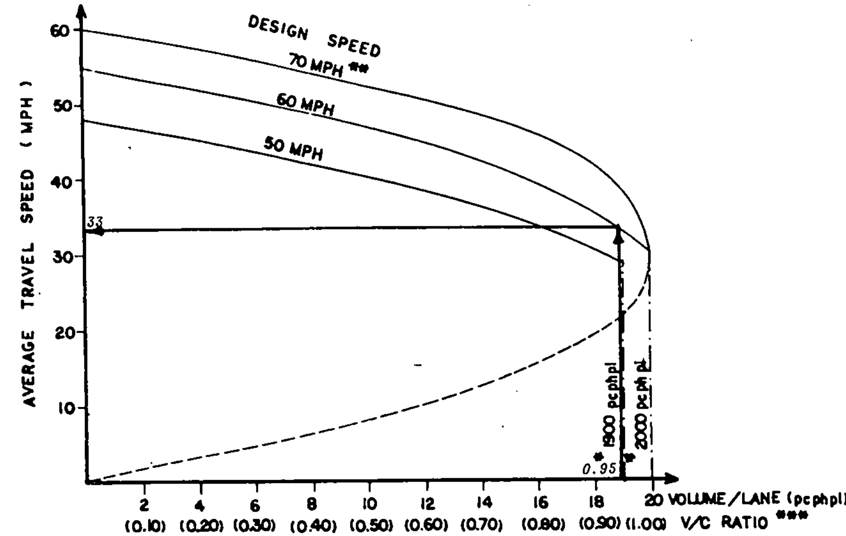
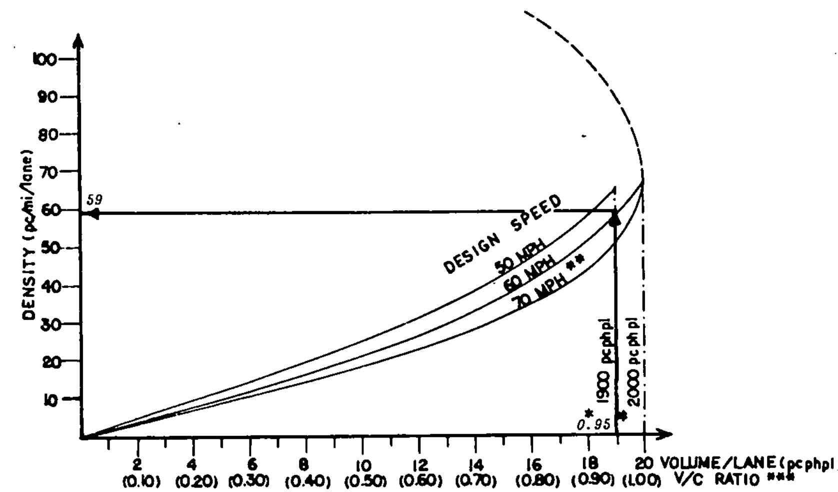
Name: *John Jones* Date:

Checked by:

Figure 7-6. Illustration of solution to Calculation 1.



* 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

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 analysis 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:

$$v/c \text{ (Upgrade)} = 2{,}588/[2{,}000 \times 3 \times 1.0 \times 0.67 \times 1.0 \times 0.82] = 0.79$$
$$v/c \text{ (Downgrade)} = 2{,}588/[2{,}000 \times 3 \times 1.0 \times 0.85 \times 1.0 \times 0.82] = 0.62$$

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.

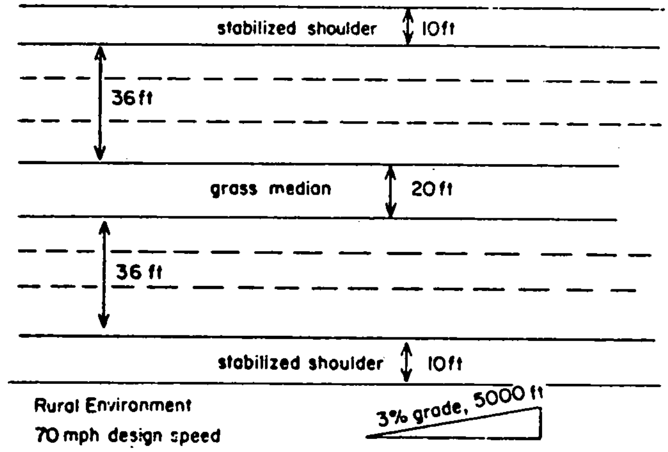


Figure 7-7. Multilane highway for Calculation 2.

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

	Highway Classification D or U, S or R	Design 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	SF=Vol./PHF	% Trucks	% Buses	% RV's	Driver Population
Dir. 1	*2,200*	*0.85*	*2,588*	*10*	*-*	*5*	☐ Commuter ☒ Other
Dir. 2	*2,200*	*0.85*	*2,588*	*10*	*-*	*5*	☐ Commuter ☒ Other

III. ANALYSIS

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

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

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

	v/c =	SF	$/[c_j$ Table 7-1	$\times N$	$\times f_w$ Table 7-2	$\times f_E$ Table 7-10	$\times f_p$ Table 7-11	$\times f_{HV}]$	E_T	E_B	E_R
Dir. 1	*0.79*	*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 (Table 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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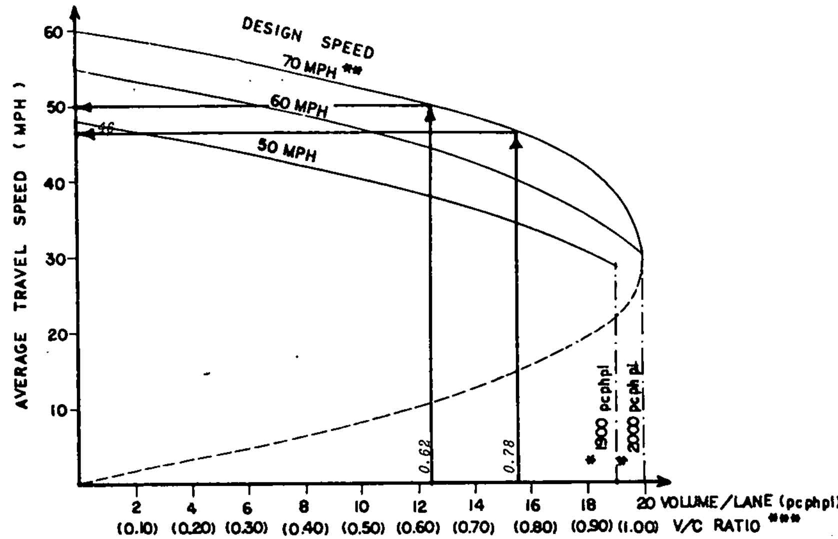
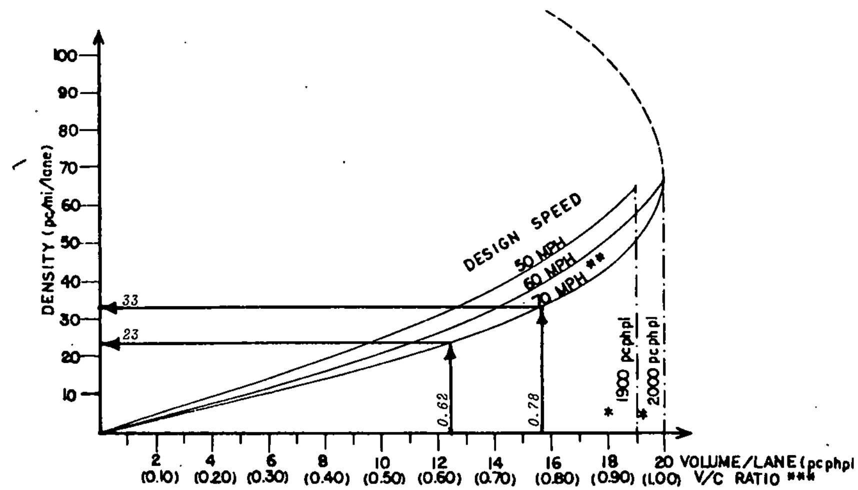
Name: *Jennifer Smith* Date: ______

Checked by: ______



* 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 50 mph and a density of 23 pc/mi/ln. (b) The *undivided* design would operate at an approximate speed of 48 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 Table 7-12	Highway Classification D or U, S or R	Design Speed	Lane Width	Lateral Clearance (ft) Roadside	Lateral Clearance (ft) Median	Terrain or Grade Type L, R, or M	(%)	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	-	-	☒ Commuter ☐ Other
Dir. 2	1,800	0.90	2,000	5	-	-	☒ 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		× (v/c)	× f_w Table 7-2	× f_E Table 7-10	× f_p Table 7-11	× f_{HV}*	E_T	E_B	E_R
			Table 7-1								
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

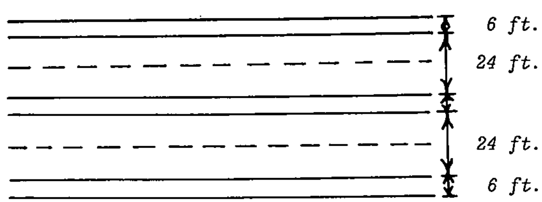
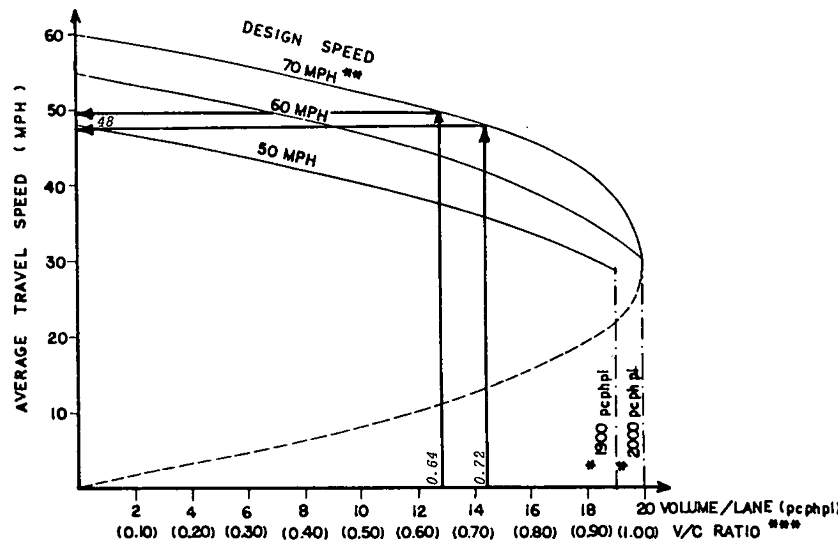


*Table 7-9 or compute as shown.

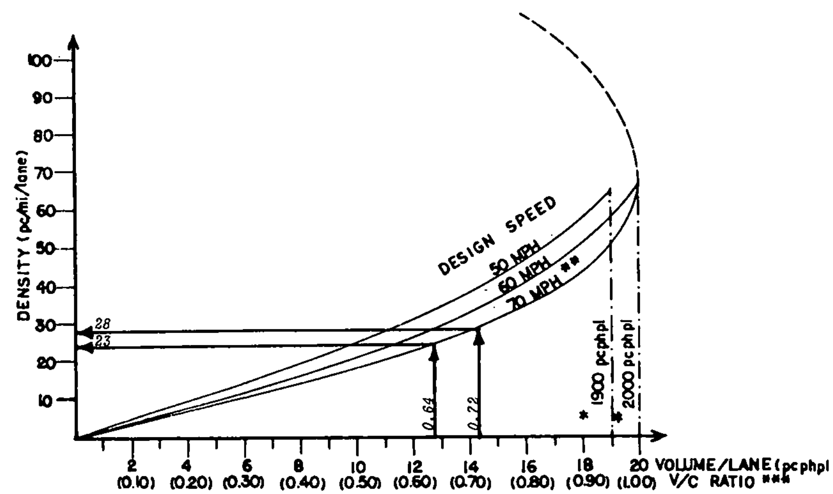
Name: Arthur Jones Date:

Checked by:

Figure 7-9. *Illustration of solution to Calculation 3.*



* 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

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:

$c = SF_E = c_j \times N \times (v/c) \times f_w \times f_{HV} \times f_E \times f_p$
$c = SF_E = 2{,}000 \times 2 \times 1.00 \times 0.80 \times 0.90 \times 0.80 \times 1.00$
$c = 2{,}304$ vph, Say 2,300 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:

$$c_I = 2{,}300 \times (40/60) = 1{,}533 \text{ vph, Say } 1{,}550 \text{ vph.}$$

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) $\times$ 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 Table 7-12	Highway Classification D or U, S or R	Design Speed	Lane Width	Lateral Clearance (ft) Roadside	Median	Terrain or Grade Type L, R, or M	(%)	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	-	-	☒ Commuter ☐ Other
Dir. 2	1,000	0.85	1,176	20	-	-	☒ Commuter ☐ Other

III. DESIGN ANALYSIS

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

$N = SF \quad /c_j \times (v/c) \times f_w \times f_E \times f_p \times {}^*f_{HV}$

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

	N	SF	c_j Table 7-1	v/c	f_w Table 7-2	f_E Table 7-10	f_p Table 7-11	$^*f_{HV}$	E_T	E_B	E_R
Dir. 1	3.3 or 2.9	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

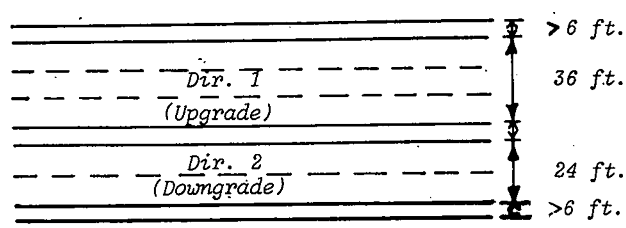


*Table 7-9 or compute as shown.

Name: *Elizabeth Duran* Date:

Checked by:

Figure 7-10. Worksheet for Calculation 4.

$SFL_B = 900$ vphpl

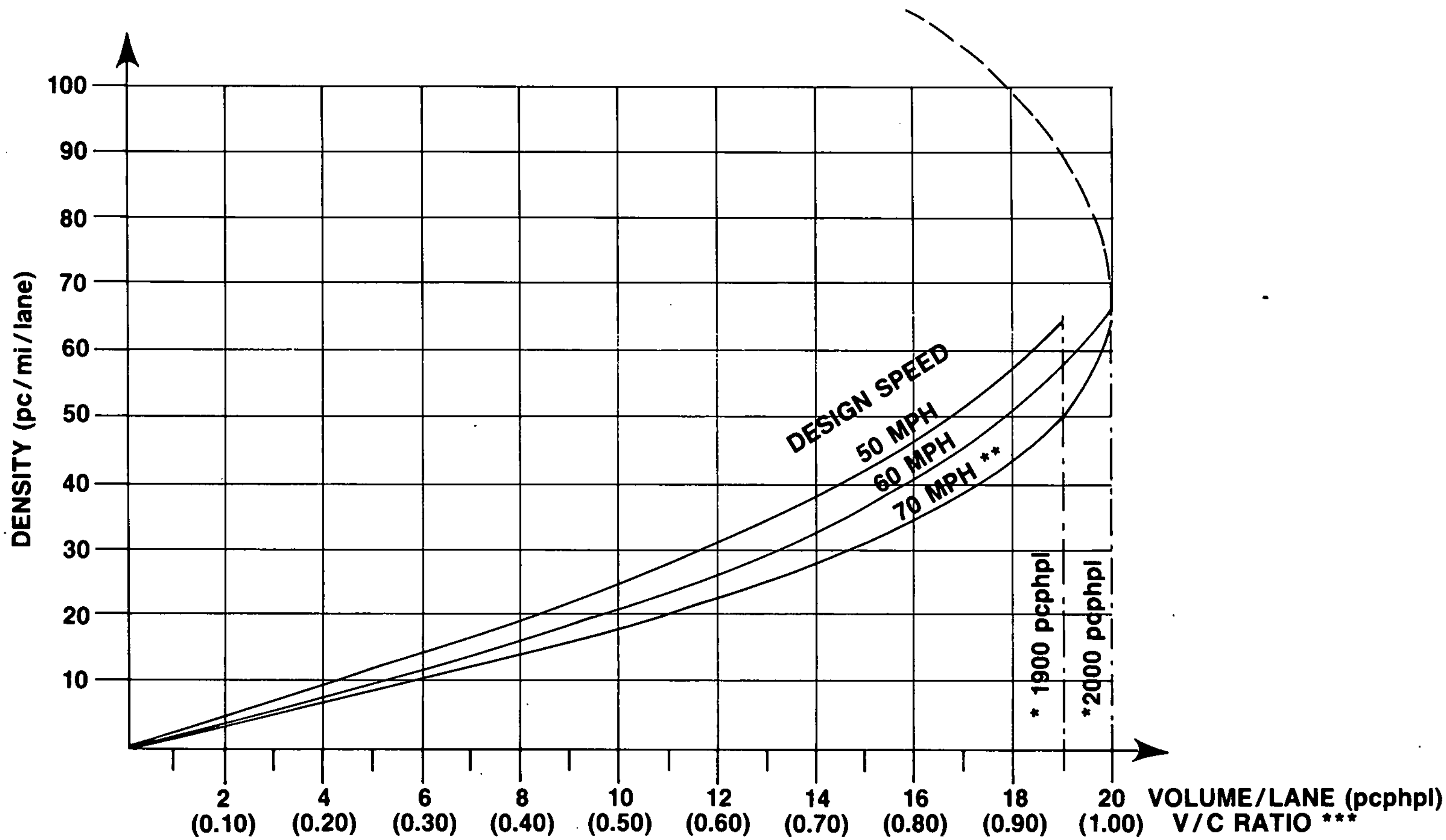
and:

$N = DDHV/[SFL \times f_E \times \text{PHF}]$
$N = 1{,}706/[900 \times 1.00 \times 0.92] = 2.1$ 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

FIGURES AND WORKSHEETS FOR USE IN THE ANALYSIS OF MULTILANE HIGHWAYS

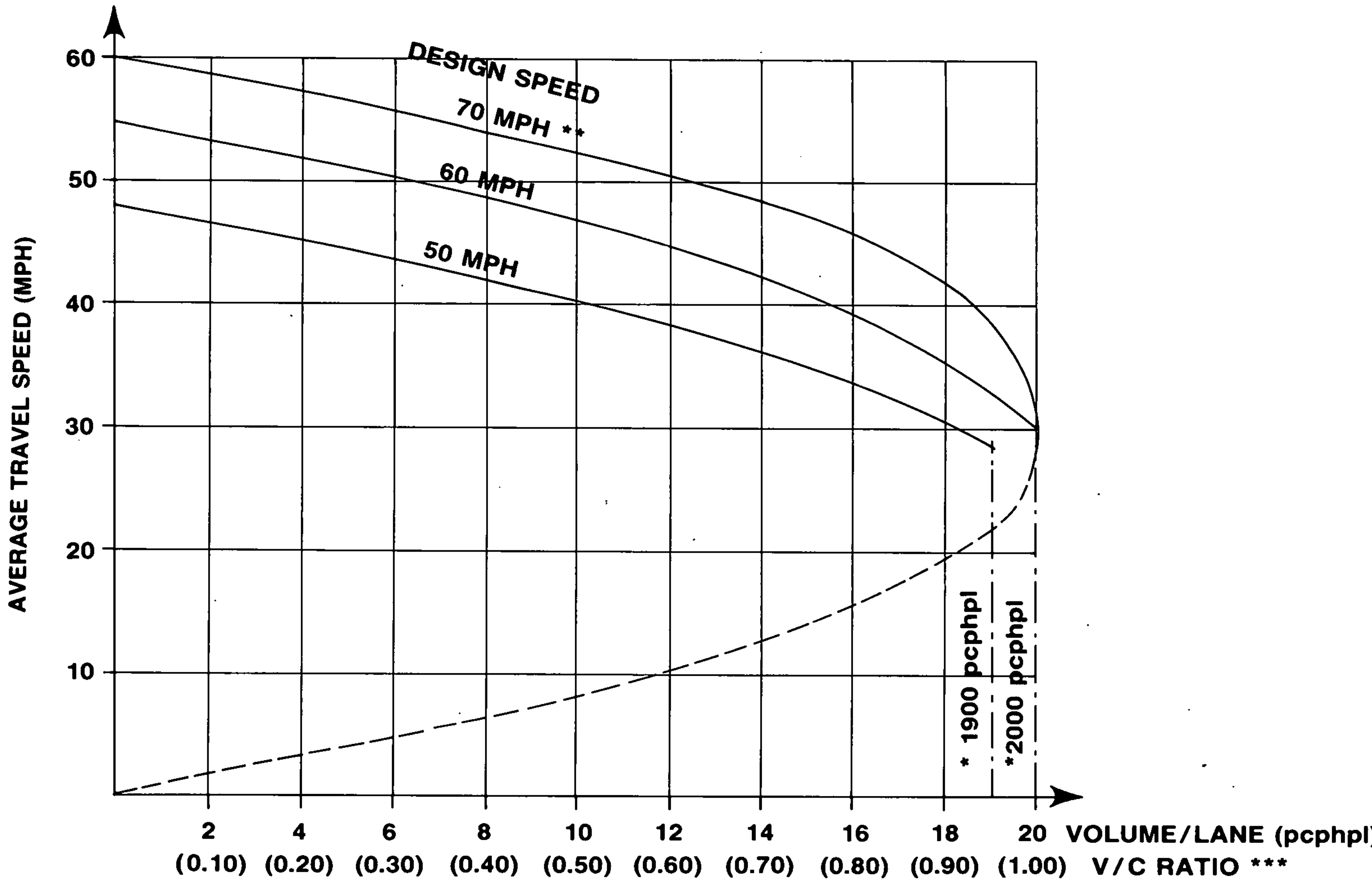


* capacity

** reflects 55 MPH speed limit

*** v/c ratio based on capacity of 2000 pcphpl, applies only to 60 and 70 MPH design speeds

Figure 7-1. 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

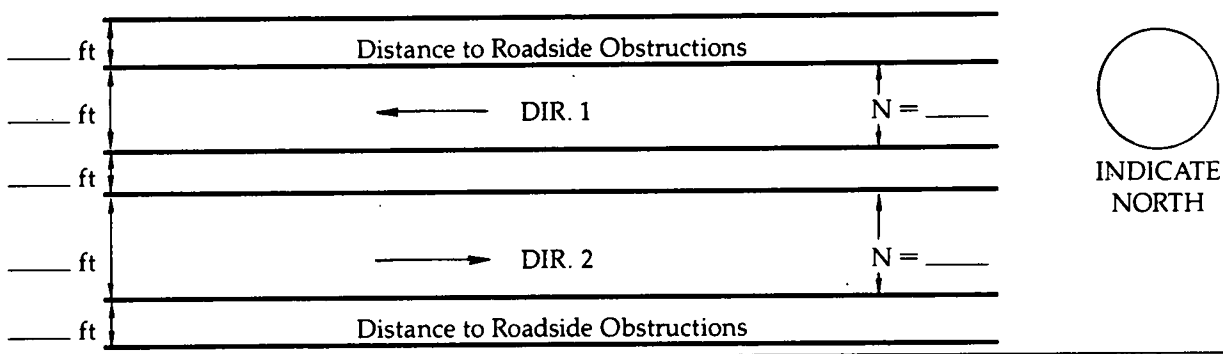
Figure 7-2. Speed-flow characteristics for uninterrupted flow segments of multilane highways.

OPERATIONAL ANALYSIS WORKSHEET

Facility Section: ______________________________

Date: ______________________ Time: ______________________ (of analysis data)

I. GEOMETRY



	Highway Classification D or U, S or R	Design 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

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

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

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

	$v/c =$	SF	$/[c_j$ Table 7-1	$\times N$	$\times f_w$ Table 7-2	$\times f_E$ Table 7-10	$\times f_p$ Table 7-11	$\times f_{HV}]$	E_T	E_B	E_R
Dir. 1											
Dir. 2											

	c	v/c	LOS (Table 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 Table 7-12	Highway Classification D or U, S or R	Design Speed	Lane Width	Lateral Clearance (ft) Roadside	Median	Terrain or Grade Type L, R, or M	(%)	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

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

$N = SF \quad /c_j \times (v/c) \times f_w \times f_E \times f_p \times {}^{*}f_{HV}$

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

	N	SF	c_j Table 7-1	v/c	f_w Table 7-2	f_E Table 7-10	f_p Table 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: ____________________

TWO-LANE HIGHWAYS

CONTENTS

I. INTRODUCTION

A two-lane highway may be defined as a two-lane roadway having one lane for use by traffic in each direction. Passing of slower vehicles requires the use of the opposing lane where sight distance and gaps in the opposing traffic stream permit. As volumes and/or geometric restrictions increase, the ability to pass decreases, resulting in the formation of platoons in the traffic stream. Motorists in these platoons are subject to delay because of the inability to pass.

Two-lane highways compose the predominant mileage of most national highway systems. They are used for a variety of functions, are located in all geographic areas, and serve a wide range of traffic requirements. Consideration of operating quality must account for these disparate traffic functions.

Efficient *mobility* is the principal function of major two-lane highways used as primary arteries connecting major traffic generators or as primary links in state and national highway networks. Such routes tend to serve long-distance commercial and recreational travelers, and may have sections of many miles through rural environments without traffic control interruptions. Consistent high-speed operations and infrequent passing delays are desirable for these facilities.

Many paved, two-lane rural roads basically serve an *accessibility* function. They provide all-weather accessibility to an area, often for relatively low traffic volumes. The provision of cost-effective access is the dominant policy consideration. High speed, while beneficial, is not the principal concern. Delay, as indicated by the formation of platoons, and the utilization of capacity are more relevant measures of service quality.

Two-lane roads also serve scenic and recreational areas where the vista and environment are to be experienced and enjoyed without traffic interruption or delay. A safe roadway is desired, but high-speed operation is neither expected nor desired.

Short sections of high-volume two-lane roads sometime serve as short connections between two major multilane roadways or urban centers. For such short links, traffic conditions tend to be better than might be expected for longer two-lane segments, and the expectations of motorists regarding service quality are generally higher than for longer sections.

For these reasons, three parameters are used to describe service quality for two-lane highways:

1. Average travel speed.
2. Percent time delay.
3. Capacity utilization.

Average travel speed reflects the mobility function of two-lane highways, and is the length of the highway segment under consideration divided by the average travel time of all vehicles traversing the segment in both directions over some designated time interval.

Percent time delay reflects both mobility and access functions, and is defined as the average percent of time that all vehicles are delayed while traveling in platoons due to the inability to pass. "Percent time delay" is difficult to measure directly in the field. The percent of vehicles traveling at headways less than 5 sec can be used as a surrogate measure in field studies.

The *utilization of capacity* reflects the access function, and is defined as the ratio of the demand flow rate to the capacity of the facility.

Level-of-service criteria utilize all three of the parameters noted above, with percent time delay being the primary measure of service quality. Speed and capacity utilization are secondary measures.

This chapter provides specific definitions and methodologies for the estimation of level of service for all types of two-lane highways. Subsequent sections provide a descriptive list of treatments for alleviating both spot and section design and/or operational problems that may arise because of high volume and/or geometric restrictions. A set of example calculations is provided to illustrate the use and application of procedures. A complete set of worksheets for all levels of analysis is also provided. Illustration 8-1 shows typical views of two-lane, two-way rural highways.

LEVELS OF ANALYSIS

This chapter is based on a comprehensive study of two-lane highway operation *(1, 2)*. Microscopic simulation combined with additional field data *(3)* and theoretical considerations were used to develop the methodology. Analysis is provided at two levels:

1. *Operational analysis*—This application is intended to determine the level of service for an existing two-lane highway with existing traffic and roadway conditions, or for projected future conditions; operational analysis applications are presented for general terrain segments and for specific grades.
2. *System planning*—This application enables planners to quickly determine the AADT volumes which can be accommodated on two-lane highways for various levels of service and terrain conditions.

Design computations cannot be readily performed for two-lane highways because the number of lanes is fixed. Modifications to grade and alignment, however, could improve the operational efficiency of a two-lane facility. For other design options, procedures for the appropriate types of facilities would be consulted. Procedures of Chapter 3, "Basic Freeway Segments," and Chapter 7, "Multilane Highways," would often be useful in investigating design alternatives.

The selection of an appropriate level of analysis is based on the objectives of the analysis, the available data base, and the accuracy requirements.

OPERATIONAL CHARACTERISTICS

Traffic operations on two-lane, two-way highways are unique. Lane-changing and passing are possible only in the face of oncoming traffic in the opposing lane. Passing demand increases rapidly as traffic volumes increase, while passing capacity in the opposing lane declines as volumes increase. Thus, unlike

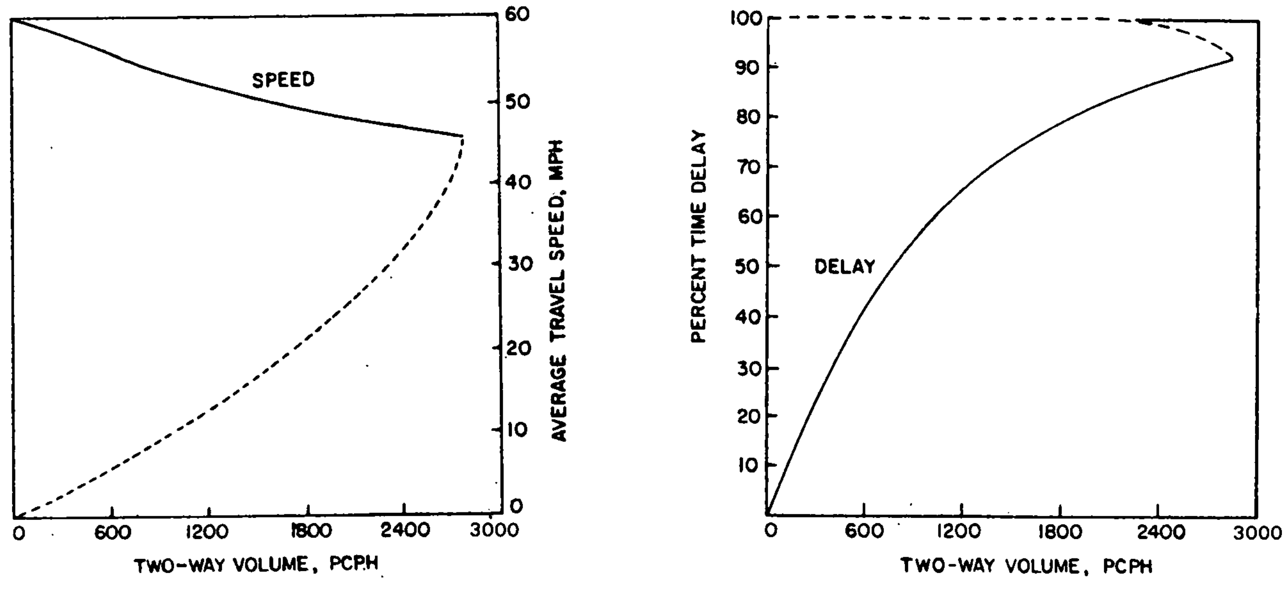
Illustration 8-1. Typical views of two-lane, two-way highways in rural environments.

other types of uninterrupted flow facilities, on two-lane highways, normal traffic flow in one direction influences flow in the other direction. Motorists are forced to adjust their individual travel speed as volume increases and the ability to pass declines. Two traffic stream characteristics, average travel speed and percent time delay, are used as operational measures describing the quality of service provided to motorists on a two-lane highway.

A relatively high running speed has become an accepted criterion for primary highway design. Mean speeds of traffic flow are frequently observed above 55 mph on primary rural highways. Research has shown that speed is fairly insensitive to volume on two-lane highways without significant grades or turning traffic *(4)*. Consequently, average speeds of less than 50 mph are judged undesirable for primary two-lane highways in level terrain because of the high percentage of time motorists would be delayed.

"Percent time delay" is the average percent of the total travel time that all motorists are delayed in platoons while traveling a given section of highway. Motorists are defined to be delayed when traveling behind a platoon leader at speeds less than their desired speed and at headways less than 5 sec. For field measurement purposes, percent time delay in a section is approximately the same as the percentage of all vehicles traveling in platoons at headways less than 5 sec *(2, 5)*.

Percent time delay reflects the changing service quality perceived by motorists under a wide range of geometric and traffic conditions. At low traffic volumes, motorists are almost never delayed because demand for passing is low, average headways are high, and the ability to pass is high. The percent time delay for such conditions is near 0 percent. As volumes approach capacity, passing demand greatly exceeds passing capacity, major platoons of traffic exist, and motorists are delayed almost



a. Relationship between average speed and flow on two-lane highways.

b. Relationship between percent time delay and flow on two-lane highways.

Figure 8-1. Speed-flow and percent time delay-flow relationships for two-lane rural highways (ideal conditions).

100 percent of the time. Even though speeds may be relatively high near capacity (40 mph or more), driver frustration would be excessive if these conditions routinely existed for long periods of time.

The basic relationships between average travel speed, percent time delay, and flow are shown in Figure 8-1. These curves assume ideal traffic and roadway conditions. The average speed represents the average travel or space mean speed of all traffic traveling in both directions over the section of highway in question. Percent time delay is the average for all vehicles in the traffic stream.

IDEAL CONDITIONS

Ideal conditions for two-lane highways are defined as no restrictive geometric, traffic, or environmental conditions. Specifically, they include:

1. Design speed greater than or equal to 60 mph.
2. Lane widths greater than or equal to 12 ft.
3. Clear shoulders wider than or equal to 6 ft.
4. No "no passing zones" on the highway.
5. All passenger cars in the traffic stream.
6. A 50/50 directional split of traffic.
7. No impediments to through traffic due to traffic control or turning vehicles.
8. Level terrain.

The capacity of two-lane rural highways under these ideal conditions is 2,800 pcph, total, in both directions. This capacity reflects the impact of opposing vehicles on passing opportunities, and therefore on the ability to efficiently fill gaps in the traffic stream. This phenomenom restricts capacity to a lower value than the 2,000 pcphpl which may be accommodated on multilane uninterrupted flow facilities.

Directional distribution is defined to be 50/50 for ideal conditions. Most directional distribution factors observed on rural two-lane highways range from 55/45 to 70/30. On recreational routes, the directional distribution may be as high as 80/20 or more during holiday or other peak periods. Some variation in speed and percent time delay occurs by direction with changing directional distribution factors and volume levels. Minor changes in average traffic stream characteristics will also occur with directional distribution.

The frequency of no passing zones along a two-lane highway is used to characterize roadway design and to define expected traffic conditions. A no passing zone is defined as any marked no passing zone or, as a surrogate, any section of road wherein the passing sight distance is 1,500 ft or less. The average percentage of no passing zones in both directions along a section is used in the procedures.

The typical percentage of no passing zones found on rural two-lane highways ranges from 20 percent to 50 percent. Values approaching 100 percent can be found on sections of winding mountainous roads. No passing zones have a greater effect in mountainous terrain than on level or rolling highway segments. Heavy platoon formation along a highway section also may cause greater than expected operational problems on an adjacent downstream section having restricted passing opportunities.

II. METHODOLOGY

LEVELS OF SERVICE

As noted previously, level-of-service criteria for two-lane highways address both mobility and accessibility concerns. The primary measure of service quality is percent time delay, with speed and capacity utilization used as secondary measures. Level-of-service criteria are defined for peak 15-min flow periods, and are intended for application to segments of significant length.

Level-of-service criteria for general terrain segments are given in Table 8-1. For each level of service, the percent time delay is shown. Average travel speed is also shown, with values varying slightly by type of terrain. The body of the table includes maximum values of v/c ratio for the various terrain categories and levels of service A through F. The v/c ratios shown in Table 8-1 are somewhat different from those used in other chapters. For two-lane highways, the values given represent the ratio of flow rate to "ideal capacity," where ideal capacity is 2,800 pcph for a level terrain segment with ideal geometrics and 0 percent no passing zones. Two-lane highways are quite complex, and capacities vary depending on terrain and the degree of passing restrictions. To simplify computational procedures, v/c ratios are given in terms of the constant "ideal capacity" of 2,800 pcph, total in both directions of flow.

The level-of-service criteria of Table 8-1 are for extended segments of two-lane rural highways where efficient mobility is the primary objective of the facility. Where speeds have been restricted by an agency, such as through a town or village, the percentage of time delay and capacity utilization are the only meaningful indicators of level of service.

Table 8-2 gives level-of-service criteria for specific grade segments. These criteria relate the average travel speed of *upgrade* vehicles to level of service. Operations on sustained two-lane grades are substantially different from extended segments of general terrain. The speed of upgrade vehicles is seriously impacted, as the formation of platoons behind slow-moving vehicles intensifies and passing maneuvers generally become more difficult. Further, unlike general terrain segments, where the approximate average travel speed at which capacity occurs can be identified, the capacity speed for a specific grade depends on the steepness and length of the grade and volume. Because of this, estimation of capacity is complex. Thus, Table 8-2 defines separate level-of-service criteria for specific grade segments. In addition, this chapter includes special computational procedures for sustained grades on two-lane highways.

Downgrade operations are not specifically addressed by these procedures. Downgrade operations on gentle grades (less than 3 percent) are generally comparable to those on a level roadway. On more severe grades, downgrade operations are about midway between those experienced on a level roadway and those experienced on an upgrade of equivalent traffic and roadway characteristics. The principal concern on steep downgrades is the potential for "runaway" trucks.

The highest quality of traffic service occurs when motorists are able to drive at their desired speed. Without strict enforcement, this highest quality, representative of *level-of-service A,* would result in average speeds approaching 60 mph on two-lane highways. The passing frequency required to maintain these speeds has not reached a demanding level. Passing demand is

TABLE 8-1. LEVEL-OF-SERVICE CRITERIA FOR GENERAL TWO-LANE HIGHWAY SEGMENTS

		v/c RATIO[a]																			
		LEVEL TERRAIN							ROLLING TERRAIN							MOUNTAINOUS TERRAIN					
	PERCENT TIME	AVG[b]	PERCENT NO PASSING ZONES						AVG[b]	PERCENT NO PASSING ZONES						AVG[b]	PERCENT NO PASSING ZONES					
LOS	DELAY	SPEED	0	20	40	60	80	100	SPEED	0	20	40	60	80	100	SPEED	0	20	40	60	80	100
A	≤ 30	≥ 58	0.15	0.12	0.09	0.07	0.05	0.04	≥ 57	0.15	0.10	0.07	0.05	0.04	0.03	≥ 56	0.14	0.09	0.07	0.04	0.02	0.01
B	≤ 45	≥ 55	0.27	0.24	0.21	0.19	0.17	0.16	≥ 54	0.26	0.23	0.19	0.17	0.15	0.13	≥ 54	0.25	0.20	0.16	0.13	0.12	0.10
C	≤ 60	≥ 52	0.43	0.39	0.36	0.34	0.33	0.32	≥ 51	0.42	0.39	0.35	0.32	0.30	0.28	≥ 49	0.39	0.33	0.28	0.23	0.20	0.16
D	≤ 75	≥ 50	0.64	0.62	0.60	0.59	0.58	0.57	≥ 49	0.62	0.57	0.52	0.48	0.46	0.43	≥ 45	0.58	0.50	0.45	0.40	0.37	0.33
E	> 75	≥ 45	1.00	1.00	1.00	1.00	1.00	1.00	≥ 40	0.97	0.94	0.92	0.91	0.90	0.90	≥ 35	0.91	0.87	0.84	0.82	0.80	0.78
F	100	< 45	—	—	—	—	—	—	< 40	—	—	—	—	—	—	< 35	—	—	—	—	—	—

[a] Ratio of flow rate to an ideal capacity of 2,800 pcph in both directions.

[b] Average travel speed of all vehicles (in mph) for highways with design speed ≥ 60 mph; for highways with lower design speeds, reduce speed by 4 mph for each 10-mph reduction in design speed below 60 mph; assumes that speed is not restricted to lower values by regulation.

TABLE 8-2. LEVEL-OF-SERVICE CRITERIA FOR SPECIFIC GRADES

LEVEL OF SERVICE	AVERAGE UPGRADE SPEED (MPH)
A	≥ 55
B	≥ 50
C	≥ 45
D	≥ 40
E	≥ 25–40[a]
F	< 25–40[a]

[a] The exact speed at which capacity occurs varies with the percentage and length of grade, traffic compositions, and volume; computational procedures are provided to find this value.

well below passing capacity, and almost no platoons of three or more vehicles are observed. Drivers would be delayed no more than 30 percent of the time by slow-moving vehicles. A maximum flow rate of 420 pcph, total in both directions, may be achieved under ideal conditions.

Level-of-service B characterizes the region of traffic flow wherein speeds of 55 mph or slightly higher are expected on level terrain. Passing demand needed to maintain desired speeds becomes significant and approximately equals the passing capacity at the lower boundary of level-of-service B. Drivers are delayed up to 45 percent of the time on the average. Service flow rates of 750 pcph, total in both directions, can be achieved under ideal conditions. Above this flow rate, the number of platoons forming in the traffic stream begins to increase dramatically.

Further increases in flow characterize *level-of-service C*, resulting in noticeable increases in platoon formation, platoon size, and frequency of passing impediment. Average speed still exceeds 52 mph on level terrain, even though unrestricted passing demand exceeds passing capacity. At higher volume levels, chaining of platoons and significant reductions in passing capacity begin to occur. While traffic flow is stable, it is becoming susceptible to congestion due to turning traffic and slow-moving vehicles. Percent time delays are up to 60 percent. A service flow rate of up to 1,200 pcph, total in both directions, can be accommodated under ideal conditions.

Unstable traffic flow is approached as traffic flows enter *level-of-service D*. The two opposing traffic streams essentially begin to operate separately at higher volume levels, as passing becomes extremely difficult. Passing demand is very high, while passing capacity approaches zero. Mean platoon sizes of 5 to 10 vehicles are common, although speeds of 50 mph can still be maintained under ideal conditions. The fraction of no passing zones along the roadway section usually has little influence on passing. Turning vehicles and/or roadside distractions cause major shockwaves in the traffic stream. The percentage of time motorists are delayed approaches 75 percent. Maximum service flow rates of 1,800 pcph, total in both directions, can be maintained under ideal conditions. This is the highest flow rate that can be maintained for any length of time over an extended section of level terrain without a high probability of breakdown.

Level-of-service E is defined as traffic flow conditions on two-lane highways having a percent time delay of greater than 75 percent. Under ideal conditions, speeds will drop below 50 mph. Average travel speeds on highways with less than ideal conditions will be slower, as low as 25 mph on sustained upgrades. Passing is virtually impossible under level-of-service E conditions, and platooning becomes intense when slower vehicles or other interruptions are encountered.

The highest volume attainable under level-of-service E defines the capacity of the highway. Under ideal conditions, capacity is 2,800 pcph, total in both directions. For other conditions, capacity is lower. Note that the v/c ratios of Table 8-1 are not all 1.00 at capacity. This is because the ratios are relative to "ideal capacity" as discussed. Operating conditions at capacity are unstable and difficult to predict. Traffic operations are seldom observed near capacity on rural highways, primarily because of a lack of demand.

Capacity of two-lane highways is affected by the directional split of traffic. As directional split moves away from the 50/50 "ideal" condition, total two-way capacity is reduced, as follows:

Directional Split	*Total Capacity (pcph)*	*Ratio of Capacity to Ideal Capacity*	Level of Service PHF
50/50	2,800	1.00	E = 1.00
60/40	2,650	0.94	D = 0.95
70/30	2,500	0.89	C = 0.94
80/20	2,300	0.83	B = 0.92
90/10	2,100	0.75	A = 0.91
100/0	2,000	0.71	

For short lengths of two-lane road, such as tunnels or bridges, opposing traffic interactions may have only a minor effect on capacity. The capacity in each direction may approximate that of a fully loaded single lane, given appropriate adjustments for the lane width and shoulder width *(5)*.

As with other highway types, *level-of-service F* represents heavily congested flow with traffic demand exceeding capacity. Volumes are lower than capacity, and speeds are below capacity speed. Level-of-service E is seldom attained over extended sections on level terrain as more than a transient condition; most often, perturbations in traffic flow as level E is approached cause a rapid transition to level-of-service F.

OPERATIONAL ANALYSIS

This section presents the methodology for operational analysis of general terrain segments and specific grades on two-lane highways. Separate procedures for general highway segments and grades are used, because the dynamics of traffic interaction on sustained two-lane grades differ from those on general terrain segments. Grades of less than 3 percent or shorter than 1/2 mile may be included in general terrain analysis. Grades both longer and steeper than these values should generally be treated as specific grades. Level, rolling, and mountainous terrain are as defined in Chapters 1 and 3.

The length of grade is taken to be the tangent length of grade plus a portion of the vertical curves at the beginning and end of the grade. About one-fourth of the length of vertical curves at the beginning and end of a grade are included in the grade length. Where two grades (in the same direction) are joined by a vertical curve, one-half the length of the curve is included in each grade segment.

The objective of operational analysis is generally the determination of level of service for an existing or projected facility operating under existing or projected traffic demand. Operational analysis may also be used to determine the capacity of a two-lane highway segment, or the service flow rate which can be accommodated at any given level of service.

Use of the Peak Hour Factor

As for other facility types, two-lane highway analysis is based on flow rates for a peak 15-min period within the hour of interest, which is usually the peak hour. The criteria of Table 8-1 refer to equivalent hourly flow rates based on the peak 15 min of flow.

These criteria are used to compute service flow rates, *SF*, which are compared to existing or projected flow rates to determine level of service. Thus, full-hour demand volumes must be converted to flow rates for the peak 15 min, as follows:

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

where:

v = flow rate for the peak 15 min, total for both directions of flow, in vph;
V = full-hour volume total for both directions of flow, in vph; and
PHF = peak hour factor.

When criteria are compared to flow rates, the predicted operating characteristics are expected to prevail for the 15-min period for which the flow rate applies. For many rural conditions, the analyst may wish to examine average conditions over a peak hour. Full-hour volumes, unadjusted for the PHF, are compared to criteria directly for these cases. It should be noted, however, that prediction of an average level-of-service C during a full hour may include portions of the hour operating at level D or E, while other portions operate at A or B.

The decision to use flow rates or full-hour volumes in an analysis is related to whether or not peaking characteristics will cause substantial fluctuation in operating conditions within the peak hour, and whether the impact of such fluctuations will impact design and/or operational policy decisions. In general, where the peak hour factor is less than 0.85, operating conditions will vary substantially within the hour.

Where the peak hour factor can be determined from local field data, this should be done. Where field data are not available, the factors tabulated in Table 8-3 may be used. These are based solely on the assumption of random flow and may be somewhat higher than those obtained from field studies. When level of service is to be determined for a given traffic volume, a value appropriate to the volume level on the subject segment is selected from the upper portion of the table. When a service flow rate is to be computed, a value is selected from the lower portion of the table, because volume is unknown.

Analysis of General Terrain Segments

The general terrain methodology estimates average traffic operational measures along a section of highway based on average terrain, geometric, and traffic conditions. Terrain is classified as level, rolling, or mountainous, as described previously. The general terrain procedure is usually applied to highway sections of at least 2 miles in length.

Highway geometric features include a general description of longitudinal section characteristics and specific roadway cross-section information. Longitudinal section characteristics are described by the average percent of the highway having no passing zones. The average for both directions is used. The percentage of roadway along which sight distance is less than 1,500 ft may be used as a surrogate for no passing zone data. Roadway cross-section data include lane width and usable shoulder width. Geometric data on design speed and specific grades are not used directly, but are reflected in the other geometric factors discussed.

TABLE 8-3. PEAK HOUR FACTORS FOR TWO-LANE HIGHWAYS BASED ON RANDOM FLOW

A. LEVEL-OF-SERVICE DETERMINATIONS

TOTAL 2-WAY HOURLY VOLUME (VPH)	PEAK HOUR FACTOR (PHF)	TOTAL 2-WAY HOURLY VOLUME (VPH)	PEAK HOUR FACTOR (PHF)
100	0.83	1,000	0.93
200	0.87	1,100	0.94
300	0.90	1,200	0.94
400	0.91	1,300	0.94
500	0.91	1,400	0.94
600	0.92	1,500	0.95
700	0.92	1,600	0.95
800	0.93	1,700	0.95
900	0.93	1,800	0.95
		≥ 1,900	0.96

B. SERVICE FLOW-RATE DETERMINATIONS

Level of Service	A	B	C	D	E
Peak Hour Factor	0.91	0.92	0.94	0.95	1.00

Traffic data needed to apply the general terrain methodology include the two-way hourly volume, a peak hour factor, and the directional distribution of traffic flow. Peak hour factors may be computed from field data, or appropriate default values may be selected from Table 8-3. Traffic data also include the proportion of trucks, recreational vehicles (RV's), and buses in the traffic stream. When estimates of the traffic mix are not available, the following default values for these fractions may be used for primary routes: HPMS Analytical

- P_T = 0.14 (trucks)
- P_R = 0.04 (RV's)
- P_B = 0.00 (buses)

Recreational routes would typically have a higher proportion of recreational vehicles than shown for primary rural routes.

1. *General relationship*—The general relationship describing traffic operations on general terrain segments is as follows:

$$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_{HV} \qquad (8\text{-}1)$$

where:

SF_i = total service flow rate in both directions for prevailing roadway and traffic conditions, for level of service *i*, in vph;

$(v/c)_i$ = ratio of flow rate to ideal capacity for level of service *i*, obtained from Table 8-1;

f_d = adjustment factor for directional distribution of traffic, obtained from Table 8-4;

f_w = adjustment factor for narrow lanes and restricted shoulder width, obtained from Table 8-5;

f_{HV} = adjustment factor for the presence of heavy vehicles in the traffic stream, computed as:

$$f_{HV} = 1/[1 + P_T(E_T - 1) + P_R(E_R - 1) + P_B(E_B - 1)] \qquad (8\text{-}2)$$

$= 1/(1+P_T$

where:

P_T = proportion of trucks in the traffic stream, expressed as a decimal;

P_R = proportion of RV's in the traffic stream, expressed as a decimal;

P_B = proportion of buses in the traffic stream, expressed as a decimal;

E_T = passenger-car equivalent for trucks, obtained from Table 8-6;

E_R = passenger-car equivalent for RV's, obtained from Table 8-6; and

E_B = passenger-car equivalent for buses, obtained from Table 8-6. = assuming 0% buses in [illegible]

Equation 8-1 takes an ideal capacity of 2,800 pcph, and adjusts it to reflect a v/c ratio appropriate for the desired level of service, directional distributions other than 50/50, lane width restrictions and narrow shoulders, and heavy vehicles in the traffic stream.

2. *Adjustment for v/c ratio*—The v/c ratios given in Table 8-1 reflect a complex relationship among speed, flow, delay, and geometric parameters for two-lane highways. Specifically, v/c values vary with level-of-service criteria, terrain type, and the magnitude of passing restrictions. Note that v/c ratios at capacity are not equal to 1.00 for rolling or mountainous terrain. This is because the ratios are based on an ideal capacity of 2,800 pcph which cannot be achieved on severe terrains. Further, as the formation of platoons is more frequent where terrain is rolling or mountainous, passing restrictions have a greater effect on capacity and service flow rate than on level terrain.

3. *Adjustment for directional distribution*—All of the v/c values in Table 8-1 are for a 50/50 directional distribution of traffic on a two-lane highway. For other directional distributions, the factors shown in Table 8-4 must be applied to Table 8-1 values.

4. *Adjustment for narrow lanes and restricted shoulder width*—Narrow lanes force motorists to drive closer to vehicles in the opposing lane than they would normally desire. Restricted or narrow shoulders have much the same effect, as drivers "shy" away from roadside objects or point restrictions perceived to be close enough to the roadway to pose a hazard. Motorists compensate for driving closer to opposing vehicles by slowing down and/or by leaving larger headways between vehicles in the same lane. Both reactions result in lower flow rates being sustained at any given speed.

Factors reflecting this behavior are shown in Table 8-5, and are applied to v/c values taken from Table 8-1. Factors at capacity are higher than those for other levels of service, as the impact of narrow lanes and restricted shoulder widths is less deleterious when vehicles are already traveling at reduced speeds which prevail under capacity operation.

5. *Adjustment for heavy vehicles in the traffic stream*—The v/c ratios of Table 8-1 are based on a traffic stream consisting of only passenger cars. All vehicles having only four wheels contacting the pavement may be considered to be passenger cars. This includes light vans and pick-up trucks.

"Heavy vehicles" are categorized as trucks, recreational vehicles, or buses, and the traffic stream is characterized by the proportion of such vehicles in the traffic mix. The adjustment factor for heavy vehicles, f_{HV}, is computed using Eq. 8-2 and the passenger-car equivalents given in Table 8-6.

A wide range in the proportions of trucks and RV's in the traffic stream are found on rural highways. Equation 8-2 will yield an adjustment factor for any given mix. In addition, there is some variation in the weight distribution between heavy (> 35,000 lb) and medium-duty (≤ 35,000 lb) trucks. The equivalents of Table 8-6 assume a 50/50 distribution between heavy and medium-duty trucks. Two-lane highways serving unusually high proportions of heavy trucks, such as in coal, gravel, or timber operations, particularly those in mountainous terrain, would have higher values of E_T than those shown in the table.

The deleterious impact of heavy vehicles on two-lane highways increases markedly as terrain becomes more severe. As heavy vehicles slow on steeper grades, platoon formation becomes more frequent and severe. This effect is compounded by passing sight distance restrictions often accompanying severe terrain and leads to serious deterioration of traffic flow.

Analysis of Specific Grades

The analysis of extended specific grades on two-lane highways is more complex than for general terrain segments. The analysis procedures assume that the approach to the grade is level. On such grades, the operation of upgrade vehicles is substantially

TABLE 8-4. ADJUSTMENT FACTORS FOR DIRECTIONAL DISTRIBUTION ON GENERAL TERRAIN SEGMENTS

Directional Distribution	100/0	90/10	80/20	70/30	60/40	50/50
Adjustment Factor, f_d	0.71	0.75	0.83	0.89	0.94	1.00

TABLE 8-5. ADJUSTMENT FACTORS FOR THE COMBINED EFFECT OF NARROW LANES AND RESTRICTED SHOULDER WIDTH, f_w

USABLE[a] SHOULDER WIDTH (FT)	12-FT LANES		11-FT LANES		10-FT LANES		9-FT LANES	
	LOS A–D	LOS[b] E	LOS A–D	LOS[b] E	LOS A–D	LOS[b] E	LOS A–D	LOS[b] E
≥ 6	1.00	1.00	0.93	0.94	0.84	0.87	0.70	0.76
4	0.92	0.97	0.85	0.92	0.77	0.85	0.65	0.74
2	0.81	0.93	0.75	0.88	0.68	0.81	0.57	0.70
0	0.70	0.88	0.65	0.82	0.58	0.75	0.49	0.66

[a] Where shoulder width is different on each side of the roadway, use the average shoulder width.
[b] Factor applies for all speeds less than 45 mph.

impacted, while downgrade vehicles experience far less impact. As a result, level-of-service criteria presented in Table 8-2 are based on the average *upgrade* travel speed. This speed is the average speed of all vehicles traveling up the grade.

Where composite grades are present, the average grade is used in analysis. The average grade is the total rise, in feet, of the composite grade divided by the horizontal length of the grade, in feet, multiplied by 100 to adjust from a decimal to a percentage.

The average upgrade speed at which capacity occurs varies between 25 and 40 mph, depending upon the percent grade, the percentage of no passing zones, and other factors. Because operating conditions at capacity vary for each grade, the finding of capacity is not as straightforward as service flow rate computations for levels-of-service A through D, where speed is established using the criteria of Table 8-2.

Research has found that grades on two-lane highways have a more significant impact on operations than similar grades on multilane highways. Platoons forming behind slow-moving vehicles can be broken up or dissipated only by passing maneuvers using the opposing lane. On two-lane highways, the same geometric features causing platoons to form also tend to restrict passing opportunities as well. It has also been found that most passenger cars, even in the absence of heavy vehicles, are affected by extended grades, and will operate less efficiently than on level terrain. Additional operational problems due to vehicle stalls, accidents, or other incidents are not accounted for in the procedure. The effects of rain, snow, ice, and other negative environmental factors are also not considered.

1. *Relationship between speed and service flow rate on specific grades*—Average upgrade speeds on two-lane highways may be estimated for specific grades of a given percent and length of grade, assuming a level approach to the grade. Two-way service flow rates, *SF*, may be calculated for a specific level of service, or correspondingly, for any designated average upgrade speed. The need to provide a climbing lane based on AASHTO's safety warrant is not part of the procedure, but sample calculation 5 illustrates the evaluation of a potential climbing lane.

TABLE 8-6. AVERAGE PASSENGER-CAR EQUIVALENTS FOR TRUCKS, RV'S, AND BUSES ON TWO-LANE HIGHWAYS OVER GENERAL TERRAIN SEGMENTS

VEHICLE TYPE	LEVEL OF SERVICE	TYPE OF TERRAIN: LEVEL	ROLLING	MOUNTAINOUS
Trucks, E_T	A	2.0	4.0	7.0
	B and C	2.2	5.0	10.0
	D and E	2.0	5.0	12.0
RV's E_R	A	2.2	3.2	5.0
	B and C	2.5	3.9	5.2
	D and E	1.6	3.3	5.2
Buses, E_B	A	1.8	3.0	5.7
	B and C	2.0	3.4	6.0
	D and E	1.6	2.9	6.5

SOURCE: Ref. *6*

The service flow rate for any given average upgrade speed is given by the following relationship:

$$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_g \times f_{HV} \quad (8\text{-}3)$$

where:

SF_i = service flow rate for level-of-service i, or speed i, total vph for both directions, for prevailing roadway and traffic conditions.

$(v/c)_i$ = v/c ratio for level-of-service i or speed i, obtained from Table 8-7;

f_d = adjustment factor for directional distribution, obtained from Table 8-8;

f_w = adjustment factor for narrow lanes and restricted shoulder width, obtained from Table 8-5;

f_g = adjustment factor for the operational effects of grades on passenger cars, computed as described below; and

f_{HV} = adjustment factor for the presence of heavy vehicles in the upgrade traffic stream, computed as described subsequently.

This relationship for specific grades is generally not applied to grades of less than 3 percent or shorter than 1/2 mile.

2. *Adjustment for v/c ratio*—Table 8-7 shows values of v/c ratio related to percent grade, average upgrade speed, and percent no passing zones. The values shown are the ratio of flow rate to an ideal capacity of 2,800 pcph, and assume that passenger cars are unaffected by extended grades. Another adjustment is applied to account for the impacts of grades on passenger-car operation. This is an important point, because a v/c ratio of 1.00 in Table 8-7 *DOES NOT* necessarily signify capacity. The solution for capacity of an extended grade is discussed later. However, solutions for capacity or service flow rate exceeding 2,000 vph total indicates that the specific grade is not affecting operations and that the general terrain methodology should be used.

Values of v/c approaching or equal to 0.00 mean that the associated average upgrade speed is difficult or impossible to achieve for the percent grade and percent no passing zones indicated.

3. *Adjustment for directional distribution*—On extended grades, the directional distribution can be a critical factor affecting operations. Table 8-8 contains adjustment factors for a range of directional distributions with a significant upgrade component.

TABLE 8-7. VALUES OF v/c RATIO[a] VS. SPEED, PERCENT GRADE, AND PERCENT NO PASSING ZONES FOR SPECIFIC GRADES

PERCENT GRADE	AVERAGE UPGRADE SPEED (MPH)	PERCENT NO PASSING ZONES 0	20	40	60	80	100
3	55	0.27	0.23	0.19	0.17	0.14	0.12
	52.5	0.42	0.38	0.33	0.31	0.29	0.27
	50	0.64	0.59	0.55	0.52	0.49	0.47
	45	1.00	0.95	0.91	0.88	0.86	0.84
	42.5	1.00	0.98	0.97	0.96	0.95	0.94
	40	1.00	1.00	1.00	1.00	1.00	1.00
4	55	0.25	0.21	0.18	0.16	0.13	0.11
	52.5	0.40	0.36	0.31	0.29	0.27	0.25
	50	0.61	0.56	0.52	0.49	0.47	0.45
	45	0.97	0.92	0.88	0.85	0.83	0.81
	42.5	0.99	0.96	0.95	0.94	0.93	0.92
	40	1.00	1.00	1.00	1.00	1.00	1.00
5	55	0.21	0.17	0.14	0.12	0.10	0.08
	52.5	0.36	0.31	0.27	0.24	0.22	0.20
	50	0.57	0.49	0.45	0.41	0.39	0.37
	45	0.93	0.84	0.79	0.75	0.72	0.70
	42.5	0.97	0.90	0.87	0.85	0.83	0.82
	40	0.98	0.96	0.95	0.94	0.93	0.92
	35	1.00	1.00	1.00	1.00	1.00	1.00
6	55	0.12	0.10	0.08	0.06	0.05	0.04
	52.5	0.27	0.22	0.18	0.16	0.14	0.13
	50	0.48	0.40	0.35	0.31	0.28	0.26
	45	0.49	0.76	0.68	0.63	0.59	0.55
	42.5	0.93	0.84	0.78	0.74	0.70	0.67
	40	0.97	0.91	0.87	0.83	0.81	0.78
	35	1.00	0.96	0.95	0.93	0.91	0.90
	30	1.00	0.99	0.99	0.98	0.98	0.98
7	55	0.00	0.00	0.00	0.00	0.00	0.00
	52.5	0.13	0.10	0.08	0.07	0.05	0.04
	50	0.34	0.27	0.22	0.18	0.15	0.12
	45	0.77	0.65	0.55	0.46	0.40	0.35
	42.5	0.86	0.75	0.67	0.60	0.54	0.48
	40	0.93	0.82	0.75	0.69	0.64	0.59
	35	1.00	0.91	0.87	0.82	0.79	0.76
	30	1.00	0.95	0.92	0.90	0.88	0.86

[a] Ratio of flow rate to ideal capacity of 2,800 pcph, assuming passenger-car operation is unaffected by grade.

NOTE: Interpolate for intermediate values of "Percent No Passing Zone"; round "Percent Grade" to the next higher integer value.

TABLE 8-8. ADJUSTMENT FACTOR FOR DIRECTIONAL DISTRIBUTION ON SPECIFIC GRADES, f_d

PERCENT OF TRAFFIC ON UPGRADE	ADJUSTMENT FACTOR
100	0.58
90	0.64
80	0.70
70	0.78
60	0.87
50	1.00
40	1.20
30	1.50

4. *Adjustment for narrow lanes and/or restricted shoulder width*—The impact of narrow lanes and/or restricted shoulder widths on grades is the same as for general terrain segments. The appropriate factor is selected from Table 8-5, presented previously.

5. *Adjustment for passenger cars on grades*—The v/c ratios of Table 8-7 assume that passenger cars will maintain their speed on grades if unimpeded. Recent studies *(1,2)* have indicated that passenger-car operation is affected by grades, even where heavy vehicles are not present in the traffic stream. The factor f_g adjusts the v/c ratios of Table 8-7 to account for this effect. The factor is computed as:

$$f_g = 1/[1 + (P_p I_p)] \tag{8-4}$$

where:

f_g = adjustment factor for the operation of passenger cars on grades;

P_p = proportion of passenger cars in the upgrade traffic stream, expressed as a decimal;

I_p = impedance factor for passenger cars, computed as:

$$I_p = 0.02\,(E - E_o) \tag{8-5}$$

E = base passenger-car equivalent for a given percent grade, length of grade, and speed, selected from Table 8-9; and

E_o = base passenger-car equivalent for 0 percent grade and a given speed, selected from Table 8-9.

The passenger-car equivalents of Table 8-9 are used for both the passenger-car and heavy vehicle adjustment factors. The passenger-car factor adjusts from the base v/c ratios, which assume no operational impact of grades on cars, to prevailing conditions of grade. The heavy vehicle adjustment factor is based on passenger-car equivalents related to passenger cars operating on the grade specified.

6. *Adjustment for heavy vehicles in the traffic stream*—The adjustment factor for heavy vehicles is computed as follows:

$$f_{HV} = 1/[1 + P_{HV}(E_{HV} - 1)] \tag{8-6}$$

where:

f_{HV} = adjustment factor for the presence of heavy vehicles in the upgrade traffic stream;

P_{HV} = total proportion of heavy vehicles (trucks + RV's + buses) in the upgrade traffic stream;

E_{HV} = passenger-car equivalent for specific mix of heavy vehicles present in the upgrade traffic stream, computed as:

$$E_{HV} = 1 + (0.25 + P_{T/HV})(E - 1) \tag{8-7}$$

$P_{T/HV}$ = proportion of trucks among heavy vehicles, i.e., the proportion of trucks in the traffic stream divided by the total proportion of heavy vehicles in the traffic stream; and

E = base passenger-car equivalent for a given percent grade, length of grade, and speed, selected from Table 8-9.

The passenger-car equivalents presented in Table 8-9 represent an average mix of trucks, recreational vehicles, and buses in the traffic stream. This average mix is for 14 percent trucks, 4 percent RV's, and no buses. The values of E_{HV} computed by this procedure yield equivalent volumes which travel at the same average overall speed as the actual mixed traffic stream under stable flow conditions. Any tendency of vehicles to stall or perform sluggishly at high volume levels and power requirements is not accounted for in these procedures.

The existence of heavy vehicles on two-lane highway grades is a particularly difficult problem, because an increase in formation of platoons is caused at the same time as passing restrictions usually also increase. Thus, the decision of whether to provide a climbing lane for heavy vehicles is often a critical one for extended grades on two-lane highways. A common criterion sometimes used in the design of grades is to include a climbing lane where the operating speed of trucks falls 10 mph or more *(11)*. Figures 8-2 and 8-3 show speed reduction curves for a 200-lb/hp truck and a 300-lb/hp truck. The former is considered indicative of a representative truck for the average mix of trucks occurring on two-lane highways. The latter is representative of a "heavy" truck, such as heavily loaded farm vehicles, coal carriers, gravel carriers, or log carriers. The choice of which type of truck should be used is based on safety considerations. Speed reduction is related to the steepness and length of the grade in Figures 8-2 and 8-3. For a more detailed depiction of the operating characteristics of trucks on extended upgrades, the truck performance curves included in Appendix I of Chapter 3 may be consulted.

In addition to the 10-mph speed reduction criterion, a climbing lane might be considered wherever a level-of-service analysis indicates a serious deterioration in operating quality on an extended grade when compared to the adjacent approach segment of the same highway.

Heavy vehicles in the traffic stream on extended grades also cause delay to other vehicles. Delay can be evaluated as the difference in travel time between what vehicles could achieve if unimpeded by heavy vehicles and the travel time actually experienced in the mixed traffic stream. Sample calculations illustrate the computation of this delay.

7. *Capacity of specific grade segments*—Sections 1 through 6 above describe the computation of service flow rates on specific two-lane highway grades. For levels-of-service A through D, this is a simple process. The speed relating to the desired LOS

TABLE 8-9. PASSENGER-CAR EQUIVALENTS FOR SPECIFIC GRADES ON TWO-LANE RURAL HIGHWAYS, E AND E_o

GRADE (%)	LENGTH OF GRADE (MI)	AVERAGE UPGRADE SPEED (MPH)					
		55.0	52.5	50.0	45.0	40.0	30.0
0	All	2.1	1.8	1.6	1.4	1.3	1.3
3	¼	2.9	2.3	2.0	1.7	1.6	1.5
	½	3.7	2.9	2.4	2.0	1.8	1.7
	¾	4.8	3.6	2.9	2.3	2.0	1.9
	1	6.5	4.6	3.5	2.6	2.3	2.1
	1½	11.2	6.6	5.1	3.4	2.9	2.5
	2	19.8	9.3	6.7	4.6	3.7	2.9
	3	71.0	21.0	10.8	7.3	5.6	3.8
	4	[a]	48.0	20.5	11.3	7.7	4.9
4	¼	3.2	2.5	2.2	1.8	1.7	1.6
	½	4.4	3.4	2.8	2.2	2.0	1.9
	¾	6.3	4.4	3.5	2.7	2.3	2.1
	1	9.6	6.3	4.5	3.2	2.7	2.4
	1½	19.5	10.3	7.4	4.7	3.8	3.1
	2	43.0	16.1	10.8	6.9	5.3	3.8
	3	[a]	48.0	20.0	12.5	9.0	5.5
	4	[a]	[a]	51.0	22.8	13.8	7.4
5	¼	3.6	2.8	2.3	2.0	1.8	1.7
	½	5.4	3.9	3.2	2.5	2.2	2.0
	¾	8.3	5.7	4.3	3.1	2.7	2.4
	1	14.1	8.4	5.9	4.0	3.3	2.8
	1½	34.0	16.0	10.8	6.3	4.9	3.8
	2	91.0	28.3	17.4	10.2	7.5	4.8
	3	[a]	[a]	37.0	22.0	14.6	7.8
	4	[a]	[a]	[a]	55.0	25.0	11.5
6	¼	4.0	3.1	2.5	2.1	1.9	1.8
	½	6.5	4.8	3.7	2.8	2.4	2.2
	¾	11.0	7.2	5.2	3.7	3.1	2.7
	1	20.4	11.7	7.8	4.9	4.0	3.3
	1½	60.0	25.2	16.0	8.5	6.4	4.7
	2	[a]	50.0	28.2	15.3	10.7	6.3
	3	[a]	[a]	70.0	38.0	23.9	11.3
	4	[a]	[a]	[a]	90.0	45.0	18.1
7	¼	4.5	3.4	2.7	2.2	2.0	1.9
	½	7.9	5.7	4.2	3.2	2.7	2.4
	¾	14.5	9.1	6.3	4.3	3.6	3.0
	1	31.4	16.0	10.0	6.1	4.8	3.8
	1½	[a]	39.5	23.5	11.5	8.4	5.8
	2	[a]	88.0	46.0	22.8	15.4	8.2
	3	[a]	[a]	[a]	66.0	38.5	16.1
	4	[a]	[a]	[a]	[a]	[a]	28.0

[a] Speed not attainable on grade specified.
NOTE: Rouhd "Percent Grade" to next higher integer value.

is selected from Table 8-2, and appropriate adjustment factors are selected for use in Eq. 8-3.

The service flow rate at capacity, i.e., SF_E, is not as easily determined, because the speed at which it occurs varies depending on the percent and length of the grade in question. For the normal range of grades, i.e., 3 to 7 percent up to 4 miles long, capacity may occur at speeds ranging from 25 to 40 mph. The speed at which capacity occurs is related to the flow rate at capacity by the following equation:

$$S_c = 25 + 3.75(v_c/1000)^2 \tag{8-8}$$

where:

S_c = speed at which capacity occurs, in mph; and

v_c = flow rate at capacity, in mixed vph.

For convenience, the equation predicts *upgrade* speeds based on total two-way flow rates. The equation is valid for speed up to 40 mph.

If the service flow rates computed for various speeds using Eq. 8-3 and the capacity speed vs. capacity flow rate relationship of Eq. 8-8 are plotted, the two curves will intersect. The inter-

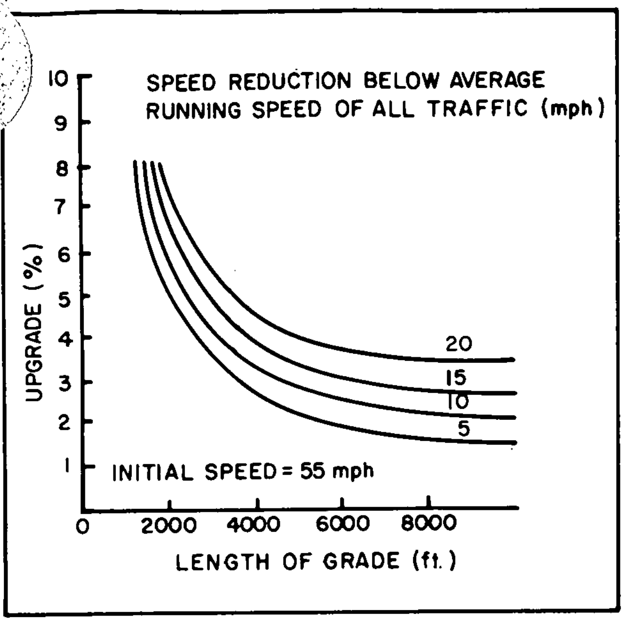


Figure 8-2. Speed reduction curve for a 200-lb/hp truck.

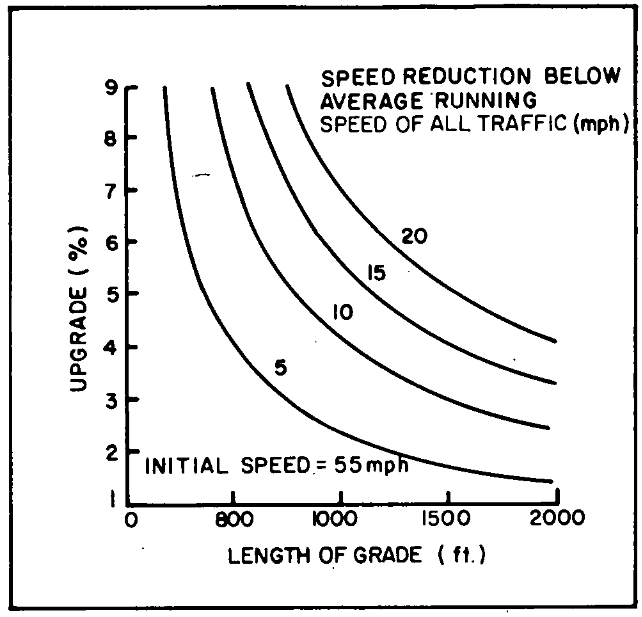


Figure 8-3. Speed reduction curve for a 300-lb/hp truck.

section defines both the speed at capacity and the flow rate at capacity for the grade in question. This procedure for determining capacity is illustrated in the sample calculations.

HIGHWAY SYSTEM PLANNING

The planning procedure enables highway operating agencies to perform very general planning and policy studies of a rural two-lane highway system. Traffic, geometric, and terrain data would be only generally classified, with traffic demand expressed in terms of an average annual daily traffic (AADT), perhaps of some future forecast year.

Table 8-10 presents estimated maximum AADT's for two-lane highways as related to:

1. Level of service.
2. Type of terrain.
3. Design hour factor, *K*.

The levels of service refer to operating conditions within the peak 15-min period of the day. In constructing Table 8-10, the default values of the peak hour factor (PHF) shown in Table 8-3 were assumed. For each level of service, the related percent time delay criteria were applied across all three types of terrain. The planning criteria also assume a typical traffic mix of 14 percent trucks, 4 percent RV's, and no buses. A 60/40 directional split is used, along with percent no passing zone values of 20 percent, 40 percent, and 60 percent for level, rolling, and mountainous terrain, respectively. Ideal geometrics of 12-ft lanes, 6-ft shoulders, and 60-mph design speed were used.

The AADT's presented in Table 8-10 illustrate a wide range of conditions, and were computed from service flow rates as follows:

$$AADT_i = SF_i \times \text{PHF}/K \qquad (8\text{-}9)$$

where:

$AADT_i$ = the maximum AADT for level-of-service *i*, based on the assumed conditions described above; vpd;
SF_i = maximum service flow rate for level-of-service *i*, computed from Eq. 8-3, based on the assumed conditions described above, in vph;
PHF = peak hour factor, selected from Table 8-3 for the indicated level of service; and
K = design hour factor, i.e., the proportion of AADT expected to occur in the design hour.

The *K*-factor is normally expressed in design problems as $DHV = AADT \times K$, where the *DHV* is the total two-way design hour volume, and *K* is estimated from the ratio of the 30th *HV* to the *AADT* from a similar site. The 30th *HV* is the 30th highest hourly volume during the year and is often used as a design volume for rural highways. Since the *DHV* should be less than SF_i for the selected level of service, the actual *AADT* for a road should be less than the maximum value shown in Table 8-10. Traffic conditions occurring during the *highest* hourly volume of the year (1st *HV*) would usually be no worse than one level of service less than that existing for the 30th *HV* for most rural highways.

TABLE 8-10. MAXIMUM AADT'S VS. LEVEL OF SERVICE AND TYPE OF TERRAIN FOR TWO-LANE RURAL HIGHWAYS

K-FACTOR	LEVEL OF SERVICE				
	A	B	C	D	E
LEVEL TERRAIN					
0.10	2,400	4,800	7,900	13,500	22,900
0.11	2,200	4,400	7,200	12,200	20,800
0.12	2,000	4,000	6,600	11,200	19,000
0.13	1,900	3,700	6,100	10,400	17,600
0.14	1,700	3,400	5,700	9,600	16,300
0.15	1,600	3,200	5,300	9,000	15,200
ROLLING TERRAIN					
0.10	1,100	2,800	5,200	8,000	14,800
0.11	1,000	2,500	4,700	7,200	13,500
0.12	900	2,300	4,400	6,600	12,300
0.13	900	2,100	4,000	6,100	11,400
0.14	800	2,000	3,700	5,700	10,600
0.15	700	1,800	3,500	5,300	9,900
MOUNTAINOUS TERRAIN					
0.10	500	1,300	2,400	3,700	8,100
0.11	400	1,200	2,200	3,400	7.300
0.12	400	1,100	2,000	3,100	6,700
0.13	400	1,000	1,800	2,900	6,200
0.14	300	900	1,700	2,700	5,800
0.15	300	900	1,600	2,500	5,400

NOTE: All values rounded to the nearest 100 vpd. Assumed conditions include 60/40 directional split, 14 percent trucks, 4 percent RV's, no buses, and PHF values from Table 8-3. For level terrain, 20 percent no passing zones were assumed; for rolling terrain, 40 percent no passing zones; for mountainous terrain, 60 percent no passing zones.

III. PROCEDURES FOR APPLICATION

The methodology described in the previous section is generally applied in either the *operational analysis* or *planning* modes.

Design computations, as used in this manual, focus on the determination of the number of lanes required for a given facility. Such computations have little significance for two-lane highways, where the number of lanes is fixed. Such design features as horizontal and vertical alignment, however, have a significant impact on operations. Operational analyses can be performed for alternative designs to document this impact. Where computations indicate that a two-lane highway is not adequate for existing or projected demands, various multilane options may be considered and analyzed using other chapters of this manual.

A separate section of this chapter deals with operational and design measures for two-lane highways, short of reconstructing the entire highway as a multilane facility. This material should be consulted where a two-lane facility presently has or is expected to experience operational difficulties.

OPERATIONAL ANALYSIS OF GENERAL TERRAIN SEGMENTS

The objective in operational analysis is to determine the level of service for a given segment or segments of roadway for a known existing set of conditions, or for a future set of conditions which are hypothesized and/or forecast. The general approach will be to compute service flow rates for each level of service and compare these values with the existing flow rate on the facility. This is done using Eq. 8-1:

$$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_{HV}$$

where all terms are as previously defined. A service flow rate for each LOS is computed because the heavy vehicle factor varies with LOS, and a direct solution of the equation for v/c ratio would be iterative. Users preferring to solve for v/c may do so, but must iterate until the assumed LOS used in computing the heavy vehicle factor is the same as that indicated by the v/c ratio found.

In general, the following computational steps are used. Computations may be conveniently performed on the worksheet illustrated in Figure 8-4.

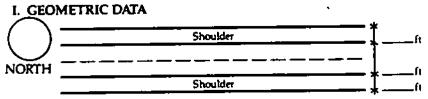
1. Summarize all input data on traffic and roadway conditions, including:
   - Existing or forecast peak hour volume, in vph.
   - Peak hour factor, PHF, from local data or default value selected from Table 8-3.
   - Traffic composition (% trucks, % RV's, % buses).

WORKSHEET FOR GENERAL TERRAIN SEGMENTS

Site Identification:________ Date:________ Time:________

Name:________ Checked by:________

I. GEOMETRIC DATA



Design Speed:________mph
% No Passing:________%
Terrain (L,R,M):________
Segment Length:________mi

II. TRAFFIC DATA

Total Volume, Both Dir.________vph
Flow Rate = Volume ÷ PHF
______ = ______ ÷ ___

Directional Distribution:________
Traffic Composition:______ %T, ______ %RV, ___%B
PHF:________

III. LEVEL OF SERVICE ANALYSIS

$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_{HV}$

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

LOS	SF	= 2,800	× (v/c) Table 8-1	× f_d Table 8-4	× f_w Table 8-5	× f_{HV}	P_T	E_T Table 8-6	P_R	E_R Table 8-6	P_B	E_B Table 8-6
A		2,800										
B		2,800										
C		2,800										
D		2,800										
E		2,800										

IV. COMMENTS Flow Rate________vph LOS = ________

Figure 8-4. Worksheet for operational analysis of general terrain segments.

- Directional distribution of traffic.
- Terrain type.
- Lane and usable shoulder widths, in ft.
- Design speed, in mph.

2. Select appropriate values of the following factors for *each* LOS from the tables indicated:
   - The v/c ratio from Table 8-1.
   - The directional distribution factor, f_d, from Table 8-4.
   - The lane width and shoulder width factor, f_w, from Table 8-5.
   - Passenger-car equivalents, E_T, E_R, and E_B, for trucks, RV's, and buses, from Table 8-6.
3. Compute the heavy vehicle factor, f_{HV}, for each LOS from:

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

4. Compute the service flow rate, *SF*, for each LOS from:

$$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_{HV}$$

5. Convert the existing or forecast volume to an equivalent flow rate, as follows: $v = V/\text{PHF}$.
6. Compare the actual flow rate of step 5 with the service flow rate of step 4 to determine the level of service.

Where the level of service is found to be inadequate, the alleviation measures presented in the next section should be considered, as well as the expansion of the facility to four or more lanes. Expansion to a multilane facility should be examined using the methodology presented in Chapter 7.

OPERATIONAL ANALYSIS OF SPECIFIC GRADES

The operational analysis of specific grades is similar to the procedure for general terrain segments. The level of service for the upgrade direction is sought, and is found by comparing an actual two-way flow rate to the service flow rates for the various levels of service. As noted in the "Methodology" section, however, the determination of capacity for specific grades requires the plotting of a service flow rate-speed curve, and a curve representing the relationship of speed at capacity to flow rate at capacity. The worksheet shown in Figure 8-5 is used to simplify the following computational steps.

1. Summarize all required input data on traffic and roadway conditions, including:
   - Existing or forecast peak hour volume, in vph.
   - Peak hour factor, PHF, from local data or default value from Table 8-3.
   - Traffic composition (% trucks, % RV's, % buses, % passenger cars).
   - Directional distribution of traffic.

WORKSHEET FOR SPECIFIC GRADES — Page 1

Site Identification:________________ Date:__________ Time:__________

Name:________________ Checked by:________________

I. GEOMETRIC DATA

NORTH

Shoulder ____ft

____ft

Shoulder ____ft

Design Speed:________mph

Grade:______%,______mi

% No Passing Zones:________

II. TRAFFIC DATA

Total Volume, Both Dir.________vph

Flow Rate = Volume ÷ PHF

______ = ______ ÷ ______

Directional Distribution:________

Traffic Composition:______%T,______%RV,______%B

PHF:________

III. SOLVING FOR ADJUSTMENT FACTORS f_g AND f_{HV}

$f_g = 1/[1 + P_p I_p]$

$I_p = 0.02\,(E - E_o)$

$f_{HV} = 1/[1 + P_{HV}(E_{HV} - 1)]$

$E_{HV} = 1 + (0.25 + P_{T/HV})(E - 1)$

Speed (mph)	P_p	I_p	E Table 8-9	E_o Table 8-9	f_g	P_{HV}	E_{HV}	$P_{T/HV}$ (P_T/P_{HV})	E Table 8-9	f_{HV}
55										
52.5										
50										
45										
40										
30										

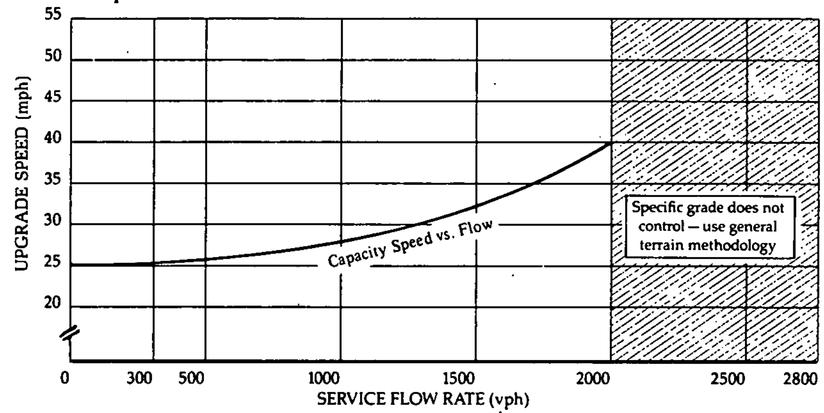
IV. SOLVING FOR SERVICE FLOW RATE

Speed (mph)	SF	2,800	× v/c	× f_d	× f_w	× f_g	× f_{HV}
			Table 8-7	Table 8-8	Table 8-5		
55 (LOS A)		2,800					
52.5		2,800					
50 (LOS B)		2,800					
45 (LOS C)		2,800					
40 (LOS D)		2,800					
30		2,800					

Figure 8-5(a). Worksheet for operational analysis of specific grades on two-lane highways (page 1).

WORKSHEET FOR SPECIFIC GRADES — Page 2

V. PLOT SF vs Speed



Intersection of Capacity Speed vs Flow curve with Service Flow Rate vs Speed curve defines Capacity, SF_E, and Speed at Capacity, S_c

VI. LEVEL OF SERVICE ANALYSIS

LOS	SF (from Worksheet)	Actual Flow Rate	Comments:
A			
B			
C			
D		Level of Service	
E			

Figure 8-5(b). Worksheet for operational analysis of specific grades on two-lane highways (page 2).

- Percent grade.
- Percent no passing zones.
- Length of grade, in miles.
- Lane and usable shoulder width, in ft.
- Design speed, in mph.

2. Select values of the following factors from the indicated tables for the following average speeds: 55 mph (LOS A), 52.5 mph, 50 mph (LOS B), 45 mph (LOS C), 40 mph (LOS D), and 30 mph. This range of speeds will allow the plotting of a service flow rate vs. speed curve to find capacity and the speed at capacity.

   - The v/c ratio from Table 8-7.
   - The directional distribution factor, f_d, from Table 8-8.
   - The lane and shoulder width factor, f_w, from Table 8-5.
   - The passenger-car equivalent, E, for the percent and length of grade, from Table 8-9.
   - The passenger-car equivalent, E_o, for a 0 percent grade, from Table 8-9.

3. Compute the grade factor, f_g, as follows:

$$f_g = 1/[1 + P_p I_p]$$
$$I_p = 0.02(E - E_o)$$

   where all values are as previously defined.
4. Compute the heavy vehicle factor, f_{HV}, for each of the speeds noted in step 2 as follows:

$$f_{HV} = 1/[1 + P_{HV}(E_{HV} - 1)]$$
$$E_{HV} = 1 + (0.25 + P_{T/HV})(E - 1)$$
$$P_{T/HV} = P_T/[P_T + P_R + P_B]$$

   where all values are as previously defined.
5. Compute the service flow rate, SF, for each of the speeds noted in step 2 as follows:

$$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_g \times f_{HV}$$

6. Plot the service flow rates vs. speeds resulting from the computations of steps 2–5 on the grid included in the worksheet of Figure 8-5. Note that the curve for speed at capacity vs. flow rate at capacity is already drawn on this grid.
7. Find the speed at capacity and the service flow rate at capacity from the intersection of the two curves on the plot of step 6.
8. Summarize the service flow rates for each level of service on the worksheet as indicated.
9. Convert the actual or forecast volume to a flow rate, as follows: $v = V/\text{PHF}$.
10. Compare the actual flow rate of step 9 with the service flow rates of step 8 to determine the level of service.

As with general terrain segments, a two-lane highway grade displaying unacceptable operating conditions would be considered for improvement. If heavy vehicles on the upgrade are the principal difficulty, the addition of a truck climbing lane should be considered. If operational problems are more broad-based, any of the alleviation techniques discussed in the next section could be considered, as well as expansion of the facility to four or more lanes. Again, the multilane option would be examined using procedures in Chapter 7.

PLANNING

The highway system planning technique described in the "Methodology" section is easily applied. Table 8-10 may be entered with a known or forecast AADT to determine expected level of service during the peak 15 min of flow, or with a known LOS to find the maximum allowable AADT. No computations are needed to use this table, although users are cautioned that any conditions varying widely from those noted in the footnotes to Table 8-10 will indicate the need to conduct an operational analysis for the facility in question.

Users may also find Table 8-10 useful in making preliminary estimates of LOS in general terrain segment analysis.

IV. DESIGN AND OPERATIONAL TREATMENTS

Addressing those operational problems that may exist on rural two-lane highways requires an understanding of the nature of two-lane highway systems. Only about 30 percent of all travel in the United States occurs on rural two-lane roads, even though this network comprises 80 percent of all paved rural highways. For the most part, two-lane highways carry light traffic and experience few operational problems. Highway agencies are typically more concerned with pavement maintenance and roadside safety issues on such highways.

Some two-lane highways, however, periodically experience severe operational and safety problems due to a variety of traffic, geometric, and environmental causes. Special treatments for such highways may be needed before capacity levels are approached. In some areas, the two-lane rural arterial system carries a disproportionately large share of rural traffic, including significant components involved in interstate commerce. Many of these highways are located near major urban areas and are experiencing rapid growth in traffic. Heavy turning movements to roadside developments can block through traffic and increase delay.

As much as 60 percent of all two-lane highway mileage is located in terrain classified as rolling or mountainous. This, coupled with occasionally high opposing volumes, is not favorable to either passing or turning maneuvers. When these and other rural highways experience increased recreational travel, major operational problems may arise. Large numbers of recreational and other heavy vehicles in the traffic stream increase the demand for passing, while at the same time, making such maneuvers more difficult. Two-lane highways serving as major routes to recreational areas may operate at or near capacity on weekends in peak seasons.

When any of the foregoing situations exist, the frequent result is a reduced level of service, increased platooning, increased delay, an increase in questionable passing maneuvers, and generally frustrated drivers. Nevertheless, many such situations do not justify the reconstruction of the two-lane highway to a full multilane facility. In these cases, one or more of the special design and/or operational treatments discussed in this section may be useful.

A wide range of design and operational solutions are needed to address the variety of problems encountered on two-lane highways. The operational and/or safety problems on a particular section may be so severe as to call for an expansion of the facility to four or more lanes. However, limited reconstruction funds, difficult terrain, and other problems may not always permit full reconstruction of a two-lane facility as a multilane highway. Less costly and less environmentally disruptive solutions may be required. Highways experiencing less severe operational and/or safety problems, together with those experiencing site-specific reductions in level of service, may be candidates for treatment with one or more of the following alleviation techniques:

1. Realignment to improve passing sight distance.
2. Use of paved shoulders.
3. Three-lane roadways with two lanes designated for travel in one direction (passing prohibited or permitted in opposing direction).
4. Three-lane road sections with continuous two-way median left-turn lanes.
5. Three-lane roadway with reversible center lane.
6. Special intersection treatments.
7. Truck or heavy vehicle climbing lanes.
8. Turnouts.
9. Short four-lane segments.

Selection of the appropriate treatment requires identification of the probable causes of the operational and safety problems existing, and the determination of cost-effectiveness of the design alternatives for a given set of highway geometric, traffic, and system constraints. The following discussions address the use of alleviation measures on two-lane highways. They are intended to provide the user with general information, and should not be construed as firm guidelines or criteria.

PASSING SIGHT DISTANCE

The opportunity to pass, given a constant volume, is a function of the availability of passing sight distance. Provision of passing sight distance is an important component in basic two-lane highway design and, as illustrated by Tables 8-1 and 8-7, has a critical impact on capacity and service flow rate. Where long queues are likely to form because of severe passing restrictions, every effort should be made to continuously and completely disperse the platoon once significant passing sight distance is regained. In these passing sections, short segments with passing sight distance restrictions should be avoided where possible. Inclusion of periodic passing lanes for each direction should be considered where the distance between segments with passing sight distance available is long and queuing extensive.

PAVED SHOULDERS

A roadway that is constructed with structurally adequate paved shoulders can be used to assist in dispersal and breakup of platoons. Slower moving vehicles may temporarily use the shoulder to permit faster vehicles to pass, returning to the travel lane when passing maneuvers have been completed. In Texas and Canada, where some agencies construct wide shoulders for a total roadway width of 40 to 44 ft, a high percentage of the driving population uses the shoulder in this manner—particularly in western Canada where long distance recreational travel is heavy during the summer. Illustration 8-2 presents a typical use of paved shoulders as described previously.

Five states allow the use of shoulders for slow-moving vehicles at all times. An additional ten states permit such use under specified conditions.

THREE-LANE HIGHWAYS

Three-lane roadways are a rational intermediate solution to four-lane expansions for two-lane highways experiencing operational problems. Because of funding and terrain constraints, three-lane roadways may be considered for spot and segment improvements. There are numerous methods for using the third travel lane on such segments.

In the 1940's and 1950's, the third (center) lane was used for passing by vehicles in either direction—the first vehicle to occupy the center lane had the right-of-way. This condition was found to be hazardous, particularly in hilly terrain. This use of three-lane highways in the United States has been generally discontinued.

Other three-lane highway treatments are being safely and efficiently applied, including the use of passing lanes, turning lanes, and climbing lanes.

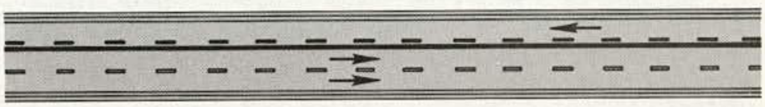
Passing Lanes

This three-lane roadway design assigns the third (center) lane to one direction of travel for a short distance (approximately 1 mile), then alternates the assignment of the passing lane to the other direction. This cyclic process may be continued along an entire highway section, or may be combined in an urban fringe area with two-way left-turn lanes and/or specific intersection turning treatments.

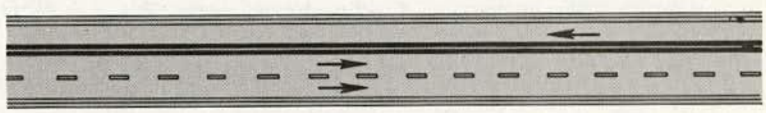
In a rural setting, intermittently spaced passing lane sections have been successfully used to break up platoons and reduce delay. Two lanes are provided for unimpeded passing in one direction for 1 to 2 miles followed by a transition to two lanes

Illustration 8-2. Slow-moving vehicle uses the shoulder of a two-lane rural highway, permitting faster vehicles to pass.

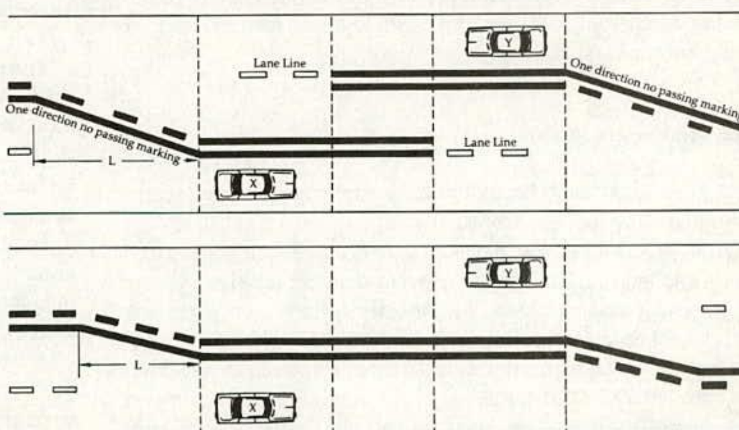
of similar design for the opposing flow. Advance signing advises motorists of the next upcoming passing lane to reduce driver anxiety and frustration. Two operational markings are practiced: passing in the single-lane direction may be permitted if passing sight distance is available, or passing in the single-lane direction may be prohibited. Figure 8-6 depicts these markings, and various methods of providing for the transition when the direction of the passing lane is changed. Permissive passing for the one-lane direction is not used by some agencies when the AADT exceeds about 3,000 vpd.



a. Typical two-way marking; passing permitted from single lane.



b. Typical two-way marking; passing prohibited from single lane.



c. Typical transition marking arrangements.

Figure 8-6. Use of third lane for passing lanes.

TABLE 8-11. SPACING OF PASSING LANES ON TWO-LANE HIGHWAYS

Two-Way Peak Hourly Volume (vph)	400	300	200
Distance to Next Passing Lane (miles)	5	6.5	9

An analytic study of passing lane requirements was conducted in Ontario, Canada (*7*). This study recommended that passing lanes should consistently be from 1.0 to 1.25 miles long. This length was found to be adequate to disperse most platoons, to provide for additional transition zones, and yet not be too long to change drivers' expectations about the true nature of the highway. Table 8-11 gives the recommended spacing between passing lanes in a given direction which resulted from the study.

Continuous Two-Way Median Left-Turn Lanes

On two-lane highways having sizable left-turn traffic, a single travel lane in each direction often becomes subject to long delays as vehicles await opportunities to complete left turns. By providing a continuous refuge area for left-turning traffic, the two-way left-turn lane can help to maintain through traffic capacity, with the added benefit of separating opposing flows. The ability to pass, however, is eliminated.

Two-way left-turn lanes are not usually used where speeds are less than 25 mph or more than 50 mph, and are most often used in urban fringe areas or on a major route passing through a small town or village.

Reversible Lane

This is another use of the third (center) lane of a three-lane highway which is most applicable where travel demands are of a tidal nature—that is, extreme directional splits occur. The center lane is reversed by time of day to match the peak flow. The center lane is controlled by overhead signs or traffic signals indicating the direction of travel assigned at the time. Passing is not permitted in this application in the direction of the single lane.

The reversible lane technique is most applicable to routes joining residential areas and high-employment centers, and for many recreational routes.

Intersection Treatments

Conventional analysis of two-lane highways assumes uninterrupted flow, which is normally representative of rural conditions. With increasing development occurring in some rural areas, and in suburban fringe areas, the demand for high-volume access and egress can grow. Major intersections along two-lane highways become more common and important to the overall quality of flow on main routes. Adequate protected turning lanes for both left and right turns are useful in minimizing disruption to through traffic. Bypass lanes for through traffic may be considered where a protected left-turn lane is not feasible, particularly where paved shoulders are provided and/or where T-intersections are involved.

Detailed analysis of intersections may be performed using the procedures of Chapter 9, "Signalized Intersections," and Chapter 10, "Unsignalized Intersections."

Climbing Lanes

Traditional climbing lanes also form three-lane cross sections when used in conjunction with two-lane highways. They are generally applied as a spot improvement, most often on steep, sustained grades which cause heavy vehicles, particularly heavy trucks, to travel at slow speeds. This reduces capacity, creates platoons, and increases delay. Additionally, safety problems may arise when the reduction in speed of heavy trucks exceeds 10 mph along the grade.

Estimated operating speed characteristics of trucks are illustrated in Figures I.3-1, I.3-2, and I.3-3 in Appendix I of Chapter 3. Resulting lengths of grade producing 10-mph speed reductions are plotted in Figures 8-2 and 8-3, presented earlier in this chapter. AASHTO presently warrants a climbing lane wherever the speed of a 300-lb/hp truck is reduced by 10 mph or more and the volume and percentage of heavy trucks justify the added cost. One set of criteria that might be applied to reflect the economic considerations is:

1. Upgrade traffic flow rate exceeds 200 vph.
2. Upgrade truck flow rate exceeds 20 vph.
3. One of the following conditions exists:
   - Level-of-service E or F exists on the grade.
   - A reduction of two or more levels of service is experienced when moving from the approach segment to the grade.
   - A 10-mph or greater speed reduction is expected for a typical heavy truck.

These general guides for the consideration of climbing lanes on grades would apply only to climbing lanes on two-lane highways and should not be used in conjunction with consideration of climbing lanes on multilane highways.

Turnouts

The use of turnouts for improving the level of service on two-lane, two-way highways is more prevalent in the rolling and mountainous terrain of the western United States. Turnouts are short segments of a third lane added to one side of the highway or the other which permit slow vehicles at the head of platoons to pull off the main roadway, allowing faster vehicles to pass. Turnouts are used satisfactorily on both upgrades and downgrades, as well as on level terrain, to improve traffic flow. Impeding motorists are legally required to use turnouts where provided under certain prescribed conditions, which vary by state.

A recent study of operational characteristics revealed that few drivers actually stop at turnouts (*8*). Several additional conclusions drawn from this study included:

1. Turnouts are safe when properly used.
2. A series of turnouts at regular intervals can provide considerable delay reduction.

3. Turnouts are not a substitute for a passing or climbing lane of adequate length.
4. About 10 percent of all platoon leaders use properly designated turnouts.
5. Large trucks tend to avoid turnouts.

Turnouts are a short but functional treatment of irritating causes of operational delay. A western state recommends that the length of turnouts vary with approach speed according to the criteria of Table 8-12 (*9*).

Approach speeds of potential turnout-users vary with prevailing traffic and roadway conditions, and differ between upgrades and downgrades. Turnout lengths of more than 500 ft are only used on downgrades exceeding 3 percent where high approach speeds are expected to exist. Lengths greater than 600 ft are never designed, as drivers may mistakenly attempt to use them as passing lanes.

TABLE 8-12. LENGTH OF TURNOUTS ON TWO-LANE HIGHWAYS

Approach Speed (mph)	25	30	40	50	55	60
Minimum Length of Turnout (ft)	200	200	250	375	450	535

SHORT FOUR-LANE SECTIONS

Short sections of four-lane cross section may be constructed along a primarily two-lane highway to break up platoons, to provide the desired frequency of safe passing zones, and to eliminate interference from low-speed vehicles. Such sections are particularly advantageous in rolling terrain, or where the alignment is winding or the profile includes critical grades from both directions. The decision to use a short four-lane segment, as compared to using a three-lane option, may be based on long-range planning objectives for the facility, availability of rights-of-way, existing cross section, topography, and on the desire to reduce platooning and passing problems.

The transition from a two-lane to a four-lane roadway should be designed to provide sufficient sight distance for passing. For the length of four-lane segments, AASHTO suggests that they be sufficiently long to permit several vehicles in line behind a slow-moving vehicle to pass before reaching the normal section of two-lane highway. Four-lane sections of 1.0 to 1.5 miles should be sufficiently long to dissipate most queues formed, depending on volume and terrain conditions. Further, it is noted that sections of four-lane highway, particularly divided sections, longer than 2 miles may cause drivers to lose their sense of awareness that the road is basically a two-lane facility.

V. SAMPLE CALCULATIONS

CALCULATION 1—FINDING SERVICE FLOW RATES FOR A GENERAL TERRAIN SEGMENT

1. *Description*—A segment of rural two-lane highway is expected to have the following characteristics:
   a. *Roadway characteristics*—70-mph design speed; 12-ft lanes; 10-ft paved shoulders; level terrain; 0 percent no passing zones; length = 5 miles.
   b. *Traffic characteristics*—70/30 directional split; 10 percent trucks; 5 percent recreational vehicles; 1 percent buses; 84 percent passenger cars.

What is the capacity of the section? What is the maximum flow rate which can be accommodated at level-of-service C?

2. *Solution*—The solution to this problem is found by computing the service flow rates for levels-of-service C and E (capacity), using Eq. 8.1:

$$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_{HV}$$

where

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

The following values are selected for use in these computations:

$(v/c)_C$ = 0.43 (Table 8-1, level terrain 0 percent no passing zones, LOS C);
$(v/c)_E$ = 1.00 (Table 8-1, level terrain, 0 percent no passing zones, LOS E);
f_d = 0.89 (Table 8-4, 70/30 split);
f_w = 1.00 (Table 8-5, 12-ft lanes, > 6-ft shoulders);
E_T = 2.2 for LOS C, 2.0 for LOS E (Table 8-6, level terrain);
E_R = 2.5 for LOS C, 1.6 for LOS E (Table 8-6, level terrain);
E_B = 2.0 for LOS C, 1.6 for LOS E (Table 8-6, level terrain);
P_T = 0.10 (Given);
P_R = 0.05 (Given); and
P_B = 0.01 (Given).

Then:

$$f_{HV}(\text{LOS C}) = 1/[1 + 0.10(2.2 - 1) + 0.05(2.5 - 1) + 0.01(2.0 - 1)] = 0.83$$

$$f_{HV}(\text{LOS E}) = 1/[1 + 0.10(2.0 - 1) + 0.05(1.6 - 1) + 0.01(1.6 - 1)] = 0.88$$

WORKSHEET FOR GENERAL TERRAIN SEGMENTS

Site Identification: State Hwy 3/7 Date: 9/18/85 Time: 5-6 PM

Name: John Jones Checked by: ____

I. GEOMETRIC DATA

NORTH

Shoulder 10 ft
24 ft
Shoulder 10 ft

Design Speed: 70 mph
% No Passing: 0 %
Terrain (L,R,M): L
Segment Length: 5 mi

II. TRAFFIC DATA

Total Volume, Both Dir. ____ vph
Flow Rate = Volume ÷ PHF
____ = ____ ÷ ____

Directional Distribution: 70/30
Traffic Composition: 10 %T, 5 %RV, 1 %B
PHF: ____

III. LEVEL OF SERVICE ANALYSIS

$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_{HV}$

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

LOS	SF	= 2,800	× (v/c) Table 8-1	× f_d Table 8-4	× f_w Table 8-5	× f_{HV}	P_T	E_T Table 8-6	P_R	E_R Table 8-6	P_B	E_B Table 8-6
A		2,800										
B		2,800										
C	889	2,800	0.43	0.89	1.00	0.83	.10	2.2	.05	2.5	.01	2.0
D		2,800										
E	2193	2,800	1.00	0.89	1.00	0.88	.10	2.0	.05	1.6	.01	1.6

IV. COMMENTS

Flow Rate ____ vph LOS = ____

Figure 8-7. Worksheet summarizing solution to Calculation 1.

WORKSHEET FOR GENERAL TERRAIN SEGMENTS

Site Identification: Overland Road Date: 9/18/85 Time: 4-5 PM

Name: Thomas Smith Checked by: ____

I. GEOMETRIC DATA

NORTH

Shoulder 2 ft
22 ft
Shoulder 2 ft

Design Speed: 60 mph
% No Passing: 80 %
Terrain (L,R,M): M
Segment Length: 10 mi

II. TRAFFIC DATA

Total Volume, Both Dir. 180 vph
Flow Rate = Volume ÷ PHF
207 = 180 ÷ 0.87

Directional Distribution: 60/40
Traffic Composition: 5 %T, 10 %RV, 0 %B
PHF: 0.87

III. LEVEL OF SERVICE ANALYSIS

$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_{HV}$

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

LOS	SF	= 2,800	× (v/c) Table 8-1	× f_d Table 8-4	× f_w Table 8-5	× f_{HV}	P_T	E_T Table 8-6	P_R	E_R Table 8-6	P_B	E_B Table 8-6
A	23	2,800	0.02	0.94	0.75	0.588	.05	7	.10	5.0		
B	127	2,800	0.12	0.94	0.75	0.535	.05	10	.10	5.2		
C	211	2,800	0.20	0.94	0.75	0.535	.05	10	.10	5.2		
D	371	2,800	0.37	0.94	0.75	0.508	.05	12	.10	5.2		
E	941	2,800	0.80	0.94	0.88	0.508	.05	12	.10	5.2		

IV. COMMENTS

Flow Rate 207 vph LOS = C

Figure 8-8. Worksheet summarizing solution to Calculation 2.

and:

$SF_C = 2{,}800 \times 0.43 \times 0.89 \times 1.00 \times 0.83 = 889\ vph$

$SF_E = 2{,}800 \times 1.00 \times 0.89 \times 1.00 \times 0.88 = 2{,}193\ vph$

Thus, the highway will have an expected capacity of 2,193 vph, total in both directions, and can accommodate a flow rate of up to 889 vph at level-of-service C. The worksheet for general terrain sections may be used to perform these computations, as shown in Figure 8-7.

CALCULATION 2—FINDING LEVEL OF SERVICE FOR A GENERAL TERRAIN SEGMENT

1. *Description*—A two-lane rural highway carries a peak hour volume of 180 vph and has the following characteristics:
   a. *Roadway characteristics*—60-mph design speed; 11-ft lanes; 2-ft shoulders; mountainous terrain; 80 percent no passing zones; length = 10 miles.
   b. *Traffic characteristics*—60/40 directional split; 5 percent trucks; 10 percent recreational vehicles; no buses; 85 percent passenger cars.

At what level of service will the highway operate during peak periods?

2. *Solution*—The solution is found by comparing the actual flow rate to service flow rates computed for each LOS. The actual flow rate is found as:

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

where:

V = 180 vph (Given)
PHF = 0.87 (Default value, Table 8-3, 200 vph)

and:

$$v = 180/0.87 = 207 \text{ vph}$$

Service flow rates are computed from Eq. 8-1:

$$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_{HV}$$
$$f_{HV} = 1/[1 + P_T(E_T - 1) + P_R(E_R - 1) + P_B(E_B - 1)]$$

where:

v/c = 0.02 for LOS A, 0.12 for LOS B, 0.20 for LOS C, 0.37 for LOS D, 0.80 for LOS E (Table 8-1, mountainous terrain, 80 percent no passing zones);

f_d = 0.94 (Table 8-4, 60/40 split);

f_w = 0.75 for LOS A through D, 0.88 for LOS E (Table 8-5, 11-ft lanes, 2-ft shoulders);

E_T = 7 for LOS A, 10 for LOS B, C, 12 for LOS D, E, (Table 8-6, mountainous terrain);

E_R = 5.0 for LOS A, 5.2 for LOS B-E (Table 8-6, mountainous terrain);

P_T = 0.05 (Given); and

P_R = 0.10 (Given).

Then:

$$f_{HV}(\text{LOS A}) = 1/[1 + 0.05(7 - 1) + 0.10(5.0 - 1)] = 0.588$$
$$(\text{LOS B, C}) = 1/[1 + 0.05(10 - 1) + 0.10(5.2 - 1)] = 0.535$$
$$(\text{LOS D, E}) = 1/[1 + 0.05(12 - 1) + 0.10(5.2 - 1)] = 0.508$$

and:

$SF_A = 2{,}800 \times 0.02 \times 0.94 \times 0.75 \times 0.588 = 23$ vph
$SF_B = 2{,}800 \times 0.12 \times 0.94 \times 0.75 \times 0.535 = 127$ vph
$SF_C = 2{,}800 \times 0.20 \times 0.94 \times 0.75 \times 0.535 = 211$ vph
$SF_D = 2{,}800 \times 0.37 \times 0.94 \times 0.75 \times 0.508 = 371$ vph
$SF_E = 2{,}800 \times 0.80 \times 0.94 \times 0.88 \times 0.508 = 941$ vph

If the actual flow rate of 207 vph (which represents the flow rate during the peak 15 min of flow) is compared to these values, it is seen that it is higher than the service flow rate for LOS B (127 vph), but is less than the service flow rate for LOS C (211 vph). Therefore, the level of service for the highway is C for the conditions described.

This problem illustrates several points. On severe terrain, such as the situation for this problem, "good" operating conditions can be sustained only at low flow rates. The capacity of the roadway is also severely limited, reaching only 941 vph, which is approximately one-third of the ideal capacity of 2,800 vph. Note that the v/c ratio used in the computation of capacity is only 0.80. This is because all v/c ratios in the two-lane methodology are referenced to the ideal capacity of 2,800 vph, which cannot be achieved in severe terrain with passing sight distance restrictions.

This solution may be summarized or done on the general terrain section worksheet, as shown in Figure 8-8.

CALCULATION 3—FINDING SERVICE FLOW RATES FOR A SPECIFIC GRADE

1. *Description*—A rural two-lane highway in mountainous terrain has a 6 percent grade of 2 miles. Other relevant characteristics include:
   a. *Roadway characteristics*—12-ft lanes; 8-ft shoulders; 60 percent no passing zones.
   b. *Traffic characteristics*—70/30 directional split; 12 percent trucks; 7 percent recreational vehicles; 1 percent buses, 80 percent passenger cars; PHF = 0.85.

What is the maximum volume which can be accommodated on the grade at a speed of 40 mph (LOS D, Table 8-2)?

2. *Solution*—Service flow rate on specific grades is computed using Eq. 8-3, as follows:

$$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_g \times f_{HV}$$

where:

$$f_g = 1/[1 + P_p I_p] \text{ from Eq. 8-4}$$

$$I_p = 0.02\,(E - E_o] \text{ from Eq. 8-5}$$

and:

$$f_{HV} = 1/[1 + P_{HV}(E_{HV} - 1)] \text{ from Eq. 8-6}$$

$$E_{HV} = 1 + (0.25 + P_{T/HV})\,(E - 1) \text{ from Eq. 8-7}$$

The following values are used in these computations:

$(v/c)_D = 0.83$ (Table 8-7, 40 mph, 6 percent grade, 60 percent no passing zones);
$f_d = 0.78$ (Table 8-8, 70/30 split, 70 percent upgrade);
$f_w = 1.00$ (Table 8-5, 12-ft lanes, > 6-ft shoulders);
$E = 10.7$ (Table 8-9, 40 mph, 6 percent for 2-mile grade);
$E_o = 1.3$ (Table 8-9, 40 mph, 0 percent grade);
$P_{HV} = P_T + P_R + P_B = 0.12 + 0.07 + 0.01 = 0.20$; and
$P_{T/HV} = P_T/P_{HV} = 0.12/0.20 = 0.60$.

Then, computing factors f_g and f_{HV}:

$$I_p = 0.02\,(10.7 - 1.3) = 0.188$$
$$f_g = 1/[1 + (0.80 \times 0.188)] = 0.87$$
$$E_{HV} = 1 + (0.25 + 0.60)\,(10.7 - 1) = 9.25$$
$$f_{HV} = 1/[1 + 0.20(9.25 - 1)] = 0.38$$

The service flow rate for the peak 15 min is now computed using Eq. 8-3:

$$SF_D = 2{,}800 \times 0.83 \times 0.78 \times 1.00 \times 0.87 \times 0.38 = \mathit{599\ vph}$$

Since the question asks for a maximum *volume,* rather than a flow rate, the service flow rate is converted to a full hour volume as follows:

$$V = SF \times \text{PHF} = 599 \times 0.85 = \mathit{509\ vph}$$

Thus, the maximum full-hour volume which can be accommodated at 40 mph, or LOS D, on the grade described is 509 vph. The maximum flow rate is 599 vph.

CALCULATION 4—FINDING LEVEL OF SERVICE AND CAPACITY OF A SPECIFIC GRADE

1. *Description*—A rural two-lane highway in mountainous terrain has a grade of 7 percent, 2 miles long. It currently carries a peak hour volume of 500 vph. Other relevant characteristics include:

a. *Roadway characteristics*—60-mph design speed; 11-ft lanes; 4-ft shoulders; 80 percent no passing zones.

b. *Traffic characteristics*—80/20 directional split; 4 percent trucks; 10 percent recreational vehicles; 2 percent buses; 84 percent passenger cars; PHF=0.85.

At what level of service does the grade operate? What upgrade speed can be expected during the peak 15 min of flow? What is the capacity of the grade? If the approach speed to the grade is 55 mph, what delay is incurred by vehicles climbing the grade?

2. *Solution*—The finding of capacity for a specific grade requires plotting of the service flow rate vs. speed curve which results from Eq. 8-3:

$$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_g \times f_{HV}$$

where:

$$f_g = 1/[1 + P_p I_p]$$

$$I_p = 0.02\,(E - E_o)$$

and:

$$f_{HV} = 1/[1 + P_{HV}(E_{HV} - 1)]$$

$$E_{HV} = 1 + (0.25 + P_{T/HV})\,(E - 1)$$

Capacity is found at the point where this curve intersects the speed at capacity vs. flow rate at capacity curve on the specific grade worksheet. The upgrade speed is found by entering this curve with the actual flow rate.

To plot the curve, the procedure recommends computing service flow rate points for the following speeds: 55 mph (LOS A), 52.5 mph, 50 mph (LOS B), 45 mph (LOS C), 40 mph (LOS D), and 30 mph. These points would be plotted on the specific grade worksheet of Figure 8-5, and a smooth curve constructed. Once capacity is determined, the service flow rates for every LOS will be known, and the actual LOS can be determined by comparing the actual flow rate to the computed values.

The following values are used in these computations:

v/c = 0.00 for 55 mph, 0.05 for 52.5 mph, 0.15 for 50 mph, 0.40 for 45 mph, 0.64 for 40 mph, 0.88 for 30 mph (Table 8-7, 7 percent grade, 80 percent no passing zones);
$f_d = 0.70$ (Table 8-8, 80/20 split);
f_w = 0.85 for 55–45 mph, 0.92 for 45–30 mph (Table 8-5, 11-ft lanes, 4-ft shoulders);
E = 88.0 for 52.5 mph, 46.0 for 50 mph, 22.8 for 45 mph, 15.4 for 40 mph, 8.2 for 30 mph (Table 8-9, 7 percent grade, 2 miles, no value given for 55 mph);
E_o = 1.8 for 52.5 mph, 1.6 for 50 mph, 1.4 for 45 mph, 1.3 for 40 mph, 30 mph (Table 8-9, 0 percent grade);
$P_p = 0.84$ (Given);
$P_{HV} = P_T + P_R + P_B = 0.04 + 0.10 + 0.02 = 0.16$; and
$P_{T/HV} = P_T/P_{HV} = 0.04/0.16 = 0.25$.

Values of f_g may now be computed as follows:

$$I_p(52.5) = 0.02(88.0 - 1.8) = 1.724$$
$$(50.0) = 0.02(46.0 - 1.6) = 0.888$$

$$\begin{aligned}(45.0) &= 0.02(22.8 - 1.4) = 0.428\\ (40.0) &= 0.02(15.4 - 1.3) = 0.282\\ (30.0) &= 0.02(8.2 - 1.3) = 0.138\end{aligned}$$

$$\begin{aligned}f_g(52.5) &= 1/[1 + 0.84(1.724)] = 0.41\\ (50.0) &= 1/[1 + 0.84(0.888)] = 0.57\\ (45.0) &= 1/[1 + 0.84(0.428)] = 0.74\\ (40.0) &= 1/[1 + 0.84(0.282)] = 0.81\\ (30.0) &= 1/[1 + 0.84(0.138)] = 0.90\end{aligned}$$

Values of f_{HV} are also computed:

$$\begin{aligned}E_{HV}(52.5) &= 1 + (0.25 + 0.25)(88.0 - 1) = 44.5\\ (50.0) &= 1 + (0.25 + 0.25)(46.0 - 1) = 23.5\\ (45.0) &= 1 + (0.25 + 0.25)(22.8 - 1) = 11.9\\ (40.0) &= 1 + (0.25 + 0.25)(15.4 - 1) = 8.2\\ (30.0) &= 1 + (0.25 + 0.25)(\ 8.2 - 1) = 4.6\end{aligned}$$

$$\begin{aligned}f_{HV}(52.5) &= 1/[1 + 0.16(44.5 - 1)] = 0.13\\ (50.0) &= 1/[1 + 0.16(23.6 - 1)] = 0.22\\ (45.0) &= 1/[1 + 0.16(11.9 - 1)] = 0.36\\ (40.0) &= 1/[1 + 0.16(\ 8.2 - 1)] = 0.46\\ (30.0) &= 1/[1 + 0.16(\ 4.6 - 1)] = 0.63\end{aligned}$$

Having computed all relevant factors, the total two-way service flow rates for the designated speeds may be computed:

SPEED	2,800 ×	v/c ×	f_d ×	f_w ×	f_g ×	f_{HV} =	SF
55.0	2,800	0.00	0.70	0.85	—	—	0 vph
52.5	2,800	0.05	0.70	0.85	0.41	0.13	4 vph
50.0	2,800	0.15	0.70	0.85	0.57	0.22	31 vph
45.0	2,800	0.40	0.70	0.85	0.74	0.36	178 vph
40.0	2,800	0.64	0.70	0.92	0.81	0.46	430 vph
30.0	2,800	0.88	0.70	0.92	0.90	0.63	900 vph

Note that the low or zero service flow rates for 55.0 and 52.5 mph indicate that these average upgrade speeds are virtually impossible to maintain on the upgrade described in this problem.

These computations are summarized on the specific grade worksheet shown in Figure 8-9. The curve defined by these points is also plotted on the worksheet. The intersection of the plotted curve with the speed at capacity vs. flow rate at capacity curve indicates that capacity is *950 vph*, total in both directions, which occurs at an average upgrade speed of *28.0 mph.*

To find the existing level of service, the volume of 500 vph is converted to a flow rate for the peak 15-min period:

$$v = V/\text{PHF} = 500/0.85 = 588 \text{ vph}$$

The plotted curve is entered on the worksheet with 588 vph, and the upgrade speed is found to be *37 mph.* Because this speed is less than 40 mph, the minimum value for LOS D (Table 8-2), but greater than the speed at capacity (28 mph), the level of service is E. This can also be determined by comparing the actual flow rate of 588 vph with the service flow rate for LOS D (40 mph) of 430 vph and capacity (950 vph).

The last part of this problem asks to find the delay incurred by vehicles traveling up the grade. "Delay" is defined as the difference in travel time experienced by vehicles traversing the upgrade at the existing speed and the travel time which would be experienced if they were able to maintain their approach speed on the grade. Thus:

$$\begin{aligned}\text{Travel time at 55.0 mph} &= (2 \text{ miles}/55 \text{ mph}) \times 3600 \text{ sec/hour}\\ &= 130.9 \text{ sec/veh}\\ \text{Travel time at 37.0 mph} &= (2 \text{ miles}/37 \text{ mph}) \times 3600 \text{ sec/hour}\\ &= 194.6 \text{ sec/veh}\\ \text{Delay} &= 194.6 - 130.9 = 63.7 \text{ sec/veh}\end{aligned}$$

WORKSHEET FOR SPECIFIC GRADES Page 1

Site Identification: *Mountain Drive* Date: *6/13/81* Time: *4-5 PM*

Name: *Ralph Hamilton* Checked by:

I. GEOMETRIC DATA

NORTH — Shoulder *4* ft; *22* ft; Shoulder *4* ft

Design Speed: *60* mph
Grade: *7* %, *2* mi
% No Passing Zones: *80*

II. TRAFFIC DATA

Total Volume, Both Dir. *500* vph
Flow Rate = Volume ÷ PHF
588 = *500* ÷ *0.85*

Directional Distribution: *80/20*
Traffic Composition: *4* %T, *10* %RV, *2* %B
PHF:

III. SOLVING FOR ADJUSTMENT FACTORS f_g AND f_{HV}

$f_g = 1/[1 + P_p I_p]$
$I_p = 0.02(E - E_o)$

$f_{HV} = 1/[1 + P_{HV}(E_{HV} - 1)]$
$E_{HV} = 1 + (0.25 + P_{T/HV})(E - 1)$

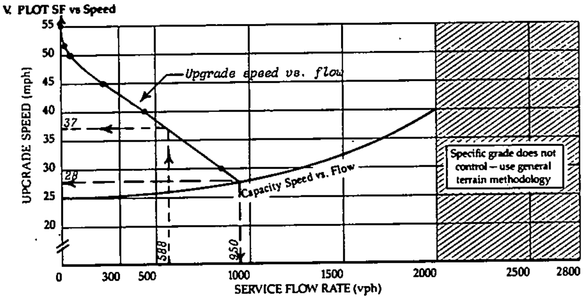
Speed (mph)	P_p	I_p	E Table 8-9	E_o Table 8-9	f_g	P_{HV}	E_{HV}	$P_{T/HV}$ (P_T/P_{HV})	E Table 8-9	f_{HV}
55										
52.5	.84	1.724	88	1.8	.41	.16	44.5	.25	88	.13
50	.84	.888	46	1.6	.57	.16	23.5	.25	46	.22
45	.84	.428	22.8	1.4	.74	.16	11.9	.25	22.8	.36
40	.84	.282	15.4	1.3	.81	.16	8.2	.25	15.4	.46
30	.84	.138	8.2	1.3	.90	.16	4.6	.25	8.2	.63

IV. SOLVING FOR SERVICE FLOW RATE

Speed (mph)	SF	2,800 ×	v/c × Table 8-7	f_d × Table 8-8	f_w × Table 8-5	f_g ×	f_{HV}
55 (LOS A)	0	2,800	.00	.70	.85	-	-
52.5	4	2,800	.05	.70	.85	.41	.13
50 (LOS B)	31	2,800	.15	.70	.85	.57	.22
45 (LOS C)	178	2,800	.40	.70	.85	.74	.36
40 (LOS D)	430	2,800	.64	.70	.92	.81	.46
30	900	2,800	.88	.70	.92	.90	.63

WORKSHEET FOR SPECIFIC GRADES Page 2

V. PLOT SF vs Speed



Intersection of Capacity Speed vs Flow curve with Service Flow Rate vs Speed curve defines Capacity, SF_c, and Speed at Capacity, S_c

VI. LEVEL OF SERVICE ANALYSIS

LOS	SF (from Worksheet)	Actual Flow Rate	Comments:
A	0	588	
B	31		
C	178		
D	430	Level of Service	
E	980	E	

Figure 8-9. Worksheet for Calculation 4 (pages 1 and 2).

CALCULATION 5—CONSIDERATION OF A CLIMBING LANE

1. *Description*—A rural two-lane highway has a 4 percent upgrade of 1½ miles, and has the following other characteristics:

   a. *Roadway characteristics*—level terrain approach; 12-ft lanes; 8-ft shoulders; 40 percent no passing zones.

   b. *Traffic characteristics*—DHV = 400 vph; 15 percent trucks; 5 percent recreational vehicles; 1 percent buses; 79 percent passenger cars; 60/40 directional split; PHF = 0.85.

Is the addition of a climbing lane justified at this location?

2. *Solution*—It is assumed that a climbing lane on a two-lane highway is generally justified when the following conditions are met:

1. Upgrade flow rate is greater than 200 vph.
2. Upgrade truck flow rate is greater than 20 vph.
3. One of the following occurs:
   a. The grade operates at LOS E or F.
   b. The typical heavy truck reduces its speed by more than 10 mph on the grade.
   c. The LOS on the grade is two or more levels poorer than on the approach to the grade.

Each of these conditions should be checked to justify the construction of the climbing lane:

Upgrade flow rate = 400 × 0.60/0.85 = 282 vph > 200 vph OK

Upgrade trucks = 400 × 0.15 × 0.60/0.85 = 42 vph > 20 vph OK

To justify a climbing lane, only one of the conditions specified in item 3 must be demonstrated. The LOS will be E or worse if the actual flow rate exceeds the service flow rate for LOS D. This value is computed using Eq. 8-3:

$$SF_D = 2{,}800 \times (v/c)_D \times f_d \times f_w \times f_g \times f_{HV}$$

where:

$$f_g = 1/[1 + P_p I_p]$$

$$I_p = 0.02\,(E - E_o)$$

and:

$$f_{HV} = 1/[1 + P_{HV}(E_{HV} - 1)]$$

$$E_{HV} = 1 + (0.25 + P_{T/HV})\,(E - 1)$$

The following values are used:

$(v/c)_D$ = 1.00 (Table 8-7, 4 percent grade, 40 mph, 40 percent no passing zones);

f_d = 0.87 (Table 8-8, 60/40 directional split);

f_w = 1.00 (Table 8-5);

E = 3.8 (Table 8-9, 4 percent, 1½-mile grade, 40 mph);

E_o = 1.3 (Table 8-9, 0 percent grade, 40 mph);

$P_{HV} = 0.15 + 0.05 + 0.01 = 0.21$; and

$P_{T/HV} = 0.15/0.21 = 0.71.$

Using these values to compute the service flow rate at level-of-service D:

$$I_p = 0.02(3.8 - 1.3) = 0.05$$

$$f_g = 1/[1 + (0.79 \times 0.05)] = 0.96$$

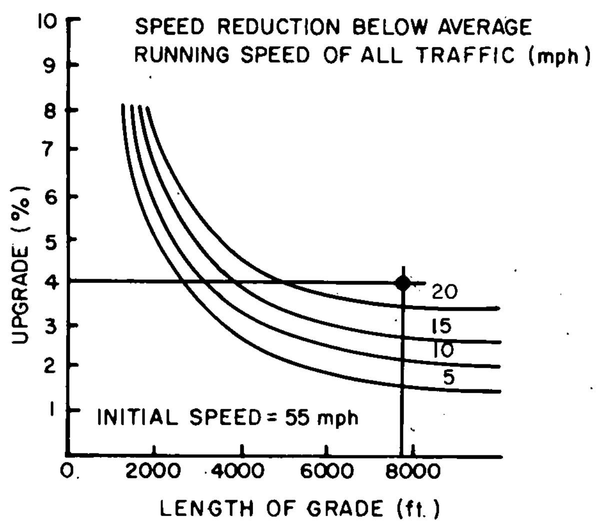
$$E_{HV} = 1 + (0.25 + 0.71)(3.8 - 1) = 3.69$$

$$f_{HV} = 1/[1 + 0.21(3.69 - 1)] = 0.64$$

$$SF_D = 2{,}800 \times 1.00 \times 0.87 \times 1.00 \times 0.96 \times 0.64 = 1{,}497 \text{ vph}$$

The actual flow rate is the DHV divided by the PHF, or 400/0.85 = 471 vph. As this is clearly *less* than the service flow rate for LOS D, the existing LOS is *not* E, and this condition is *not* met.

The next condition to investigate is whether a 10-mph speed reduction of heavy trucks would exist on the grade described. Based on the assumption that the typical truck on this grade has a weight/horsepower ratio of 200 lb/hp, Figure 8-2 is used to estimate the speed reduction experienced as shown below:



It can be seen that the speed reduction will be well in excess of 20 mph, which is greater than 10 mph, fulfilling the last required condition for justifying a climbing lane. Note that because only one of the conditions in item 3 needs to be satisfied, it is not necessary to investigate the third condition.

It can be concluded that a climbing lane is justified on the basis of the stated criteria.

CALCULATION 6—PLANNING APPLICATION 1

1. *Description*—A rural two-lane highway in mountainous terrain is located in an area where the design hour factor, *K*, is 0.14. What is the maximum AADT which can be accommodated without the LOS falling below D during the peak 15-min flow period?

2. *Solution*—The solution is simply found by entering Table 8-10 with mountainous terrain, LOS D, and a *K*-factor of 0.14. The maximum permissible AADT is found to be *2,700 vpd*.

CALCULATION 7—PLANNING APPLICATION 2

1. *Description*—A rural two-lane highway is located in rolling terrain in an area where the design hour factor, *K*, is 0.12. Its current AADT is 5,000 vpd. What is the likely LOS during the peak 15 min of flow?

2. *Solution*—Again, the solution is straightforward using Table 8-10. The maximum AADT's for the various levels of service are found for rolling terrain and a *K*-factor of 0.12. The 5,000 AADT is seen to fall between the maximum values for LOS C (4,400 vpd) and LOS D (6,600 vpd). The LOS is therefore expected to be D during the peak 15 min of flow.

CALCULATION 8—PLANNING APPLICATION 3

1. *Description*—A two-lane highway carrying an AADT of 6,600 vpd is located in level terrain in an area where the design hour factor, *K*, is 0.12. The area has a traffic growth rate of 5 percent per year. The responsible highway agency's policy is to expand two-lane highways to four lanes before the level of service becomes E during peak periods. In how many years will expansion of the facility have to be completed under this policy? If it will take 7 years to construct a four-lane highway, how long will it be before the construction project should begin?

2. *Solution*—The policy requires that expansion of the highway be completed before the AADT exceeds the maximum allowable value for LOS D. From Table 8-10, the maximum AADT for LOS D, for level terrain and a *K*-factor of 0.12, is 11,200 vpd.

The question now becomes: How many years will it take an AADT of 6,600 vpd to grow to 11,200 vpd at a rate of 5 percent per year? Therefore:

$$11{,}200 = 6{,}600(1 + 0.05)^n$$
$$n = 10.9 \text{ years}$$

Construction should begin in 10.9 − 7 years, or in 3.9 years.

VI. REFERENCES

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3. KRUMMINS, I., "Capacity and Level of Service of Two-Lane Rural Highways in Alberta." Thesis, University of Calgary, Calgary, Alberta, Canada (Sept. 1981).
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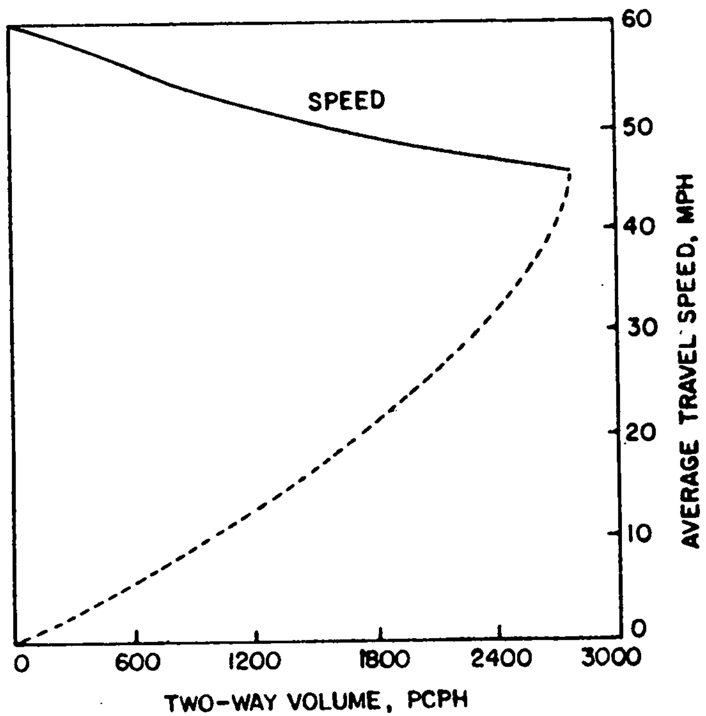
8. ROONEY, F., *Turnouts: Traffic Operational Report No. 2.* Office of Traffic, California Department of Transportation, Sacramento, Calif. (1976).
9. *Theoretical Analysis: Slow Moving Vehicle Turnouts.* Oregon Department of Transportation (1978).
10. ST. JOHN, A.D. AND KOBETT, D.R., "Grade Effects on Traffic Flow Stability and Capacity." *NCHRP Report 185* (1978) 110 pp.
11. *A Policy on Geometric Design of Highways and Streets.* American Association of State Highway and Transportation Officials, Washington, D.C. (1984).

APPENDIX I

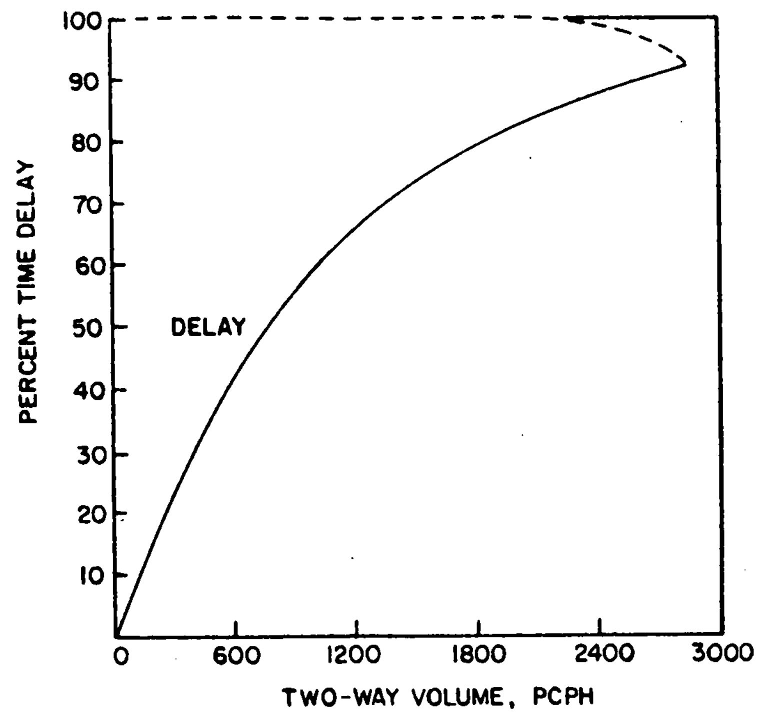
FIGURES AND WORKSHEETS FOR USE IN ANALYSIS OF TWO-LANE HIGHWAYS

FIGURES *PAGE*

WORKSHEETS



a. Relationship between average speed and flow on two-lane highways.



b. Relationship between percent time delay and flow on two-lane highways.

Figure 8-1. Speed-flow and percent time delay-flow relationships for two-lane rural highways (ideal conditions).

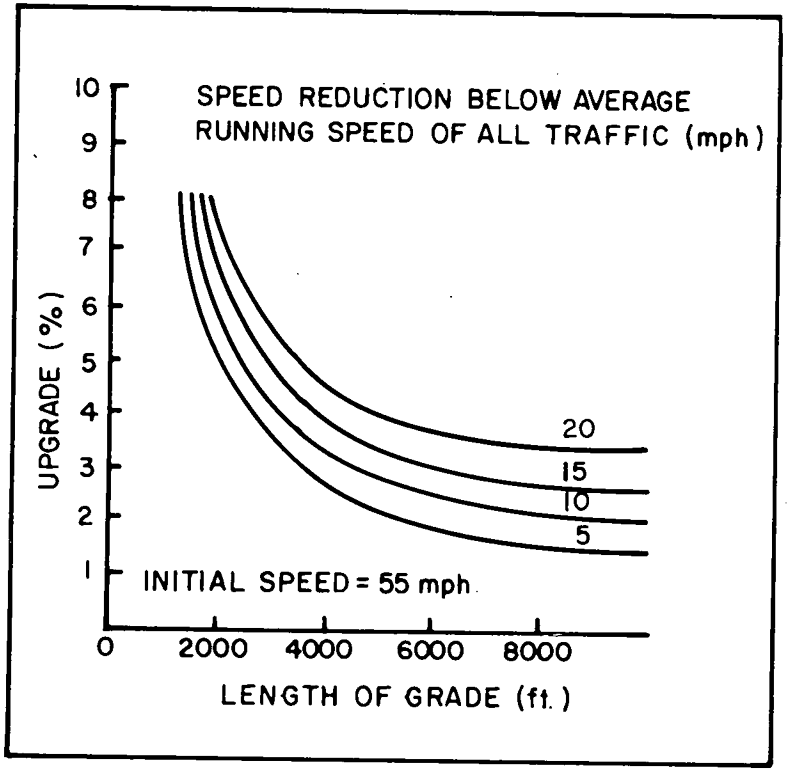


Figure 8-2. Speed reduction curve for a 200-lb/hp truck.

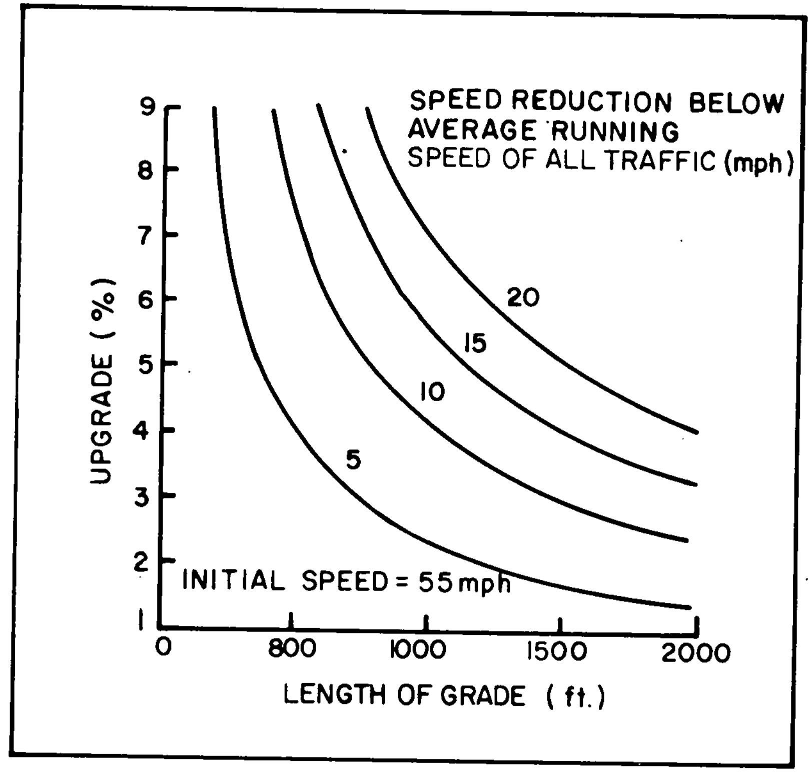


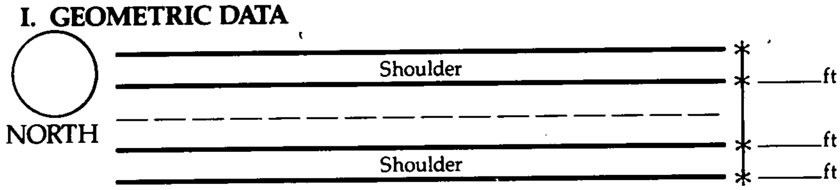
Figure 8-3. Speed reduction curve for a 300-lb/hp truck.

WORKSHEET FOR GENERAL TERRAIN SEGMENTS

Site Identification:__________ Date:__________ Time:__________

Name:__________ Checked by:__________

I. GEOMETRIC DATA



Design Speed:__________mph
% No Passing:__________%
Terrain (L,R,M):__________
Segment Length:__________mi

II. TRAFFIC DATA

Total Volume, Both Dir.__________vph
Flow Rate = Volume ÷ PHF
_____ = _____ ÷ _____

Directional Distribution:__________
Traffic Composition:_____%T, _____%RV, _____%B
PHF:__________

III. LEVEL OF SERVICE ANALYSIS

$SF_i = 2{,}800 \times (v/c)_i \times f_d \times f_w \times f_{HV}$

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

LOS	SF	= 2,800	× (v/c) Table 8-1	× f_d Table 8-4	× f_w Table 8-5	× f_{HV}	P_T	E_T Table 8-6	P_R	E_R Table 8-6	P_B	E_B Table 8-6
A		2,800										
B		2,800										
C		2,800										
D		2,800										
E		2,800										

IV. COMMENTS

Flow Rate__________vph LOS = __________

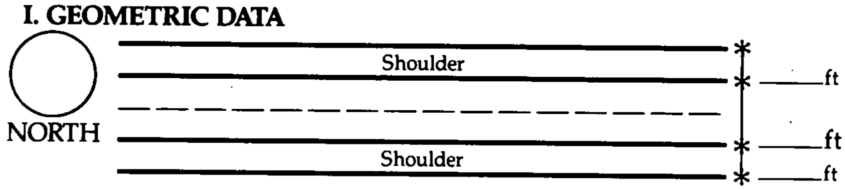
WORKSHEET FOR SPECIFIC GRADES

Page 1

Site Identification: ______________________ Date: ______________ Time: ______________

Name: ______________________ Checked by: ______________________

I. GEOMETRIC DATA



Design Speed: ______________ mph
Grade: ______________ %, ________ mi
% No Passing Zones: ______________

II. TRAFFIC DATA

Total Volume, Both Dir. ______________ vph
Flow Rate = Volume ÷ PHF
________ = ______ ÷ ______

Directional Distribution: ______________
Traffic Composition: ______ %T, ______ %RV, ______ %B
PHF: ______________

III. SOLVING FOR ADJUSTMENT FACTORS f_g AND f_{HV}

$f_g = 1 / [1 + P_p I_p]$
$I_p = 0.02 (E - E_o)$

$f_{HV} = 1 / [1 + P_{HV} (E_{HV} - 1)]$
$E_{HV} = 1 + (0.25 + P_{T/HV}) (E - 1)$

Speed (mph)	P_p	I_p	E Table 8-9	E_o Table 8-9	f_g	P_{HV}	E_{HV}	$P_{T/HV}$ (P_T/P_{HV})	E Table 8-9	f_{HV}
55										
52.5										
50										
45										
40										
30										

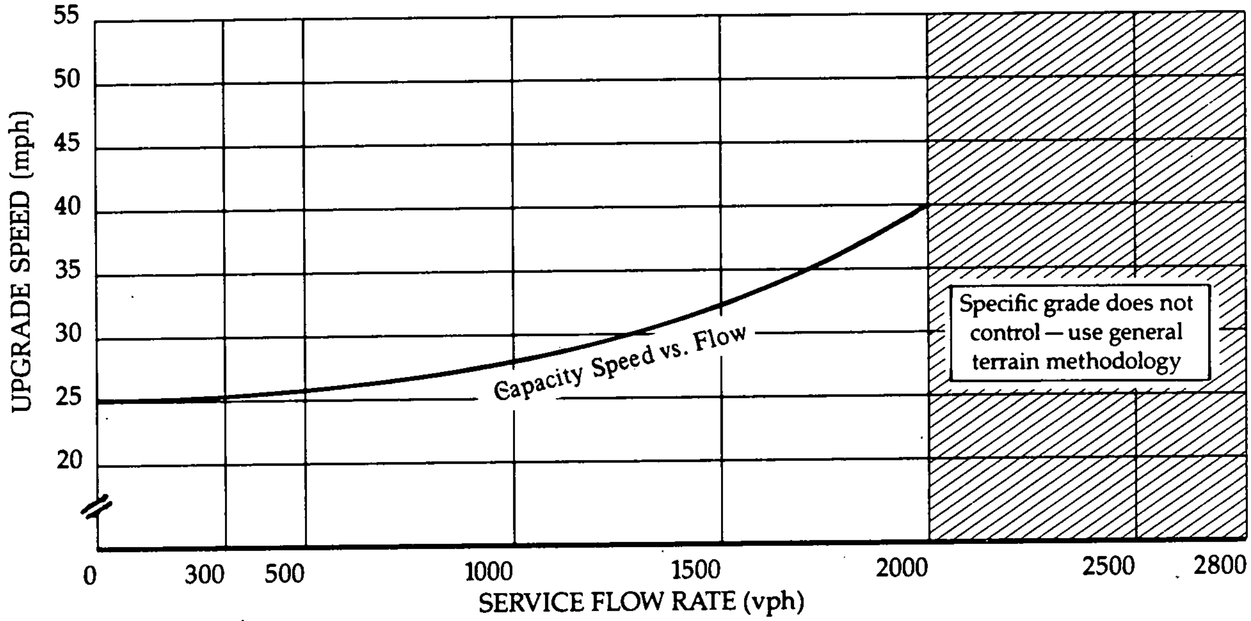
IV. SOLVING FOR SERVICE FLOW RATE

Speed (mph)	SF	2,800	× v/c Table 8-7	× f_d Table 8-8	× f_w Table 8-5	× f_g	× f_{HV}
55 (LOS A)		2,800					
52.5		2,800					
50 (LOS B)		2,800					
45 (LOS C)		2,800					
40 (LOS D)		2,800					
30		2,800					

WORKSHEET FOR SPECIFIC GRADES

Page 2

V. PLOT SF vs Speed



Intersection of Capacity Speed vs Flow curve with Service Flow Rate vs Speed curve defines Capacity, SF_E, and Speed at Capacity, S_c

VI. LEVEL OF SERVICE ANALYSIS

LOS	SF (from Worksheet)	Actual Flow Rate	Comments:
A			
B			
C			
D		Level of Service	
E			

SIGNALIZED INTERSECTIONS

CONTENTS

I. INTRODUCTION

This chapter contains procedures for the analysis of signalized intersection capacity and level of service. The signalized intersection is one of the most complex locations in a traffic system. Signalized intersection analysis must consider a wide variety of prevailing conditions, including the amount and distribution of traffic movements, traffic composition, geometric characteristics, and the details of intersection signalization. The methodology of this chapter focuses on the determination of level of service for known or projected prevailing conditions, but presents computational alternatives for determining other variables using an assumed or desired level of service.

In other chapters of this manual, the capacity of a highway is related primarily to the geometric characteristics of the facility, as well as to the composition of the traffic stream on the facility. Geometrics are a fixed, or nonvarying, characteristic of a facility. Thus, allowing for some variation in traffic composition over time, the capacity of a facility is generally a stable value which can be significantly improved only by initiating geometric improvements.

At the signalized intersection, an additional element is introduced into the concept of capacity: time allocation. A traffic signal essentially allocates time among conflicting traffic movements seeking use of the same physical space. The way in which time is allocated has a significant impact on the operation of the intersection and on the capacity of the intersection and its approaches.

The methodology presented herein addresses the capacity and level of service of intersection approaches, and the level of service of the intersection as a whole. Capacity is evaluated in terms of the ratio of demand flow rate to capacity (v/c ratio), while level of service is evaluated on the basis of average stopped delay per vehicle (sec/veh). The capacity of the intersection as a whole is not addressed, because the design and signalization of intersections focuses on the accommodation of major movements and approaches comprising the intersection. Capacity is, therefore, only meaningful as applied to these major movements and approaches.

TRAFFIC SIGNALS

Modern traffic signals allocate time in a variety of ways, from the most simple two-phase pretimed mode to the most complex multiphase actuated mode. This section describes the basic terminology of traffic signals, and briefly describes the various types of signal operation and their impact on capacity.

The following terms are commonly used to describe traffic signal operation:

Cycle—Any complete sequence of signal indications.

Cycle length—The total time for the signal to complete one cycle, stated in seconds, and given the symbol C.

Phase—The part of a cycle allocated to any combination of traffic movements receiving the right-of-way simultaneously during one or more intervals.

Interval—A period of time during which all signal indications remain constant.

Change interval—The "yellow" plus "all red" intervals that occur between phases to provide for clearance of the intersection before conflicting movements are released; stated in seconds, and given the symbol Y.

Green time—The time within a given phase during which the "green" indication is shown; stated in seconds, and given the symbol G_i (for phase i).

Lost time—Time during which the intersection is not effectively used by any movement; these times occur during the change interval (when the intersection is cleared), and at the beginning of each phase as the first few cars in a standing queue experience start-up delays.

Effective green time—The time during a given phase that is effectively available to the permitted movements; this is generally taken to be the green time plus the change interval minus the lost time for the designated phase; stated in seconds, and given the symbol g_i (for phase i).

Green ratio—The ratio of effective green time to the cycle length; given the symbol g_i/C (for phase i).

Effective red—The time during which a given movement or set of movements is effectively not permitted to move; stated in seconds, it is the cycle length minus the effective green time for a specified phase; given the symbol r_i.

Traffic signals may operate in three basic modes, depending on the type of control equipment used:

1. *Pretimed operation*—In pretimed operation, the cycle length, phases, green times, and change intervals are all preset. The signal rotates through this defined cycle in a constant fashion: each cycle is the same, with the cycle length and phases constant. Depending on the equipment available, several preset timing patterns may be used, each being implemented automatically at fixed times of the day.

2. *Semiactuated operation*—In semiactuated operation, the designated main street has a "green" indication at all times until detectors on the side street determine that a vehicle or vehicles have arrived on one or both of the minor approaches. The signal then provides a "green" phase for the side street, after an appropriate change interval, which is retained until all vehicles are served, or until a preset maximum side-street green is reached. In progressive signal systems, the initiation of side-street green phases can be limited to prespecified times within the cycle.

In this type of operation, the cycle length and green times may vary from cycle to cycle in response to demand. Because the green is always on the main street unless needed by side-street vehicles, side-street green times are virtually fully used, while all "excess" green time is allocated to the main street.

3. *Full-actuated operation*—In full-actuated operation, all signal phases are controlled by detector actuations. In general, minimum and maximum green times are specified for each phase, as is the phase sequence. In this form of control, cycle lengths and green times may vary considerably in response to demand. Certain phases in the cycle may be optional, and may be skipped entirely if no demand is sensed by detectors.

Many signal systems are now controlled by computers. Where such computer systems are used, the individual intersections generally operate under pretimed control, with the phasing plan and signal coordination being selected and controlled by the computer. In such systems, the computer serves as a master or supervisory controller.

It is not only the allocation of green time that has a significant impact on capacity and operations at a signalized intersection, but the manner in which turning movements are accommodated within the phase sequence as well. Signal phasing can provide for either *protected* or *permitted* turning movements.

A *permitted* turning movement is made through a conflicting pedestrian or opposing vehicle flow. Thus, a left-turn movement that is made at the same time as the opposing through movement is considered to be "permitted," as is a right-turn movement made at the same time as pedestrian crossings in a conflicting crosswalk.

Protected turns are those made without these conflicts, such as turns made during an exclusive left-turn phase or a right-turn phase during which conflicting pedestrian movements are prohibited.

Permitted turns experience the friction of selecting and passing through gaps in a conflicting vehicle or pedestrian flow. Thus, a single permitted turn often consumes more of the available green time than a single protected turn. Either permitted or protected turning phases may be more "efficient" in a given situation, depending on the turning and opposing volumes, intersection geometry, and other factors.

The preceding discussion emphasizes this primary concept: the capacity of an intersection is highly dependent on the signalization present. Given the range of potential signal control schemes, this capacity is far more variable than for other types of facilities, where capacity is mainly dependent on the physical geometry of the roadway. In effect, signalization, which can be changed frequently and quickly, allows considerable latitude in the "management" of the physical capacity of the intersection space and geometry. Thus, the concept of "capacity" is somewhat different from that discussed in previous chapters.

The capacity analysis procedures of this chapter are based on known or projected signalization plans. Appendixes are provided to assist the analyst in establishing signalization plans. State and local policies or methods should also be consulted in making such determinations. The appendixes herein are provided to assist in capacity analysis, and *should not* be construed to suggest nationally accepted standards, criteria, or guidelines for signalization.

CAPACITY AND LEVEL OF SERVICE

The concepts of capacity and level of service are central to the analysis of intersections, as they are for all types of facilities. In intersection analysis, however, the two concepts are not as strongly correlated as they are for other facility types. In previous chapters, the same analysis results yielded a determination of both the capacity and level of service of the facility. For signalized intersections, the two are analyzed separately, and are not simply related to each other. It is critical to note at the outset, however, that both *capacity* and *level of service* must be fully considered to evaluate the overall operation of a signalized intersection.

Capacity analysis of intersections results in the computation of v/c ratios for individual movements and a composite v/c ratio for the sum of critical movements or lane groups within the intersection. The v/c ratio is the actual or projected rate of flow on an approach or designated group of lanes during a peak 15-min interval divided by the capacity of the approach or designated group of lanes. Level of service is based on the average stopped delay per vehicle for various movements within the intersection. While v/c affects delay, there are other parameters that more strongly affect it, such as the quality of progression, length of green phases, cycle lengths, and others. Thus, for any given v/c ratio, a range of delay values may result, and vice-versa. For this reason, both the capacity and level of service of the intersection must be carefully examined. These two concepts are discussed in detail in the following sections.

Capacity of Signalized Intersections

Capacity at intersections is defined for each approach. Intersection approach capacity is the maximum rate of flow (for the subject approach) which may pass through the intersection under prevailing traffic, roadway, and signalization conditions. The rate of flow is generally measured or projected for a 15-min period, and capacity is stated in vehicles per hour.

Traffic conditions include volumes on each approach, the distribution of vehicles by movement (left, through, right), the vehicle type distribution within each movement, the location of and use of bus stops within the intersection area, pedestrian crossing flows, and parking movements within the intersection area.

Roadway conditions include the basic geometrics of the intersection, including the number and width of lanes, grades, and lane-use allocations (including parking lanes).

Signalization conditions include a full definition of the signal phasing, timing, type of control, and an evaluation of signal progression on each approach.

The capacity of designated lanes or groups of lanes within an approach may also be evaluated and determined using the procedures of this chapter. This may be done to isolate lanes serving a particular movement or movements, such as an exclusive right- or left-turn lane. Lanes so designated for separate analysis are referred to as "lane groups." The procedure herein contains guidelines for when and how separate lanes groups should be designated in an approach.

Capacity at signalized intersections is based on the concept of saturation flow and saturation flow rates. *Saturation flow rate* is defined as the maximum rate of flow that can pass through a given intersection approach or lane group under prevailing traffic and roadway conditions, assuming that the approach or lane group had 100 percent of real time available as effective green time. Saturation flow rate is given the symbol s, and is expressed in units of vehicles per hour of effective green time (vphg).

The *flow ratio* for a given approach or lane group is defined as the ratio of the actual flow rate for the approach or lane group, v, to the saturation flow rate. The flow ratio is given the symbol, $(v/s)_i$, for approach or lane group i.

The *capacity* of a given lane group or approach may be stated as:

$$c_i = s_i \times (g/C)_i \qquad (9\text{-}1)$$

where:

c_i = capacity of lane group or approach i, in vph;
s_i = saturation flow rate for lane group or approach i, in vphg; and
$(g/C)_i$ = green ratio for lane group or approach i.

The ratio of flow rate to capacity, v/c, is given the symbol X in intersection analysis. This new symbol is introduced in this chapter to emphasize the strong relationship of capacity to signalization conditions, and for consistency with the literature, which also refers to this variable as the "degree of saturation."

For a given lane group or approach i:

$$X_i = (v/c)_i = v_i/[s_i \times (g/C)_i] \qquad (9\text{-}2)$$
$$X_i = v_iC/s_ig_i = (v/s)_i/(g/C)_i$$

where:

X_i = v/c ratio for lane group or approach i;
v_i = actual flow rate for lane group or approach i, in vph;
s_i = saturation flow rate for lane group or approach i, in vphg; and
g_i = effective green time for lane group i or approach i, in sec.

Values of X_i range from 1.00 when the flow rate equals capacity to 0.00 when the flow rate is zero.

The capacity of the full intersection is not a significant concept and is not specifically defined herein. Rarely do all movements at an intersection become saturated at the same time of day. It is the ability of individual movements to move through the intersection with some efficiency which is the critical concern.

Another capacity concept of utility in the analysis of signalized intersections is, however, the *critical v/c ratio, X_c*. This is a v/c ratio for the intersection as a whole, considering only the lane groups or approaches that have the highest flow ratio, v/s, for a given signal phase.

For example, in a two-phase signal, opposing approaches move during the same green time. Generally, one of these two approaches will require more green time than the other (i.e., it will have a higher flow ratio). This would be the "critical" approach for the subject signal phase. Each signal phase will have a critical lane group or approach that determines the green time requirements for the phase. Where signal phases overlap, the identification of these critical lane groups or approaches is somewhat complex, and is discussed in the "Methodology" section of this chapter.

The critical v/c ratio for the intersection is defined in terms of critical lane groups or approaches:

$$X_c = \sum_i (v/s)_{ci} \times [C/(C-L)] \qquad (9\text{-}3)$$

where:

X_c = critical v/c ratio for the intersection;
$\sum_i (v/s)_{ci}$ = the summation of flow ratios for all critical lane groups or approaches, i;
C = cycle length, in sec; and
L = total lost time per cycle; computed as the sum of "start-up" and change interval lost time minus the portion of the change interval used by vehicles for each critical signal phase.

This equation is useful in evaluating the overall intersection with respect to the geometrics and total cycle length provided, and is also useful in estimating signal timings where they are not known or specified by local policies or procedures. It gives the v/c ratio for all critical movements, assuming that green time has been appropriately or proportionally allocated. It is therefore possible to have a critical v/c ratio of less than 1.00, and still have individual movements oversaturated within the signal cycle. A critical v/c ratio less than 1.00, however, does indicate that all movements in the intersection can be accommodated within the defined cycle length and phase sequence by proportionally allocating green time. In essence, the total available green time in the phase sequence is adequate to handle all movements if properly allocated.

The analysis of capacity in this chapter focuses on the computation of saturation flow rates, v/c ratios, and capacities for various approaches or lane groups of the intersection. Procedures for these computations are described in greater detail in the "Methodology" and "Procedures for Application" sections of this chapter.

Level of Service for Signalized Intersections

Level of service for signalized intersections is defined in terms of *delay*. Delay is a measure of driver discomfort, frustration, fuel consumption, and lost travel time. Specifically, level-of-service criteria are stated in terms of the average stopped delay per vehicle for a 15-min analysis period. The criteria are given in Table 9-1.

Delay may be measured in the field, or may be estimated using procedures presented later in this chapter. Delay is a complex measure, and is dependent on a number of variables, including the quality of progression, the cycle length, the green ratio, and the v/c ratio for the lane group or approach in question.

Level-of-service A describes operations with very low delay, i.e., less than 5.0 sec per vehicle. This occurs when progression is extremely favorable, and most vehicles arrive during the green phase. Most vehicles do not stop at all. Short cycle lengths may also contribute to low delay.

TABLE 9-1. LEVEL-OF-SERVICE CRITERIA FOR SIGNALIZED INTERSECTIONS

LEVEL OF SERVICE	STOPPED DELAY PER VEHICLE (SEC)
A	≤ 5.0
B	5.1 to 15.0
C	15.1 to 25.0
D	25.1 to 40.0
E	40.1 to 60.0
F	> 60.0

Level-of-service B describes operations with delay in the range of 5.1 to 15.0 sec per vehicle. This generally occurs with good progression and/or short cycle lengths. More vehicles stop than for LOS A, causing higher levels of average delay.

Level-of-service C describes operations with delay in the range of 15.1 to 25.0 sec per vehicle. These higher delays may result from fair progression and/or longer cycle lengths. Individual cycle failures may begin to appear in this level. The number of vehicles stopping is significant at this level, although many still pass through the intersection without stopping.

Level-of-service D describes operations with delay in the range of 25.1 to 40.0 sec per vehicle. At level D, the influence of congestion becomes more noticeable. Longer delays may result from some combination of unfavorable progression, long cycle lengths, or high v/c ratios. Many vehicles stop, and the proportion of vehicles not stopping declines. Individual cycle failures are noticeable.

Level-of-service E describes operations with delay in the range of 40.1 to 60.0 sec per vehicle. This is considered to be the limit of acceptable delay. These high delay values generally indicate poor progression, long cycle lengths, and high v/c ratios. Individual cycle failures are frequent occurrences.

Level-of-service F describes operations with delay in excess of 60.0 sec per vehicle. This is considered to be unacceptable to most drivers. This condition often occurs with oversaturation, i.e., when arrival flow rates exceed the capacity of the intersection. It may also occur at high v/c ratios below 1.00 with many individual cycle failures. Poor progression and long cycle lengths may also be major contributing causes to such delay levels.

Relating Capacity and Level of Service

Because delay is a complex measure, its relationship to capacity is also complex. The levels of service of Table 9-1 have been established based on the acceptability of various delays to drivers. It is important to note that this concept is not related to capacity in a simple one-to-one fashion.

In previous chapters, the lower bound of LOS E has always been defined to be capacity, i.e., the v/c ratio is, by definition, 1.00. This *is not the case* for the procedures of this chapter. It is possible, for example, to have delays in the range of LOS F (unacceptable) while the v/c ratio is below 1.00, perhaps as low as 0.75–0.85. Very high delays can occur at such v/c ratios when some combination of the following conditions exists: (1) the cycle length is long, (2) the lane group in question is disadvantaged (has a long red time) by the signal timing, and/or (3) the signal progression for the subject movements is poor.

The reverse is also possible: a saturated approach or lane group (i.e., v/c ratio = 1.00) may have low delays if: (1) the cycle length is short, and/or (2) the signal progression is favorable for the subject movement. Thus, the designation of LOS F *does not* automatically imply that the intersection, approach, or lane group is overloaded, nor does a level of service in the A to E range automatically imply that there is unused capacity available.

The procedures and methods of this chapter require the analysis of both capacity and level-of-service conditions to fully evaluate the operation of a signalized intersection. It is imperative that the analyst recognize the unique relationship of these two concepts as they apply to signalized intersections.

LEVELS OF ANALYSIS

This chapter presents two levels of analysis for use. The primary methodology used is *operational analysis.* At this level, detailed information on all prevailing traffic, roadway, and signalization conditions must be provided. The method provides for a full analysis of capacity and level of service, and can be used to evaluate alternative geometric designs and/or signal plans.

A second method is provided for *planning analysis.* At this level, only capacity is addressed, because the detailed information needed to estimate delay is not available. Information on intersection geometrics and turning movements is required, but the details of signalization and vehicle type distributions are not needed. The method provides broad results that allow a projection of whether or not the intersection is likely to be oversaturated. Inasmuch as delay estimates cannot be made in planning analysis, level of service cannot be addressed at this level.

Operational analysis would be used in most analyses of existing intersections or of future situations in which traffic, geometric, and control parameters were well established by projections and trial designs. The planning procedure is useful in testing general design alternatives for new intersections in areas of new development, where details of signalization and demand characteristics are not yet under consideration.

The operational analysis methodology provided considers the full details of each of four components: demand or service flow rates at the intersection, signalization of the intersection, geometric design or characteristics of the intersection, and the delay or level of service that results from these. The methodology is capable of treating any of these four as an "unknown," to be determined knowing the details of the other three. Thus the method can be used to:

1. Solve for *level of service,* knowing details of intersection flows, signalization, and geometrics.
2. Solve for allowable *service flow rates* for selected levels of service, knowing the details of signalization and geometrics.
3. Solve for *signal timing* (for an assumed phase plan), knowing the desired level of service and the details of flows and geometrics.
4. Solve for *basic geometrics* (number or allocation of lanes), knowing the desired level of service and the details of flows and signalization.

While the methodology is capable of computations in all four modes, specific procedures and worksheets herein are designed for the first of these, i.e., a solution for level of service. In developing alternative signal and geometric designs, it is often necessary to consider simultaneous changes in both. Rarely can signalization be considered in isolation from geometric design and vice-versa. Thus, the most frequent type of analysis would consider such alternatives on a trial-an-error basis, and would not attempt to hold one constant and "solve" for the other. Sample calculations, however, illustrate alternative uses of the methodology.

II. METHODOLOGY

OPERATIONAL ANALYSIS

Operational analysis results in the determination of capacity and level of service for each lane group or approach, as well as the level of service for the intersection as a whole. It requires that detailed information be provided concerning geometric, traffic, and signalization conditions at the intersection. These may be known for existing cases or projected for future situations.

Because the operational analysis of signalized intersections is complex, it is divided into five distinct modules, as follows:

1. *Input module*—This analysis module focuses on the definition of all required information on which subsequent computations are based. It includes all necessary data on intersection geometry, traffic volumes and conditions, and signalization. It is used to provide a convenient summary for the remainder of the analysis.
2. *Volume adjustment module*—Demand volumes are generally stated in terms of vehicles per hour for a peak hour. The volume adjustment module converts these to flow rates for a peak 15-min analysis period, and accounts for the effects of lane distribution. The definition of lane groups for analysis also takes place in this module.
3. *Saturation flow rate module*—This module is used to compute the saturation flow rate for each of the lane groups established for analysis. It is based on the adjustment of an "ideal" saturation flow rate to reflect a variety of prevailing conditions.
4. *Capacity analysis module*—In this module, volumes and saturation flow rates are manipulated to compute the capacity and v/c ratios for each lane group and the critical v/c ratio for the intersection.
5. *Level-of-service module*—Delay is estimated for each lane group established for analysis. Delay measures are aggregated for approaches and for the intersection as a whole, and levels of service are determined.

Figure 9-1 provides a diagrammatic illustration of the modules, and of the analysis procedure. Each of these modules is discussed in detail in the sections that follow.

Input Module

Figure 9-2 provides a summary of the input information required to conduct an operational analysis. The data needed are detailed and varied, and fall into three main categories:

1. *Geometric conditions*—Intersection geometry is generally

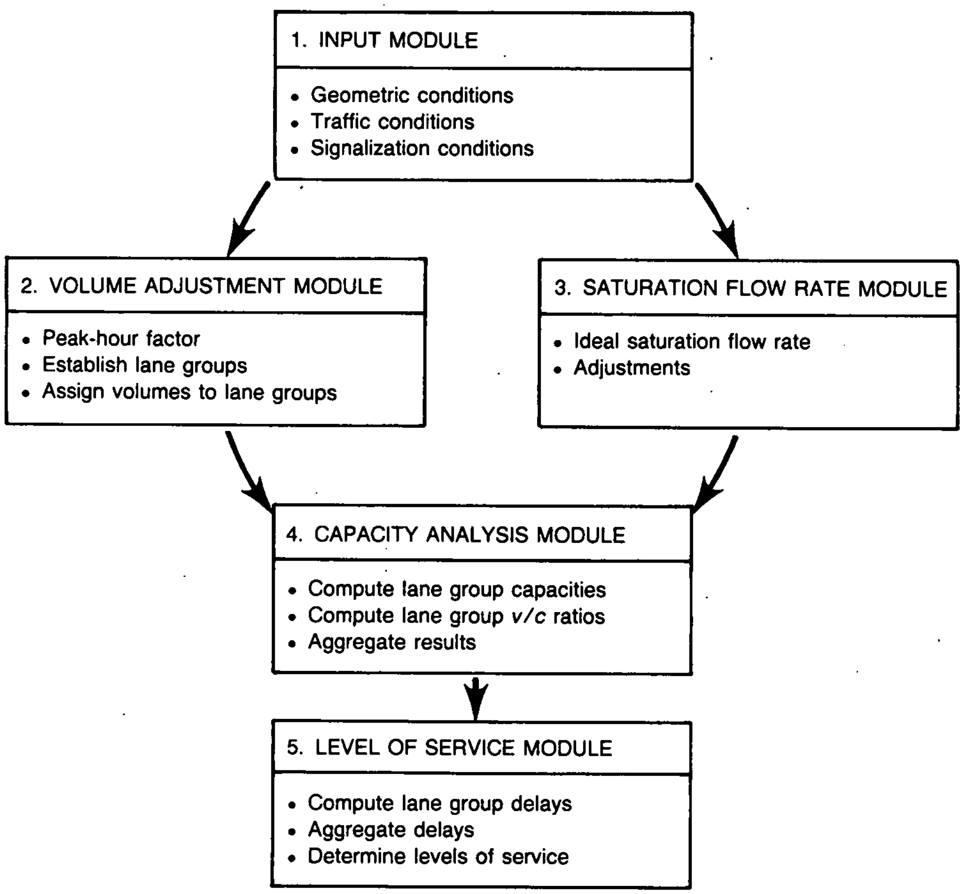


Figure 9-1. Operational analysis procedure.

presented in diagrammatic form, and must include all of the relevant information, including approach grades, the number and width of lanes, and parking conditions. The existence of exclusive left- or right-turn lanes should be noted, along with the storage lengths of such lanes.

Where the specifics of geometry are to be designed, these features must be assumed for the analysis to continue. State or local policies and guidelines should be used in establishing the trial design. Where these are not readily available, Appendix I of this chapter contains suggestions for geometric design that may be useful in preparing a preliminary design for analysis.

2. *Traffic conditions*—Traffic volumes for the intersection must be specified for each movement on each approach. Vehicle type distribution is quantified as the percent of heavy vehicles (%HV) in each movement, where all vehicles with more than four wheels touching the pavement are considered to be "heavy vehicles." The number of local buses on each approach should also be identified. Only those buses making stops to pick up or discharge passengers at the intersection (either on the approach or departure side of the intersection) are included in this number. Buses not making such stops are considered to be heavy vehicles.

Pedestrian flows are needed, as these will interfere with permitted right-turn and left-turn movements. The pedestrian flow for a given vehicular approach is the flow in the crosswalk interfering with right turns from the approach. Thus, for a westbound approach, the pedestrian flow in the north crosswalk would be used. For an eastbound approach, the south crosswalk flow is used; for a northbound approach, the east crosswalk flow is used; and for a southbound approach, the west crosswalk flow is used.

One of the most critical traffic characteristics is the designation of "arrival type" on each approach. This is a general categorization that attempts to approximately quantify the quality of progression on the approach. Five arrival types are defined for the dominant arrival flow as follows:

- Type 1—This condition is defined as a dense platoon arriving at the intersection at the beginning of the red phase. This is the worst platoon condition.
- Type 2—This condition may be a dense platoon arriving during the middle of the red phase or a dispersed platoon arriving throughout the red phase. Better than Type 1, this is still an unfavorable platoon condition.
- Type 3—This condition represents totally random arrivals. This occurs when arrivals are widely dispersed throughout the red and green phases, and/or where the approach is totally uncoordinated with other signals—either because it is at an isolated location or because nearby signals operate on different cycle lengths. This is an average condition.
- Type 4—This condition is defined as a dense platoon arriving during the middle of the green phase, or a dispersed platoon arriving throughout the green phase. This is a moderately favorable platoon condition.
- Type 5—This condition is defined as a dense platoon ar-

TYPE OF CONDITION	PARAMETER	SYMBOL
Geometric Conditions	Area Type	CBD or other
	Number of Lanes	N
	Lane Widths, ft	W
	Grades, %	+ (Upgrade) − (Downgrade)
	Existence of Exclusive LT or RT Lanes	
	Length of Storage Bay, LT or RT Lanes	L_s
	Parking Conditions	Y or N
Traffic Conditions	Volumes by Movement, vph	V_i
	Peak-Hour Factor	PHF
	Percent Heavy Vehicles	%HV
	Conflicting Pedestrian Flow Rate, peds/hr	*PEDS*
	Number of Local Buses Stopping in Intersection	N_B
	Parking Activity, pkg maneuvers/hr	N_m
	Arrival Type	
Signalization Conditions	Cycle Length, sec	C
	Green Times, sec	G_i
	Actuated vs Pretimed Operation	A or P
	Pedestrian Push-Button?	Y or N
	Minimum Pedestrian Green	G_p
	Phase Plan	

Figure 9-2. Input data needs for each analysis lane group.

riving at the beginning of the green phase. It is the most favorable platoon condition.

The arrival type is best observed in the field, but could be approximated by examining time-space diagrams for the arterial or street in question. The arrival type should be determined as accurately as possible, because it will have a significant impact on delay estimates and level-of-service determination. Although there are no definitive parameters to precisely quantify arrival type, the following ratio is a useful value:

$$R_p = PVG/PTG \tag{9-4}$$

where:

R_p = platoon ratio;
PVG = percentage of all vehicles in the movement arriving during the green phase; and
PTG = percentage of the cycle that is green for the movement; $PTG = (G/C) \times 100$.

PVG must be observed in the field, while PTG is computed from the signal timing. Table 9-2 gives approximate ranges of R_p related to arrival type.

Another traffic condition of interest is the activity in parking lanes adjacent to analysis lane groups. Parking activity is measured in terms of the number of parking maneuvers per hour within 250 ft of the intersection, N_m. Each vehicle entering or leaving a parking place is considered to be a parking maneuver.

3. *Signalization conditions*—Complete information regarding signalization is needed. This includes a phase diagram illustrating the phase plan, cycle length, green times, and change intervals. Actuated phases must be identified, including the existence of push-button pedestrian-actuated phases. Where pedestrian push-buttons do not exist, the minimum green time for the phase should be indicated and must be provided for in the signal timing. The minimum green time for a phase may be estimated as:

$$G_p = 7.0 + (W/4.0) - Y \tag{9-5}$$

where:

G_p = minimum green time, in sec;
W = distance from the curb to the center of the farthest travel lane on the street being crossed, or to the nearest pedestrian refuge island, in ft; and
Y = change interval (yellow + all red time), in sec.

It is assumed that the 15th percentile walking speed of pedestrians crossing a street is 4.0 fps in this computation. This is lower than the average walking speed of pedestrians of 4.5 fps cited in Chapter 13, "Pedestrians." The lower value is intended to accommodate crossing pedestrians who walk at speeds slower than the average. Where local policy uses different criteria for estimating minimum pedestrian crossing requirements, they should be used in lieu of Eq. 9-5.

Where signal phases are actuated, the cycle length and green times will vary from cycle-to-cycle in response to demand. To establish values for analysis, the operation of the signal should be observed in the field during the same period as volumes are observed. Average values of cycle length and green times may then be used.

TABLE 9-2. RELATIONSHIP BETWEEN ARRIVAL TYPE AND PLATOON RATIO

ARRIVAL TYPE	RANGE OF PLATOON RATIO, R_p
1	0.00 to 0.50
2	0.51 to 0.85
3	0.86 to 1.15
4	1.16 to 1.50
5	$\geq$ 1.51

Where signalization is to be established as part of the analysis, state or local policies and procedures should be applied where appropriate in designing the signalization for analysis. Appendix II contains suggestions for the design of a trial signalization that may also be useful. These should not be construed to be standards or criteria for signal design. It should be noted that a trial signalization cannot be designed until the "volume adjustment" and "saturation flow rate" modules have been completed. In some cases, the computations will be iterative, because left-turn adjustments for permitted turns used in the "saturation flow rate" module depend on signal timing. Appendix II also contains suggestions for estimating the timing of an actuated signal if field observations are unavailable.

4. *Default values*—Occasionally, some of the field data noted in Figure 9-2 will not be available. Where critical variables are missing, it may be necessary to conduct a planning analysis. However, default values may be used for some of the variables without seriously compromising computations. Caution should be used when applying such values, and it must be recognized that results become more approximate as more default values are used.

Table 9-3 presents default values for use where field data are not available.

The input module summarizes the information needed to conduct subsequent analysis. This information forms the basis for selecting computational values and procedures in the modules that follow.

TABLE 9-3. DEFAULT VALUES FOR USE IN OPERATIONAL ANALYSIS

PARAMETER	DEFAULT VALUE	
Conflicting Pedestrian Flow Rate, peds/hr	Low Ped. Flow	50 peds/hr
	Moderate Ped. Flow	200 peds/hr
	High Ped. Flow	400 peds/hr
Percent Heavy Vehicles, %HV	2%	
Peak-Hour Factor, PHF	0.90	
Grade	0%	
Number of Buses, N_B	0 buses/hr	
Number of Parking Maneuvers, N_m	20 maneuvers/hr (where parking exists)	
Arrival Type	3	

Volume Adjustment Module

Three major analytical steps are performed in the volume adjustment module: (1) movement volumes are adjusted to flow rates for a peak 15-min period of analysis, (2) lane groups for analysis are established, and (3) lane group flows are adjusted to account for unbalanced lane utilization.

1. *Adjustment of movement volumes to reflect peak flow rates*—As with other chapters and procedures of this manual, the initial computational process is to convert demands stated as hourly volumes to flow rates for the peak 15-min period within the hour. This is done by dividing the movement volumes by an appropriate peak-hour factor, PHF, which may be defined for the intersection as a whole, for each approach, or for each movement. Then:

$$v_p = V/\mathrm{PHF}$$

where:

v_p = flow rate during peak 15-min period, in vph;
V = hourly volume, in vph; and
PHF = peak-hour factor.

Because not all intersection movements may peak at the same time, it is valuable to observe 15-min flows directly, and select critical periods for analysis. The conversion of hourly volumes to peak flow rates using the PHF assumes that all movements peak during the same 15-min period, and is therefore a conservative approach.

2. *Determination of lane groups for analysis*—The operational analysis procedure is disaggregate, i.e., it is designed to consider individual intersection approaches and individual lane groups within approaches. It is therefore necessary to determine appropriate lane groups for analysis.

A "lane group" is defined as one or more lanes on an intersection approach serving one or more traffic movements. Segmenting the intersection into lane groups is generally a relatively obvious process that considers both the geometry of the intersection and the distribution of traffic movements. In general, the smallest number of lane groups is used which adequately describes the operation of the intersection. The following guidelines may be applied:

a. An exclusive left-turn lane or lanes should be designated as a separate lane group. The same is true of an exclusive right-turn lane.

b. On approaches with exclusive left-turn and/or right-turn lanes, all other lanes on the approach would generally be included in a single lane group.

c. Where an approach with more than one lane includes a lane that may be used by both left-turning vehicles and through vehicles, it is necessary to determine whether conditions permit equilibrium conditions to exist, or whether there are so many left-turns that the lane essentially acts as an exclusive left-turn lane.

A simple approach is used to make this determination. The left-turn flow rate is converted to an approximate equivalent flow of through vehicles:

$$v_{LE} = v_L \times \frac{1{,}800}{1{,}400 - v_o} \qquad (9\text{-}6)$$

where:

v_{LE} = approximate equivalent left-turn flow rate, in vph;
v_L = actual left-turn flow rate, in vph; and
v_o = total opposing flow rate, in vph.

Note that when v_o is equal to or greater than 1,400 vph, v_{LE} has no meaning. In such cases, left-turn movement against the opposing flow is not feasible, and inclusion of a protected LT phase in the signal cycle should be considered.

It is assumed that under the most extreme conditions, the equivalent left-turn flow, v_{LE}, completely occupies the left-most lane of the approach. Remaining flow is then assumed to use remaining lanes equally. If the equivalent flow rate in the left-most lane exceeds the average flow rate in remaining lanes, it is assumed that the lane acts as an exclusive left-turn lane, and a separate lane group is established. If the equivalent left-lane flow rate is less than the average flow rate in remaining lanes, it is assumed that through vehicles will share the left lane to establish equilibrium, and the entire approach is considered as a single lane group. Thus, if:

$$v_{LE} \geq (v - v_L)/(N - l)$$

where:

v_a = total flow rate on the approach, in vph; and
N = total number of lanes on the approach

then assume the left lane acts as an exclusive left-turn lane, and analyze the lane as a separate lane group. If:

$$v_{LE} < (v_a - v_L)/(N - l)$$

then assume that shared use of the left lane will take place. Include the lane as part of the total approach for analysis.

Where two or more lanes are included in a lane group for analysis purposes, all subsequent computations treat these lanes as a single entity. Figure 9-3 illustrates some common lane group schemes for analysis.

The operation of a shared left-turn and through lane with permitted left-turn phasing is quite complex. Left-turning vehicles execute their turning maneuvers through gaps in the opposing traffic stream. The first gap, however, does not appear until the queue of opposing vehicles clears the intersection. If a left-turner arrives during the interval in which the opposing queue is clearing, it effectively blocks the lane *for both through and turning vehicles* until the first gap appears. Thereafter, left-turn vehicles may move through gaps in the opposing traffic stream until the green phase terminates, at which time as many as two left-turning vehicles may be able to execute turns during the change interval. Any lane blockages or congestion in the shared lane will influence lane distribution, as vehicles move to adjacent lanes to avoid turbulence and delays. Another factor also influences lane distribution. If a through vehicle arrives at the intersection at the time a gap appears in the opposing traffic stream, no left-turning vehicle will be able to use it. A large number of through vehicles in the shared lane may block so

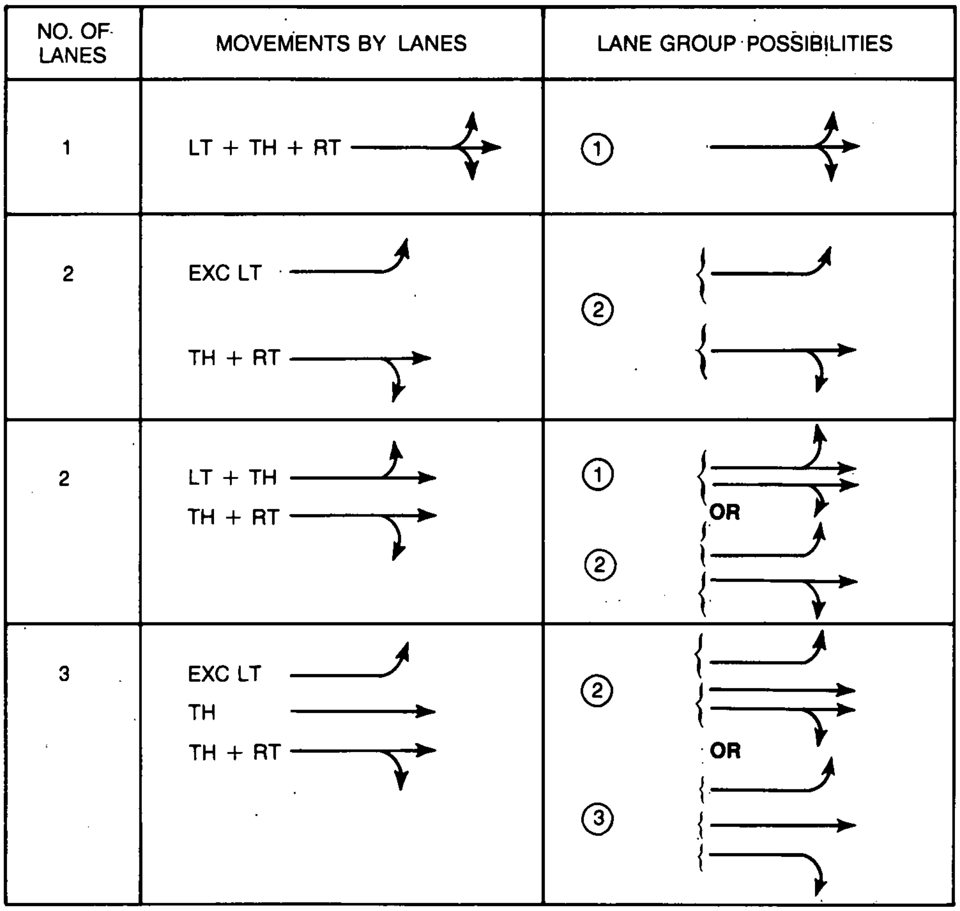


Figure 9-3. Typical lane groups for analysis.

many of the available gaps as to leave insufficient capacity for left-turning vehicles. The interaction of all these mechanisms results in vehicles establishing an equilibrium through their selection of lanes. The procedures of this chapter address this equilibrium state, and allow approaches containing shared left-turn and through lanes to be analyzed as a single lane group.

3. *Adjustment for lane distribution*—Movement volumes have been adjusted to peak 15-min flow rates, and lane groups for analysis have been established. Flow rates in each lane group are now adjusted to reflect unequal lane utilization. Where more than one lane exists, flow will not divide equally. The lane utilization adjustment reflects this, and increases the analysis flow rate to reflect the flow in the lane with the highest utilization. Thus:

$$v = v_g \times U \tag{9-7}$$

where:

v = adjusted demand flow rate for the lane group, in vph;
v_g = unadjusted demand flow rate for the lane group, in vph; and
U = lane utilization factor.

The lane utilization factor (Table 9-4) is only used when it is desirable to analyze the worst of two or more lanes in a lane group. Where average conditions for the lane group are desired, the factor is set at 1.00. The factor may also be set at 1.00 when the v/c ratio for the lane group approaches 1.0, as lanes tend to be more equally utilized in such situations. When used, the factor assumes that the most heavily used lane in a group of two serves 52.5 percent of the total flow, while the most heavily used lane in a group of three serves 36.7 percent of the total flow.

TABLE 9-4. LANE UTILIZATION FACTORS

NO. OF THROUGH LANES IN GROUP (EXCLUDING LANES USED BY LEFT-TURNING VEHICLES)	LANE UTILIZATION FACTOR, U
1	1.00
2	1.05
≥ 3	1.10

Saturation Flow Module

In this module, a saturation flow rate for each lane group is computed. The saturation flow rate is the flow in vehicles per hour which could be accommodated by the lane group assuming that the green phase was always available to the approach—i.e., that the green ratio, g/C, was 1.00. Computations begin with the selection of an "ideal" saturation flow rate, usually 1,800 passenger cars per hour of green time per lane (pcphgpl), and adjustment of this value for a variety of prevailing conditions that are not ideal.

$$s = s_o N f_w f_{HV} f_g f_p f_{bb} f_a f_{RT} f_{LT} \quad (9\text{-}8)$$

where:

s = saturation flow rate for the subject lane group, expressed as a total for all lanes in the lane group under prevailing conditions, in vphg;
s_o = ideal saturation flow rate per lane, usually 1,800 pcphgpl;
N = number of lanes in the lane group.
f_w = adjustment factor for lane width; 12-ft lanes are standard; given in Table 9-5;
f_{HV} = adjustment factor for heavy vehicles in the traffic stream, given in Table 9-6;
f_g = adjustment factor for approach grade, given in Table 9-7;
f_p = adjustment factor for the existence of a parking lane adjacent to the lane group and the parking activity in that lane, given in Table 9-8;
f_{bb} = adjustment factor for the blocking effect of local buses stopping within the intersection area, given in Table 9-9;
f_a = adjustment factor for area type, given in Table 9-10;
f_{RT} = adjustment factor for right turns in the lane group, given in Table 9-11; and
f_{LT} = adjustment factor for left turns in the lane group, given in Table 9-12, or computed as described in following sections.

Where detailed data defining each of the above factors is not available, a default value for s may be taken to be 1,600 vphgpl $\times N$. When this is done, however, the analysis becomes highly approximate. Appendix IV gives a procedure for measuring the prevailing saturation flow rate, s, directly.

1. *Adjustment factors*—The use of adjustment factors is similar to that of previous chapters. Each factor accounts for the impact of one or several prevailing conditions that are different from the ideal conditions for which the ideal saturation flow rate of 1,800 pcphgpl applies.

The *lane width factor, f_w,* accounts for the deleterious impact of narrow lanes on saturation flow rate, and allows for an increased flow on wide lanes. Twelve-foot lanes are the standard.

The effects of heavy vehicles and grades are treated by separate factors, f_{HV} and f_g respectively. Their separate treatment recognizes that passenger cars are affected by approach grades, as are heavy vehicles. The *heavy vehicle factor* accounts for the additional space occupied by these vehicles and for the differential in the operating capabilities of heavy vehicles with respect to passenger cars.

The *grade factor* accounts for the effect of grades on the operation of all vehicles.

The *parking factor, f_p,* accounts for the frictional effect of a parking lane on flow in adjacent lanes, as well as for the occasional blocking of an adjacent lane by vehicles moving into and out of parking spaces.

The *bus blockage factor, f_{bb},* accounts for the impacts of local transit buses stopping to discharge or pick up passengers at a near-side or far-side bus stop. Where local transit buses are believed to be a major factor in intersection performance, Chapter 12, "Transit," may be consulted for a more precise method of quantifying this effect.

The *area type factor, f_a,* accounts for the relative inefficiency of business area intersections in comparison to those in other locations. This is primarily due to the complexity and general congestion of the environment in business areas.

Turning factors depend on a number of parameters. The most important characteristic is the manner in which turns are accommodated in the intersection. Turns may operate out of exclusive or shared lanes, with protected or permitted signal phasing, or some combination of these conditions. The impact of turns on saturation flow rates is very much dependent on the mode of turning operations.

The *right-turn factor, f_{RT},* depends on a number of variables, including:

a. Whether right turns are made from an exclusive or shared lane.
b. Type of signal phasing (protected, permitted, or protected plus permitted); a protected right-turn phase has no conflicting pedestrian movements.
c. Volume of pedestrians using the conflicting crosswalk.
d. Proportion of right turns using a shared lane.
e. Proportion of right turns using the protected portion of a protected plus permitted phase.

Item e should be determined by field observation, but can be grossly estimated from the signal timing. This is done by assuming that the proportion of right-turning vehicles using the protected phase is approximately equal to the proportion of the turning phase which is protected.

Where right-turn-on-red (RTOR) is permitted, the right-turn volume may be reduced by the volume of right-turning vehicles moving on the red phase. This is generally done on the basis of hourly volumes, before converting to flow rates.

The *left-turn factor, f_{LT},* is based on similar variables, including:

a. Whether left turns are made from exclusive or shared lanes.
b. Type of phasing (protected, permitted, or protected plus permitted).
c. Proportion of left-turning vehicles using a shared lane.
d. Opposing flow rate when permitted left turns are made.

The turn factors basically account for the fact that these movements cannot be made at the same saturation flow rates as through movements. They consume more of the available green time, and consequently, more of the intersection's available capacity.

These adjustment factors are given in Tables 9-5 through 9-12.

TABLE 9-5. ADJUSTMENT FACTOR FOR LANE WIDTH

Lane Width, ft	8	9	10	11	12	13	14	15	≥ 16
Lane Width Factor, f_w	0.87	0.90	0.93	0.97	1.00	1.03	1.07	1.10	Use 2 Lanes

TABLE 9-6. ADJUSTMENT FACTOR FOR HEAVY VEHICLES

Percent Heavy Vehicles, %HV	0	2	4	6	8	10	15	20	25	30
Heavy Vehicle Factor, f_{HV}	1.00	0.99	0.98	0.97	0.96	0.95	0.93	0.91	0.89	0.87

TABLE 9-7. ADJUSTMENT FACTOR FOR GRADE

	DOWNHILL			LEVEL	UPHILL		
Grade, %	−6	−4	−2	0	+2	+4	+6
Grade Factor, f_g	1.03	1.02	1.01	1.00	0.99	0.98	0.97

TABLE 9-8. ADJUSTMENT FACTOR FOR PARKING, f_p

NO. OF LANES IN LANE GROUP	NO PKG	NUMBER OF PARKING MANEUVERS PER HOUR, N_m				
		0	10	20	30	40
1	1.00	0.90	0.85	0.80	0.75	0.70
2	1.00	0.95	0.92	0.89	0.87	0.85
3	1.00	0.97	0.95	0.93	0.91	0.89

TABLE 9-9. ADJUSTMENT FACTOR FOR BUS BLOCKAGE, f_{bb}

NO. OF LANES IN LANE GROUP	NUMBER OF BUSES STOPPING PER HOUR, N_B				
	0	10	20	30	40
1	1.00	0.96	0.92	0.88	0.83
2	1.00	0.98	0.96	0.94	0.92
3	1.00	0.99	0.97	0.96	0.94

TABLE 9-10. ADJUSTMENT FACTOR FOR AREA TYPE

TYPE OF AREA	FACTOR f_a
CBD	0.90
All other areas	1.00

TABLE 9-11. ADJUSTMENT FACTOR FOR RIGHT TURNS

CASE	TYPE OF LANE GROUP	RIGHT-TURN FACTORS, f_{RT}
1	EXCLUSIVE RT LANE; PROTECTED RT PHASING	0.85
2	EXCLUSIVE RT LANE; PERMITTED RT PHASING	$f_{RT} = 0.85 - (\text{peds}/2{,}100)$ peds $\leq 1{,}700$ $f_{RT} = 0.05$ peds $> 1{,}700$

Case 2 (continued):

No. of Conf. Pedestrians (peds)	0	50 (Low)	100	200 (Mod)	300	400 (High)	500
Factor	0.85	0.83	0.80	0.75	0.71	0.66	0.61
No. of Conf. Pedestrians (peds)	**600**	**800**	**1,000**	**1,200**	**1,400**	**1,600**	**≥ 1,700**
Factor	0.56	0.47	0.37	0.28	0.18	0.05	0.05

CASE	TYPE OF LANE GROUP	RIGHT-TURN FACTORS, f_{RT}
3	EXCLUSIVE RT LANE; PROTECTED PLUS PERMITTED PHASING	$f_{RT} = 0.85 - (1 - P_{RTA})(\text{peds}/2{,}100)$ $f_{RT} = 0.05$ (minimum)

Case 3 (continued):

No. of Conf. Pedestrians (peds)	Prop. of RT Using Prot. Phase, P_{RTA}					
	0.00	0.20	0.40	0.60	0.80	1.00
0	0.85	0.85	0.85	0.85	0.85	0.85
50 (Low)	0.83	0.83	0.84	0.84	0.85	0.85
100	0.80	0.81	0.82	0.83	0.84	0.85
200 (Mod)	0.75	0.77	0.79	0.81	0.83	0.85
300	0.71	0.74	0.76	0.79	0.82	0.85
400 (High)	0.66	0.70	0.74	0.77	0.81	0.85
600	0.56	0.62	0.68	0.74	0.79	0.85
800	0.47	0.55	0.62	0.70	0.77	0.85
1,000	0.37	0.47	0.56	0.66	0.75	0.85
1,400	0.18	0.32	0.45	0.58	0.72	0.85
≥ 1,700	0.05	0.20	0.36	0.53	0.69	0.85

CASE	TYPE OF LANE GROUP	RIGHT-TURN FACTORS, f_{RT}
4	SHARED RT LANE; PROTECTED PHASING	$f_{RT} = 1.0 - 0.15\,P_{RT}$

Case 4 (continued):

Prop. of RT in Lane, P_{RT}	0.00	0.20	0.40	0.60	0.80	1.00
Factor	1.00	0.97	0.94	0.91	0.88	0.85

CASE	TYPE OF LANE GROUP	RIGHT-TURN FACTORS, f_{RT}
5	SHARED RT LANE; PERMITTED PHASING	$f_{RT} = 1.0 - P_{RT}[0.15 + (\text{peds}/2{,}100)]$ $f_{RT} = 0.05$ (minimum)

Case 5 (continued):

No. of Conf. Pedestrians (peds)	Prop. of RT in Lane Group, P_{RT}					
	0.00	0.20	0.40	0.60	0.80	1.00
0	1.00	0.97	0.94	0.91	0.88	0.85
50 (Low)	1.00	0.97	0.93	0.90	0.86	0.83
100	1.00	0.96	0.92	0.88	0.84	0.80
200 (Mod)	1.00	0.95	0.90	0.85	0.80	0.75
400 (High)	1.00	0.93	0.86	0.80	0.73	0.66
600	1.00	0.91	0.83	0.74	0.65	0.56
800	1.00	0.89	0.79	0.68	0.58	0.47
1,000	1.00	0.87	0.75	0.62	0.50	0.37
1,400	1.00	0.84	0.67	0.51	0.35	0.18
≥ 1,700	1.00	0.81	0.62	0.42	0.23	0.05

TABLE 9-11. ADJUSTMENT FACTOR FOR RIGHT TURNS CONTINUED

CASE	TYPE OF LANE GROUP	RIGHT-TURN FACTORS, f_{RT}
6	SHARED RT LANE; PROTECTED PLUS PERMITTED PHASING	$f_{RT} = 1.0 - P_{RT}[0.15 + (\text{peds}/2{,}100)(1 - P_{RTA})]$ $f_{RT} = 0.05$ (minimum)

Prop. RT's Using Prot. Phase P_{RTA}	No. of Conf. Peds. (peds)	Prop. of RT's in Lane Group P_{RT}: 0.00	0.20	0.40	0.60	0.80	1.00
0.00	All	Same as Case 5					
0.20	0	1.00	0.97	0.94	0.91	0.88	0.85
	50	1.00	0.97	0.93	0.90	0.86	0.83
	200	1.00	0.95	0.91	0.86	0.82	0.77
	400	1.00	0.94	0.88	0.82	0.76	0.70
	600	1.00	0.92	0.85	0.77	0.70	0.62
	1,000	1.00	0.89	0.79	0.68	0.58	0.47
	1,400	1.00	0.86	0.73	0.59	0.45	0.32
	≥ 1,700	1.00	0.81	0.62	0.42	0.23	0.20
0.40	0	1.00	0.97	0.94	0.91	0.88	0.85
	50	1.00	0.97	0.94	0.91	0.87	0.84
	200	1.00	0.96	0.92	0.88	0.83	0.79
	400	1.00	0.95	0.89	0.84	0.79	0.74
	600	1.00	0.94	0.87	0.81	0.74	0.68
	1,000	1.00	0.91	0.83	0.74	0.65	0.56
	1,400	1.00	0.89	0.78	0.67	0.56	0.45
	≥ 1,700	1.00	0.87	0.75	0.62	0.49	0.36
0.60	0	1.00	0.97	0.94	0.91	0.88	0.85
	50	1.00	0.97	0.94	0.90	0.87	0.84
	200	1.00	0.96	0.92	0.89	0.85	0.81
	400	1.00	0.95	0.91	0.86	0.82	0.77
	600	1.00	0.94	0.89	0.84	0.79	0.74
	1,000	1.00	0.93	0.86	0.80	0.73	0.66
	1,400	1.00	0.92	0.83	0.75	0.67	0.58
	≥ 1,700	1.00	0.91	0.81	0.72	0.62	0.53
0.80	0	1.00	0.97	0.94	0.91	0.88	0.85
	50	1.00	0.97	0.94	0.91	0.88	0.85
	200	1.00	0.97	0.93	0.90	0.86	0.83
	400	1.00	0.96	0.92	0.89	0.85	0.81
	600	1.00	0.96	0.92	0.88	0.83	0.79
	1,000	1.00	0.95	0.90	0.85	0.80	0.75
	1,400	1.00	0.94	0.89	0.83	0.77	0.72
	≥ 1,700	1.00	0.94	0.88	0.81	0.75	0.69
1.00	All	Same as Case 4					

CASE	TYPE OF LANE GROUP	RIGHT-TURN FACTORS, f_{RT}
7	SINGLE LANE APPROACH	$f_{RT} = 0.90 - P_{RT}[0.135 + (\text{peds}/2{,}100)]$ $f_{RT} = 0.05$ (minimum)

No. of Conf. Peds. (peds)	Prop. of RT's in Single Lane P_{RT}: 0.00	0.20	0.40	0.60	0.80	1.00
0	1.00	0.87	0.85	0.82	0.79	0.77
50 (Low)	1.00	0.87	0.84	0.81	0.77	0.74
100	1.00	0.86	0.83	0.79	0.76	0.72
200 (Mod)	1.00	0.86	0.81	0.77	0.72	0.68
300	1.00	0.85	0.79	0.74	0.69	0.64
400 (High)	1.00	0.84	0.78	0.72	0.65	0.59
600	1.00	0.82	0.74	0.66	0.59	0.51
800	1.00	0.80	0.71	0.61	0.52	0.42
1,000	1.00	0.79	0.67	0.56	0.45	0.34
1,200	1.00	0.77	0.64	0.51	0.38	0.25
1,400	1.00	0.75	0.61	0.46	0.31	0.16
≥ 1,700	1.00	0.73	0.55	0.38	0.21	0.05

CASE	TYPE OF LANE GROUP	RIGHT-TURN FACTORS, f_{RT}
8	DOUBLE EXCLUSIVE RT LANE; PROTECTED PHASING	0.75

TABLE 9-12. ADJUSTMENT FACTOR FOR LEFT TURNS

<table>
<tr><th>CASE</th><th>TYPE OF LANE GROUP</th><th colspan="7">LEFT-TURN FACTOR, f_{LT}</th></tr>
<tr><td>1</td><td>EXCLUSIVE LT LANE; PROTECTED PHASING</td><td colspan="7">0.95</td></tr>
<tr><td>2</td><td>EXCLUSIVE LT LANE; PERMITTED PHASING</td><td colspan="7">Special Procedure; See Worksheet Fig. 9-9</td></tr>
<tr><td>3</td><td>EXCLUSIVE LT LANE; PROTECTED PLUS PERMITTED PHASING</td><td colspan="7">0.95[a]</td></tr>
<tr><td rowspan="3">4</td><td rowspan="3">SHARED LT LANE; PROTECTED PHASING</td><td colspan="7">$f_{LT} = 1.0/(1.0 + 0.05\ P_{LT})$</td></tr>
<tr><td>Prop. of LT's in Lane, P_{LT}</td><td>0.00</td><td>0.20</td><td>0.40</td><td>0.60</td><td>0.80</td><td>1.00</td></tr>
<tr><td>Factor</td><td>1.00</td><td>0.99</td><td>0.98</td><td>0.97</td><td>0.96</td><td>0.95</td></tr>
<tr><td>5</td><td>SHARED LT LANE; PERMITTED PHASING</td><td colspan="7">Special Procedure; See Worksheet Fig. 9-9</td></tr>
<tr><td rowspan="11">6</td><td rowspan="11">SHARED LT LANE; PROTECTED PLUS PERMITTED PHASING</td><td colspan="7">$f_{LT} = (1{,}400 - V_o)/[(1{,}400 - V_o) + (235 + 0.435\ V_o)P_{LT}]\quad V_o \leq 1{,}220$ vph
$f_{LT} = 1/[1 + 4.525\ P_{LT}]\quad V_o > 1{,}220$ vph</td></tr>
<tr><td rowspan="2">Opposing Volume V_o</td><td colspan="6">Prop. of Left Turns, P_{LT}</td></tr>
<tr><td>0.00</td><td>0.20</td><td>0.40</td><td>0.60</td><td>0.80</td><td>1.00</td></tr>
<tr><td>0</td><td>1.00</td><td>0.97</td><td>0.94</td><td>0.91</td><td>0.88</td><td>0.86</td></tr>
<tr><td>200</td><td>1.00</td><td>0.95</td><td>0.90</td><td>0.86</td><td>0.82</td><td>0.78</td></tr>
<tr><td>400</td><td>1.00</td><td>0.92</td><td>0.85</td><td>0.80</td><td>0.75</td><td>0.70</td></tr>
<tr><td>600</td><td>1.00</td><td>0.88</td><td>0.79</td><td>0.72</td><td>0.66</td><td>0.61</td></tr>
<tr><td>800</td><td>1.00</td><td>0.83</td><td>0.71</td><td>0.62</td><td>0.55</td><td>0.49</td></tr>
<tr><td>1,000</td><td>1.00</td><td>0.74</td><td>0.58</td><td>0.48</td><td>0.41</td><td>0.36</td></tr>
<tr><td>1,200</td><td>1.00</td><td>0.55</td><td>0.38</td><td>0.29</td><td>0.24</td><td>0.20</td></tr>
<tr><td>≥ 1,220</td><td>1.00</td><td>0.52</td><td>0.36</td><td>0.27</td><td>0.22</td><td>0.18</td></tr>
<tr><td>7</td><td>SINGLE LANE APPROACH</td><td colspan="7">Special Procedures; See Worksheet Fig. 9-9</td></tr>
<tr><td>8</td><td>DOUBLE EXCLUSIVE LT LANE; PROTECTED PHASING</td><td colspan="7">0.92</td></tr>
</table>

[a] This value is a starting estimate. Solutions are iterated for this case.

2. *Special procedure: left-turn adjustment factor for permitted phasing*—When a lane group includes permitted left turns, the left-turn adjustment factor must be computed using a complex series of equations. The equations approximate the effect of equilibrium flows that result from the interaction of left-turning vehicles, through vehicles, and opposing flows. The procedure is used for all permitted left turns, whether made from an exclusive or shared lane. A worksheet for these computations is presented in Section III of this chapter, "Procedures for Application."

The following variables are used in equations determining the left-turn adjustment factor, f_{LT}:

C = cycle length, in sec;

g = effective green time, in sec;

g_q = portion of green phase blocked to left-turning vehicles by the clearing of an opposing queue of vehicles, $g_q = g - g_u$, in sec;

g_u = portion of green not blocked by the clearing of an opposing queue of vehicles, in sec;

g_f = initial portion of green phase, during which through vehicles may move in a shared LT/TH lane; movement continues until arrival of first LT vehicle, which waits until opposing queue clears, blocking the lane for the remaining portion of g_q, in sec;

E_L = through-vehicle equivalent for opposed left turns;

v_a = total approach flow rate, in vph;
v_M = mainline approach flow rate; total approach flow rate less left turns from an exclusive lane or on a one-lane approach, in vph; the maximum value of v_M is 1,399; this value is used for all $v_M \geq 1{,}399$;
v_{LT} = left-turn flow rate, in vph;
P_{LT} = proportion of left turns in lane group flow;
P_{LTo} = proportion of left turns in opposing flow;
P_L = proportion of left turns in shared median or left-turn lane;
P_T = proportion of through vehicles in shared median or left-turn lane;
N = number of lanes in lane group or approach;
N_o = number of mainline lanes opposing the permitted left turn;
v_o = opposing flow rate, discounting left turns from an exclusive lane or one-lane approach; the maximum value of v_o is 1,399; this value is used for all $v_o \geq 1{,}399$.
s_{op} = saturation flow rate for opposing approach, in vphg;
Y_o = flow ratio for opposing approach; $Y_o = v_o/s_{op}$; and
f_s = left-turn saturation factor.

The left-turn adjustment factor reflects three component flows during any given green phase: (1) through flow in a shared lane at the start of the green until a left-turning vehicle arrives, blocking the lane while waiting to turn, (2) shared-lane or left-turn lane flow during the unsaturated period of opposing flow, and (3) left-turns made at the end of the green phase by vehicles already waiting in the intersection for an appropriate gap in opposing flow.

The computation of an appropriate left-turn adjustment factor proceeds using the following sequence:

a. The saturation flow rate for the opposing flow is estimated as:

$$s_{op} = \frac{1{,}800\ N_o}{1 + P_{LTo}\,(400 + v_M)/(1{,}400 - v_M)} \qquad (9\text{-}9)$$

where N_o does not include exclusive LT or RT lanes on the opposing approach, v_M is the mainline flow on the subject approach, not including left turns from an exclusive lane or a one-lane approach, and the denominator of the equation represents a weighted average through-vehicle equivalent for the opposing flow.

b. The flow ratio for the opposing flow may now be computed as;

$$Y_o = v_o/s_{op} \qquad (9\text{-}10)$$

c. The portion of the green phase that is *not* blocked by an opposing queue of vehicles is estimated as:

$$g_u = \frac{C\,(g/C - Y_o)}{1 - Y_o} = \frac{(g - CY_o)}{1 - Y_o} \qquad (9\text{-}11)$$
$$g_u = 0 \text{ if } Y_o \geq g/C$$

For there to be any left-turn capacity other than at the end of the green phase, $g_u \geq 0$ and $g/C > Y_o$. The opposing green ratio must exceed the opposing flow ratio. This requirement may be useful when the signalization must be assumed, as in a design problem.

d. The left-turn saturation factor is calculated from consideration of the opposing flow as:

$$f_s = (875 - 0.625\,v_o)/1{,}000 \qquad (9\text{-}12)$$

where v_o is the total opposing traffic flow. It should include left-turning vehicles only when left turns are made from a shared lane on a multilane approach during a permissive phase. Left turns are not included in opposing flow when made from a single-lane approach or from an exclusive LT lane.

e. Where the subject left turn is made from a shared lane, the proportion of left-turn flow in the shared lane is determined from:

$$P_L = P_{LT}[1 + (N - 1)g/(f_s g_u + 4.5)] \qquad (9\text{-}13)$$

Where the subject left turn is made from an exclusive LT lane, $P_L = 1.00$, because 100 percent of the traffic in the lane turns left.

f. The duration of the green phase during which through vehicles may move in shared lane until a left-turning vehicle arrives is estimated as:

$$g_f = \frac{2P_T}{P_L}\,[1 - P_T^{0.5g_q}] \qquad (9\text{-}14)$$

where $g_q = g - g_u$. If a separate left-turn lane is being analyzed, $P_T = 0$ and $g_f = 0$.

g. During the portion of the phase when opposing flow is unsaturated, g_u, the approximate through-vehicle equivalent for opposed left turns is:

$$E_L = \frac{1{,}800}{1{,}400 - v_o} \qquad (9\text{-}15)$$

h. The left-turn factor for a shared LT/TH lane or an exclusive LT lane is then given by:

$$f_m = \frac{g_f}{g} + \frac{g_u}{g}\left\{\frac{1}{1 + P_L(E_L - 1)}\right\} + \frac{2}{g}\{1 + P_L\} \text{ (Note: } f_m \leq 1.0) \qquad (9\text{-}16)$$

This factor applies only to the single lane from which left turns are made, however. Thus, where a left-turn lane is being considered, or for a single-lane approach, $f_{LT} = f_m$. For shared lanes on a multilane approach, the left-turn factor for the lane group or approach is:

$$f_{LT} = (f_m + N - 1)/N \qquad (9\text{-}17)$$

Although this procedure for determining the left-turn factor for permitted left turns is somewhat complex, it is a reasonable analytical representation of a complex equilibrium process. A worksheet presented in the "Procedures for Application" section of this chapter simplifies computations.

It should be noted that exact determination of the left-turn adjustment factor requires that signal timing parameters, specifically cycle length and green times, be known. Where signal timing is not known, a 60 to 90-sec cycle may be assumed, with green times proportional to average per lane flows during each phase. These assumptions can be iterated after a more definitive timing is established, but this is often not necessary, because the impact of these on the final factor is not substantial.

Capacity Analysis Module

In the capacity analysis module, computational results of previous modules are manipulated to compute key capacity variables, including the:

1. Flow ratio for each lane group.
2. Capacity of each lane group.
3. v/c Ratio of each lane group.
4. Critical v/c ratio for the overall intersection.

Flow ratios are computed by dividing the adjusted demand flow, v, computed in the "volume adjustment module," by the adjusted saturation flow rate, s, computed in the "saturation flow rate module."

The capacity of each lane group is computed from Eq. 9-1:

$$c_i = s_i \times (g/C)_i$$

If the signal timing is not known, a timing plan will have to be estimated or assumed to make these computations. Appendix II contains suggestions for making these estimates, but state or local policies and guidelines should also be consulted wherever possible.

The v/c ratio for each lane group is computed directly, by dividing the adjusted flows by the capacities computed above, as in Eq. 9-2:

$$X_i = v_i/c_i$$

The final capacity parameter of interest is the critical v/c ratio, X_c, for the intersection. It is computed from Eq. 9-3, as follows:

$$X_c = \sum_i (v/s)_{ci} \times [C/(C - L)]$$

This ratio indicates the proportion of available capacity which is being used by vehicles in critical lane groups. If this ratio exceeds 1.00, one or more of the critical lane groups will be oversaturated. It is an indication that the intersection design, cycle length, phase plan, and/or signal timing is inadequate for the existing or projected demand. A ratio of less than1.00 indicates that the design, cycle length, and phase plan are adequate to handle all critical flows without demand exceeding capacity, assuming that green times are proportionally assigned. Where phase splits are not proportional, some movement demands may exceed movement capacities even where the critical v/c ratio is less than 1.00.

The computation of this ratio requires that *critical lane groups* be identified. Where there are no overlapping signal phases in the signal design, the determination of critical lane groups is straightforward. Overlapping phases (concurrent phase timing) complicate matters, as various lane groups may move in several phases of the signal. The following guidelines may be used in determining critical lane groups:

1. Where phases *do not overlap:*

a. There will be one critical lane group for each signal phase.

b. The lane group with the highest flow ratio, v/s, of those lane groups moving in a given signal phase is critical.

c. Where signal timing is to be estimated or assumed, the critical lane groups are used to determine the timing.

2. Where phases *overlap:*

a. Based on the phase plan, combinations of lane groups that may consume the largest amount of available capacity must be identified. These are the same lane groups that will control the signal timing if it is to be estimated. This principle is best illustrated by example.

Consider the case of a leading and lagging green phasing plan on an arterial with exclusive left-turn lanes. Lane groups and signal phasing are shown in Figure 9-4.

During Phase 1, only two lane groups move, the NB LT/TH/RT group and the SB LT/TH/RT group. These move during no other phases. Thus the selection of a critical lane group is straightforward—the group with the highest flow ratio (v/s) is critical. For this phase, the critical lane group is either:

NB LT/TH/RT or SB LT/TH/RT

The second phase includes overlaps: the EB TH/RT lane group moves during Phases 2A and 2B, the WB TH/RT lane group moves during Phases 2B and 2C, the EB LT lane group moves during Phase 2A, and the WB LT lane group moves during Phase 2C. Thus, the EB TH/RT lane group could be critical *for the sum of phases* 2A AND 2B, with the WB LT being critical *for phase* 2C. The WB TH/RT lane group could be critical *for the sum of phases* 2B *and* 2C, with the EB LT being critical *for phase* 2A. Thus, two potential combinations of lane groups could be critical for the full Phase 2:

EB TH/RT + WB LT or WB TH/RT + EB LT

In determining the sum of critical lane flow ratios for the intersection, there are four possibilities:

NB LT/TH/RT + EB TH/RT + WB LT
SB LT/TH/RT + EB TH/RT + WB LT
NB LT/TH/RT + WB TH/RT + EB LT
SB LT/TH/RT + WB TH/RT + EB LT

The maximum sum would be used, and would identify the critical lane groups for the intersection.

A second example is given in Figure 9-5. For this example both streets have similar phasing. Left turns receive an exclusive phase which is followed by a leading phase for the direction of flow with the larger left-turn flow. During each of these portions of each phase, left turns are protected. The final portion of each phase is for through and right-turning movements, with left turns made on a permitted basis.

For the E-W street, the sum of Phases 1A and 1B could be controlled by either the EB LT or the WB LT, whichever is the heavier movement. If the EB LT controls Phases 1A and 1B, the WB TH/RT controls Phase 1C. If the WB LT controls

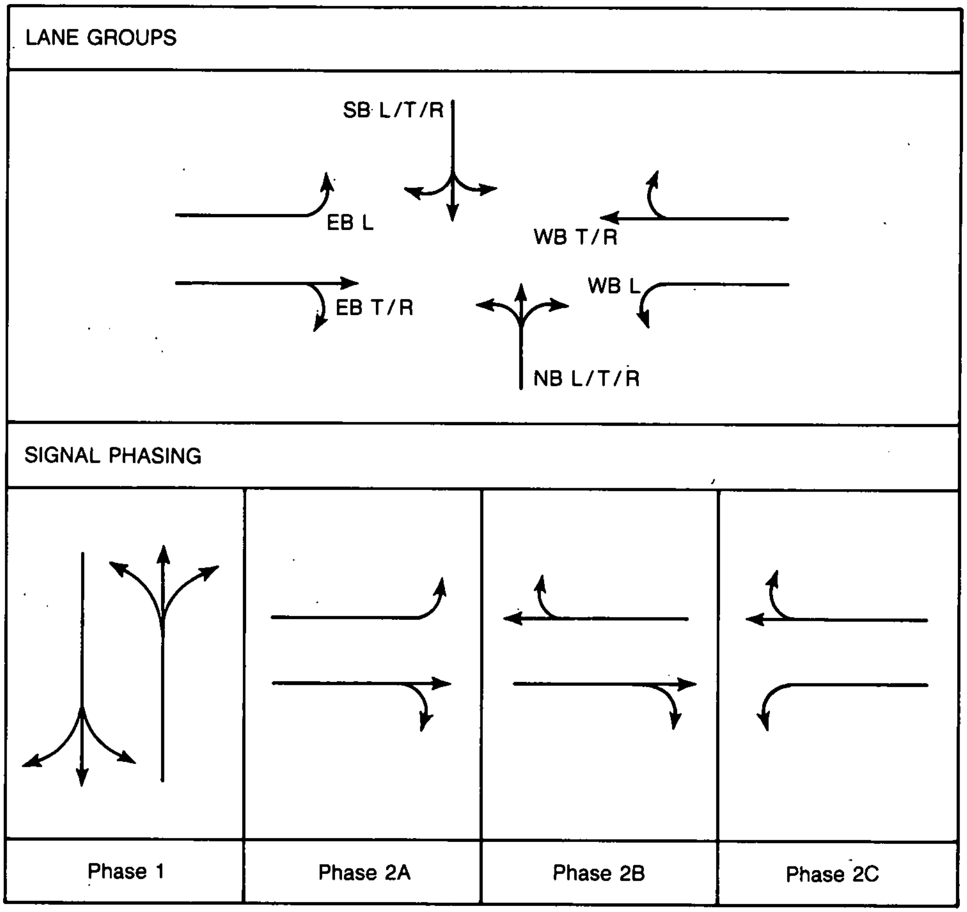


Figure 9-4. Illustrative example of determining critical lane groups for leading and lagging green phasing.

Phases 1A and 1B, the EB TH/RT controls Phase 1C. Thus, the potential combinations of critical movements are:

EB TH/RT + WB LT or WB TH/RT + EB LT

The N-S street has similar phasing, with similar results for the possible combinations of critical flows:

NB TH/RT + SB LT or SB TH/RT + NB LT

Again, the critical movements for the overall intersection may be any one of the following:

EB TH/RT + WB LT + NB TH/RT + SB LT
EB TH/RT + WB LT + SB TH/RT + NB LT
WB TH/RT + EB LT + NB TH/RT + SB LT
WB TH/RT + EB LT + SB TH/RT + NB LT

b. In examining phase plans for potential critical lane group combinations, no phase or portion of a phase can have more than *one* critical lane group. Thus, if a lane group is critical for the sum of Phases x and y, neither Phase x nor y can have another lane group which is critical for the individual phase, or for any other combination of phases containing Phase x or y.

c. Where signal timing is to be estimated, the critical lane groups are used to determine the timing.

Level-of-Service Module

In the level of-service module, the average stopped delay per vehicle is estimated for each lane group, and averaged for approaches and the intersection as a whole. Level of service is directly related to the delay value, and is found from Table 9-1.

1. *Delay assuming random arrivals*—The delay for each lane group is found using the following relationship.

$$d = 0.38\, C \frac{[1 - g/C]^2}{[1 - (g/C)\,(X)]} + 173\, X^2 \left[(X - 1) + \sqrt{(X - 1)^2 + (16\, X/c)}\right] \tag{9-18}$$

where:

d = average stopped delay per vehicle for the lane group, in sec/veh;
C = cycle length, in sec;
g/C = green ratio for the lane group; the ratio of effective green time to cycle length;
X = v/c ratio for the lane group; and
c = capacity of the lane group.

Equation 9-18 predicts the average stopped delay per vehicle for an assumed random arrival pattern for approaching vehicles.

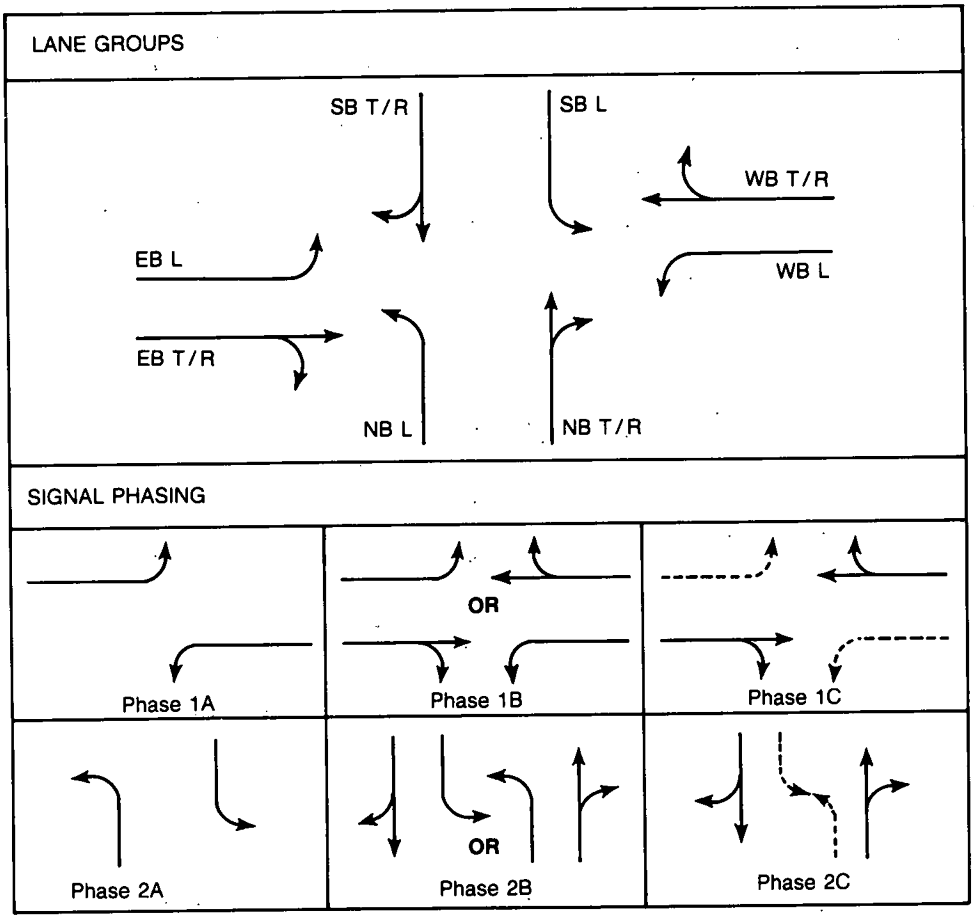


Figure 9-5. Illustrative example of determining critical lane groups for a complex multiphase signal.

The first term of the equation accounts for *uniform delay,* i.e., the delay that occurs if arrival demand in the subject lane group is uniformly distributed over time. The second term of the equation accounts for *incremental delay* of random arrivals over uniform arrivals, and for the additional delay due to cycle failures. The equation yields reasonable results for values of X between 0.0 and 1.0. Where oversaturation occurs for long periods (> 15 min), it is difficult to accurately estimate delay, because spillbacks may extend to adjacent intersections. The equation may be used with caution for values of X up to 1.2, but delay estimates for higher values are not recommended. Oversaturation, i.e., $X > 1.0$, is an undesirable condition that should be ameliorated if possible.

It is often useful to compute the uniform delay and incremental delay terms as separate values. This allows the analyst to see the relative contribution of individual cycle failures to total delay. Then:

$$d = d_1 + d_2 \tag{9-19}$$

where:

d_1 = first-term uniform delay, in sec/veh; and
d_2 = second-term incremental delay, in sec/veh.

2. *Progression adjustment factor*—As noted, the delay estimate obtained from Eq. 9-18 or Eq. 9-19 is for an assumed random arrival condition. In most cases, arrivals are not random, but are platooned as a result of signal progression and other factors. As part of the input data for an operational analysis, five arrival types are defined, and one would be specified for each lane group. The delay obtained from Eq. 9-18 or Eq. 9-19 is multiplied by the progression adjustment factor, given in Table 9-13.

When the signal progression is favorable to the subject lane group, delay will be considerably less than that for random arrivals. Similarly, when signal progression is unfavorable, delay can be considerably higher than that for random arrivals. The variation of delay with progression quality decreases as the v/c ratio (X) approaches 1.00, and is greater for pretimed signals than for other types of signalization. Left-turn movement delays are generally unaffected by progression: protected left-turn phases are rarely progressed, and permitted left-turn delay is most dependent on opposing traffic.

Delay is a complex variable that is sensitive to a variety of local and environmental conditions. These procedures provide reasonable estimates for delays expected for average conditions. They are most useful when used to compare operational conditions for various geometric or signalization designs. When evaluating existing conditions, it is advisable to measure delay in the field. Appendix III contains guidelines for intersection delay measurements using lane occupancy and volume counts.

3. *Aggregating delay estimates*—The procedure for delay estimation yields the average stopped delay per vehicle for each

TABLE 9-13. PROGRESSION ADJUSTMENT FACTOR, PF

TYPE OF SIGNAL	LANE GROUP TYPES	v/c RATIO, X	ARRIVAL TYPE[a]				
			1	2	3	4	5
Pretimed	TH, RT	≤ 0.6 0.8 1.0	1.85 1.50 1.40	1.35 1.22 1.18	1.00 1.00 1.00	0.72 0.82 0.90	0.53 0.67 0.82
Actuated	TH, RT	≤ 0.6 0.8 1.0	1.54 1.25 1.16	1.08 0.98 0.94	0.85 0.85 0.85	0.62 0.71 0.78	0.40 0.50 0.61
Semiactuated	Main St. TH, RT[b]	≤ 0.6 0.8 1.0	1.85 1.50 1.40	1.35 1.22 1.18	1.00 1.00 1.00	0.72 0.82 0.90	0.42 0.53 0.65
Semiactuated	Side St. TH, RT[b]	≤ 0.6 0.8 1.0	1.48 1.20 1.12	1.18 1.07 1.04	1.00 1.00 1.00	0.86 0.98 1.00	0.70 0.89 1.00
	All LT[c]	all	1.00	1.00	1.00	1.00	1.00

[a] See Table 9-2.
[b] Semiactuated signals are typically timed to give all extra green time to the main street. This effect should be taken into account in the allocation of green times.
[c] This category refers to exclusive LT lane groups with protected phasing only. When LT's are included in a lane group encompassing an entire approach, use factor for the overall lane group type. Where heavy LT's are intentionally coordinated, apply factors for the appropriate through movement.

lane group. It is also desirable to aggregate these values to provide average delay for an intersection approach and for the intersection as a whole. In general, this is done by computing weighted averages, where the lane group delays are weighted by the adjusted flows in the lane groups.

Thus, the delay for an approach is computed as:

$$d_A = \sum_i d_i v_i \Big/ \sum_i v_i \qquad (9\text{-}20)$$

where:

d_A = delay for approach A, in sec/veh;
d_i = delay for lane group i (on approach A), in sec/veh; and
v_i = adjusted flow for lane group i, in vph.

Approach delays can then be further averaged to provide the average delay for the intersection:

$$d_I = \sum_A d_A v_A \Big/ \sum_A v_A \qquad (9\text{-}21)$$

where:

d_I = average delay per vehicle for the intersection, in sec/veh; and
v_A = adjusted flow for approach A, in vph.

5. *Level of service determination*—Intersection level of service is directly related to the average stopped delay per vehicle. Once delays have been estimated for each lane group and aggregated for each approach and the intersection as a whole, Table 9-1 is consulted, and the appropriate levels of service are determined.

Interpretation of Results

The results of an operational analysis will yield two key results which must be considered:

1. The v/c ratios for each lane group and for the intersection as a whole.
2. Average stopped-time delays for each lane group and approach, and for the intersection as a whole, and the levels of service which correspond.

Any v/c ratio greater than 1.00 is an indication of actual or potential breakdowns, and is a condition requiring amelioration. Where the critical v/c ratio is less than 1.00, but some lane groups have v/c ratios greater than 1.00, the green time is generally not appropriately apportioned, and a retiming using the existing phasing should be attempted. Appendix II may be consulted for suggestions in this regard.

Where the critical v/c ratio is greater than 1.00, this is an indication that the overall signal and geometric design have inadequate capacity for the existing or projected flows. Improvements that might be considered include:

1. Basic changes in intersection geometry (number and use of lanes).
2. Lengthening the signal cycle.
3. Changing the signal phase plan.

Appendixes I and II may be consulted for suggestions with regard to these items. Existing state and local policies or standards should also be consulted in the development of potential improvements.

It should also be noted that v/c ratios near 1.00 represent situations with little available capacity to absorb demand increases. Particularly where projected volumes are being used, normal inaccuracies in such projections can cause an intersection projected to operate near capacity to oversaturate.

Level of service is a measure of the acceptability of stopped delay levels to motorists at a given intersection. Where delays are unacceptable, the causes of delay should be carefully examined. If an unfavorable progression is the largest contributor to delay, changes in intersection design and intersection signalization will have little impact—offsets and arterial coordination should be examined for possible improvement. Where progression is reasonable, and unacceptable delays still exist, provision of greater capacity through geometric or signal design changes should be examined.

In some cases, delay will be high even where v/c ratios are low. In these situations, poor progression and/or an inappropriately long cycle length is generally present.

The following point must be emphasized: unacceptable delay can exist where capacity is a problem, as well as in cases where it is adequate. Further, acceptable delay levels do not automatically ensure that capacity is sufficient. The analysis must consider the results of both the capacity analysis module and the level-of-service module to obtain a complete picture of existing or projected intersection operations.

Because of the complexity of this methodology, detailed worksheets are provided for the computations of each analysis module. These are presented and discussed in the "Procedures for Application" section of this chapter.

PLANNING ANALYSIS

Planning analysis of intersections is a broad evaluation of the capacity of an intersection without considering the details of signalization. It provides a basic assessment of whether or not capacity is likely to be exceeded for a given set of demand volumes and geometrics.

Because signalization is not considered in planning analysis, it is not possible to assess delay or level of service.

Input Information

Planning analysis requires basic data concerning:

1. Geometrics—number and use of lanes on each approach.
2. Volumes—given in total vph for each movement.

The procedure does not consider the details of lane width, parking conditions, or other features, nor does it consider the number of trucks and buses in the traffic stream.

Planning analysis identifies critical movements by individual lanes rather than lane groups, as in the operational analysis procedure. Thus, volumes must be assigned by lane:

1. Where exclusive turning lanes are present, all turns are assigned to the appropriate turning lane.
2. For shared and/or through lanes when permitted left turns are not present, volume is distributed equally among the available lanes.
3. When permitted left turns are included in shared lanes, vehicles are assigned to available through and shared lanes in equal numbers of passenger car equivalents. All right-turning and through vehicles have a passenger car equivalent (PCE) of 1.00, while permitted left turns have the following PCE values:

Opposing Through and Right-Turn Volume, V_o (vph)	Passenger Car Equivalent (PCE)
0 to 199	1.1
200 to 599	2.0
600 to 799	3.0
800 to 999	4.0
$\leq$ 1,000	5.0

This distribution is subject to the requirement that all left turns be assigned to the leftmost shared lane.

Capacity Analysis

Capacity analysis is carried out entirely in mixed vehicles per hour. No conversions are made to account for vehicle type distribution, turning volumes, specific geometrics (such as lane widths, grades, etc.), or other detailed characteristics. The one exception to this simple technique is for one-lane approaches, where permitted left-turning volumes are considered in terms of total numbers of passenger car equivalents. This exception is explained in the "Procedures for Application" section of this chapter.

As signal design is not known in the planning analysis, combinations of critical lane volumes are identified by considering conflicting movements. For a north-south street, critical conflicts are the NB left-turn movement with the SB through movement and the SB left-turn movement with the NB through movement. The *critical volume* for the north-south street is the largest sum among:

NB left-turn volume plus the maximum single-lane volume for the SB through plus right-turn movement, or
SB left-turn volume plus the maximum single-lane volume for the NB through plus right-turn movement.

Similarly, the critical volume for the east-west street is the greatest sum among:

EB left-turn volume plus the maximum single-lane volume for the WB through plus right-turn movement, or
WB left-turn volume plus the maximum single-lane volume for the EB through plus right-turn movement.

The total critical volume for the intersection is the sum of the critical volumes for the north-south and east-west streets. The critical volume for the intersection is compared to the criteria in Table 9-14.

As capacity cannot be precisely defined when signal design and the details of geometric and traffic conditions are not fully defined, the results of planning analysis are general determinations of the probable traffic conditions at the intersection.

TABLE 9-14. CAPACITY CRITERIA FOR PLANNING ANALYSIS OF SIGNALIZED INTERSECTIONS

CRITICAL VOLUME FOR INTERSECTION, VPH	RELATIONSHIP TO PROBABLE CAPACITY
0 to 1,200	Under Capacity
1,201 to 1,400	Near Capacity
$\geq$ 1,401	Over Capacity

Capacity will vary considerably with the cycle length, number of phases, lost times, grades, lane widths, presence of heavy vehicles, and numerous other factors. The values in Table 9-14 represent a range of normally occurring situations, including:

1. Cycle lengths from 30 to 120 sec.
2. Lost times from 6 sec/cycle to 14 sec/cycle.
3. Percent heavy vehicles from 0 percent to 10 percent.
4. Level terrain.
5. Standard lane widths from 10 ft to 12 ft.

For this range of conditions, critical volumes of less than 1,200 vph will virtually always be below the capacity of the intersection, while values greater than 1,400 vph will be more than the capacity of the intersection in most cases. For critical volumes between 1,200 and 1,400 vph, judgment is difficult, as the specific characteristics noted above will be important determining whether or not capacity is exceeded. For such cases, the only possible general evaluation is that the volume is "near" the capacity of the intersection, and could be less than or more than capacity, depending on prevailing conditions.

Planning analysis is a useful tool in evaluating the overall adequacy of proposed intersection designs or for comparing alternative designs. Such analyses are, however, preliminary and general. As the planning process proceeds to the design stage and more detailed information becomes available, operational analysis should be performed to gain a more definitive analysis of both capacity and delay at the intersection.

Worksheets are provided for computations in planning analysis. These are introduced and explained in the "Procedures for Application" section of this chapter.

OTHER ANALYSES

As noted previously, the computational procedures in this chapter emphasize the estimation of level of service (delay) based on known or projected traffic demands, signalization, and geometric design. Other computational applications include:

1. Determination of v/c ratios and service flow rates associated with selected levels of service, given a known signalization and geometric design.
2. Determination of signal timing parameters when known inputs are a selected level of service, demand flow rates, and geometric design.
3. Determination of geometric parameters (number of lanes, lane use allocations, etc.), given a selected level of service, demand flow rates, and signalization.

These alternative computational approaches are discussed in the "Procedures for Application" section, and illustrated among the "Sample Calculations."

III. PROCEDURES FOR APPLICATION

This section presents detailed worksheets for computations in both operational and planning analyses, and step-by-step instructions for their use and interpretation.

OPERATIONAL ANALYSIS

Operational analysis is divided into five modules: (1) Input Module, (2) Volume Adjustment Module, (3) Saturation Flow Rate Module, (4) Capacity Analysis Module, and (5) Level of Service Module.

The computations for each of these modules are conducted and/or summarized on the appropriate worksheet, as presented. The following sections give instructions for each module of the analysis.

Input Module

The input module is essentially a summary of the geometric, traffic, and signalization characteristics needed to conduct other computations. Where an existing case is under study, most of these data will be obtained from field studies. Where future conditions are under consideration, traffic data will be forecast, while geometric and signal designs will be based on existing conditions or will be proposed. The worksheet for the input module is shown in Figure 9-6.

The upper half of the worksheet contains a schematic intersection drawing on which basic volume and geometric data are recorded:

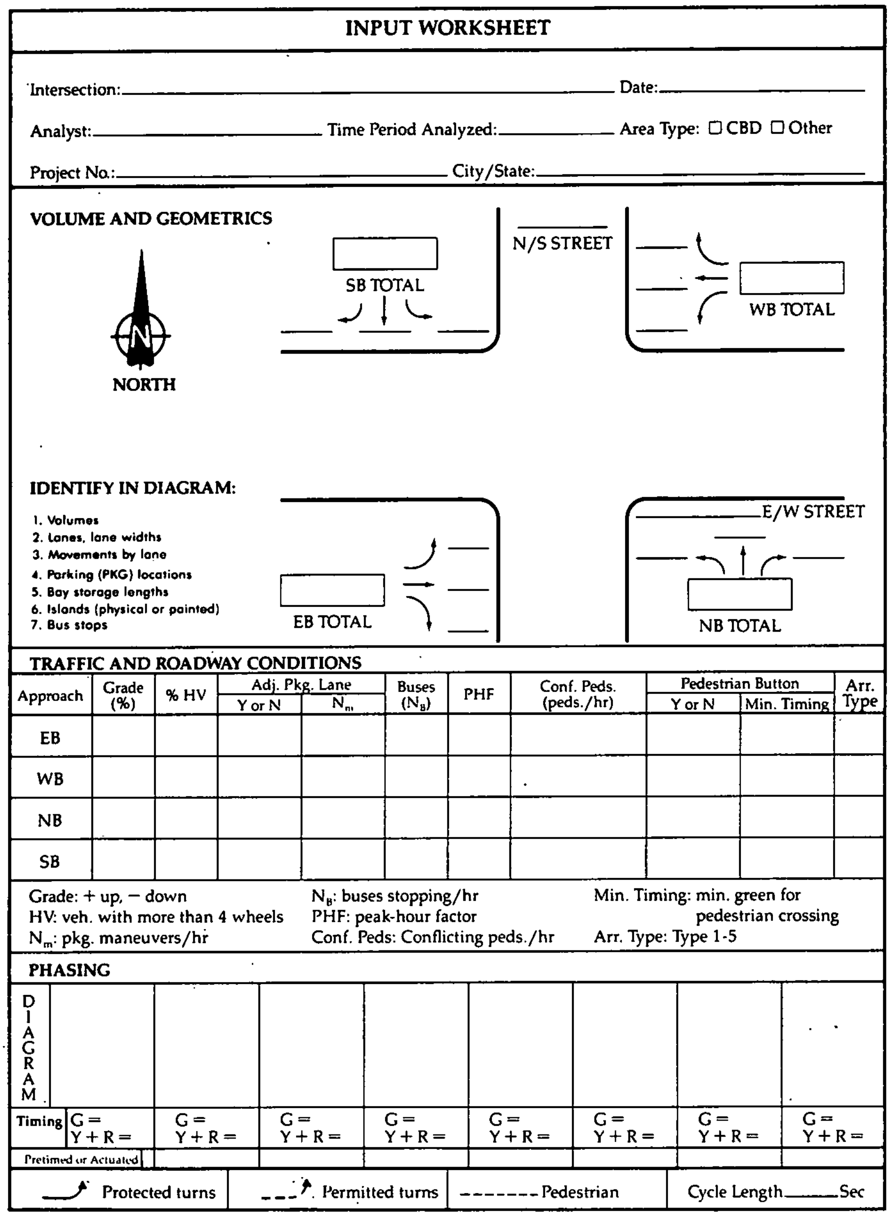
Step 1—Record Traffic Volumes

Full hourly volumes are entered for each approach into the appropriate boxes shown in each corner of the intersection diagram. Left-turn, through, and right-turn volumes are shown below these boxes, at the head of the appropriate directional arrow. The sum of the left, through, and right movements on each approach should equal the value shown in the approach volume box.

Step 2—Record Geometrics

The details of lane geometrics should be shown within the intersection diagram. Details should include:

1. Number of lanes.
2. Lane widths.
3. Traffic movements using each lane (shown by arrows).
4. Existence and location of curb parking lanes.
5. Existence and length of storage bays.
6. Existence of islands.
7. Existence and location of bus stops.



INPUT WORKSHEET

Intersection: ______ Date: ______

Analyst: ______ Time Period Analyzed: ______ Area Type: ☐ CBD ☐ Other

Project No.: ______ City/State: ______

VOLUME AND GEOMETRICS

IDENTIFY IN DIAGRAM:

1. Volumes
2. Lanes, lane widths
3. Movements by lane
4. Parking (PKG) locations
5. Bay storage lengths
6. Islands (physical or painted)
7. Bus stops

TRAFFIC AND ROADWAY CONDITIONS

Approach	Grade (%)	% HV	Adj. Pkg. Lane		Buses (N_B)	PHF	Conf. Peds. (peds./hr)	Pedestrian Button		Arr. Type
			Y or N	N_m				Y or N	Min. Timing	
EB										
WB										
NB										
SB										

Grade: + up, − down
HV: veh. with more than 4 wheels
N_m: pkg. maneuvers/hr
N_B: buses stopping/hr
PHF: peak-hour factor
Conf. Peds: Conflicting peds./hr
Min. Timing: min. green for pedestrian crossing
Arr. Type: Type 1-5

PHASING

DIAGRAM								
Timing	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =
Pretimed or Actuated								

Protected turns --- Permitted turns ------- Pedestrian Cycle Length ______ Sec

Figure 9-6. Worksheet for the input module.

Where geometric conditions are not known, a design should be proposed based on state or local practice. Appendix I may be consulted to assist in establishing a design for analysis. Where separate left-turn lanes exist, procedures assume that the storage length is adequate. This should be checked against the criteria in Appendix I.

The middle portion of the worksheet consists of a tabulation of additional geometric and traffic conditions for each approach.

Step 3—Enter Geometric and Traffic Conditions

The following parameters are entered into the tabulation in the middle of the worksheet. Separate entries are required for each approach:

1. Percent grade is entered in the first column; "+" indicates upgrades, while "−" indicates downgrades.
2. Percent heavy vehicles is entered in the second column. Normally the average for the entire approach is used. Where heavy vehicle presence varies significantly between movements, separate percentages may be used for LT, TH, and RT movements. A "heavy vehicle" is any vehicle with more than four tires touching the pavement.
3. The third and fourth columns describe parking characteristics for the approach. The third column indicates the presence of an adjacent parking lane at the intersection; "Y" or "N" is entered as appropriate. The fourth column indicates the number of parking maneuvers per hour occurring into and out of the parking lane within 250 ft of the intersection.
4. The number of local buses stopping per hour to discharge or pick up passengers within the confines of the intersection is listed in the fifth column. Any bus stop within 250 ft of the intersection should be considered to be "within the confines of the intersection."
5. The peak-hour factor is entered into the sixth column.
6. The number of pedestrians per hour using the crosswalk

conflicting with right turns from the subject approach is listed in the seventh column. For the NB approach, this is the east crosswalk; for the SB approach, the west crosswalk; for the EB approach, the south crosswalk; and for the WB approach, the north crosswalk.

7. The eighth and ninth columns describe pedestrian controls at the intersection. The eighth column indicates the existence of a pedestrian push-button detector on the subject approach with a "Y" or "N" entry. The ninth column gives the minimum green time required for a pedestrian to cross the street, as computed from Eq. 9-5:

$$G_p = 7.0 + W/4.0 - Y$$

8. The tenth and last column is used to identify the arrival type, which describes the platoon and progression characteristics in general terms. Arrival types are identified by number, from 1 to 5, as described in the "Methodology" section of this chapter.

Where data for some of these variables are not available, or where forecasts cannot be adequately established, default values may be used as an approximation. These may be established by judgment. Table 9-3 contains recommended default values where they cannot be established by other means.

The bottom portion of the worksheet is used to diagrammatically indicate the signal design for the intersection.

Step 4—Enter Signal Design

The sequence of signal phases is diagrammatically shown in the eight boxes at the bottom of the Input Module Worksheet. Up to an eight-phase signal design may be shown. Each box is used to show a single phase or subphase during which the permitted movements remain constant.

1. For each phase, show the allowable movements with arrows. Permitted turns are shown with a dashed arrow, while protected turns are shown with solid arrows. Conflicting pedestrian flows should be shown with dashed lines.
2. For each phase, the actual green time and the actual yellow + red times should be shown (in seconds) in the line labeled "Timing."
3. Each phase should be identified as pretimed (P) or actuated (A) in the appropriate box.

Where signal design is not known, two basic decisions should be made at this point: What type of control is going to be assumed for analysis, and what phase sequence will be used? These two questions are important, because they will influence the determination of lane groups for analysis. This portion of the signal design should be projected based on state or local practice. For additional suggestions on establishing the type of control and phase sequence, Appendix II may be consulted.

The timing of the signal will not be known where signal design is to be established. It may or may not be known where actuated signals are in place, depending on whether or not average phase durations were observed in the field. Appendix II contains recommendations for establishing phase times based on an assumed signal type and phase sequence, and for estimating the average phase lengths of actuated signals where observations are not available. These estimates, however, cannot be computed until the first half of the capacity analysis module is complete. Other computations may proceed without this information.

Volume Adjustment Module

The second major analysis module focuses on (1) adjusting hourly movement volumes to flow rates for a peak 15-min period within the hour, (2) establishing lane groups for analysis, and (3) adjusting demand flows to reflect lane distribution.

A worksheet for volume adjustment computations is shown in Figure 9-7.

Step 1—Enter Hourly Volumes

Hourly movement volumes are entered in column 3 of the worksheet. These are taken directly from the intersection diagram on the Input Worksheet.

Step 2—Convert Hourly Volumes to Peak Flow Rates

The peak-hour factor for each movement is entered in column 4 of the worksheet. Hourly volumes are divided by the PHF to compute peak flow rates:

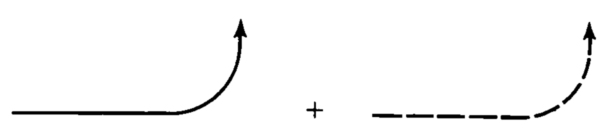
$$v_p = V/\text{PHF}$$

where v_p is the flow rate for the peak 15-min analysis period. The result is entered in column 5 of the worksheet.

Step 3—Establish Lane Groups for Analysis

Lane groups for analysis should be established based on the recommendations cited in the "Methodology" section of the chapter. Exclusive turn lanes are always established as separate lane groups. Where shared LT/TH lanes exist on an approach, they should be analyzed to determine whether they operate in a shared equilibrium mode or as effective left-turn lanes. In the latter case, they would be established as a separate lane group. If operating in equilibrium, the approach would be taken as a single lane group.

Lane groups are shown in column 6 of the worksheet by entering arrows illustrating the lanes and movements included in the group. Permitted turning movements are shown with dashed arrows, while protected turning movements are shown with solid arrows. Where a turn has a protected and a permitted phase, arrows should be shown as:



Step 4—Enter Lane Group Flow Rate

Once lane groups are established, the flow rates for included

VOLUME ADJUSTMENT WORKSHEET

① Appr.	② Mvt.	③ Mvt. Volume (vph)	④ Peak Hour Factor PHF	⑤ Flow Rate v_p (vph) ③ ÷ ④	⑥ Lane Group	⑦ Flow rate in Lane Group v_g (vph)	⑧ Number of Lanes N	⑨ Lane Utilization Factor U Table 9-4	⑩ Adj. Flow v (vph) ⑦ × ⑨	⑪ Prop. of LT or RT P_{LT} or P_{RT}
EB	LT									
	TH									
	RT									
WB	LT									
	TH									
	RT									
NB	LT									
	TH									
	RT									
SB	LT									
	TH									
	RT									

Figure 9-7. Worksheet for the volume adjustment module.

movements must be added and entered in column 7 of the worksheet as the lane group flow rate v_g.

Step 5—Enter the Number of Lanes in the Lane Group

The number of lanes in each lane group is entered in column 8 of the worksheet.

Step 6—Enter the Lane Utilization Factor

The lane utilization factor for each lane group is found from Table 9-4 and entered in column 9 of the worksheet. It is based on the number of lanes in the lane group, and accounts for unequal use of available lanes by vehicles.

Step 7—Compute the Adjusted Lane Group Flow Rate

The adjusted lane group flow rate is computed as:

$$v = v_g \times U$$

where v is the adjusted flow rate for the lane group. The result is entered in column 10 of the worksheet.

Step 8—Enter the Proportion of Left and/or Right Turns in the Lane Group

Column 11 of the worksheet is provided for entering the proportion of left and/or right turns in the lane group demand. These values may be computed as:

SATURATION FLOW ADJUSTMENT WORKSHEET

LANE GROUPS				ADJUSTMENT FACTORS								
① Appr.	② Lane Group Movements	③ Ideal Sat. Flow (pcphgpl)	④ No. of Lanes N	⑤ Lane Width f_w Table 9-5	⑥ Heavy Veh f_{HV} Table 9-6	⑦ Grade f_g Table 9-7	⑧ Pkg. f_p Table 9-8	⑨ Bus Blockage f_{bb} Table 9-9	⑩ Area Type f_a Table 9-10	⑪ Right Turn f_{RT} Table 9-11	⑫ Left Turn f_{LT} Table 9-12	⑬ Adj. Sat. Flow Rate s (vphg)
EB												
WB												
NB												
SB												

Figure 9-8. Worksheet for the saturation flow rate module.

$$P_{LT} = v_{LT}/v_g$$

$$P_{RT} = v_{RT}/v_g$$

where P_{LT} and P_{RT} are the proportions of left- and right-turning vehicles using the lane group, expressed as a decimal. Left- and right-turn flow rates are obtained from column 5 of the worksheet, while the total lane group flow rate, v_g, is given in column 7 of the worksheet.

The volume adjustment module results in the preparation of demand flows in a form amenable to further analysis, and it provides values used in subsequent analysis modules.

Saturation Flow Rate Module

In this module, the total saturation flow rate that can be accommodated by the lane group under prevailing conditions is computed. A worksheet for this module is shown in Figure 9-8.

Step 1—Enter Description of Lane Groups

Column 2 of the worksheet is used to identify the lanes and movements included in each lane group. These are the same as the entries in column 6 of the Volume Adjustment Worksheet, where lane groups have been established.

Step 2—Enter the Ideal Saturation Flow Rate

The ideal saturation flow rate per lane is entered in column 3 of the worksheet. For most computations, this value will be taken to be 1,800 pcphgpl, unless local data indicate that another value is appropriate. Appendix IV contains guidelines for conducting local studies to calibrate the saturation flow rate.

Step 3—Enter Adjustment Factors

The ideal saturation flow rate is multiplied by the number of

SUPPLEMENTAL WORKSHEET FOR LEFT-TURN ADJUSTMENT FACTOR, f_{LT}				
INPUT VARIABLES	**EB**	**WB**	**NB**	**SB**
Cycle Length, C (sec)				
Effective Green, g (sec)				
Number of Lanes, N				
Total Approach Flow Rate, v_a (vph)				
Mainline Flow Rate, v_M (vph)				
Left-Turn Flow Rate, v_{LT} (vph)				
Proportion of LT, P_{LT}				
Opposing Lanes, N_o				
Opposing Flow Rate, v_o (vph)				
Prop. of LT in Opp. Vol., P_{LTO}				
COMPUTATIONS	**EB**	**WB**	**NB**	**SB**
$S_{op} = \dfrac{1800\ N_o}{1 + P_{LTO}\left[\dfrac{400 + v_M}{1400 - v_M}\right]}$				
$Y_o = v_o / S_{op}$				
$g_u = (g - CY_o) / (1 - Y_o)$				
$f_s = (875 - 0.625\ v_o) / 1000$				
$P_L = P_{LT}\left[1 + \dfrac{(N-1)g}{f_s g_u + 4.5}\right]$				
$g_q = g - g_u$				
$P_T = 1 - P_L$				
$g_f = 2\ \dfrac{P_T}{P_L}\left[1 - P_T^{\ 0.5\ g_q}\right]$				
$E_L = 1800 / (1400 - v_o)$				
$f_m = \dfrac{g_f}{g} + \dfrac{g_u}{g}\left[\dfrac{1}{1 + P_L\ (E_L - 1)}\right] + \dfrac{2}{g}(1 + P_L)$				
$f_{LT} = (f_m + N - 1) / N$				

Figure 9-9. Supplemental worksheet for computation of left-turn adjustment factors for permissive left turns.

lanes in the lane group and by eight separate adjustment factors, as follows:

1. Enter the number of lanes in the lane group in column 4 of the worksheet.
2. Enter the lane width factor, f_w, in column 5 of the worksheet. It is obtained from Table 9-5.
3. Enter the heavy vehicle factor, f_{HV}, in column 6 of the worksheet. It is obtained from Table 9-6.
4. Enter the grade factor, f_g, in column 7 of the worksheet. It is obtained from Table 9-7.
5. Enter the parking factor, f_p, in column 8 of the worksheet. It is obtained from Table 9-8.
6. Enter the bus blockage factor, f_{bb}, in column 9 of the worksheet. It is obtained from Table 9-9.
7. Enter the area type factor, f_a, in column 10 of the worksheet. It is obtained from Table 9-10.
8. Enter the right-turn factor, f_{RT}, in column 11 of the worksheet. It is obtained from Table 9-11.
9. Enter the left-turn factor, f_{LT}, in column 12 of the worksheet. It is obtained from Table 9-12, or computed using the special procedure described in the "Methodology" section of this chapter for permitted left turns made from exclusive or shared lanes.

Factors for each lane group are separately determined from the prevailing conditions for the lane group. Information for these determinations is taken from the Input Module Worksheet. The proportions of left and/or right turns are taken from the last column of the Volume Adjustment Module Worksheet. Determination of right-turn factors for protected plus permitted phasing will require an assumption of the proportion of right-turning vehicles using the protected portion of the phase. This is basically judgmental, and should be guided by field observations where possible.

Step 4—Special Procedure for Permitted Left-Turn Adjustment Factors

As noted above, the left-turn adjustment factor for permitted left turns must be computed using the methodology described earlier. Because the computational process is reasonably complex, a special worksheet for these computations is provided in Figure 9-9.

1. *Enter input variables*—The first 11 rows of the worksheet are for the summarizing of input data needed to estimate the left-turn factor. Columns are provided for each approach, but only those including permitted left turns are used. The following input variables are entered:

a. Cycle Length, C. Where the signal timing is unknown, this value is taken to be 90 sec for the purpose of computing the factor.

b. Effective Green, g. The effective green for the lane group under study is entered. Where not yet known, it is estimated on the basis of average per lane flows in critical movements at the intersection. Thus, if a two-phase signal existed, and the largest N-S street per lane flow was 500 vphpl and the largest E-W street per lane flow was 300 vphpl, the green times (based on an assumed 90-sec cycle) would be estimated as:

$$g_{N\text{-}S} = (90 - 6)\,(500/800) = 52.5 \text{ sec}$$
$$g_{E\text{-}W} = (90 - 6)\,(300/800) = 31.5 \text{ sec}$$

where 6 sec is based on an assumption of 3 sec lost time per phase. Even if the result of the total analysis is to be a consideration of signalization, this rough estimation at this point is sufficient for the computation of an appropriate left-turn adjustment factor.

c. Number of Lanes, N. Enter the number of lanes in the lane group.

d. Total Approach Flow Rate, v_a. Enter the total flow rate on the approach under study. This includes the flow on both exclusive turning lanes and through lanes.

e. Mainline Flow Rate, v_M. The mainline flow rate is the total approach flow rate, v_a, minus left-turn flow rate from an exclusive lane or from a single-lane approach.

f. Left-Turn Flow Rate, v_{LT}. Enter the left-turn flow rate for the approach under study.

g. Proportion of Left Turns in Lane Group, P_{LT}. Enter the proportion of left turns in the lane group under study. This is obtained from column 11 of the Volume Adjustment Worksheet. For exclusive LT lanes, the value is 1.00.

h. Number of Opposing Lanes, N_o. Enter the number of lanes on the opposing approach. This would not include exclusive turning lanes on the opposing approach.

i. Opposing Flow Rate, v_o. The mainline flow rate v_M, on the opposing approach is entered.

j. Proportion of Left Turns in Opposing Flow, P_{LTo}. Enter the proportion of left turns included in v_o. This value will be 0.0 where opposing left turns are in an exclusive lane.

2. *Perform computations*—The worksheet lists Eqs. 9-7 to 9-17, which are used to compute the left-turn adjustment factor from the input data noted above. In sequence, the following variables are computed.

a. Opposing Saturation Flow rate, s_{op}. This is the approximate saturation flow rate for the opposing flow. Where there are no left turns in the opposing flow, the value is simply $1{,}800 \times N_o$.

b. Opposing Flow Ratio, Y_o. Compute as shown on worksheet.

c. Unsaturated Green, g_u. This is the portion of the green phase not blocked by the clearance of an opposing queue. Compute as shown on worksheet.

d. Left-Turn Saturation Factor, f_s. Compute as shown on worksheet. This step may be omitted where a left-turn factor for an exclusive LT lane group is being considered.

e. Proportion of Left-Turns in Shared Lane, P_L. Compute as shown on worksheet. Where an exclusive LT lane is under consideration, this value is 1.00.

f. Saturated Green, g_q. This is the portion of the green phase that is blocked by the clearance of an opposing queue. Compute as shown on worksheet.

g. Proportion of Through Vehicles in Shared Lane, P_T. Compute as shown on worksheet. Where an exclusive LT lane is under consideration, this value is 0.0.

h. Initial Green, g_f. This is the portion of the green for a shared lane during which through vehicles move until the arrival of the first left-turning vehicle. Compute as shown on worksheet. For exclusive LT lanes, this value is 0.0.

i. Through Equivalent of Left Turns, E_L. Compute as shown on worksheet.

j. Left-Turn Factor for Lane, f_m. This is the left-turn factor applied to the single-lane (either shared or exclusive) from which left turns are made. Compute as shown on the worksheet.

k. Left-Turn Factor for Lane Group, f_{LT}. This is the left-turn factor for the lane group under study. Where the lane group has only one lane, $f_{LT} = f_m$.

After completing this worksheet, enter the appropriate left-turn factors in column 12 of the Saturation Flow Adjustment Worksheet shown in Figure 9-8.

Step 5—Compute Adjusted Saturation Flow Rates

The adjusted saturation flow rate for each lane group is computed by multiplying the ideal saturation flow rate by the number of lanes in the lane group, and by each of the eight adjustment factors determined in Step 3. This is done in accordance with Eq. 9-8: $s = s_o\, N f_w f_{HV} f_g f_p f_{bb} f_a f_{RT} f_{LT}$.

Where information is insufficient to allow a detailed determination of adjustment factors, an approximate analysis may be conducted using an adjusted saturation flow rate of $1{,}600 \times N$ vphg. It must be remembered, however, that use of such a default value significantly reduces the accuracy of the analysis.

Capacity Analysis Module

In this module, information and computational results from the first three modules are combined to compute the capacity of each approach and to compute v/c ratios for each lane group and for the intersection as a whole. A worksheet for these computations is shown in Figure 9-10.

Step 1—Enter Lane Group Description

Column 2 of the worksheet is once again for the description of lane groups. Lanes and movements included in each lane group are entered, as on the Saturation Flow Rate Worksheet. For this worksheet, however, exclusive left-turn lane groups involving protected plus permitted left-turn phasing are divided, using separate lines to show the protected portion of the phase and the permitted portion of the phase. This is to allow this case to be iteratively examined, as indicated in later steps. Initially, it is assumed that the entire left-turn volume occurs during the *protected* portion of the phase.

CAPACITY ANALYSIS WORKSHEET								
LANE GROUP		③ Adj. Flow Rate v (vph)	④ Adj. Sat. Flow Rate s (vphg)	⑤ Flow Ratio v/s ③÷④	⑥ Green Ratio g/C	⑦ Lane Group Capacity c (vph) ④×⑥	⑧ v/c Ratio X ③÷⑦	⑨ Critical ? Lane Group
① Appr.	② Lane Group Movements							
EB								
WB								
NB								
SB								

Cycle Length, C ______ sec

Lost Time Per Cycle, L ______ sec

$\sum_i (v/s)_{ci} =$ ______

$X_c = \frac{\sum_i (v/s)_{ci} \times C}{C - L} =$ ______

Figure 9-10. Worksheet for the capacity analysis module.

Step 2—Enter the Adjusted Flow Rate for Each Lane Group

The adjusted flow rate for each lane group is obtained from the Volume Adjustment Module Worksheet and entered in column 3 of the worksheet. As noted above, where a protected plus permitted left-turn lane group has been separated, it is initially assumed that the entire flow uses the protected portion of the phase.

Step 3—Enter the Adjusted Saturation Flow Rate for Each Lane Group

The adjusted saturation flow rate for each lane group is obtained from the Saturation Flow Rate Worksheet and entered in column 4 of the worksheet. Where a protected plus permitted left-turn lane group has been separated, the saturation flow rate is entered for the protected portion of the phase. No saturation flow rate is shown for the permitted portion of the phase.

Step 4—Compute the Flow Ratio for Each Lane Group

The flow ratio for each lane group is computed as v/s and entered in column 5 of the worksheet.

Step 5—Identify Critical Lane Groups

At this point in the computations, critical lane groups may be identified according to the guidelines discussed in the "Methodology" section of this chapter. A critical lane group is defined as the lane group with the highest flow ratio in each phase or set of phases. Where overlapping phases exist, all possible combinations of critical lane groups must be examined for the combination producing the largest sum of flow ratios, as discussed previously. Critical lane groups are identified by a check placed in the last column of the worksheet.

The flow ratios for critical lane groups are summed. The result is entered in the appropriate space at the bottom of the fourth column of the worksheet.

Step 6—Enter the Green Ratio for Each Lane Group

The g/C ratio for each lane group is computed and entered in column 6 of the worksheet. The g/C ratio is the effective green time divided by the cycle length. Actual green times and the cycle length are obtained from the Input Module Worksheet. Where change intervals are in the range of 3 to 5 sec, the effective green time can be assumed to be equal to the actual green time.

Where longer change intervals are present, efffective green times can be taken to be equal to the actual green time plus the change interval minus the start-up and clearance lost times per phase. These lost times are normally assumed to be in the range of 3 to 5 sec per phase.

For those cases in which signal timing has not yet been established, information is now available to permit the estimation of signal timing. Consult Appendix II for guidelines on estimating the timing, so that g/C values may be found and entered on the worksheet.

Step 7—Compute Capacity of Each Lane Group

The capacity of each lane group is computed as the saturation flow rate times the green ratio: $c_i = s_i \times (g/C)_i$. The result is entered in column 7 of the worksheet.

Step 8—Compute v/c *Ratios for Each Lane Group*

The v/c ratio for the lane group is the ratio of adjusted flow to capacity: $X_i = v_i/c_i$. These values are computed and entered in column 8 of the worksheet.

Step 9—Compute the Critical v/c *Ratio*

The critical v/c ratio, X_c, is computed according to Eq. 9-3 as:

$$x_c = \frac{\sum_i (v/s)_{ci}\, C}{C - L}$$

This computation is made and the result is entered in the appropriate space at the bottom of the worksheet.

The cycle length required for these computations is obtained from the Input Module Worksheet. The total lost time per cycle must be consistent with the assumed effective green times. The lost time per cycle is the cycle length minus the sum of the nonoverlapping effective greens. It is generally in the range of 3 to 5 sec times the number of discrete phases.

Step 10—Iteration of Protected Plus Permitted Left-Turn Lane Groups

In this module, exclusive left-turn lane groups with protected/permitted phasing are separated, with all flow assumed to occur in the protected phase. This assumption may result in a critical v/c ratio or a protected-phase v/c ratio that is considered to be too high. Rather than consider major changes in signal or geometric design at this point, it is advisable to assign a portion of the left-turn movement to the permitted portion of the phase. The maximum flow rate that may be assigned to the permitted portion of the phase is the capacity of the permitted phase, computed as the maximum of:

$$c_{LT} = (1{,}400 - v_o)(g/C)_{PLT} \tag{9-22}$$

or

$$c_{LT} = 2 \text{ vehicles per signal cycle}$$

where:

c_{LT} = capacity of the left-turn permitted phase, in vph;
v_o = opposing through plus right-turn flow rate, in vph; and
$(g/C)_{PLT}$ = effective green ratio for the permitted left-turn phase, in sec.

The analyst may assign up to this flow rate to the permitted phase, deducting a similar amount from the protected phase. All other computations must now be redone, using the new flow rate. The flow and v/c ratios for the phase will be altered, and the critical v/c ratio for the intersection may also be affected. If signal timing was estimated based on v/c ratios as recommended in Appendix II, g/C ratios will have to be recomputed and all lane group capacities and v/c ratios will be affected.

At the completion of this module, the capacity characteristics of the intersection are now defined. These characteristics must be evaluated in their own right, as well as in conjunction with the delays and levels of service resulting from the next module. While the "Methodology" section of this chapter discusses the interpretation of capacity results, some key points are summarized here:

1. A critical v/c ratio of greater than 1.00 indicates that the signal and geometric design cannot accommodate the combination of critical flows at the intersection. The actual or projected demand in these movements exceeds the capacity of the intersection to handle them. The condition can be ameliorated by some combination of increased cycle length, changes in the phasing plan, and/or basic changes in geometrics.

Note, however, that computations should be conducted using *arrival volumes.* Where the v/c ratios are under 1.00, arrival and departure volumes are the same. Where v/c ratios are greater than 1.00, either for an individual phase or for the overall intersection, departure volumes are less than arrival volumes. Future volume forecasts are also arrival volumes, by definition. Where counts of actual *departure volumes* are used in analysis, the actual v/c ratio *cannot be greater than* 1.00. Observed departure volumes cannot exceed capacity. In such cases, computations should be checked for errors. If v/c ratios of greater than 1.00 persist, it is an indication that the intersection operates more efficiently than anticipated by these computational techniques.

2. When the critical v/c ratio is acceptable, but the v/c ratios for critical lane groups vary widely, the green time allocation

should be reexamined, as disproportionate distribution of available green is indicated.

3. If permitted left turns result in extreme reductions in saturation flow rate for applicable lane groups, consideration of protected phasing might be considered.

4. If the sum of critical lane group flow ratios exceeds 0.90 to 0.95, it is unlikely that the existing geometric and signal design can accommodate the demand. Changes in either or both should be considered.

5. Where v/c ratios are unacceptable, and signal phasing already includes protective phasing for significant turning movements, it is probable that geometric changes will be required to ameliorate the condition.

The capacity of an intersection is a complex variable depending on a large number of prevailing traffic, roadway, and signalization conditions. Suggestions on interpretation are not meant to be exhaustive or complete, but merely to point out some of the more common problems that can be identified as a result of the capacity analysis module results.

Level-of-Service Module

The level-of-service module combines the results of the volume adjustment, saturation flow rate, and capacity analysis modules to find the average stopped-time delay per vehicle in each lane group. The level of service is directly related to delay, and is found from Table 9-1. The worksheet for this module is shown in Figure 9-11.

Delay is found from Eqs. 9-18 and 9-19. These equations are restated for convenience.

$$d_1 = 0.38\, C \frac{[1 - (g/C)]^2}{[1 - (g/C)(X)]}$$

$$d_2 = 173\, X^2 \left[(X-1) + \sqrt{(X-1)^2 + (16X/c)}\right] \tag{9-18}$$

$$d = d_1 + d_2 \tag{9-19}$$

The worksheet is designed to compute the first and second term delays separately. Their sum is then multiplied by the progression adjustment factor (PF) to account for the impact

LEVEL-OF-SERVICE WORKSHEET

Lane Group		First Term Delay				Second Term Delay				Total Delay & LOS		
① Appr.	② Lane Group Movements	③ v/c Ratio X	④ Green Ratio g/C	⑤ Cycle Length C (sec)	⑥ Delay d_1 (sec/veh)	⑦ Lane Group Capacity c (vph)	⑧ Delay d_2 (sec/veh)	⑨ Progression Factor PF Table 9-13	⑩ Lane Group Delay (sec/veh) (⑥+⑧) × ⑨	⑪ Lane Group LOS Table 9-1	⑫ Approach Delay (sec/veh)	⑬ Appr. LOS Table 9-1
EB												
WB												
NB												
SB												

Intersection Delay ______ sec/veh   Intersection LOS ______ (Table 9-1)

Figure 9-11. Worksheet for the level-of-service module.

of progression on delay. This factor is obtained from Table 9-13.

Step 1—Enter Lane Group Description

As in the case of previous worksheets, column 2 is used to enter the description of the lanes and movements included in the lane group. This description will be the same as shown on the volume adjustment worksheet. Protected plus permitted left-turn lane groups need not be separated on this worksheet.

Step 2—Find First Term Delay

The first term of the delay equation (Eq. 9-18) accounts for "uniform delay," i.e., that delay which would result in a lane group if arrivals were uniformly distributed, and if no cycles experienced oversaturation. It is dependent on the v/c ratio, X, for the lane group, the green ratio, g/C, for the lane group, and the cycle length, C. It is found as follows:

1. Enter the v/c ratio for each lane group in column 3 of the worksheet. These may be obtained from the Capacity Analysis Worksheet.
2. Enter the green ratio for each lane group in column 4 of the worksheet. This value is obtained from the Capacity Analysis Worksheet.
3. Enter the cycle length in column 5 of the worksheet. This value is also found on the Capacity Analysis Worksheet.
4. Compute the first term delay in accordance with Eq. 9-18. Enter the result in column 6 of the worksheet.

Step 3—Find Second Term Delay

The second term of the delay equation accounts for the "incremental delay," i.e., the delay over and above uniform delay due to arrivals being random rather than uniform, and due to cycles which overflow. It is based on the v/c ratio, X, and the capacity, c, for the lane group. It is found as follows:

1. Enter the capacity for each lane group in column 7 of the worksheet. It is found from the Capacity Analysis Worksheet.
2. Compute the second-term delay from Eq. 9-18. Enter the result in column 8 of the worksheet.

Step 4—Find the Delay and Level of Service for Each Lane Group

The delay for each lane group is the sum of the first- and second-term delays multiplied by the progression factor. Delay and level of service are found as follows:

1. Find the progression factor PF for each lane group from Table 9-13. Enter this value in column 9 of the worksheet.
2. Compute the average stopped-time delay per vehicle for each lane group as follows: Delay = $(d_1 + d_2) \times$ PF. Enter the result in column 10 of the worksheet.
3. Find the level of service for each lane group from Table 9-1. Enter the result in column 11 of the worksheet.

Step 5—Find the Delay and Level of Service for Each Approach

The average delay per vehicle is found for each approach by adding the product of lane group flow rate and delay for each lane group on the approach and dividing by the total approach flow rate. The weighted-average delay is entered in column 12 of the worksheet for each approach. Level of service is determined from Table 9-1 and entered in column 13 of the worksheet.

Step 6—Find the Delay and Level of Service for the Intersection

The average delay per vehicle for the intersection as a whole is found by adding the product of approach flow rate and approach delay for all approaches and dividing the sum by the total intersection flow rate. This weighted-average delay is entered in the appropriate space at the bottom of the worksheet. The overall intersection level of service is found from Table 9-1, and entered in the appropriate space at the bottom of the worksheet.

The result of this module is an estimation of the average stopped-time delay per vehicle in each lane group, as well as average values for each approach and for the intersection as a whole. Level of service is directly related to delay values and is assigned on that basis.

Level of service and delay values are best analyzed in conjunction with the results of the capacity analysis module. While clearly not exhaustive, some of the more common situations are discussed as follows:

1. The level of service is an indication of the general acceptability of delay to drivers. It should be noted that this is somewhat subjective: what is "acceptable" in a large central business district is not necessarily "acceptable" in a less-dense environment.
2. Where delay levels for the intersection as a whole are acceptable, but are unacceptable for certain lane groups, the phase plan and/or allocation of green time might be examined to provide for more efficient handling of the disadvantaged movement(s).
3. Where delay levels are unacceptable, but v/c ratios are relatively low (capacity analysis module), the cycle length may be too long for prevailing conditions and/or the phase plan may be inefficient. It should be noted, however, that where signals are part of a coordinated system, the cycle length at individual intersections is determined by system considerations, and alterations at isolated locations may not be practical.
4. Where both delay levels and v/c ratios are unacceptable, the situation is most critical. Delay is already high, and demand is near or over capacity. In such situations, the delay may increase rapidly with small changes in demand. The full range of potential geometric and signal design improvements should be considered in the search for improvements to such cases.

Delay and level of service, like capacity, are complex variables depending on a wide range of traffic, roadway, and signalization conditions. The operational analysis techniques presented herein are useful in estimating the performance characteristics of the intersection, and in providing basic insights into probable causal factors.

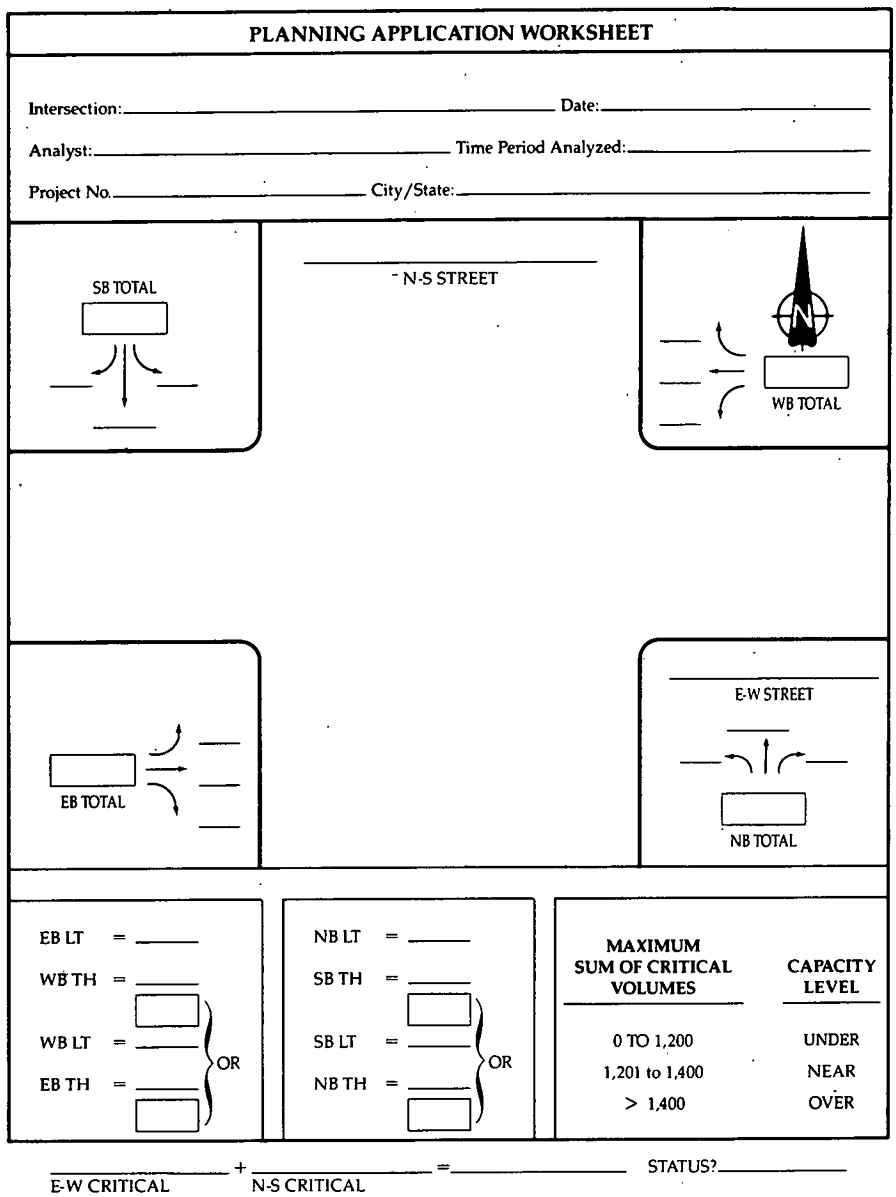
These procedures do not, however, account for all possible conditions. The influences of such characteristics of specific curb-corner radii, intersection angle, combinations of grades on various approaches, odd geometric features (offset intersections, narrowing on the departure lanes, etc.), and other unusual site-specific conditions are not addressed in the methodology. Field studies may be conducted in such cases to determine delay directly (see Appendix III), and or to calibrate the prevailing saturation flow rate (see Appendix IV). Unusual delays may result from blockages, such as illegally parked or stopped vehicles or other factors. The analyst may also gain additional insights into intersection operations by observing them in the field, in addition to making the analytical analyses prescribed in this chapter.

PLANNING ANALYSIS

The planning application is intended for use in sizing the overall geometrics of the intersection or in identifying the general capacity sufficiency of an intersection for planning purposes. It is based on the "sum of critical lane volumes" and requires minimum effort and minimum input information.

The basic input consists of demand volumes and intersection geometrics. Two worksheets are provided for planning analysis. Figure 9-12 is the basic worksheet on which all input information and analysis are conducted. Figure 9-13 is used to distribute approach volume among available lanes where shared left-turn/through lanes are involved.

The following steps describe the detailed application of the planning analysis technique.


PLANNING APPLICATION WORKSHEET

Intersection: ______ Date: ______

Analyst: ______ Time Period Analyzed: ______

Project No. ______ City/State: ______

EB LT = ______
WB TH = ______
WB LT = ______ OR
EB TH = ______

NB LT = ______
SB TH = ______
SB LT = ______ OR
NB TH = ______

MAXIMUM SUM OF CRITICAL VOLUMES	CAPACITY LEVEL
0 TO 1,200	UNDER
1,201 to 1,400	NEAR
> 1,400	OVER

______ + ______ = ______ STATUS? ______
E-W CRITICAL N-S CRITICAL

Figure 9-12. Basic worksheet for planning analysis.

LANE DISTRIBUTION FOR SHARED LEFT/THRU LANES ON A MULTILANE APPROACH WITH PERMITTED LEFT TURN LANES (OPTIONAL WORKSHEET)										
①	②	③	④	⑤	⑥	⑦	⑧	⑨	⑩	⑪
V_O Opposing Volume (vph)	PCE_{LT}	V_{LT}	LT Equiv. PCE's	Total Volume (TH+RT)	Total	No. of Lanes on Approach	Equiv. Volume Per Lane	Thru Vehicles in LT+TH Lane	Vol. in LT+TH Lane	Vol. in ea. of the Remaining Lanes
0 — 199 = 1.1 200 — 599 = 2.0 600 — 799 = 3.0 800 — 999 = 4.0 ≥ 1,000 = 5.0 APPR.			②×③		④+⑤		⑥÷⑦	⑧−④	③+⑨	$\frac{(③+⑤)-⑩}{⑦-1.0}$
EB LT										
WB LT										
NB LT										
SB LT										

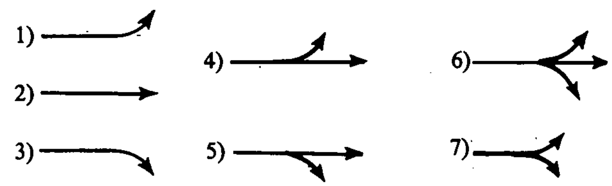
Figure 9-13. Planning worksheet for lane distribution of volume.

Step 1—Record Demand Volumes

The Planning Worksheet provides specific locations for recording the basic turning and through movement demand volumes to be evaluated. The terminology "demand volumes" is used to emphasize the planning nature of this application; because these volumes are projections of expected traffic at some future time, the analyst must decide whether to use an hourly volume or a peak 15-min flow rate in the analysis. Peak 15-min rates of flow are equal to the peak-hour volume divided by a projected peak-hour factor. Volumes are stated in terms of mixed traffic, including trucks, buses, and recreational vehicles. Enter the given volumes for each movement on each approach in the appropriate spaces provided in the corners of the worksheet. Add and record the total volume on each approach in the appropriate boxes.

Step 2—Record Geometrics

If the intersection geometrics have been determined, sketch the approach lanes and lane configurations on the Planning Worksheet within the intersection schematic diagram provided. Next, identify the traffic movement or combination of movements expected for each traffic lane. Possible movements by lane are as follows:



If the intersection geometrics have not been determined, they must be estimated or assumed. State or local practice should guide the estimation. Appendix I may be consulted for additional suggestions.

Step 3—Identify Lane Impedance

Identify those left-turn movements in shared lanes that interfere with an opposing through flow. These left-turn movements will impede through vehicles using the same lane. Identify those median lanes containing left-turning and through traffic impeded by an opposing through flow by marking these lanes with an asterisk (*).

Step 4—Assign Lane Volumes

Distribute the through and turning movement volumes to each lane as uniformly as flow conditions permit. The basic premise is that traffic will redistribute at the intersection yielding an equal number of passenger car equivalents (PCE's) on each main approach lane. This concept is particularly important in shared-lane situations. Lane volumes can be determined as follows:

1. If an approach has an exclusive left-turn, through, or right-turn lane, assign the total movement volume to this lane. Where multiple lanes exist for a single movement, uniformly distribute the volume to each lane. Short turn lanes may cause some imbalance to occur which may be accounted for where field experience dictates that judgment be exercised. Appendix I discusses the subject of turn-lane design in detail.
2. Where an approach has an exclusive left-turn lane, but does not have an exclusive right-turn lane, the through and right-turning vehicles are distributed equally to available lanes, subject to the constraint that all right-turning vehicles must be assigned to the right lane. Right-turning vehicles are assumed to have a PCE value of 1.0 for planning purposes.
3. If an approach with two or more lanes has a shared left-turn/through lane, the volume distribution will be impacted by impedance. Such lanes should have been denoted by an asterisk in Step 3. Vehicles are distributed among the available lanes such that the number of PCE's in each lane is equal, subject to the restriction that all left-turning vehicles must be assigned to the leftmost lane. PCE values for impeded left-turning vehicles are as follows:

Opposing Volume, V_o (vph)	Passenger Car Equivalent (PCE)
0 to 199	1.1
200 to 599	2.0
600 to 799	3.0
800 to 999	4.0
$\geq$ 1,000	5.0

The worksheet shown in Figure 9-13 may be used to make PCE computations and distribute the volume to available lanes. The worksheet is self-explanatory, and results in the assignment of equal PCE volumes to each lane of the approach. If the left-turn PCE volume exceeds the average lane PCE volume, *all* left turns are assigned to the leftmost lane, and the remaining through and right-turn volumes are assigned equally to remaining lanes. Synthetic "PCE volumes" are converted back to actual vph before entering the lane volumes on the Planning Worksheet.

Step 5—Special Procedure for Single-Lane Approaches

If an approach has a single lane, record all turning movement volumes on the Planning Worksheet as previously described. Sum the movement volumes, and record this as the total flow per lane.

The lane distribution worksheet of Figure 9-13 is filled out for the single-lane approach through column 8. This allows modifications to the basic procedure to account for the unique way in which single-lane approaches operate, as follows:

1. Record the PCE volume from column 8 of the lane distribution worksheet on the Planning Worksheet in place of the actual volume for the combined movement.
2. Subtract any opposing left-turn volume made from a single approach lane from the left-turn volume for the subject single-lane approach and record the remainder, if any, in place of the left-turn volume. Left turns are assumed to be made into gaps created by left turns from the opposing single lane approach until all such gaps are exhausted. It is critical that the analyst determine whether the opposing approach will operate as a single-lane or as a left-turn lane with a bypass for through vehicles. In the latter case, the modification presented in this step is eliminated, since the approach essentially operates as two lanes. For the single-lane approach, analysis is carried out in PCE volumes to account for the impact of impeded left turns blocking the approach to through and right-turning vehicles, as well as to following left-turners.

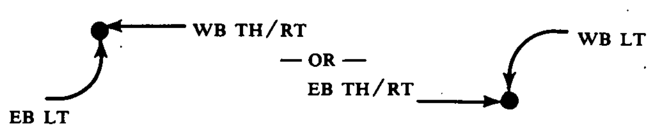
Step 6—Find the Sum of Critical Lane Volumes

The sum of conflicting critical lane volumes represents the total demand volume per lane at the intesection. It is assumed that the signalization at the intersection will be optimal and that protected phases would be provided where appropriate.

The sum of the critical volumes for the intersection equals the sum of the critical lane volumes on each street. Identify the critical lane flows, calculate the critical volumes for each street, and record these volumes in the appropriate boxes provided on the Planning Worksheet.

The critical volume on the east-west street is the maximum of two sums each calculated as the conflicting volume of a left-turn movement and the opposing through plus right-turn lane flow, as illustrated below:

CASE 1 CASE 2



1. The first possible conflicting flow is the sum of the EB left-turn volume plus the largest WB main lane volume per lane. The WB lane may be composed of westbound through vehicles only, westbound through and right-turn vehicles, or westbound through, right-turn, and left-turn vehicles in the case of a single-lane approach. The second possible conflicting flow is the sum of the WB left-turn volume and the largest EB main-lane volume. The larger of the two sums is the critical volume for the east-west street.

In considering conflicting volumes, use the following guidelines:

- Left turns made from a separate lane are not considered as part of opposing main-lane traffic.
- Left turns made from a single approach lane designed such that through traffic cannot readily bypass left-turning traffic should be considered as contributing to opposing volume, based on their predicted PCE volumes. Any unsatisfied left-turn volume (that left-turn volume which exceeds any opposing left-turn volume made from a single-lane approach) is considered to conflict with the opposing left-thru movement.
- Right-turn volumes in a separate lane are usually not considered to conflict with opposing left turns. Only main lanes composed partly of through vehicles should be used in computing the sum of critical lane volumes.

2. Repeat the above process for the north-south street, identifying the critical lane volumes, and entering them in the appropriate boxes provided at the bottom of the Planning Worksheet.

3. Determine the sum of critical lane volumes for the intersection by adding the critical lane volumes for the east-west and north-south streets, and record in the appropriate box provided on the Planning Worksheet.

Step 7—Check Intersection Capacity

As discussed in the "Methodology" section of this chapter, the sum of critical lane volumes may be checked against the capacity criteria of Table 9-14 to determine the likelihood that capacity will be exceeded at the intersection. Three results are possible:

1. Traffic demand is expected to be *under* the physical capacity of the intersection. Excessive delays are not anticipated.
2. Traffic demand is expected to be *near* the physical capacity of the intersection. The ranges of estimated demand and capacity are such that true demand may be more or less than the capacity of the intersection. Unstable traffic flow having a wide range of delay is possible.
3. Traffic demand is expected to be *over* the physical capacity of the intersection. Excessive delays are anticipated during the analysis period.

The results of the planning analysis give a general indication of the acceptability of the capacity of the intersection for a forecast future demand condition. Where results are unacceptable, the need for geometric modifications is indicated. State or local practice will determine the kinds of modifications that should be considered. Appendix I may also be consulted for some general suggestions on intersection geometrics.

The use of these procedures is illustrated with sample calculations in the next section.

PROCEDURES FOR OTHER ANALYSES

As noted in the "Methodology" section of this chapter, it is possible to sequence the computations of the "Operational Analysis" procedure to solve for (1) v/c ratios and/or service flow rates, (2) signalization, or (3) geometric features by starting with a known or desired level of service. In such computations, the steps of an operational analysis are rearranged in recognition of the fact that LOS, and therefore average stopped delay per vehicle, is a known quantity. Given knowledge of any two of the other three variables previously noted, the remaining variable may then be calculated.

Solutions for any of the above may be handled through iterative computations using the standard sequence of calculations. Delay results are then tabulated vs. various trial values of the variable of interest. It is also possible, though computationally difficult, to work "backwards" through the procedure, starting with a known delay. This is complex because relationships deal primarily with individual lane groups, and changes to one virtually always imply changes in the operation of others at the intersection. Further, geometric and signalization parameters must often change in relation to one another, such as an exclusive left-turn phase requiring an exclusive left-turn lane. Nevertheless, reverse computations are feasible and are best carried out using computer programs designed by the analyst for the specific objective in mind.

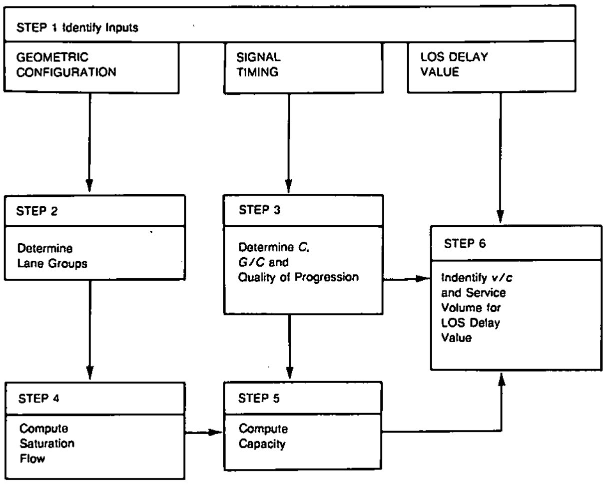
Figure 9-14 illustrates the computational path for such alternative analyses. In Figure 9-14(a), a v/c ratio or service flow rate is calculated for a given level of service. Calculations are made in the normal sequence through the computation of capacity for each lane group. Delay equations, however, are solved for a known delay commensurate with the selected LOS with the v/c ratio, X, as the unknown. Service flow rates may be computed as the v/c ratio times the capacity of the lane group.

In Figure 9-14(b), the signal timing for a given LOS (delay) is desired. In this case, computations through the saturation flow rate adjustment module are done in the normal sequence. As in all signal timing exercises, the phase plan must be established before computations are made. As indicated in the figure, however, determination of the signal timing for a given LOS requires some iterative calculations. This is because signal timing affects both capacity and delay, while capacity also impacts delay. Further, the delay equations include g/C, C, c, and X, all of which are influenced by signal timing. Thus, no one variable can be directly computed without checking its effect on the other. In this approach, signal timing is estimated based on the recommendations of Appendix II or local practice, and iterations are pursued to produce the desired delay value.

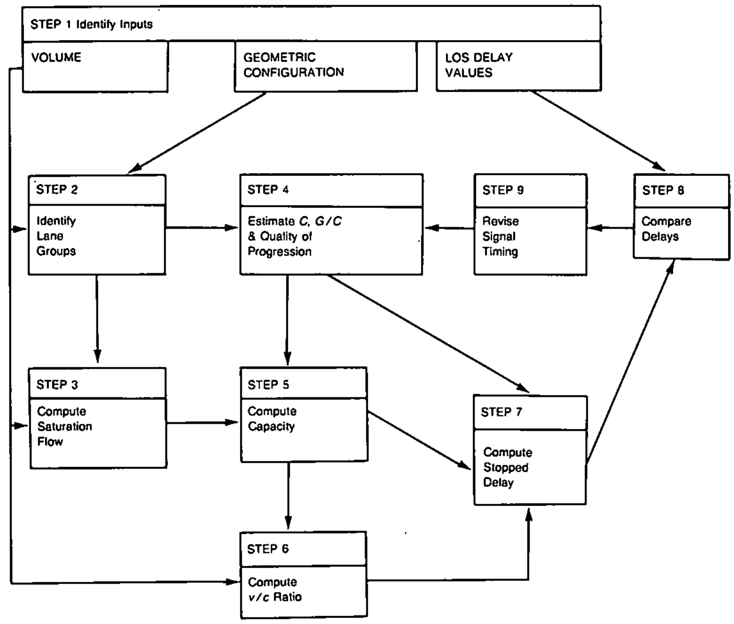
In Figure 9-14(c), the number of lanes in a given lane group is to be computed. This is also an interative process. For any given signal timing, the capacity of the lane group may be estimated using the delay equations (with c as the unknown). The delay equations, however, also require v/c ratios that depend heavily on capacity. Once again, therefore, it is more practical to iterate the number of lanes, comparing the resulting delay for several trial values.

The relative complexity of these other approaches makes a manual solution difficult, and is the reason this manual presents the operational analysis procedure in the mode of solving for LOS. A sample calculation is, however, included, illustrating how these alternative approaches may be accomplished.

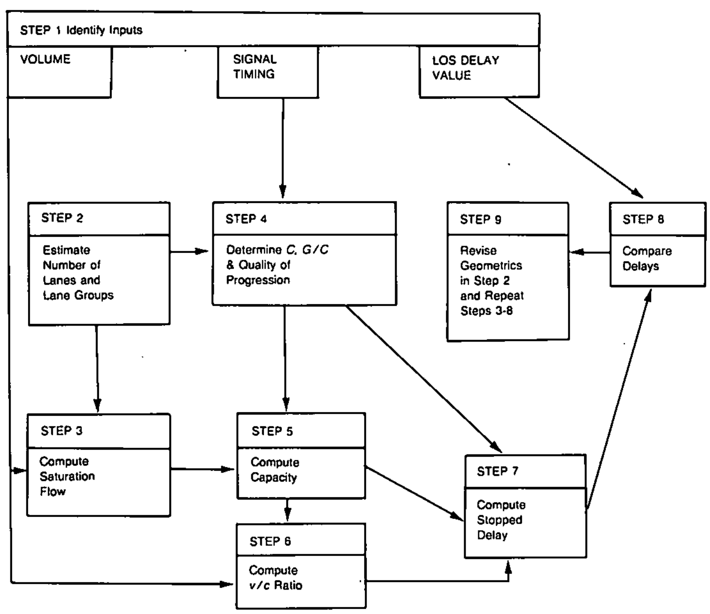
As with any analysis, v/c ratio and LOS must be considered as two important measures of performance. Any analysis yielding v/c ratio exceeding 1.00 should immediately trigger consideration of alternatives. High v/c ratios in the 0.95 to 1.00 range may also cause such consideration. This is an important point that can save a good deal of analysis effort. In many analyses (Figs. 9-14(b) and (c)), v/c ratios will be obtained before delays and LOS. If an intersection is operating in an unacceptable v/c range, completing computations to find delay and LOS may be a fruitless exercise.



(a) Determining v/c Ratios and Service Flow Rates



(b) Determining Signal Timing



(c) Determining Number of Lanes

Figure 9-14. Alternative computation using operational analysis.

IV. SAMPLE CALCULATIONS

CALCULATION 1—OPERATIONAL ANALYSIS OF AN EXISTING PRETIMED, TWO-PHASE SIGNAL

1. *Description*—The intersection of Third Avenue and Main Street is illustrated in Figure 9-15, which is the Input Module Worksheet for this calculation. It is a simple four-leg intersection with a two-phase, pretimed signal on a 70-sec cycle. Main Street has two lanes in each direction, while Third Avenue has one lane in each direction.

The objective is to analyze the capacity and level of service of the existing intersection for a projected set of future demand volumes that will result from new development in the area, and to recommend changes to the signal and/or geometric design if the current situation is not able to handle the new traffic in an acceptable manner.

2. *Solution*—The solution is discussed on a module-by-module basis, as follows:

a. Input module—The Input Module Worksheet for this calculation is shown in Figure 9-15. All relevant volumes and geometric conditions are illustrated in the diagram on the upper half of the worksheet. Note that turning volumes are not extremely high, but that general volume levels are heavy.

Other relevant characteristics are shown in the center of the worksheet. The intersection is on level grade, traffic includes 5 percent heavy vehicles on Main Street and 8 percent on Third Avenue, and there are no buses or parking lanes on any of the approaches. Pedestrian volumes are estimated to be 100 peds/hr in all crosswalks, and the peak-hour factor is 0.90 for all approaches. As there are no pedestrian push-buttons, the minimum green time for pedestrians may be computed from Eq. 9-5:

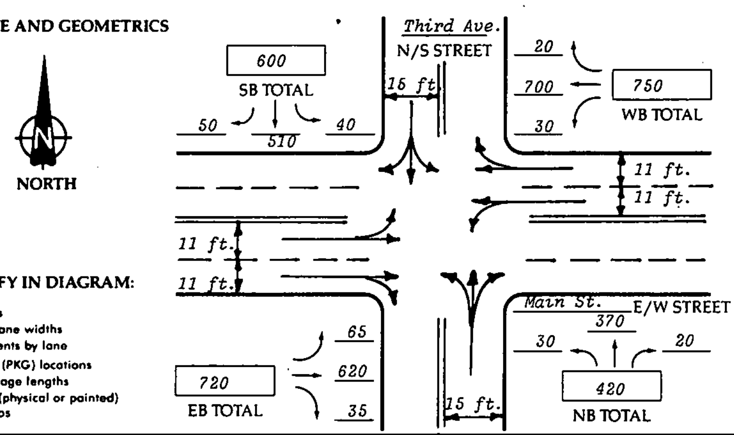
INPUT WORKSHEET

Intersection: *Third Ave. and Main St.* Date: *8/12/85*

Analyst: *RPR* Time Period Analyzed: *4-5 PM* Area Type: ☒ CBD ☐ Other

Project No.: City/State: *Central City*

VOLUME AND GEOMETRICS



IDENTIFY IN DIAGRAM:

1. Volumes
2. Lanes, lane widths
3. Movements by lane
4. Parking (PKG) locations
5. Bay storage lengths
6. Islands (physical or painted)
7. Bus stops

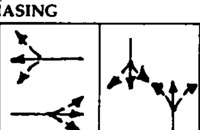
TRAFFIC AND ROADWAY CONDITIONS

Approach	Grade (%)	% HV	Adj. Pkg. Lane Y or N	Adj. Pkg. Lane N_m	Buses (N_B)	PHF	Conf. Peds. (peds./hr)	Pedestrian Button Y or N	Pedestrian Button Min. Timing	Arr. Type
EB	0	5	N	-	0	0.90	100	N	9.8	4
WB	0	5	N	-	0	0.90	100	N	9.8	2
NB	0	8	N	-	0	0.90	100	N	13.8	3
SB	0	8	N	-	0	0.90	100	N	13.8	3

Grade: + up, − down
HV: veh. with more than 4 wheels
N_m: pkg. maneuvers/hr
N_B: buses stopping/hr
PHF: peak-hour factor
Conf. Peds: Conflicting peds./hr
Min. Timing: min. green for pedestrian crossing
Arr. Type: Type 1-5

PHASING

DIAGRAM



Timing	G = 27 Y+R = 43	G = 37 Y+R = 33	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =
Pretimed or Actuated	P	P						

Protected turns | Permitted turns | ------- Pedestrian | Cycle Length *70* Sec

Figure 9-15. Input module worksheet for Calculation 1.

$$G_p = 7.0 + W/4.0 - Y$$

$$G_p(\text{Main St.}) = 7.0 + 23/4.0 - 3.0 = 9.8 \text{ sec}$$

$$G_p(\text{Third Ave.}) = 7.0 + 39/4.0 - 3.0 = 13.8 \text{ sec}$$

The arrival types are also given: Main Street is on a progression plan that favors the EB approach (Type 4) and disadvantages the WB approach (Type 2). Third Avenue has essentially random arrivals (Type 3).

The signal timing is illustrated at the bottom of Figure 9-15. It shows a simple two-phase plan, with Main Street receiving 27 sec of green time and Third Avenue receiving 37 sec of green time in a 70-sec cycle.

b. Volume adjustment module—The Volume Adjustment Module Worksheet for this calculation is shown in Figure 9-16.

Movement volumes are entered in column 3 from the Input Worksheet. Each is divided by the PHF of 0.90 to produce the movement flow rate indicated in column 5.

At this point, the lane groups for analysis must be established. Clearly, the NB and SB approaches form one lane group each, as only one lane is present. For the EB and WB approaches, it is now necessary to determine whether or not equilibrium shared-lane operation exists for the left turn, or whether a *de facto* left-turn lane exists. This is done following the procedure outlined in the "Methodology" section of the chapter.

First, the approximate equivalent through flow rate is computed for both EB and WB left turns, using Eq. 9-6. It is assumed that under the worst conditions this flow fully occupies the left-most lane of the approach:

$$v_{LE} = v_L\,[1{,}800/(1{,}400 - v_o)]$$

As neither approach includes a left-turn or single lane, v_o is taken to be the total flow on the opposing approach, including left and right turns. Thus:

$$v_{LE}(\text{EB}) = 72\,[1{,}800/(1{,}400 - 833)] = 229 \text{ vph}$$

VOLUME ADJUSTMENT WORKSHEET										
① Appr.	② Mvt.	③ Mvt. Volume (vph)	④ Peak Hour Factor PHF	⑤ Flow Rate v_p (vph) ③ ÷ ④	⑥ Lane Group	⑦ Flow rate in Lane Group v_g (vph)	⑧ Number of Lanes N	⑨ Lane Utilization Factor U Table 9-4	⑩ Adj. Flow v (vph) ⑦ × ⑨	⑪ Prop. of LT or RT P_{LT} or P_{RT}
	LT	65	0.90	72						
EB	TH	620	0.90	689		800	2	1.05	840	.09 LT .05 RT
	RT	35	0.90	39						
	LT	30	0.90	33						
WB	TH	700	0.90	778		833	2	1.05	875	.04 LT .03 RT
	RT	20	0.90	22						
	LT	30	0.90	33						
NB	TH	370	0.90	411		466	1	1.00	466	.07 LT .05 RT
	RT	20	0.90	22						
	LT	40	0.90	44						
SB	TH	510	0.90	567		667	1	1.00	667	.07 LT .08 RT
	RT	50	0.90	56						

Figure 9-16. Volume adjustment module worksheet for Calculation 1.

SATURATION FLOW ADJUSTMENT WORKSHEET												
LANE GROUPS				ADJUSTMENT FACTORS								
① Appr.	② Lane Group Movements	③ Ideal Sat. Flow (pcphgpl)	④ No. of Lanes N	⑤ Lane Width f_w Table 9-5	⑥ Heavy Veh f_{HV} Table 9-6	⑦ Grade f_g Table 9-7	⑧ Pkg. f_p Table 9-8	⑨ Bus Blockage f_{bb} Table 9-9	⑩ Area Type f_a Table 9-10	⑪ Right Turn f_{RT} Table 9-11	⑫ Left Turn f_{LT} Table 9-12	⑬ Adj. Sat. Flow Rate s (vphg)
EB		1800	2	0.97	0.975	1.00	1.00	1.00	0.90	0.99	0.75	2275
WB		1800	2	0.97	0.975	1.00	1.00	1.00	0.90	0.99	0.85	2579
NB		1800	1	1.10	0.96	1.00	1.00	1.00	0.90	0.89	0.86	1309
SB		1800	1	1.10	0.96	1.00	1.00	1.00	0.90	0.89	0.95	1446

Figure 9-17. Saturation flow rate module worksheet for Calculation 1.

$$v_{LE}(\text{WB}) = 33\ [1{,}800/(1{,}400 - 800)] = 99 \text{ vph}$$

For equilibrium to exist, these values must be less than the average flow per lane assuming that all right-turning and through vehicles must use remaining approach lanes. Thus:

$$v_{LE} < (v - v_L)/(N - 1), \text{ and:}$$

$$229 < (800 - 72)/(2 - 1) = 728 \text{ vphpl} \quad \text{OK}$$

$$99 < (833 - 33)/(2 - 1) = 800 \text{ vphpl} \quad \text{OK}$$

Therefore, equilibrium will exist on both the EB and WB approaches, and a single lane group is established for each. These computations use the flow rates of column 5 on the Volume Adjustment Worksheet. The lane group flow rate (sum of the three movements in column 5 in this example), v_g, is entered in column 7.

The lane utilization factor is selected from Table 9-4. For Third Avenue, with one-lane approaches, the value is 1.00, while for Main Street, with two-lane approaches, the value is 1.05. These are entered in column 9 of the worksheet, and are multiplied by the flow rates of column 7 to produce the adjusted flow rates of column 10, that is: $v = v_g \times U$.

The proportion of left and/or right turns in the lane group is computed by taking the turning flow rates of column 5 and dividing by the total unadjusted flow in the lane group from column 7. These values are generally rounded to the nearest 0.01 for use in analysis.

c. Saturation flow rate module—The worksheet for the saturation flow rate module is shown in Figure 9-17.

Lane group descriptions are repeated in column 2 of the worksheet. The ideal saturation flow rate will be assumed to be the usual value of 1,800 pcphgpl. The columns that follow contain the number of lanes in the lane group and all adjustments to the ideal saturation flow rate, as follows:

- No. of lanes. EB and WB lane groups have two lanes. NB and SB lane groups have one lane.
- Lane width factor. This factor is selected from Table 9-5. For the 11-ft lanes on Main Street, the value is 0.97; for the 15-ft lanes on Third Avenue, the value is 1.10.
- Heavy vehicle factor. This factor is found in Table 9-6. For 5 percent heavy vehicles on Main Street, the value is 0.975 (interpolated between 0.97 and 0.98); for 8 percent heavy vehicles on Third Avenue, the value is 0.96.
- Grade factor. Selected from Table 9-7, all values are 1.00, because the grades are level for all approaches.
- Parking factor. Found in Table 9-8, all values are 1.00, because no parking lanes exist on any approach.
- Bus blockage factor. This factor is taken from Table 9-9; because there are no buses on any approach, all values are 1.00.
- Area type factor. This factor is taken from Table 9-10, and is 0.90, because the intersection is located in a CBD.
- Right-turn factor. This factor is taken from Table 9-11. Right turns on Main Street are permissive from shared lanes (Case 5 in Table 9-11), while those from Third Avenue are on single-lane approaches (Case 7 in Table 9-11). The factor is based on the proportion of right turns in the lane group (see Fig. 9-16) and the number of conflicting pedestrians per hour (100 for all approaches, see Fig. 9-15).
- Left-turn factor. Left turns from all four approaches are permissive; left-turn factors must be computed using the special procedures for such cases. The supplemental worksheet used for these computations is shown in Figure 9-18.

Although lengthy, the worksheet is self-explanatory. Input variables are entered for each approach. They are selected from the Input Worksheet and the Volume Adjustment Worksheet. Note that as single-lane approaches, the left-turn flow is discounted when computing mainline and opposing flows ($v_m + v_o$) for the NB and SB approaches. The resulting factors are entered on the Saturation Flow Rate Worksheet.

The ideal saturation flow rate is multiplied by all adjustments, with the resulting saturation flow rates for prevailing conditions shown in the last column of the Saturation Flow Rate Worksheet.

SUPPLEMENTAL WORKSHEET FOR LEFT-TURN ADJUSTMENT FACTOR, f_{LT}				
INPUT VARIABLES	EB	WB	NB	SB
Cycle Length, C (sec)	70	70	70	70
Effective Green, g (sec)	27	27	37	37
Number of Lanes, N	2	2	1	1
Total Approach Flow Rate, v_a (vph)	800	833	466	667
Mainline Flow Rate, v_M (vph)	800	833	433	623
Left-Turn Flow Rate, v_{LT} (vph)	72	33	33	44
Proportion of LT, P_{LT}	0.09	0.04	0.07	0.07
Opposing Lanes, N_o	2	2	1	1
Opposing Flow Rate, v_o (vph)	833	800	623	433
Prop. of LT in Opp. Vol., P_{LTO}	0.04	0.09	0.07	0.07
COMPUTATIONS	EB	WB	NB	SB
$S_{op} = \dfrac{1800\,N_o}{1 + P_{LTO}\left[\dfrac{400 + v_M}{1400 - v_M}\right]}$	3333	3012	1698	1648
$Y_o = v_o / S_{op}$	0.250	0.266	0.367	0.263
$g_u = (g - CY_o) / (1 - Y_o)$	12.67	11.42	17.87	25.24
$f_s = (875 - 0.625\,v_o) / 1000$	0.354	0.375	—	—
$P_L = P_{LT}\left[1 + \dfrac{(N-1)\,g}{f_s g_u + 4.5}\right]$	0.360	0.163	0.070	0.070
$g_q = g - g_u$	14.33	15.58	19.13	11.76
$P_T = 1 - P_L$	0.640	0.837	0.930	0.930
$g_f = 2\,\dfrac{P_T}{P_L}\left[1 - P_T^{\,0.5\,g_q}\right]$	3.41	7.70	13.29	9.22
$E_L = 1800 / (1400 - v_o)$	3.17	3.00	2.32	1.86
$f_m = \dfrac{g_f}{g} + \dfrac{g_u}{g}\left[\dfrac{1}{1 + P_L(E_L - 1)}\right] + \dfrac{2}{g}(1 + P_L)$	0.490	0.690	0.859	0.950
$f_{LT} = (f_m + N - 1) / N$	0.75	0.85	0.86	0.95

Figure 9-18. Supplemental worksheet for computation of left-turn adjustment factors for Calculation 1.

d. Capacity analysis module—The Capacity Analysis Module Worksheet is shown in Figure 9-19.

Lane group descriptions are again repeated in column 2 of the worksheet. In the subsequent two columns, the adjusted flow for each lane group is entered from the Volume Adjustment Worksheet (Fig. 9-16), and the saturation flow rate is entered from the Saturation Flow Rate Worksheet (Fig. 9-17). From these values, flow ratios are computed as v/s, and entered in column 5 of the worksheet.

At this point, a search is made for the critical lane groups. For Main Street, the E-W street, all lane groups move on the same phase. Thus, the maximum flow ratio among the four EB and WB lane groups is critical for the first signal phase. This value is the 0.369 ratio on the EB lane group. For Third Avenue, both lane groups move on the same phase, and the critical lane group is the one with the highest flow ratio on the N-S street. This is the SB approach, which has a flow ratio of 0.437. Thus, the sum of critical lane flow ratios is 0.369 + 0.437 = 0.806.

Green ratios are entered in column 6 of the worksheet, and are found by dividing the effective green time for the lane group by the cycle length. For this calculation, it is assumed that the effective green is equal to the actual green, and:

$$g/C \text{ (Main St.)} = 27/70 = 0.386$$
$$g/C \text{ (Third Ave.)} = 37/70 = 0.528$$

Lane group capacities are computed by multiplying the green ratio, g/C, by the saturation flow rate for the lane group, s. Finally, the v/c ratio, X, for each lane group is computed by dividing the adjusted lane group flow rate, v, by the capacity of the lane group, c.

Intersection values are computed at the bottom of Figure 9-19. As the effective green time was assumed equal to actual green time, the lost time is taken as equal to the change intervals, which are assumed to be 3.0 sec per phase. Thus, the cycle length is 70 sec, with 6 sec of lost time per cycle.

The critical v/c ratio, X_c, is computed by Eq. 9-3, shown on the worksheet, as:

$$X_c = 0.806(70)/(70 - 6) = 0.881$$

CAPACITY ANALYSIS WORKSHEET

LANE GROUP ① Appr.	② Lane Group Movements	③ Adj. Flow Rate v (vph)	④ Adj. Sat. Flow Rate s (vphg)	⑤ Flow Ratio v/s ③÷④	⑥ Green Ratio g/C	⑦ Lane Group Capacity c (vph) ④×⑥	⑧ v/c Ratio X ③÷⑦	⑨ Critical ? Lane Group
EB		840	2275	0.369	0.386	878	0.956	✓
WB		875	2579	0.339	0.386	995	0.879	
NB		466	1309	0.357	0.528	692	0.673	
SB		667	1446	0.461	0.528	765	0.872	✓

Cycle Length, C 70 sec

$\sum_i (v/s)_{ci} =$ 0.835

Lost Time Per Cycle, L 6 sec

$$X_c = \frac{\sum_i (v/s)_{ci} \times C}{C - L} = 0.84$$

Figure 9-19. Capacity analysis module worksheet for Calculation 1.

The results of this module indicate that the existing signal and geometric design of the intersection will be adequate to handle the projected demands, albeit at a reasonable high v/c ratio. The intersection operates at about 90 percent of its capacity for the critical movements, and the EB approach operates dangerously near its capacity, $X = 0.956$.

Given these results, it may be reasoned that green time could be reallocated to produce more equitable operations on all critical movements, but that there is little "room to spare" on any of the critical approach lane groups. Given this result, delay and level of service on these approaches are now considered.

e. Level-of-service module—The Level-of-Service Module Worksheet is shown in Figure 9-20. Lane group descriptions are entered in column 2.

Values of X, g/C, C, and c are entered into the appropriate columns, as these will be needed to compute delay. They are obtained from the capacity analysis worksheet.

The first-term delay is computed from the first term of Eq. 9-18:

$$d_1 = 0.38\,C\,(1 - g/C)^2/[1 - (g/C)\,(X)]$$

$$d_1\,(\text{EB}) = 0.38(70)\,(1 - 0.386)^2/[1 - (0.386)\,(0.956)] = 15.89 \text{ sec/veh}$$

$$d_1\,(\text{WB}) = 0.38(70)\,(1 - 0.386)^2/[1 - (0.386)\,(0.879)] = 15.18 \text{ sec/veh}$$

$$d_1\,(\text{NB}) = 0.38(70)\,(1 - 0.528)^2/[1 - (0.528)\,(0.673)] = 9.16 \text{ sec/veh}$$

$$d_1\,(\text{SB}) = 0.38(70)\,(1 - 0.528)^2/[1 - (0.528)\,(0.822)] = 10.95 \text{ sec/veh}$$

The second term delay is computed from the second term of Eq. 9-18:

$$d_2 = 173\,X^2\,[(X - 1) + \sqrt{(X - 1)^2 + (16X/c)}]$$

$$d_2\,(\text{EB}) = 15.04 \text{ sec/veh } (X = 0.956,\ c = 878)$$

$$d_2\,(\text{WB}) = 6.50 \text{ sec/veh } (X = 0.879,\ c = 995)$$

$$d_2\,(\text{NB}) = 1.80 \text{ sec/veh } (X = 0.673,\ c = 753)$$

$$d_2\,(\text{SB}) = 7.64 \text{ sec/veh } (X = 0.872,\ c = 807)$$

These values are entered in the appropriate columns of the

LEVEL-OF-SERVICE WORKSHEET

Lane Group		First Term Delay				Second Term Delay				Total Delay & LOS		
① Appr.	② Lane Group Movements	③ v/c Ratio X	④ Green Ratio g/C	⑤ Cycle Length C (sec)	⑥ Delay d_1 (sec/veh)	⑦ Lane Group Capacity c (vph)	⑧ Delay d_2 (sec/veh)	⑨ Progression Factor PF Table 9-13	⑩ Lane Group Delay (sec/veh) (⑥+⑧) × ⑨	⑪ Lane Group LOS Table 9-1	⑫ Approach Delay (sec/veh)	⑬ Appr. LOS Table 9-1
EB		.956	.386	70	15.89	878	15.04	0.90	27.84	D	27.84	D
WB		.879	.386	70	15.18	995	6.50	1.18	25.58	D	25.58	D
NB		.673	.528	70	9.16	753	1.80	1.00	10.96	B	10.96	B
SB		.872	.528	70	10.95	807	7.64	1.00	18.59	C	18.59	C

Intersection Delay 22.22 sec/veh Intersection LOS C (Table 9-1)

Figure 9-20. Level-of-service module worksheet for Calculation 1.

Level of Service Worksheet. Progression factors are now selected from Table 9-13. For the EB approach, the factor is 0.90 (arrival type 4); for the WB approach, 1.18 (arrival type 2); and for the NB and SB approaches, 1.00 (arrival type 3). The EB and WB factors depend on the v/c ratio. Values were selected for a v/c ratio of 1.0, because both v/c ratios are above 0.80, the next lower category. Intermediate values could be interpolated, but the accuracy of the delay prediction usually does not warrant this precision.

The delay in each lane group is now computed as:

$$\text{Delay} = (d_1 + d_2)\ \text{PF}$$

$$\text{Delay (EB)} = (15.89 + 15.04)(0.90) = 27.84,$$
Say 27.8 sec/veh

$$\text{Delay (WB)} = (15.18 + 6.50)(1.18) = 25.58,$$
Say 25.6 sec/veh

$$\text{Delay (NB)} = (9.16 + 1.80)(1.00) = 10.96,$$
Say 11.0 sec/veh

$$\text{Delay (SB)} = (10.95 + 7.64)(1.00) = 18.59,$$
Say 18.6 sec/veh

The average stopped delay per vehicle for the intersection as a whole is now computed as a weighted average of the values for each approach:

$$\text{Delay (Int.)} = [(840 \times 27.84) + (875 \times 25.58) + (466 \times 10.96) + (667 \times 18.59)]/[840 + 875 + 466 + 667]$$

$$\text{Delay (Int.)} = 22.22, \text{ Say } 22.2 \text{ sec/veh}$$

Levels of service may be assigned by comparing the computed delay values with the criteria of Table 9-1.

It is seen from these results that the intersection as a whole operates at LOS C, with individual approaches operating a range of LOS from B to D. The EB approach, which has the highest v/c ratio (0.956) also has the highest delay (27.84 sec/veh). The delay value, however, is well below the LOS E boundary of 60 sec/veh, and does not in itself suggest that the approach is virtually at its capacity under the projected demands.

Given these results, some reallocation of green time is called for, both to reduce the v/c ratio in the EB lane to a value closer to the critical v/c for the intersection (0.84), and to reduce delay on the approach. Given the critical v/c ratio of 0.84, the cycle and phase plan do not allow for any significant reallocations. A phase plan incorporating short leading and lagging greens for the left turns on Main Street might also be considered, although these would doubtless lead to the requirement of a longer cycle length.

CALCULATION 2—OPERATIONAL ANALYSIS OF A THREE-PHASE, PRETIMED SIGNAL

1. *Description*—The intersection of Sixth Street and Western Blvd. is expected to experience increased demand because of proposed developments in the area. Sixth Street is a one-way facility having two moving lanes and two parking lanes. Western Blvd. is a divided arterial with no parking lanes, with two moving lanes in each direction, and a left-turn lane for the EB approach. The intersection and the anticipated demand volumes are shown in Figure 9-21, the Input Module Worksheet for the calculation.

A retiming of the existing three-phase signal plan is to be attempted, and a determination made if this will be sufficient to accommodate the anticipated demand without geometric improvements.

2. *Solution*—The solution is described on a module-by-module basis, as follows:

a. Input module—The Input Module Worksheet is shown in Figure 9-21. The upper portion of the worksheet shows existing geometric conditions and the expected demand volumes, with turning movements. Note that there are bus stops on Western Blvd. located within the confines of the intersection.

The central portion of the worksheet contains additional geometric and traffic data. Note that the NB approach is on a 2 percent downgrade, and that 20 parking maneuvers per hour are experienced from parking lanes on the NB approach. Twenty buses per hour use each bus stop on the EB and WB approaches. The PHF is 0.95 for all approaches. Pedestrian volumes are assumed to be light, and the default value of 50 peds/hr is used for all approaches. Because there are no pedestrian push-buttons, the minimum green times for pedestrians are computed from Eq. 9-5:

$$G_p = 7.0 + W/4.0 - Y$$

$$G_p \text{ (Western Blvd, E-W)} = 7.0 + 38/4.0 - 3.0 = 13.5 \text{ sec}$$

$$G_p \text{ (Sixth Street, N-S)} = 7.0 + 54/4.0 - 3.0 = 17.5 \text{ sec}$$

The signal is not coordinated in any progressive system, and the arrival type is therefore taken to be Type 3.

The signal phase plan is shown on the lower portion of the Input Module Worksheet. It shows a leading green for the EB flow, followed by an EB-WB through phase, during which permitted EB left turns are continued. The third phase is for all NB movements.

b. Volume adjustment module—The Volume Adjustment Module Worksheet is shown in Figure 9-22. Movement volumes are entered in column 3 of the worksheet and divided by the PHF to obtain the movement flow rates of column 5. The establishment of lane groups may now be considered.

The EB approach must have a separate lane group for the exclusive LT lane. The two RT/TH lanes would form a separate lane group. The WB and NB approaches can both be represented by single lane groups, because neither involves opposed left turns that might prevent an equilibrium lane distribution from occurring. Lane groups are indicated in column 6 of the worksheet.

The total flow rate in each lane group v_g is entered in column 7 of the worksheet. All EB left turns are assigned to the EB LT lane group.

The number of lanes in each lane group is entered in column 8 of the worksheet. The EB LT lane group has one lane, while all other groups consist of two lanes.

The lane use factor from Table 9-4 is entered in column 9 of the worksheet. For one-lane lane groups, the factor is 1.00; for two-lane lane groups, it is 1.05.

The adjusted flow rate for each lane group is computed in column 10 of the worksheet. The lane group flow rate is multiplied by the lane use factor, U. The last column contains the proportions of left and/or right turns in each lane group, rounded to the nearest 0.01.

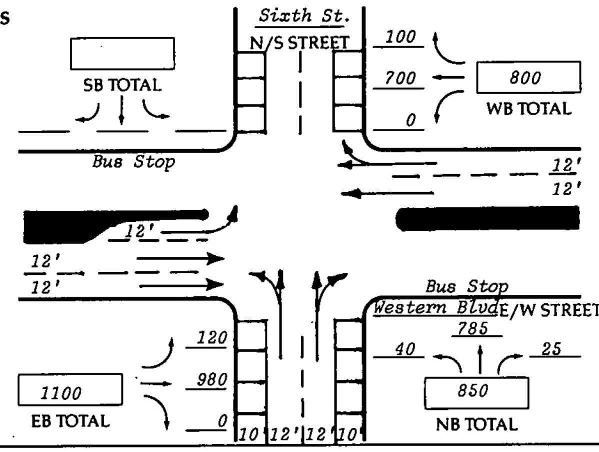
INPUT WORKSHEET

Intersection: *Sixth St. and Western Blvd.* Date: *9/27/85*

Analyst: *RPR* Time Period Analyzed: *8-9 AM* Area Type: ☐ CBD ☒ Other

Project No.: City/State: *Metropolis*

VOLUME AND GEOMETRICS



IDENTIFY IN DIAGRAM:

1. Volumes
2. Lanes, lane widths
3. Movements by lane
4. Parking (PKG) locations
5. Bay storage lengths
6. Islands (physical or painted)
7. Bus stops

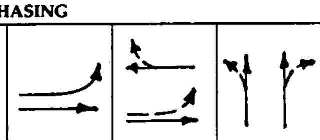
TRAFFIC AND ROADWAY CONDITIONS

Approach	Grade (%)	% HV	Adj. Pkg. Lane Y or N	Adj. Pkg. Lane N_m	Buses (N_B)	PHF	Conf. Peds. (peds./hr)	Pedestrian Button Y or N	Pedestrian Button Min. Timing	Arr. Type
EB	0	10	N	-	20	0.95	50	N	13.5	3
WB	0	10	N	-	20	0.95	50	N	13.5	3
NB	-2	5	Y	20	0	0.95	50	N	17.5	3
SB	-	-	-	-	-	-	-	-	-	-

Grade: + up, − down
HV: veh. with more than 4 wheels
N_m: pkg. maneuvers/hr
N_B: buses stopping/hr
PHF: peak-hour factor
Conf. Peds: Conflicting peds./hr
Min. Timing: min. green for pedestrian crossing
Arr. Type: Type 1-5

PHASING

DIAGRAM



Timing	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =
Pretimed or Actuated	P	P	P					

Protected turns | Permitted turns | ------- Pedestrian | Cycle Length ____ Sec

Figure 9-21. Input module worksheet for Calculation 2.

c. Saturation flow rate module—The worksheet for the saturation flow rate module is shown in Figure 9-23. The lane group descriptions are repeated in column 2 of the worksheet. The third column contains the ideal saturation flow rate for each lane group. The usual value of 1,800 pcphgpl will be used for all lane groups in this computation. Subsequent columns record the number of lanes in the lane group and adjustments to the ideal saturation flow rate:

- No. of lanes. The number of lanes in each lane group is recorded in the fourth column of the worksheet.
- Lane width factor. The lane width adjustment factor is obtained from Table 9-5. All lane widths are 12 ft; therefore all factors are 1.00.
- Heavy vehicle factor. A heavy vehicle factor is selected from Table 9-6 for 10 percent heavy vehicles on the EB and WB approaches (0.950), and for 5 percent heavy vehicles on the NB approach (0.975).
- Grade factor. The grade factor is selected from Table 9-7. It is 1.00 for the level grade of the EB and WB approaches, and 1.01 for the 2 percent downgrade of the NB approach.
- Parking factor. The parking factor is selected from Table 9-8. For the EB and WB approaches, where there is no parking, the factor is 1.00. For the NB approach, the factor is 0.89 for a two-lane group and 20 parking maneuvers per hour.
- Bus blockage factor. The bus blockage factor is 0.96 for the EB TH/RT and WB lane groups, as each has 20 buses per hour. The factor is 1.00 for other maneuvers where no buses exist. These factors are obtained from Table 9-9.
- Area type factor. The area type factor is selected from Table 9-10 for a non-CBD location. The factor is 1.00 for all movements.
- Right-turn factor. The right-turn factor is selected from Table 9-11. For the EB TH/RT, WB, and NB lane groups, Case 5 (for permitted right turns from shared lanes) is used.

VOLUME ADJUSTMENT WORKSHEET										
① Appr.	② Mvt.	③ Mvt. Volume (vph)	④ Peak Hour Factor PHF	⑤ Flow Rate v_p (vph) ③÷④	⑥ Lane Group	⑦ Flow rate in Lane Group v_g (vph)	⑧ Number of Lanes N	⑨ Lane Utilization Factor U Table 9-4	⑩ Adj. Flow v (vph) ⑦×⑨	⑪ Prop. of LT or RT P_{LT} or P_{RT}
EB	LT	120	0.95	126		126	1	1.00	126	1.00
	TH	980	0.95	1032		1032	2	1.05	1084	0.0 LT 0.0 RT
	RT									
WB	LT									
	TH	700	0.95	737		842	2	1.05	884	0.0 LT .125 RT
	RT	100	0.95	105						
NB	LT	40	0.95	42						
	TH	785	0.95	826		894	2	1.05	939	.05 LT .03 RT
	RT	25	0.95	26						
SB	LT									
	TH									
	RT									

Figure 9-22. Volume adjustment module worksheet for Calculation 2.

The factor is based on the proportion of right-turners in each lane group (obtained from the Volume Adjustment Worksheet), and the number of conflicting predestrians per hour (50 for all approaches).

- Left-turn factor. Left-turn factors are found from Table 9-12. For the EB LT lane group, Case 3 (exclusive lanes, protected + permitted phasing) is used. For the NB approach, a unique situation arises. As a one-way street, the "permitted" left turn does not have an opposing vehicular flow, but merely an opposing pedestrian flow. For this case, the equivalent *right-turn factor* from Table 9-11 would be used (Case 5).

The ideal saturation flow rate is multiplied by all of the above adjustments, with the result entered in the last column of the worksheet.

d. Capacity analysis module—In this module, flow ratios will be computed which will allow a retiming of the signal to determine appropriate green times. Once the timing is estimated, capacities and v/c ratios for each lane group may be computed. The worksheet for this module is shown in Figure 9-24.

Lane group descriptions are repeated in column 2 of the worksheet. Note, however, that the protected + permitted LT phase is divided on the worksheet, although all demand is initially assumed to take place in the protected portion of the phase. In column 3 of the worksheet, adjusted flows for each lane group are entered from the Volume Adjustment Worksheet. In column 4, saturation flow rates are entered from the Saturation Flow Rate Worksheet. Column 5 computes the flow ratio for each lane group as the adjusted flow divided by the saturation flow rate.

At this point, critical lane groups are identified. For the leading green phase plan shown, the sum of critical lane groups may be either: EB LT + WB + NB *or* EB TH/RT + NB.

The maximum sum occurs when the EB LT, WB, and NB lane groups are used, and these are identified as the critical lane groups. The sum of critical lane group flow ratios is then computed as 0.078 + 0.275 + 0.304 = 0.657.

The signal timing may now be estimated. The procedure recommended in Appendix II will be used. It will be assumed that lost time equals the change intervals, or 3 sec per phase.

SATURATION FLOW ADJUSTMENT WORKSHEET												
LANE GROUPS				ADJUSTMENT FACTORS								
① Appr.	② Lane Group Movements	③ Ideal Sat. Flow (pcphgpl)	④ No. of Lanes N	⑤ Lane Width f_w Table 9-5	⑥ Heavy Veh f_{HV} Table 9-6	⑦ Grade f_g Table 9-7	⑧ Pkg. f_p Table 9-8	⑨ Bus Blockage f_{bb} Table 9-9	⑩ Area Type f_a Table 9-10	⑪ Right Turn f_{RT} Table 9-11	⑫ Left Turn f_{LT} Table 9-12	⑬ Adj. Sat. Flow Rate s (vphg)
		1800	1	1.00	0.95	1.00	1.00	1.00	1.00	1.00	0.95	1625
EB		1800	2	1.00	0.95	1.00	1.00	0.96	1.00	1.00	1.00	3283
WB		1800	2	1.00	0.95	1.00	1.00	0.96	1.00	0.98	1.00	3218
NB		1800	2	1.00	0.975	1.01	0.89	1.00	1.00	0.99	0.99	3092
SB												

Figure 9-23. Saturation flow rate module worksheet for Calculation 2.

The signalization plan includes three phases. The leading green for the EB left turn, however, is permitted to proceed in the following through phase. Thus, there is no change interval for this phase, and the cycle contains a total of 2 × 3, or 6 sec of lost time.

The cycle length is estimated from Eq. II.9-1:

$$C = L\,X_c/\left[X_c - \sum_i (v/s)_{ci}\right]$$

The minimum, cycle length is obtained by using a value of $X_c = 1.0$, or

$$C = 6\ (1.0)/[1.0 - 0.657] = 17.49 \text{ sec}$$

If a critical v/c ratio of 0.80 were desired, the cycle length would have to be at least:

$$C = 6\ (0.80)/[0.80 - 0.657] = 33.6 \text{ sec}$$

As this is still rather short for a three-phase signalization, a critical v/c ratio of 0.75 will be attempted:

$$C = 6\ (0.75)/[0.75 - 0.657] = 48.4 \text{ sec}$$

This appears to be reasonable, and a cycle length of 50 sec will be adopted. Cycle lengths are generally available in even increments of 5 or 10 sec. If 50 sec is used, the exact critical v/c ratio will be:

$$X_c = 0.657\ (50)/(50 - 6) = 0.746$$

Green times may be estimated from Eq. II.9-2. A policy of timing each critical movement to an equal v/c ratio of 0.746 will be adopted.

$$g_i = v_i\,C/s_i\,X_i = (v/s)_i\,(C/X_i)$$

and:

CAPACITY ANALYSIS WORKSHEET

LANE GROUP ① Appr.	② Lane Group Movements	③ Adj. Flow Rate v (vph)	④ Adj. Sat. Flow Rate s (vphg)	⑤ Flow Ratio v/s ③÷④	⑥ Green Ratio g/C	⑦ Lane Group Capacity c (vph) ④×⑥	⑧ v/c Ratio X ③÷⑦	⑨ Critical ? Lane Group
EB		126	1625	0.078	0.104	169	0.745	✓
		1084	3283	0.330	0.472	1550	0.699	
WB								
		884	3218	0.275	0.368	1184	0.746	✓
NB								
		939	3092	0.304	0.408	1262	0.744	✓
SB								

Cycle Length, C 50 sec

Lost Time Per Cycle, L 6 sec

$\Sigma (v/s)_{ci} =$ 0.657

$X_c = \frac{\sum_i (v/s)_{ci} \times C}{C - L} =$ 0.746

Figure 9-24. Capacity analysis module worksheet for Calculation 2.

			g/C
g_1 (EB LT)	= 0.078 (50/0.746) =	5.2 sec	0.104
g_2 (E-W TH/RT)	= 0.275 (50/0.746) =	18.4 sec	0.368
g_3 (NB)	= 0.304 (50/0.746) =	20.4 sec	0.408
		44.0 sec	
Lost Time		6.0 sec	
		50.0 sec	

Note that the through and right-turn movements on the EB approach move during both the first and second signal phases. Thus:

g(EB TH/RT) = 5.2 + 18.4 = 23.6 sec, and g/C = 0.472

This allocation assumes that each phase is intended to operate at a similar v/c ratio, X. The green times should be checked against the minimum pedestrian green times listed on the Input Module Worksheet. It is seen that all green phases are long enough to cover minimum pedestrian times.

These green times and ratios all appear to be acceptable, and the g/C values estimated are entered into the fifth column of the Capacity Analysis Worksheet.

Lane group capacities may be computed by multiplying the green ratio by the saturation flow rate, and the v/c ratio for each lane group is found by dividing the adjusted flow rate by the capacity. Note that no lane group v/c ratio exceeds 0.746, a condition guaranteed by the estimation procedure for signal timing. The critical v/c ratio is 0.746, also set by the signal timing computations.

There is nothing in the capacity analysis results to suggest that the recommended signal timing is not adequate for the existing geometrics and anticipated flows. Further, it appears that all EB left-turns can be accommodated in the leading green phase, although there is no guarantee that they will all use this phase. The level of service module will therefore be examined for delays and levels of service.

e. Level-of-service module—The Level of Service Module Worksheet is shown in Figure 9-25. Values of X, g/C, C, and

LEVEL-OF-SERVICE WORKSHEET

Lane Group		First Term Delay				Second Term Delay				Total Delay & LOS		
① Appr.	② Lane Group Movements	③ v/c Ratio X	④ Green Ratio g/C	⑤ Cycle Length C (sec)	⑥ Delay d_1 (sec/veh)	⑦ Lane Group Capacity c (vph)	⑧ Delay d_2 (sec/veh)	⑨ Progression Factor PF Table 9-13	⑩ Lane Group Delay (sec/veh) (⑥+⑧)×⑨	⑪ Lane Group LOS Table 9-1	⑫ Approach Delay (sec/veh)	⑬ Appr. LOS Table 9-1
EB		.745	.472	50	8.15	169	10.85	1.00	19.00	D		
		.699	.472	50	7.90	1550	0.99	1.00	8.90	B	9.95	B
WB		.746	.368	50	10.46	1184	1.84	1.00	12.30	B	12.30	B
NB		.744	.408	50	9.56	1262	1.70	1.00	11.27	B	11.27	B
SB												

Figure 9-25. Level-of-service module worksheet for Calculation 2.

Intersection Delay 11.04 sec/veh   Intersection LOS B (Table 9-1)

c are entered from the Capacity Analysis Worksheet for all lane groups.

First-term delays, d_1, are computed from the first term of Eq. 9-18:

$$d_1 = 0.38\, C\,(1 - g/C)^2/[1 - (g/C)(X)]$$

The results of these computations are shown in column 6 of the worksheet.

Second-term delays, d_2, are computed from the second term of Eq. 9-18:

$$d_2 = 173\, X^2 \left[(X - 1) + \sqrt{(X - 1)^2 + (16X/c)}\right]$$

The results of these computations are shown in column 8 of the worksheet.

The progression factor (PF) is selected from Table 9-13, based on type of signal control, v/c ratio, and arrival type. For arrival type 3, the factor is 1.00.

The average stopped delay per vehicle is found by adding the first- and second-term delays, and multiplying by the progression factor:

Delay = $(d_1 + d_2)$ PF
Delay (EB LT) = (8.15 + 10.85) 1.00 = 19.0 sec/veh
Delay (EB TH) = (7.90 + 0.99) 1.00 = 8.9 sec/veh
Delay (WB) = (10.46 + 1.84) 1.00 = 12.3 sec/veh
Delay (NB) = (9.56 + 1.70) 1.00 = 11.27,
Say 11.3 sec/veh

These delay values must be averaged to find the delay for the EB approach (both lane groups), and for the intersection as a whole. The average is weighted by the adjusted volume in each lane group.

Delay (EB Approach) = [19.0(126) + 8.9(1084)]/[126 + 1084]
= 9.95, Say 10.0 sec/veh

Delay (Intersection) =

$$\frac{[9.95(1210) + 12.3(884) + 11.27(939)]}{1210 + 884 + 939}$$

= 11.04, Say 11.0 sec/veh

Levels of service are found by comparing delay values to the criteria in Table 9-1. These are shown in the appropriate columns of the worksheet. Because all levels of service for all lane groups, approaches, and the intersection as a whole are LOS B, except for the EB LT lane, for which the LOS is D, the operation is generally acceptable, and the suggested signal timing may be adopted for the existing intersection geometry and the forecast traffic volumes. No geometric improvements are needed.

CALCULATION 3—OPERATIONAL ANALYSIS OF A MULTIPHASE ACTUATED SIGNAL

1. *Description*—The intersection of Fifth Avenue and 12th Street is a heavily loaded location. Both facilities are four-lane divided arterials, with left-turn lanes for each approach. There is parking permitted on 12th Street, but none is permitted on Fifth Avenue. Because the left-turns from Fifth Avenue are heavy, and an exclusive LT phase is included in the phase plan for this street, followed by a leading green for the approach with the heavier left-turn demand, the signal is fully actuated. The intersection is described in Figure 9-26, the Input Module Worksheet for the calculation. Determine the adequacy of the geometric and signal design to accommodate existing arrival volumes.

2. *Solution*—The solution is described on a module-by-module basis, as follows:

a. Input module—The Input Module Worksheet is shown in Figure 9-26. The upper portion of the worksheet depicts all geometric conditions and existing demand volumes at the intersection.

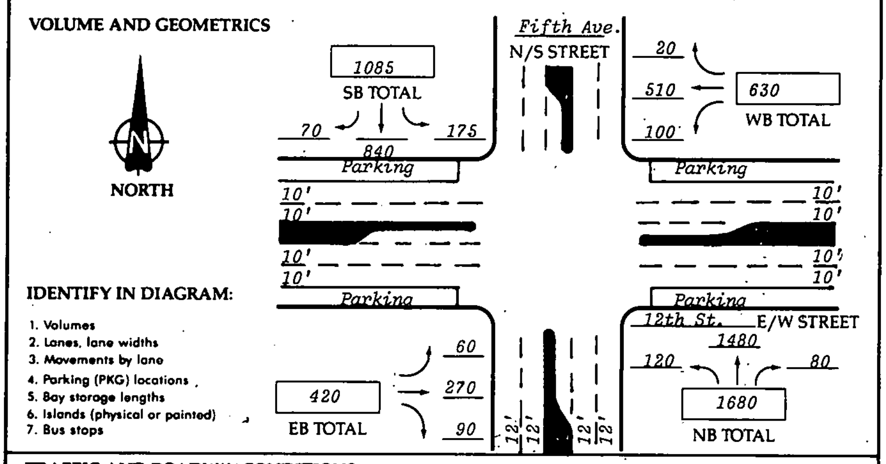
The central portion of the worksheet contains additional information. Grades are level on all approaches. The percent of heavy vehicles is 5 percent on the EB and WB approaches, and

INPUT WORKSHEET

Intersection: *Fifth Ave. and 12th Street* Date: *12/12/85*

Analyst: *RPR* Time Period Analyzed: *5-6 PM* Area Type: ☒ CBD ☐ Other

Project No.: City/State: *Mudville*



TRAFFIC AND ROADWAY CONDITIONS

Approach	Grade (%)	% HV	Adj. Pkg. Lane Y or N	Adj. Pkg. Lane N_m	Buses (N_B)	PHF	Conf. Peds. (peds./hr)	Pedestrian Button Y or N	Pedestrian Button Min. Timing	Arr. Type
EB	0	5	Y	5	0	0.85	200	Y	22	3
WB	0	5	Y	5	0	0.85	200	Y	22	3
NB	0	2	N	0	0	0.90	50	Y	22	3
SB	0	2	N	0	0	0.90	50	Y	22	3

Grade: + up, − down
HV: veh. with more than 4 wheels
N_m: pkg. maneuvers/hr
N_B: buses stopping/hr
PHF: peak-hour factor
Conf. Peds: Conflicting peds./hr
Min. Timing: min. green for pedestrian crossing
Arr. Type: Type 1-5

PHASING

Timing	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =	G = Y+R =
Pretimed or Actuated	A	A	A	A	A			

Protected turns — Permitted turns — ------- Pedestrian — Cycle Length ____ Sec

Figure 9-26. Input module worksheet for Calculation 3.

it is 2 percent on the NB and SB approaches. Parking is permitted only on the EB and WB approaches, with 5 maneuvers/hour experienced in and out of the parking lane within 250 ft of the intersection. There are no buses, and the PHF is 0.85, for the EB and WB approaches, and 0.90, for the NB and SB approaches. There is a moderate number of pedestrians interfering with EB and WB right turns (200 peds/hr), and a low number interfering with NB and SB right turns (50 peds/hr), the intersection has pedestrian push buttons, and the pedestrian green time resulting from an actuation is shown as 22 sec for all approaches. The signal is not coordinated in a progressive system, and arrival type 3 prevails on all approaches.

The signal phasing described in the project statement is illustrated on the lower portion of the worksheet.

b. Volume adjustment module—The worksheet for this module is shown in Figure 9-27. Movement volumes are entered in column 3, and divided by the appropriate PHF to obtain movement flow rates, which are shown in column 5.

Lane groups are straightforward, as left-turn lanes exist on all approaches. Thus, each approach consists of a single left-turn lane group, with remaining lanes forming a second lane group for through and right-turning vehicles.

Lane utilization factors are selected from Table 9-4, and they are entered in column 9 of the worksheet. Lane group flow rates are multiplied by this factor to obtain the adjusted lane group flow rates shown in column 10. Column 11 shows the proportion of left and/or right turns in each lane group.

c. Saturation flow rate module—The worksheet for this module is shown in Figure 9-28. Lane group descriptions are repeated in column 2, and the ideal saturation flow rate is given in column 3. An ideal saturation flow rate of 1,800 pcphg will be used for all lane groups.

Subsequent columns contain the number of lanes in the group, and adjustments to the ideal saturation flow, which are obtained from the following tables, as indicated:

- No. of lanes. Obtained from the Volume Adjustment Worksheet.

VOLUME ADJUSTMENT WORKSHEET

① Appr.	② Mvt.	③ Mvt. Volume (vph)	④ Peak Hour Factor PHF	⑤ Flow Rate v_p (vph) ③ ÷ ④	⑥ Lane Group	⑦ Flow rate in Lane Group v_g (vph)	⑧ Number of Lanes N	⑨ Lane Utilization Factor U Table 9-4	⑩ Adj. Flow v (vph) ⑦ × ⑨	⑪ Prop. of LT or RT P_{LT} or P_{RT}
EB	LT	60	0.85	71		71	1	1.00	71	1.0 LT
	TH	270	0.85	318		424	2	1.05	445	0.25 RT
	RT	90	0.85	106						
WB	LT	100	0.85	118		118	1	1.00	118	1.0 LT
	TH	510	0.85	600		624	2	1.05	655	0.04 RT
	RT	20	0.85	24						
NB	LT	120	0.90	133		133	1	1.00	133	1.0 LT
	TH	1480	0.90	1644		1733	2	1.05	1820	0.05 RT
	RT	80	0.90	89						
SB	LT	175	0.90	194		194	1	1.00	194	1.0 LT
	TH	840	0.90	933		1011	2	1.05	1062	0.08 RT
	RT	70	0.90	78						

Figure 9-27. Volume adjustment module worksheet for Calculation 3.

SATURATION FLOW ADJUSTMENT WORKSHEET

LANE GROUPS				ADJUSTMENT FACTORS								
① Appr.	② Lane Group Movements	③ Ideal Sat. Flow (pcphgpl)	④ No. of Lanes N	⑤ Lane Width f_w Table 9-5	⑥ Heavy Veh f_{HV} Table 9-6	⑦ Grade f_g Table 9-7	⑧ Pkg. f_p Table 9-8	⑨ Bus Blockage f_{bb} Table 9-9	⑩ Area Type f_a Table 9-10	⑪ Right Turn f_{RT} Table 9-11	⑫ Left Turn f_{LT} Table 9-12	⑬ Adj. Sat. Flow Rate s (vphg)
EB		1800	1	.93	.975	1.00	1.00	1.00	.90	1.00	.31	455
		1800	2	.93	.975	1.00	.935	1.00	.90	.94	1.00	2582
WB		1800	1	.93	.975	1.00	1.00	1.00	.90	1.00	.48	705
		1800	2	.93	.975	1.00	.935	1.00	.90	.99	1.00	2719
NB		1800	1	1.00	.99	1.00	1.00	1.00	.90	1.00	.95	1524
		1800	2	1.00	.99	1.00	1.00	1.00	.90	.99	1.00	3176
SB		1800	1	1.00	.99	1.00	1.00	1.00	.90	1.00	.95	1524
		1800	2	1.00	.99	1.00	1.00	1.00	.90	.99	1.00	3176

Figure 9-28. Saturation flow rate module worksheet for Calculation 3.

- Lane width factor. Obtained from Table 9-5, based on given lane widths.
- Heavy vehicle factor. Obtained from Table 9-6, based on given percents of heavy vehicles.
- Grade factor. Obtained from Table 9-7, based on given grades.
- Parking factor. Obtained from Table 9-8, based on given data on parking lanes, parking maneuvers, and the number of lanes in the lane group.
- Bus blockage factor. Obtained from Table 9-9, based on the number of buses per hour.
- Area type factor. Obtained from Table 9-10, based on the type of area in which the intersection is located.
- Right-turn factor. Obtained from Table 9-11, based on the proportion of right turns in the lane group, the conflicting pedestrian volume, and the type of right-turn phasing and lane group. All right turns at this intersection are of a type described by Case 5.
- Left-turn factor. Obtained from Table 9-12, based on the proportion of left turns in the lane group, the opposing vehicular flow, and the type of left-turn phasing and lane group. The NB and SB LT lane groups are described by Case 3 (LT lane group, protected plus permitted phasing).

The EB and WB approaches contain a permitted left-turn phase, and the adjustment factor must be computed using the special procedure on the worksheet shown in Figure 9-29. Note that the signal timing has not yet been estimated. For the purposes of this computation, the methodology recommends using a 90-sec cycle, with green time allocated in proportion to the average critical flow per lane for each phase. Examination of the phase plan indicates that the WB through movement will be critical in the E-W signal phase, and has an average per lane flow of 655/2 = 327 vphpl. The NB through movement will be critical for the N-S through phase, and has a per lane flow of 1,820/2 = 910 vphpl. The SB LT is critical for the exclusive turning phase, with a value of 194/1 = 194 vphpl. The total of these is 327 + 910 + 194 = 1,431 vphpl, and the green time for the E-W phase may be estimated as (327/1,431)

SUPPLEMENTAL WORKSHEET FOR LEFT-TURN ADJUSTMENT FACTOR, f_{LT}				
INPUT VARIABLES	**EB**	**WB**	**NB**	**SB**
Cycle Length, C (sec) *(Estimated)*	90	90		
Effective Green, g (sec) *(Estimated)*	18.5	18.5		
Number of Lanes, N	1	1		
Total Approach Flow Rate v_a (vph)	495	742		
Mainline Flow Rate, v_M (vph)	424	624		
Left-Turn Flow Rate, v_{LT} (vph)	71	118		
Proportion of LT, P_{LT}	1.0	1.0		
Opposing Lanes, N_o	2	2		
Opposing Flow Rate, v_o (vph)	624	424		
Prop. of LT in Opp. Vol., P_{LTO}	0.0	0.0		
COMPUTATIONS	**EB**	**WB**	**NB**	**SB**
$S_{op} = \dfrac{1800\, N_o}{1 + P_{LTO}\left[\dfrac{400 + v_M}{1400 - v_M}\right]}$	3600	3600		
$Y_o = v_o / S_{op}$	0.173	0.118		
$g_u = (g - CY_o)/(1 - Y_o)$	3.54	8.93		
$f_s = (875 - 0.625\, v_o)/1000$	-	-		
$P_L = P_{LT}\left[1 + \dfrac{(N-1)\,g}{f_s g_u + 4.5}\right]$	1.0	1.0		
$g_q = g - g_u$	-	-		
$P_T = 1 - P_L$	0.0	0.0		
$g_f = 2\,\dfrac{P_T}{P_L}\left[1 - P_T^{\,0.5\, g_q}\right]$	0.0	0.0		
$E_L = 1800/(1400 - v_o)$	2.32	1.84		
$f_m = \dfrac{g_f}{g} + \dfrac{g_u}{g}\left[\dfrac{1}{1 + P_L(E_L - 1)}\right] + \dfrac{2}{g}(1 + P_L)$	0.31	0.48		
$f_{LT} = (f_m + N - 1)/N$	0.31	0.48		

Figure 9-29. Supplemental worksheet for computation of left-turn adjustment factors for Calculation 3.

(90 − 9) = 18.5 sec, assuming 9 sec of lost time in the cycle. Depending on the combination of critical flows determining the actual signal timing, the cycle may only involve 6 sec of lost time. Nine seconds is assumed at this point to be conservative in this rough estimate. Note that this approximation is used solely for the purpose of estimating the left-turn factor for the EB and WB exclusive turning lanes.

The estimated values of C and g are entered on the worksheet, along with other input information, and the left-turn factors are computed as shown on the worksheet. The results are entered on the Saturation Flow Adjustment Worksheet of Figure 9-28.

The ideal saturation flow rate is multiplied by all adjustment factors, with the resulting saturation flow rate being entered in the last column of Figure 9-28.

d. Capacity analysis module—The lane group descriptions are repeated in column 2 of the Capacity Analysis Worksheet, shown in Figure 9-30. Note that for this module, the protected and permitted portions of the NB and SB LT phases are divided, although it is initially assumed that all of the demand for these movements occurs within the protected portion of the phase.

The adjusted flow rates for each lane group are entered in column 3 of the worksheet from the Volume Adjustment Worksheet. Saturation flow rates for each lane group are obtained from the Saturation Flow Adjustment Worksheet, and entered in column 4. Flow ratios are then computed as v/s and entered in column 5 of the worksheet. At this point, critical lane groups must be identified, and the average timing of the phases estimated.

Given the phase plan illustrated in Figure 9-26, the combination of critical lane groups is found from:

$$\begin{Bmatrix} \text{EB LT or TH/RT} \\ \text{or} \\ \text{WB LT or TH/RT} \end{Bmatrix} + \begin{Bmatrix} \text{NB LT + SB TH/RT} \\ \text{or} \\ \text{SB LT + NB TH/RT} \end{Bmatrix}$$

$$\begin{Bmatrix} 0.156 \text{ or } 0.172 \\ \text{or} \\ 0.167 \text{ or } 0.241 \end{Bmatrix} + \begin{Bmatrix} 0.087 + 0.334 \\ \text{or} \\ 0.127 + 0.573 \end{Bmatrix}$$

CAPACITY ANALYSIS WORKSHEET

LANE GROUP ① Appr.	② Lane Group Movements	③ Adj. Flow Rate v (vph)	④ Adj. Sat. Flow Rate s (vphg)	⑤ Flow Ratio v/s ③÷④	⑥ Green Ratio g/C	⑦ Lane Group Capacity c (vph) ④×⑥	⑧ v/c Ratio X ③÷⑦	⑨ Critical ? Lane Group
EB		71	455	0.156	0.254	116	0.612	
		445	2582	0.172	0.254	648	0.678	
WB		118	705	0.167	0.254	181	0.659	
		655	2719	0.241	0.254	682	0.948	✓
NB		133 / 73	1524	0.087 / 0.048	0.051	78	0.936	
		0 / 60						
		1820	3176	0.573	0.598	1915	0.950	✓
SB		194 / 134	1524	0.127 / 0.088	0.093	142	0.944	✓
		0 / 60						
		1062	3176	0.334	0.645	2049	0.518	

Cycle Length, C 118.8 sec

Lost Time Per Cycle, L 6 sec

$\sum_i (v/s)_{ci} =$ 0.902

$X_c = \dfrac{\sum_i (v/s)_{ci} \times C}{C - L} =$ 0.95

Figure 9-30. Capacity analysis module worksheet for Calculation 3.

It is seen that the maximum sum of flow ratios occurs using the SB LT + NB TH/RT + WB TH/RT, and this sum is 0.941.

In estimating the actual average timing of an actuated signal, it is assumed that the critical v/c ratio is 0.95. Using this assumption, the average cycle length may be estimated from Eq. II.9-1 as:

$$C = L\,X_c/\left[X_c - \sum_i (v/s)_{ci}\right]$$

At this point, a closer examination of the cycle is needed to determine the appropriate value of lost time. As all E-W flows move on the same phase, a single lost time of 3.0 sec is experienced on this phase. The N-S approaches have overlapping phases, including an exclusive LT phase, a leading green, and a through phase, during which permitted LT's are included. Given this sequence, the controlling, or critical SB left turn, will move continuously through all three portions, and will experience the same 3.0 sec of lost times as the through movements. Thus, the cycle contains only 6.0 total seconds of lost time, and:

$$C = 6\,(0.95)/(0.95 - 0.941) = 633 \text{ sec}$$

This value is obviously unreasonably high, and would doubtless exceed the maximum green limitations of the controller. At this point, it would be advisable to revise the assumption that all NB and SB left turns occur during the protected portions of the phasing.

It is now assumed that some of the left turns on the NB and SB approaches are made during the permitted portion of the phase. The minimum number of left turns that could be made during the permitted phase is 2 vehicles per cycle. If a 120-sec cycle is assumed for the moment, a minimum left-turn flow of $(3{,}600/120) \times 2 = 60$ vph could be handled by the permitted left-turn phases. The NB LT and SB LT lane groups are adjusted to reflect this, assigning 60 vph to the permitted phase, with the remaining left turns assigned to the protected portion of the phase plan. Flow ratios are now recomputed. The same combination of critical lane groups remains, but the sum of critical lane flow ratios is now: 0.088 + 0.573 + 0.241 = 0.902.

The average cycle length of the actuated signal is reestimated from Eq. II. 9-1, using an assumed critical saturation flow ratio, X_c, of 0.95 (see Appendix II):

$$C = 6\,(0.95)/[0.95 - 0.902] = 118.8 \text{ sec}$$

This is a reasonable value, within the limits of most controllers. As the signal is actuated, there is no need to round the cycle length to an increment of 5 or 10 sec. The value of 118.8 sec will be taken as the average cycle length.

Green times are computed from Eq. II. 9-2, as follows:

$$g_i = v_i\, C/s_i\, X_i = (v/s)_i\,(C/X_i)$$

The green time for the first phase (exclusive NB and SB LT) is controlled by the NB LT movement, which is not critical and uses only this phase. A v/c of 0.95 is used, because this is a minor movement, and excess green time may be assigned to higher flows:

$$g_1 = 0.048\,(118.8/0.95) = 6.0 \text{ sec}$$

$$g/C = 6.0/118.8 = 0.050$$

The green time for the total of phases 1 and 2 (the exclusive NB and SB LT plus the leading green for the SB LT, which is the heavier of the two LT movements) is controlled by the SB LT movement, which uses both phases (and the change interval between them). Thus:

$$g_1 + g_2 = 0.088\,(118.8/0.95) = 11.0 \text{ sec}$$

$$g/C = 11.0/118.8 = 0.092$$

This suggests that the average green time for phase 2 (leading SB green) is:

$$g_2 = 11.0 - 6.0 - 3.0 = 2.0 \text{ sec}$$

where 3.0 is the assumed change interval between phases 1 and 2. This is a very small value. It is, however, an average, as is the length of g_1. The actual lengths of these phases vary on a cycle-by-cycle basis. It should also be noted that the SB LT is the heavier LT movement, while the NB TH/RT is the heavier of the N-S through movements. Thus, the inclusion of the leading green portion of the N-S phase is not efficient. The leading green would be most useful where both the LT and TH/RT movements are heaviest on the same approach, so that both are provided with additional green.

The third phase (NB and SB TH/RT) is controlled by the NB TH/RT movement, which uses only this phase:

$$g_3 = 0.573\,(118.8/0.95) = 71.7 \text{ sec}$$

$$g/C = 71.7/118.8 = 0.603$$

The last phase (EB, WB all mvts) is controlled by the EB TH/RT lane group, and:

$$g_4 = 0.241\,(118.8/0.95) = 30.1 \text{ sec}$$

$$g/C = 30.1/118.8 = 0.254$$

The SB TH/RT movement is permitted in phases 2 and 3, as well as the change interval between them. The green time for this movement is, therefore:

$$g \text{ (SB TH/RT)} = 2.0 + 3.0 + 71.7 = 76.7 \text{ sec}$$

$$g/C = 76.7/118.8 = 0.646$$

These estimated g/C values are entered in column 6 of the worksheet. Lane group capacities may now be computed by multiplying the g/C ratios by the saturation flow rates for each group. Volume-to-capacity ratios, X, for each lane group are the ratio of adjusted flow rate to capacity.

Note that critical lane groups operate near capacity, with little excess capacity or green time available. The critical v/c ratio was set at 0.95 by the assumed average signal timing. Note, however, that actuated signals are intended to operate in this mode under heavy demands, i.e., they are designed to minimize unutilized green time. The level of service module will investigate the delays that result from this condition.

e. Level-of-service module—The Level of Service Module Worksheet is shown in Figure 9-31. Values of X, g/C, C, and c are obtained from the Capacity Analysis Worksheet, and are entered in the appropriate columns.

First-term delays, d_1, are computed from the first term of Eq. 9-18:

$$d_1 = 0.38\, C\,(1 - g/C)^2/[1 - (g/C)\,(X)]$$

The results of these computations are shown in column 6 of the worksheet.

Second-term delays, d_2, are computed from the second term of Eq. 9-18:

$$d_2 = 173X^2\,[(X - 1) + \sqrt{(X - 1)^2 + (16X/c)}]$$

The results of these computations are shown in column 8 of the worksheet.

The progression factor is selected from Table 9-13, based on the type of control, arrival type, and v/c ratio for the lane group.

Delay for each lane group is computed as the first + second-term delays multiplied by the progression factor, or:

Delay = $(d_1 + d_2)$ PF

Delay (EB LT) = (29.74 + 6.27) 1.00 = 36.01 sec/veh

Delay (EB TH/RT) = (30.34 + 1.97) 0.85 = 27.46 sec/veh

Delay (WB LT) = (30.16 + 5.83) 1.00 = 35.99 sec/veh

Delay (WB TH/RT) = (33.08 + 16.33) 0.85 = 42.00 sec/veh

Delay (NB LT) = (13.94 + 57.42) 1.00 = 71.36 sec/veh

Delay (NB TH/RT) = (16.65 + 8.14) 0.85 = 21.07 sec/veh

Delay (SB LT) = (12.17 + 42.43) 1.00 = 54.60 sec/veh

LEVEL-OF-SERVICE WORKSHEET

Lane Group		First Term Delay				Second Term Delay				Total Delay & LOS		
① Appr.	② Lane Group Move-ments	③ v/c Ratio X	④ Green Ratio g/C	⑤ Cycle Length C (sec)	⑥ Delay d_1 (sec/veh)	⑦ Lane Group Capacity c (vph)	⑧ Delay d_2 (sec/veh)	⑨ Progression Factor PF Table 9-13	⑩ Lane Group Delay (sec/veh) (⑥+⑧) × ⑨	⑪ Lane Group LOS Table 9-1	⑫ Approach Delay (sec/veh)	⑬ Appr. LOS Table 9-1
EB		.612	.254	118.8	29.74	116	6.27	1.00	36.01	D	28.6	D
		.678	.254	118.8	30.34	648	1.97	0.85	27.46	D		
WB		.659	.254	118.8	30.61	181	5.83	1.00	35.99	D	41.1	E
		.948	.254	118.8	33.08	682	16.33	0.85	42.00	E		
NB		.936	.653	118.8	13.94	78	57.54	1.00	71.36	F	24.5	C
		.950	.603	118.8	16.65	1915	8.14	0.85	21.07	C		
SB		.944	.690	118.8	12.17	142	42.43	1.00	54.60	E	14.7	C
		.518	.645	118.8	8.54	2049	0.19	0.85	7.42	B		

Intersection Delay 25.1 sec/veh Intersection LOS D (Table 9-1)

Figure 9-31. Level-of-service module worksheet for Calculation 3.

Delay (SB TH/RT) = (8.54 + 0.19) 0.85 = 7.42 sec/veh

Note, in the computation of delay for protected plus permitted left-turn phases with exclusive lanes, that g/C ratios for the entire protected plus permitted phase are used. The v/c ratios for the protected portions of the phase are based on an arbitrary assumption concerning the split of demand between the two phase portions, and are estimated only for the protected portion on the Capacity Analysis Worksheet. As an approximation, the v/c ratio computed for the protected portion of the phase (for the assumed demand split) is used in delay computations.

These delays must now be averaged for each lane group, and then for the intersection as a whole. The average is weighted by the adjusted volume in each lane group. Then:

Delay (EB) = [36.01(71) + 27.46(445)]/[71 + 445]
= 28.63, Say 28.6 sec/veh

Delay (WB) = [35.99(118) + 42.00(655)]/[118 + 655]
= 41.08, Say 41.1 sec/veh

Delay (NB) = [71.36(133) + 21.07(1,820)]/[133 + 1,820]
= 24.49, Say 24.5 sec/veh

Delay (SB) = [54.60(194) + 7.42(1,062)]/[194 + 1,062]
= 14.71, Say 14.7 sec/veh

Delay (Int.) = [28.63(516) + 41.08(773) + 24.49(1,953) + 14.71(12.56)]/[516 + 773 + 1,953 + 1,256] = 25.08, Say 25.1 sec/veh

All delays are in terms of seconds of stopped delay per vehicle.

Levels of service are determined by comparing these delay values to the criteria in Table 9-1. Results are shown in the appropriate columns.

While the overall intersection operates at LOS D, the critical lane groups are experiencing LOS E, and the NB LT lane group is operating at LOS F. Thus, the delays to these groups are undesirably high. The high delay to the NB LT, which is not a critical movement, could be partially ameliorated by lengthening the green phase at the expense of SB TH + RT, also not a critical movement. Note that the timing policy kept the v/c for the NB LT at 0.95 assigning excess green to the SB TH + RT. This was because the NB LT is a minor movement. Green time could be reallocated to this movement to reduce delay, which will then increase delay to SB TH + RT drivers. Aggregate delay, however, should be considered, given that many more vehicles are in the SB TH + RT movement.

The phasing plan is relatively efficient, and the heavy NB and SB left-turn movements already have exclusive phases. While delays are undesirably high, the intersection operates at a v/c ratio of 0.95, under capacity. Geometric improvements appear to be the only reasonable direction for significant improvements. As a first step, parking might be eliminated on the E-W street. This would enable a shortening of the green phase for this street, and reallocation of additional time to the N-S street. The delay and v/c ratio for the E-W left turns, however, would have to be carefully evaluated under this option. A N-S left-turn phase might be considered, but this might actually increase delay, and the cycle length of 118.8 sec (estimated) does not appear to provide enough flexibility for addition of another phase. Construction of additional lanes on the N-S street might also be considered if right-of-way is available.

CALCULATION 4—PLANNING ANALYSIS OF AN INTERSECTION WITH MULTILANE APPROACHES

1. *Description*—The intersection of Tenth Avenue and First Street is currently a minor intersection of two 2-lane, lightly used streets. In 20 years, major development is expected to cause both streets to be reconstructed as multilane divided facilities, and the intersection will have substantial demand. Figure 9-32, the Planning Analysis Worksheet, contains a diagram of the expected intersection geometry and the forecast volumes for the intersection. Note that left-turn lanes are expected to be incorporated on each approach. Will the capacity of the proposed design be adequate?

2. *Solution*—Given the level of information available, the planning analysis technique will be applied for an approximate

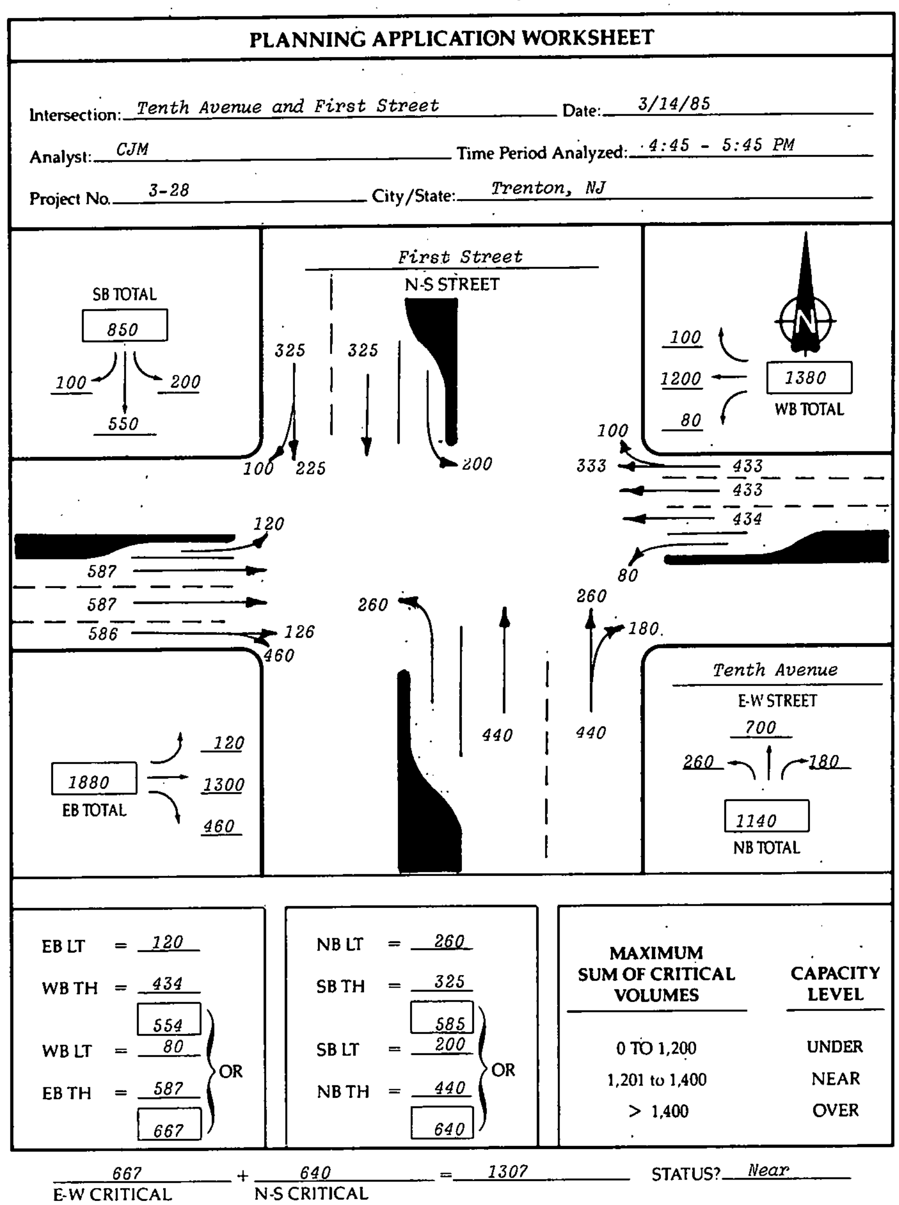


Figure 9-32. Planning analysis worksheet for Calculation 4.

evaluation of the capacity of the intersection. The solution is illustrated on Figure 9-32, and is explained in a step-by-step fashion.

a. Step 1: Record Demand Volumes—The turning movement volumes for the evening rush hour have been entered on the Planning Worksheet of Figure 9-32 in the appropriate quadrants.

b. Step 2: Record Geometrics—The expected geometry has been sketched on the Planning Worksheet.

c. Step 3: Identify Lane Impedance—Left turns from shared lanes that conflict with an opposing vehicle flow are marked with an asterisk (*), indicating that the movement causes lane impedance. As all left turns are made from exclusive lanes in the proposed design, none are so marked.

d. Step 4: Assign Lane Volumes—All left turns are assigned to the appropriate left-turn lanes. The sum of right turn plus through movements on each approach is equally divided among available through lanes, and is shown on the intersection diagram of Figure 9-32.

e. Step 5: Special Procedure for Single Lane Approaches—There are no such approaches in this calculation.

f. Step 6: Determine the Sum of Critical Volumes—The critical volume for each street is the maximum sum of the left-turn movement plus the opposing per lane through or through plus right-turn movement.

Thus, sum of critical volumes is:

$$\left\{\begin{matrix}\text{EB LT + WB TH}\\ \text{or}\\ \text{WB LT + EB TH}\end{matrix}\right\} + \left\{\begin{matrix}\text{NB LT + SB TH}\\ \text{or}\\ \text{SB LT + NB TH}\end{matrix}\right\}$$

$$\left\{\begin{matrix}120 + 434\\ \text{or}\\ 80 + 587\end{matrix}\right\} + \left\{\begin{matrix}260 + 325\\ \text{or}\\ 200 + 440\end{matrix}\right\}$$

The maximum sum is given by the WB LT + EB TH (80 + 587 = 667 vph) and the SB LT + NB TH (200 + 440 = 640 vph), for a total critical volume of 667 + 640 = 1,307 vph. These values are shown in the appropriate boxes at the bottom of the worksheet.

g. Step 7: Check Capacity—The total critical volume is checked vs. the criteria of Table 9-14, which is also shown in the lower right-hand corner of the worksheet. It is seen that the critical volume is *near* capacity, i.e., in a range where it is uncertain whether or not demand will exceed capacity.

It would be desirable to provide a design which lowered the sum of critical volumes to a value under 1,200 vph to ensure that capacity will most probably not be exceeded.

The geometric suggestions of Appendix I indicate that intersection design should attempt to keep per lane volumes to 450 vph or less. This is not the case for the EB approach on the proposed design of Figure 9-32. Note that the right-turn volume is extremely high on this approach. If a right-turn lane were provided, lane volumes on the remainder of the approach could be brought below the 450-vph suggestion. Such a design is depicted in Figure 9-33, which is the worksheet for planning analysis of this proposed revision.

h. Analysis of Revised Intersection—The analysis of the revised intersection is similar to that outlined above. The only values that change are the per lane volumes on the EB approach, which are lowered because of the addition of the right-turn lane. This alters the determination of the critical volume, which is now the maximum sum among:

$$\left\{\begin{matrix}\text{EB LT + WB TH}\\ \text{or}\\ \text{WB LT + EB TH}\end{matrix}\right\} + \left\{\begin{matrix}\text{NB LT + SB TH}\\ \text{or}\\ \text{SB LT + NB TH}\end{matrix}\right\}$$

$$\left\{\begin{matrix}120 + 434\\ \text{or}\\ 80 + 434\end{matrix}\right\} + \left\{\begin{matrix}260 + 325\\ \text{or}\\ 200 + 440\end{matrix}\right\}$$

The maximum sum among these is given by the EB LT + WB TH (120 + 434 = 554 vph) and the SB LT + NB TH (200 + 440 = 640 vph) for a total of 554 + 640 = 1,194 vph. Note that the critical movements on the E-W street have been altered by the proposed design change. This critical volume is *under* capacity, and is therefore acceptable.

The proposed addition of the right-turn lane would be recommended. As the design process proceeds, the volume forecasts are refined, and a signal design is developed, the intersection should be subjected to detailed operational analysis.

CALCULATION 5—PLANNING ANALYSIS OF AN INTERSECTION WITH ONE-LANE APPROACHES

1. *Description*—A large area of a semirural community has been rapidly developing, requiring a considerable planning effort to provide additional capacity at numerous intersections of low-type, formerly rural, highway facilities. The intersection of Eighth Avenue and Main Street is one such location. It is the intersection of a two-lane roadway with a four-lane roadway. No turning lanes are present on any approach. The intersection is illustrated in Figure 9-34, along with projected traffic volumes. Is it likely that capacity will be exceeded at this location?

2. *Solution*—As in calculation 4, the solution is presented in a step-by-step fashion:

a. Step 1: Record Demand Volumes—Afternoon rush-hour movements are recorded on Figure 9-34. All turning movements are noted.

b. Step 2: Record Geometrics—The geometrics are sketched on Figure 9-34. Eighth Avenue is a four-lane street with two lanes in each direction. Main Street is a two-way street with one lane on each approach.

c. Step 3: Identify Lane Impedance—Lane impedance is experienced in a shared left-turn/through lane with an opposing vehicular flow. This exists in the left lane of the EB and WB approaches and on the one-lane NB and SB approaches. Each of these movements is marked with an asterisk (*).

d. Step 4: Assign Lane Volumes—All approaches have lane impedance or shared left-turn/through lanes. Passenger car equivalent computations are performed on Figure 9-35. The assignment of lane volumes on the EB and WB approaches is in terms of equal PCE's per lane. The total volume on the NB and SB approaches is assigned to the one lane available on each. PCE computations for these movements proceed through column 8 of Figure 9-35, with the results being used in Step 5, where special adjustments are made to account for the unique operating characteristics of single-lane approaches. Lane volumes are entered on the planning worksheet of Figure 9-34.

e. Step 5: Special Procedure for Single-Lane Approaches—The SB and NB approaches are narrow single lanes with unprotected turning. The first 8 columns of Figure 9-35 are used to determine that the PCE flow is 640/hour for the SB approach (an increase of 120 vph over the actual volume of 520 vph) and

370/hour for the NB approach (an increase of 80 vph over the actual volume of 290 vph for the approach). The new PCE volumes are entered on the planning worksheet of Figure 9-34.

The number of conflicting left turns for a single-lane approach opposed by a single-lane approach is now calculated. It is assumed that left turns from one approach can be made through gaps created by opposing left turns. Thus, conflicting left turns are assumed to be only the difference between the subject left-turn volume and the opposing left-turn volume. Thus, for the SB approach, the number of left turns that conflict with the opposing through movement is estimated to be:

$$120 \text{ vph} - 80 \text{ vph} = 40 \text{ vph}$$

where it is assumed that 80 vph turn through gaps created by the opposing 80 left-turns. Similarly, the number of NB left-turns that conflict with the opposing through movement is:

$$80 \text{ vph} - 120 \text{ vph} = -40, \text{ Say } 0 \text{ vph}$$

where it is assumed that all 80 left-turns are executed through gaps created by the opposing 120 left-turns.

These adjusted conflicting left-turn volumes are entered on the planning worksheet of Figure 9-34.

f. Step 6: Calculate Sum of Critical Lane Volumes—The possible combinations of critical volumes are given as follows:

$$\begin{Bmatrix} \text{EB LT + WB TH/RT} \\ \text{or} \\ \text{WB LT + EB TH/RT} \end{Bmatrix} + \begin{Bmatrix} \text{NB LT + SB} \\ \text{or} \\ \text{SB LT + NB} \end{Bmatrix}$$

$$\begin{Bmatrix} 120 + 470 \\ \text{or} \\ 170 + 605 \end{Bmatrix} + \begin{Bmatrix} 0 + 640 \\ \text{or} \\ 40 + 370 \end{Bmatrix}$$

The maximum sum is given by the WB LT + EB TH/RT + NB LT + SB, which yields a critical volume of 1,415 vph (170 + 605 + 0 + 640).

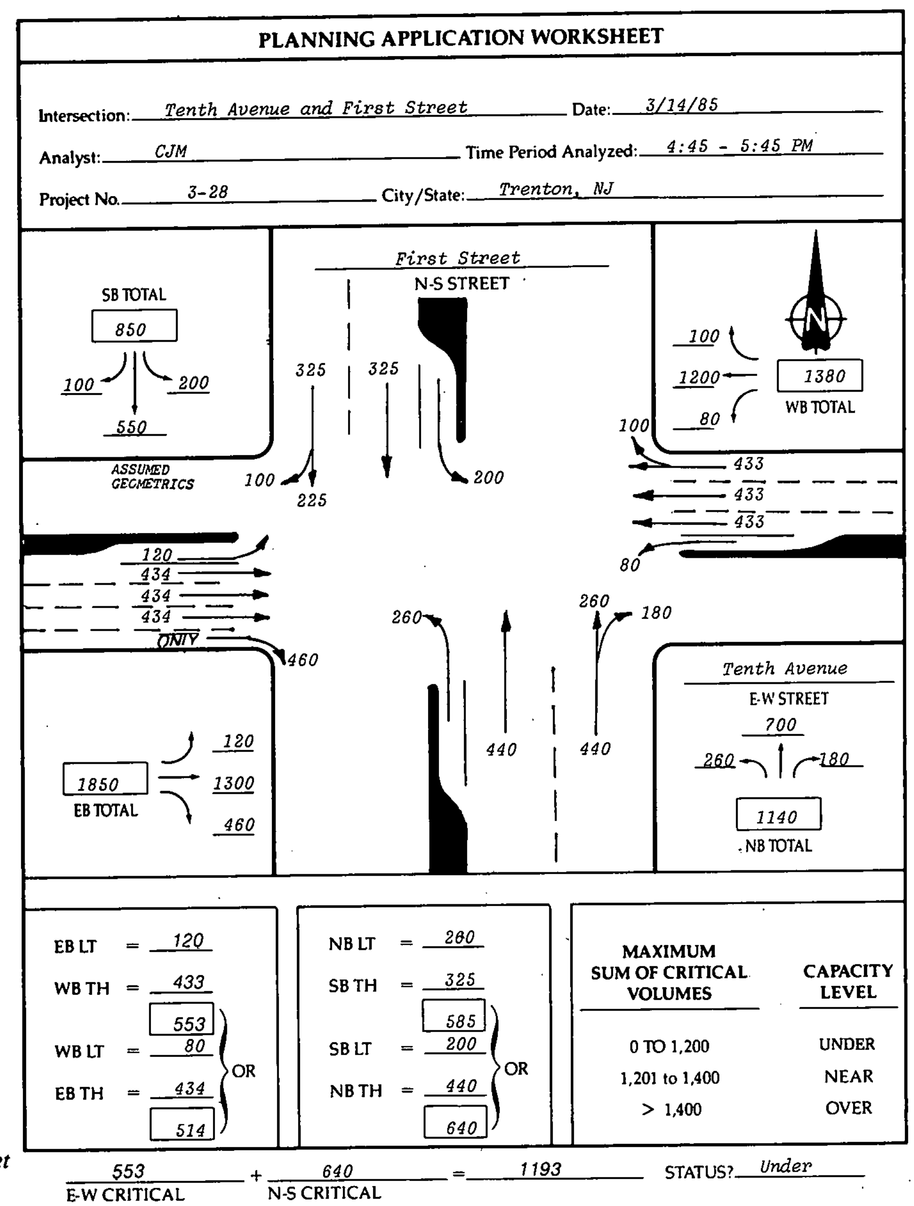


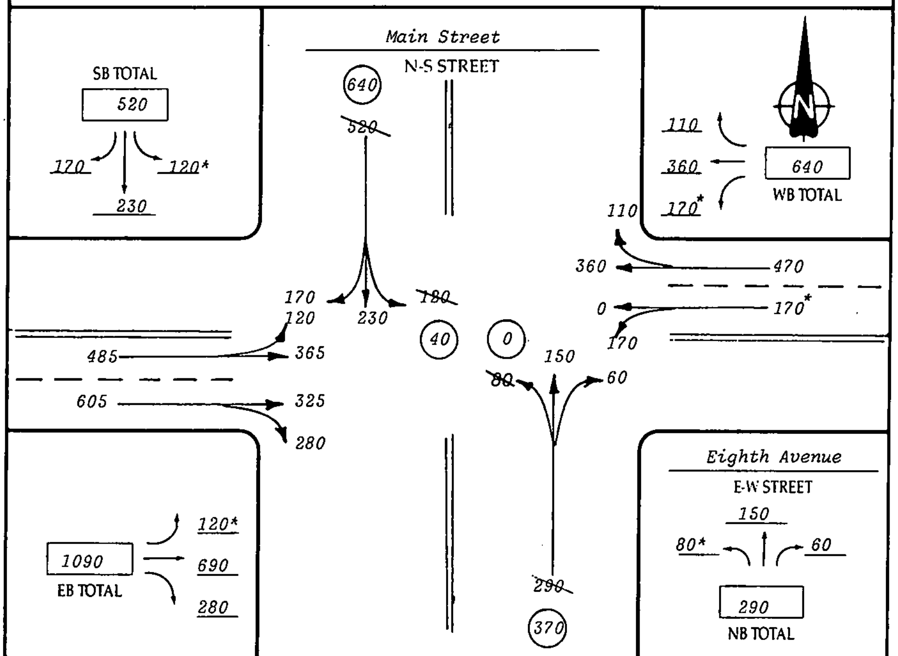
Figure 9-33. Planning analysis worksheet for revised design of Calculation 4.

PLANNING APPLICATION WORKSHEET

Intersection: *Eighth Avenue and Main Street* Date: *3/15/85*

Analyst: *CJM* Time Period Analyzed: *4:30 - 5:30 PM*

Project No. *3-28* City/State: *Fairfax, VA*



EB LT = 120	NB LT = 0	**MAXIMUM SUM OF CRITICAL VOLUMES** / **CAPACITY LEVEL**
WB TH = 470	SB TH = 640	
590	640	
WB LT = 170 OR	SB LT = 40 OR	0 TO 1,200 — UNDER
EB TH = 605	NB TH = 370	1,201 to 1,400 — NEAR
775	370	> 1,400 — OVER

775 (E-W CRITICAL) + 640 (N-S CRITICAL) = 1415 STATUS? *Over*

Figure 9-34. Planning analysis worksheet for Calculation 5.

g. Step 7: Check Capacity—The critical volume is checked vs. the criteria of Table 9-14, which is also shown in the bottom right-hand corner of Figure 9-34. It is seen that this intersection will probably be *over* capacity, and will be subject to breakdowns during the study period unless improvements to capacity are made. Given the volume of left-turn movements, separate left-turn lanes might be considered for each approach, subject to physical constraints.

CALCULATION 6—DETERMINING *v/c* AND SERVICE FLOW RATES, AN ALTERNATIVE USE OF THE OPERATIONAL ANALYSIS PROCEDURE

1. *Description*—A two-lane through movement at one approach to a signalized intersection has a cycle length of 90 sec, with a g/C ratio of 0.50. The arrival type is currently 3 (random), but could be improved by altering the progression. What is the maximum service flow rate that could be accommodated at level-of-service B (15 sec/veh of delay) on this approach?

2. *Solution*—Delay is based on the v/c ratio, X; the green ratio, g/C, the cycle length, C; the lane group capacity, c; and the progression factor, PF. The latter value may be computed as the saturation flow rate for the lane group times the g/C ratio which is known. Assume that a standard analysis using the Saturation Flow Adjustment Worksheet has been conducted, and that the saturation flow rate for the lane group is found to be 3,200 vphg, and the capacity 3,200 × 0.50 = 1,600 vph.

If delay is set at 15.0 sec/veh, and the known values of C, g/C, and c are inserted into Eq. 9-18, the following relationship is established:

$$15.0 = (d_1 + d_2)\,\mathrm{PF}$$

$$d_1 = 0.38(90)\,(1 - 0.50)^2/(1 - 0.50X)$$

$$d_2 = 173\,X^2\left[(X-1) + \sqrt{(X-1)^2 + (16X/1{,}600)}\right]$$

Various combinations of PF (based on arrival type) and X may now be solved for, which result in 15.0 sec of delay. If the

LANE DISTRIBUTION FOR SHARED LEFT/THRU LANES ON A MULTILANE APPROACH WITH PERMITTED LEFT TURN LANES (OPTIONAL WORKSHEET)

APPR.	①	②	③	④	⑤	⑥	⑦	⑧	⑨	⑩	⑪
	V_O Opposing Volume (vph)	PCE_{LT}	V_{LT}	LT Equiv. PCE's	Total Volume (TH+RT)	Total	No. of Lanes on Approach	Equiv. Volume Per Lane	Thru Vehicles in LT+TH Lane	Vol. in LT+TH Lane	Vol. in ea. of the Remaining Lanes
	0 — 199 = 1.1 200 — 599 = 2.0 600 — 799 = 3.0 800 — 999 = 4.0 ≥ 1,000 = 5.0			②×③		④+⑤		⑥÷⑦	⑧−④	③+⑨	[(③+⑤)−⑩] / (⑦−1.0)
EB LT	470	2.0	120	240	970	1210	2	605	365	485	605
WB LT	970	4.0	170	680	470	1150	2	575	0	170	470
NB LT	400	2.0	80	160	210	370	1	370			
SB LT	210	2.0	120	240	400	640	1	640			

Figure 9-35. Lane distribution worksheet for Calculation 5.

level of service (delay) were to be allowed to vary as well, a tabular array of X vs. delay and arrival type could be developed for the subject approach. Such a tabular array is shown in Figure 9-36. It was generated using a computer program to simplify computations.

Two presentation formats are shown in Figure 9-36. The upper portion tabulates delay for various arrival types and v/c ratios, X. The lower portion tabulates v/c ratio vs. delay and arrival type. For the solution to this problem, the lower display is most useful. For a delay of 15 sec, maximum v/c ratios are given for arrival types 2–5. A delay of 15 sec/veh cannot be achieved with arrival type 1. Service flow rates, SF are computed as the v/c ratio times the lane group capacity of 1,600 vph. Thus, for LOS B, the approach can carry a maximum service flow rate of 1,183 vph under the existing arrival type (3), but could be improved to as much as 1,418 vph for arrival type 5 with improved progression.

This calculation is intended to illustrate the potential for alternative computational sequences using the basic operational analysis format. It should be noted, however, that this calculation addresses only one lane group and that computations become far more complex where multiple lane groups are to be considered simultaneously. Nevertheless, the procedure is capable of determining service flow rates, as herein, or geometric or signal parameters based on a desired level of service.

Average Stopped Delay For Pretimed Signal
sat flow = 3,200; C = 90; g/C = 0.5; c = 1,600; two through lanes

X	d_1	d_2	FLOW RATE	ARRIVAL TYPE 1	2	3	4	5
0	8.55	0.00	0	15.82	11.54	8.55	6.16	4.53
0.1	9.00	0.00	160	16.65	12.15	9.00	6.48	4.77
0.2	9.50	0.01	320	17.59	12.84	9.51	6.85	5.04
0.3	10.06	0.03	480	18.67	13.62	10.09	7.27	5.35
0.4	10.69	0.09	640	19.94	14.55	10.78	7.76	5.71
0.5	11.40	0.22	800	21.49	15.68	11.62	8.36	6.16
0.6	12.21	0.46	960	23.45	17.11	12.68	9.13	6.72
0.7	13.15	0.97	1,120	23.73	18.08	14.12	10.88	8.47
0.8	14.25	2.11	1,280	24.55	19.96	16.36	13.42	10.96
0.9	15.55	5.30	1,440	30.23	25.02	20.85	17.93	15.64
1	17.10	17.30	1,600	48.16	40.59	34.40	30.96	28.21

Service Flow Rate and v/c Ratio

LOS	MAXIMUM STOPPED DELAY	ARRIVAL TYPE 1 MAX SF	1 X	2 MAX SF	2 X	3 MAX SF	3 X	4 MAX SF	4 X	5 MAX SF	5 X
A	5									296	0.19
	10					455	0.28	1,040	0.65	1,218	0.76
B	15			703	0.44	1,183	0.74	1,336	0.84	1,418	0.89
	20	646	0.40	1,281	0.80	1,410	0.88	1,465	0.92	1,496	0.93
C	25	1,293	0.81	1,439	0.90	1,489	0.93	1,527	0.95	1,559	0.97
	30	1,434	0.90	1,491	0.93	1,548	0.97	1,588	0.99		
	35	1,483	0.93	1,543	0.96						
D	40	1,527	0.95	1,594	1.00						
	45	1,572	0.98								
	50										
	55										
E	60										

Figure 9-36. Tabular presentation of service flow rate solutions for Calculation 6.

V. REFERENCES

The methodology of this chapter is based in part on the results of an NCHRP study conducted by JHK&Associates *(1,2)*. The development of critical movement capacity analysis techniques has occurred in the United States *(3-5)*, Australia *(6)*, Great Britain *(7)*, and Sweden *(8)*. Background for delay estimation procedures was developed in Great Britain *(7)*, Australia *(9, 10)*, and the United States *(11)*.

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11. REILLY, W. R., GARDNER, C. C., and KELL, J. H., "A Technique for Measurement of Delay at Intersections." *FHWA Report No. RD-76-135/137,* Federal Highway Administration, Washington, D.C. (1976).

APPENDIX I

INTERSECTION GEOMETRICS—SUGGESTIONS FOR ESTIMATING DESIGN ELEMENTS

This appendix summarizes suggestions for establishing the geometric design of an intersection, where it is not defined by existing conditions or by state and/or local practice. These suggestions may also be applied where analysis indicates intersection deficiencies that are to be corrected by changes in geometric design. Nothing in this appendix, however, should be construed as constituting strict guidelines or standards. This material should not be used in place of applicable state and local standards, guidelines, policies, or practice.

The geometric design of an intersection involves several critical decisions involving the number and use of lanes to be provided on each approach. The following discussions address these determinations.

EXCLUSIVE LEFT-TURN LANES

Left-turn lanes are provided to accommodate heavy left-turn movements without disruption to through and right-turning vehicles. The provision of an exclusive left-turn lane (or lanes) allows for the use of protected left-turn phasing and provides storage for queued left-turn vehicles without disrupting other flows. The following suggestions are made concerning the provision of exclusive left-turn lanes:

1. Where fully protected left-turn phasing is to be provided, an exclusive left-turn lane should be provided.
2. Where space permits use of a left-turn lane, it should be considered where left-turn volumes exceed 100 vph. Left-turn lanes may be provided for lower volumes as well, based on the judged need and state and/or local practice.
3. Where left-turn volumes exceed 300 vph, provision of a double left-turn lane should be considered.
4. The length of the storage bay should be sufficient to handle the turning traffic without reducing the safety or capacity of the approach. A method for estimating the required length of the storage bay is summarized in Figure I.9-1 and Table I-9-1.

Figure I.9-1 shows the relationship between the left-turn volume (expressed in PCE's) and the length of the turn storage bay. The relationship is based on random arrivals and 5 per-

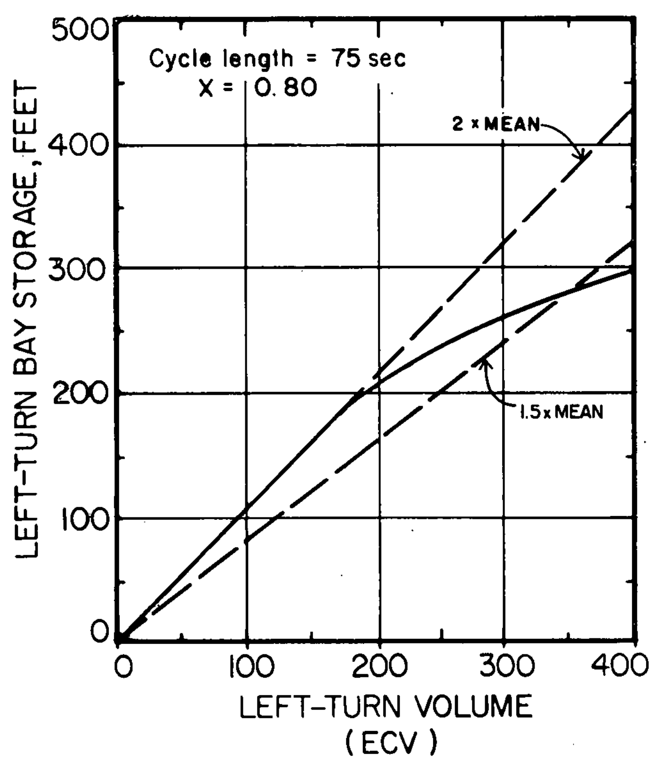


Figure I.9-1. Left-turn bay length vs. turning volume. (Source: C. J. Messer, "Guidelines for Signalized Left-Turn Treatments, *Implementation Package FHWA-IP-81-4,* Federal Highway Administration, Washington, D.C. 1981, Fig. 2)

TABLE I.9-1. LEFT-TURN BAY LENGTH ADJUSTMENT FACTORS

v/c RATIO, X	CYCLE LENGTH, C (SEC)				
	60	70	80	90	100
0.50	0.70	0.76	0.84	0.89	0.94
0.55	0.71	0.77	0.85	0.90	0.95
0.60	0.73	0.79	0.87	0.92	0.97
0.65	0.75	0.81	0.89	0.94	1.00
0.70	0.77	0.84	0.92	0.98	1.03
0.75	0.82	0.88	0.98	1.03	1.09
0.80	0.88	0.95	1.05	1.11	1.17
0.85	0.99	1.06	1.18	1.24	1.31
0.90	1.17	1.26	1.40	1.48	1.56
0.95	1.61	1.74	1.92	2.03	2.14

SOURCE: C. J. Messer, "Guidelines for Signalized Left-Turn Treatments," *Implementation Package FHWA-IP-81-4,* Federal Highway Administration, Washington, D.C., 1981, Table 1.

cent probability of storage bay overflow. Left-turn PCE's are as follows:

TYPE OF TURN	OPPOSING VOLUME (VPH)	PASSENGER CAR EQUIVALENT (PCE)
Protected	—	1.05
Permitted	0 to 199	1.1
	200 to 599	2.0
	600 to 799	3.0
	800 to 999	4.0
	$\geq$ 1,000	5.0

The value obtained from Figure I.9-1 is for a cycle length of 75 sec and a v/c ratio of 0.80. For other values, the length obtained from Figure I.9-1 is multiplied by a correction factor obtained from Table I.9-1. The v/c ratio for left-turn lane groups is computed on the capacity analysis worksheet in operational analysis.

EXCLUSIVE RIGHT-TURN LANES

Exclusive right-turn lanes are provided for many of the same reasons that left-turn lanes are used. Right turns are, however, generally more efficiently made than left-turn movements. Right turns may face a conflicting pedestrian flow, but do not face a conflicting vehicular flow. As a general suggestion, an exclusive right-turn lane should be considered when the right-turn volume exceeds 300 vph and the adjacent main-lane volume also exceeds 300 vphpl.

NUMBER OF LANES

The number of lanes required on an approach depends on a variety of factors, including the signal design. In general, enough main roadway lanes should be provided such that the total of through plus right-turn volume (plus left-turn volume, if present) does not exceed 450 vphpl. This is a very broad suggestion. Higher volumes can be accommodated on major approaches where a substantial portion of available green time can be allocated to the subject approach. Where the number of lanes is unknown, this represents a reasonable starting point for analysis computations.

OTHER FEATURES

Where lane widths are unknown, the 12-ft standard lane width should be assumed, unless known restrictions prevent this. Parking conditions should be assumed to be consistent with local practice in the area. Where no information exists, no curb parking and no local buses should be assumed for analysis purposes.

APPENDIX II

SIGNALIZATION—SUGGESTIONS FOR ESTABLISHING SIGNAL DESIGN IN ANALYSIS

The design of a signal is a complex issue involving three primary determinations:

1. The type of signal controller to be used, i.e., actuated vs. pretimed.
2. The phase plan to be adopted.
3. The allocation of green time among the various phases.

Each of these determinations is heavily influenced by state and local policies, guidelines, and standards—all of which may vary considerably from location-to-location. This appendix presents the alternatives available to the analyst, along with some general discussion of the range of situations in which they are employed. These discussions are intended only to assist the analyst in establishing an initial signal design for study, and do not represent recognized standards or guidelines.

TYPE OF SIGNAL

There are three general types of signal control that are available:

1. *Pretimed control*—In pretimed control, the signal times out a preset sequence of phases in repetitive order. Each phase has a fixed green time and change interval that is repeated in each cycle. The cycle length is constant.

2. *Semiactuated control*—In semiactuated control, vehicle detectors are located on the minor street only. The signal is set such that the green is *always* on the major street unless a minor street actuation is received. Once actuated, the green remains on the side street if additional actuations are received within a preset time interval, subject to a maximum green limitation. When the green is returned to the major street, it remains until another side-street actuation is received, subject to a minimum green time before the side street can recapture the green. Semiactuated signals generally operate on a simple two-phase plan, although major-street left-turn phasing can be utilized in unusual cases. The cycle length for a semiactuated signal may vary from cycle to cycle.

3. *Actuated control*—In fully actuated control, all approaches to the intersection have vehicle detectors. Each phase is subject to a minimum and a maximum green time, and some phases may be "skipped" if no demand is detected. A phase is terminated when (a) there are no further actuations for the phase within the specified time interval, or (b) when the maximum green time has been reached. The cycle length for an actuated signal varies from cycle to cycle.

Pretimed control is the simplest and most inexpensive form of signalization. In certain situations, it will be less efficient than actuated control, because it cannot respond to changes in demand as they occur. Where signals are part of a coordinated progressive signal system, however, pretimed control is often effective, because the cycle length and the initiation of green phases must remain fixed for progression to exist. Thus, pretimed signals are most often used in urban and suburban areas where signals are closely spaced and interconnection is provided. Pretimed signals may operate on different timing plans during different times of the day. "Three-dial controllers" provide for three different timing plans (usually AM, off-peak, and PM) which can be initiated by time-clock at preset times of the day.

Semiactuated control is most often used on intersections of major streets with minor side-streets with low-to-moderate demand. Signals are generally installed at such locations because there are insufficient gaps in the major traffic stream to allow even a low side-street demand to safely cross the intersection. Semiactuated control allows for stopping the major street flow to permit side-street crossings only when there is demand present. Semiactuated control can be used in an overall progressive signal system, but where this is done, the initiation of side-street green phases must be restricted to preset times, and the signals must be coordinated.

Fully actuated control is the most flexible form of signal control. It allows for a cycle-by-cycle adjustment of timing (and even phase sequence) in response to demand changes. Thus, the actuated signal makes most efficient use of available green time by not allowing excess or unused green time in one phase if demand is present on another. Fully actuated control is most often used at isolated intersections that are not coordinated with other signals as part of a progressive system. Occasionally, actuated signals will be placed at individual intersections within a progressive system where maximum efficiency is required. In such cases, it is expected that the platooned arrivals resulting from the progressive system will "drive" the actuated signal in concert with the progression.

In *computer-controlled* signal systems, individual intersections generally operate according to a pretimed plan that is selected and initiated by the computer. Some computer systems provide both coordinated and actuated control.

The selection of a type of control is very much subject to local policy and practice, which should be the primary factor in selecting a controller.

PHASE PLANS

The most critical aspect of any signal design is the selection of an appropriate phase plan. The phase plan involves the determination of the number of phases to be used, and the sequence in which they are implemented. As a general guideline, simple two-phase control should be used unless conditions dictate the need for additional phases. Because the change interval between phases contributes to lost time in the cycle, as the number of phases increases, the percentage of the cycle made up of lost time generally also increases.

Figure II.9-1 shows a number of common phase plans that may be used with either pretimed or actuated controllers, and Figure II.9-2 illustrates an "optional" phasing scheme that typically can be implemented only with actuated controllers. These and other phase plans are discussed in the sections that follow.

Two-Phase Control

Two-phase control is the most straightforward and simple of the available phase plans. Each of two intersecting streets is given a green phase during which all movements on the street are allowed to move. All left and right turns are made on a permitted basis against an opposing vehicle or pedestrian flow. The two-phase plan is shown in Figure II.9-1(a). This phase plan is generally used unless turn volumes require protective phasing.

Multiphase Control

Multiphase control is adopted at any intersection where one or more left or right turns are determined to require protected phasing. It is generally the left-turn movement which would require a partially or fully protected left-turn phase. Local policy and practice are again critical determinants of this need. Most agencies have guidelines on left-turn volumes that require protective phasing. These threshold volumes are generally in the range of 100 to 200 vph turning left. Left-turn phasing is also considered where the speed of opposing traffic is greater than 40 mph.

Multiphase control can be provided in a wide variety of ways, depending on the number of turns requiring protected phasing, and the sequence and overlaps used. Figures II.9-1(b), (c), and (d) present three common plans for multiphase control.

Figure II.9-1(b) illustrates a three-phase plan in which an exclusive left-turn phase is provided for both left-turn movements on the major street. It is followed by a through and right-turn phase for the major street, during which left turns may be permitted on an optional basis.

The use of permitted left-turn phases following protected left-

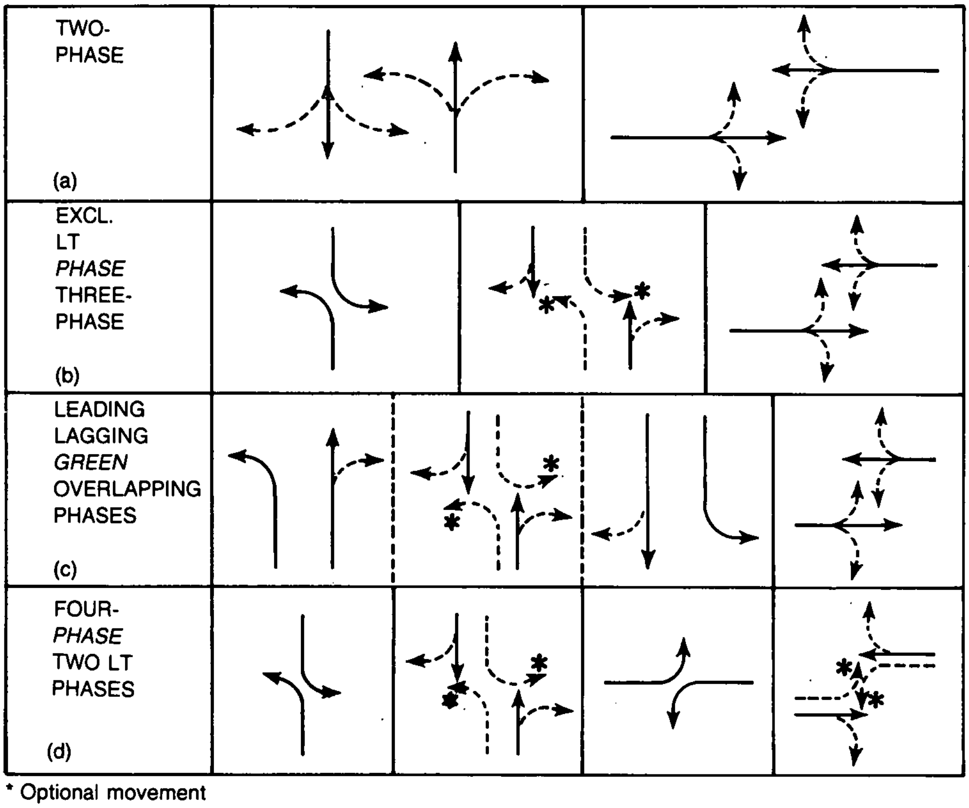


Figure II.9-1. Phase plans for pretimed and actuated control.

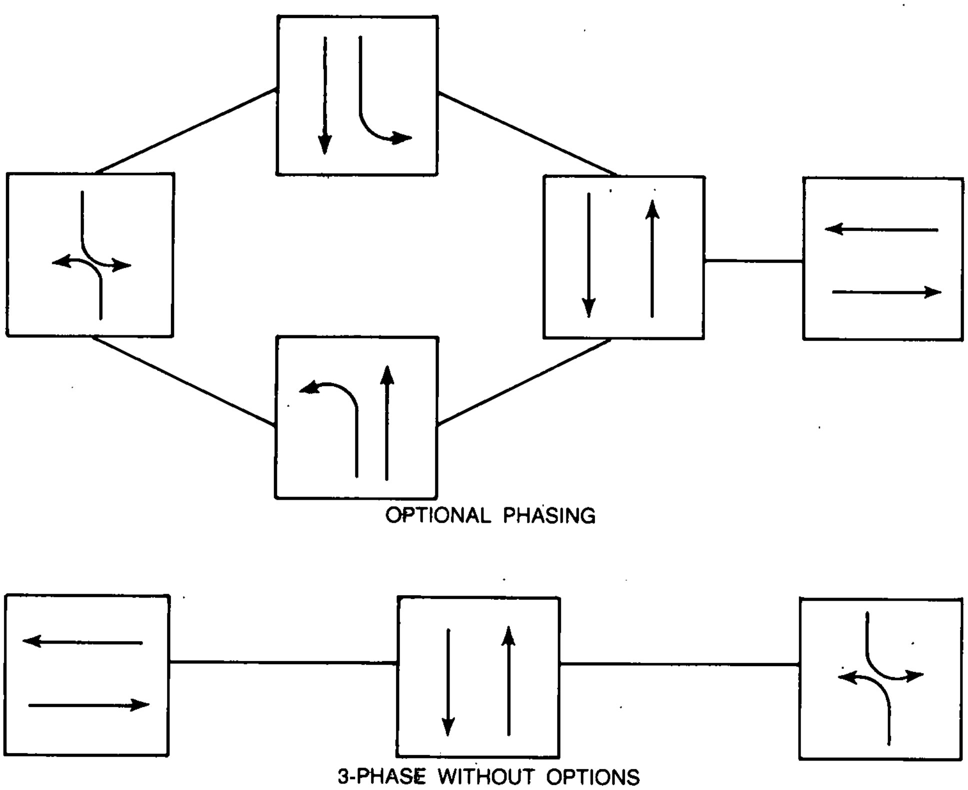


Figure II.9-2. An optional phase plan for actuated control.

turn phases is very much a matter of local practice. Some agencies make extensive use of protected/permitted phasing, while others prefer protected-only phase plans. The phasing illustrated in Figure II.9-1(b) can be used in either mode.

Exclusive left-turn phases provide for simultaneous movement of opposing left-turns, and are most efficient where the opposing left-turn volumes are nearly equal. Where volumes are unequal, or in cases where only one left turn requires protected phasing, other phase plans are more efficient.

The three-phase plan may be expanded to a four-phase sequence if both streets require left-turn phases. Such a sequence is illustrated in Figure II.9-1(d). As previously, left-turns may be continued on a permitted basis during the through phases if desired.

It should be noted that all approaches having an exclusive left-turn phase should have an exclusive left-turn lane as well.

Figure II.9-1(c) illustrates what is commonly referred to as "leading and lagging green" phasing. The initial phase is a through + left-turn phase for one direction of the major street. This is followed by a through phase for both directions of the major street, during which left turns in both directions may be permitted on an optional basis. The direction of flow started in the first phase is then stopped, providing the opposing direction with a through + left-turn phase. The final phase accommodates all movements on the minor street.

Such phasing is extremely flexible. Where only one left turn requires a protected phase, a leading green can be provided without a lagging green phase. Where left-turn volumes are unequal, the lengths of the leading and lagging green can be adjusted to avoid excessive green time for one or both left-turn movements. Leading and/or lagging green phases can even be used where no left-turn lane exists, as long as turns are permitted to continue during the through phase. The phasing of Figure II.9-1(c) may also be expanded to incorporate leading and/or lagging green phases on both streets.

All of the phase plans discussed to this point can be implemented with pretimed or actuated controllers. The only difference in operation would be the manner in which green time is allocated to the various phases: for pretimed controllers, green times are preset, while for actuated controllers, green times vary based on detector actuations.

Figure II.9-2 shows a multiphase plan that can be implemented only with actuated controllers, because it includes an optional phase. The initial phase is an exclusive left-turn phase for the major street. It is followed by a leading green for the major street direction of flow having the larger left-turn demand. Detector actuations will determine which direction gets this phase on a cycle-by-cycle basis. This is followed by a major street through phase and a side-street phase accommodating all side-street movements. The advantage of such phasing over traditional leading and lagging greens is that the left-turn movements are not split, i.e., they are initiated at the same time. This is more flexible than the simple three-phase approach also illustrated in Figure II.9-2.

Once again, this type of phasing could be expanded to provide protected left-turns on both streets.

The establishment of a phase plan is the most creative part of signal design, and deserves the careful attention of the analyst. A good phase plan can achieve great efficiency in the use of available space and time, while an inappropriate plan can cause great inefficiency. The phase plans presented and discussed in this appendix represent a sampling of the more common forms used. They may be combined in a large number of innovative ways on various approaches of an intersection.

Again, local practice is an important determinant in the selection of a phase plan. Phasing throughout an area should generally be relatively uniform. The introduction of protected plus permitted phasing at one location in an area where left turns are generally handled in exclusive left-turn phases, for example, may confuse drivers. Thus, system considerations should also be included when phase plans are established.

ALLOCATION OF GREEN TIME

Once a phase plan and signal type have been established, the allocation of green time may be estimated using Eqs. 9-2 and 9-3:

$$X_c = \sum_i (v/s)_{ci} C/(C - L)$$

$$X_i = v_i C/s_i g_i$$

Equation 9-3 may be manipulated to solve for the cycle length, C:

$$C = LX_c \Big/ \Big[X_c - \sum_i (v/s)_{ci} \Big] \qquad \text{(II.9-1)}$$

Equation 9-2 may be manipulated to solve for the green time for a particular phase, g_i:

$$g_i = v_i C/s_i X_i = (v/s)_i (C/X_i) \qquad \text{(II.9-2)}$$

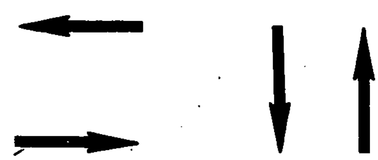
where:

C = cycle length, in sec;
L = lost time per cycle, in sec;
X_c = critical v/c ratio for the intersection;
X_i = v/c ratio for lane group i;
$(v/s)_i$ = flow ratio for lane group i; and
g_i = effective green time for lane group i, in sec.

Cycle lengths and green times may be estimated using these relationships, flow ratios computed as part of the capacity analysis module, and desired v/c ratios.

Pretimed Signals

For pretimed signals, fixed green times and cycle lengths may be estimated using Eqs. II.9-1 and II.9-2. The procedure will be illustrated using a sample calculation. Consider the two-phase signal shown below. Flow ratios are shown, and it is assumed that lost times equal the change intervals, which are 4 sec for each phase or 8 sec/cycle.



v/s = 0.45 $\qquad$ v/s = 0.35

The cycle length is computed from Eq. II.9-1 for the desired v/c ratio, X_c, which must be selected by the analyst. The absolute minimum cycle length may be computed using $X_c = 1.00$:

$$C(\text{minimum}) = LX_c/\left[X_c - \sum_i (v/s)_{ci}\right]$$

$$C(\text{minimum}) = 8\ (1.0)/[1.0 - (0.45 + 0.35)]$$
$$= 8/0.2 = 40 \text{ sec}$$

If a v/c ratio of no more than 0.8 were desired, the computation would become:

$$C = 8\ (0.80)/[0.8 - (0.45 + 0.35)] = 8/0 = \text{infinite}$$

This computation indicates that a critical v/c ratio of 0.8 cannot be provided for the demand levels existing at the intersection. Any cycle length of greater than 40 sec may be selected. For the purposes of illustration, assume a cycle length of 60 sec. In all cases, the cycle length assumed would be rounded to the nearest 5 sec for values between 30 and 90 sec, and to the nearest 10 sec for higher values.

The actual critical v/c ratio provided by a 60-sec cycle is:

$$X_c = \sum_i (v/s)_i C/(C - L)$$

$$X_c = (0.45 + 0.35)(60)/(60 - 8) = 0.923$$

A number of different policies may be employed in allocating the available green time. A common policy for two-phase signals is to allocate the green such that the v/c ratios for critical movements in each phase are equal. Thus, for the example problem, the v/c ratio for each phase would be 0.923, and:

$$\begin{array}{lll}
g_i & = (v/s)_i(C/X_i) & \\
g_1 & = 0.45\ (60/0.923) & = 29.3 \text{ sec} \\
g_2 & = 0.35\ (60/0.923) & = \underline{22.7 \text{ sec}} \\
 & & \ 52.0 \text{ sec} \\
\text{Lost time} & & = \underline{\ 8.0 \text{ sec}} \\
 & & \ 60.0 \text{ sec}
\end{array}$$

Another common policy would be to allocate the minimum required green time to the minor approach, and assign all remaining green to the major approach. In this case, the v/c ratio for phase 2 would be 1.0, and:

$$\begin{array}{lll}
g_2 & = 0.35\ (60/1.0) & = 21.0 \text{ sec} \\
g_1 & = 60 - 8 - 21 & = \underline{31.0 \text{ sec}} \\
 & & \ 52.0 \text{ sec} \\
\text{Lost time} & & = \underline{\ 8.0 \text{ sec}} \\
 & & \ 60.0 \text{ sec}
\end{array}$$

Note that in both cases the entire 60-sec cycle is fully allocated among the green times and lost time.

The procedure for timing may be summarized as:

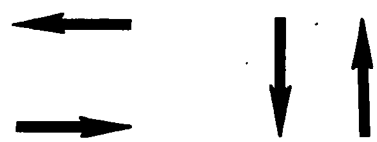
1. Estimate the minimum cycle length using Eq. II.9-1 and $X_c = 1.0$.
2. Estimate the cycle length for the desired critical v/c ratio, X_c, using Eq. II.9-1.
3. From the results of steps 1 and 2, select an appropriate cycle length for the signal. Where system constraints determine the cycle length, steps 1 and 2 may be eliminated.
4. Estimate the green times using Eq. II.9-2 and v/c ratios, X_i, appropriate to the proportioning policy adopted.
5. Check the timing to ensure that the sum of green times + the lost time equals the cycle length. Include overlapping green times only once in this summation.

Semiactuated Signals

In cases where a semiactuated signal is proposed, the timing is estimated in a manner similar to the procedure outlined previously. Where existing semiactuated signals are in place, the average timing can be determined by field observation, or estimated using the procedure outlined herein. Note that for semiactuated signals, the cycle length and green times vary from cycle to cycle, and computations estimate the *average* timing for the flow period under study.

The procedure is the same as that for pretimed signals, except for the fact that the minor street is always timed for the minimum possible green time, i.e., X(side-street) is 1.00. Note that it is not the purpose of this appendix to provide suggestions on setting minimum and maximum green times, but it is to estimate the average timing of the signal for inclusion in capacity analysis procedures.

Consider the following problem. Lost time is again assumed to be 4 sec/phase or 8 sec/cycle.



$v/s = 0.55$ $v/s = 0.15$

A semiactuated signal is assumed to make reasonably efficient use of available green time. Excess green time, however, can exist on the major street. To estimate a cycle length, it may be assumed that the signal will operate at a critical v/c ratio, X_c, in the range of 0.80 to 0.90. A value of 0.85 will be used for the example given:

$$C = LX_c/[X_c - \sum_i (v/s)_i]$$

$$C = 8\ (0.85)/[0.85 - (0.55 + 0.15)] = 45.3 \text{ sec}$$

Because the signal is semiactuated, the cycle length is not rounded off. The green time for the side street is estimated using a value of $X_2 = 1.0$:

$$\begin{array}{lll}
g_2 & = 0.15\ (45.3/1.0) & = \ 6.8 \text{ sec} \\
g_1 & = 45.3 - 8 - 6.8 & = \underline{30.5 \text{ sec}} \\
 & & \ 37.3 \text{ sec} \\
\text{Lost time} & & = \underline{\ 8.0 \text{ sec}} \\
 & & \ 45.3 \text{ sec}
\end{array}$$

The steps of this procedure can be generalized as follows:

1. Estimate the average cycle length for the signal using Eq. II.9-1 and a critical v/c ratio, X_c, in the range of 0.80 to 0.90.
2. Allocate green time to the side-street using Eq. II.9-2 and a v/c ratio, X, of 1.0.
3. Assign all remaining green time to the major street.
4. Check the timing to ensure that the sum of green times + lost time is equal to the cycle length. Include overlapping green times only once in this summation.

Actuated Signals

The average timing of fully actuated signals can also be estimated using Eqs. II.9-1 and II.9-2. The procedure may be applied where the timing of a future actuated signal is being estimated and/or where the average timing of an existing signal has not been observed in the field.

Figure II.9-3 illustrates a problem with complex overlapping phases, along with the flow ratios for each of the lane groups served. Note that the cycle contains an optional phase: the second phase is a leading green for either the eastbound or westbound approaches. As the westbound left-turn flow ratio is higher than that for the eastbound left-turn flow ratio, it will be assumed that (on the average) this phase is a *westbound* leading green.

For this sequence of overlapping phases, the sum of critical lane flow ratios is not obvious. The lower portion of Figure II.9-3 illustrates the possible combinations, and indicates that the critical lane groups are the EB TH, the WB LT, and the NB LT/TH/RT.

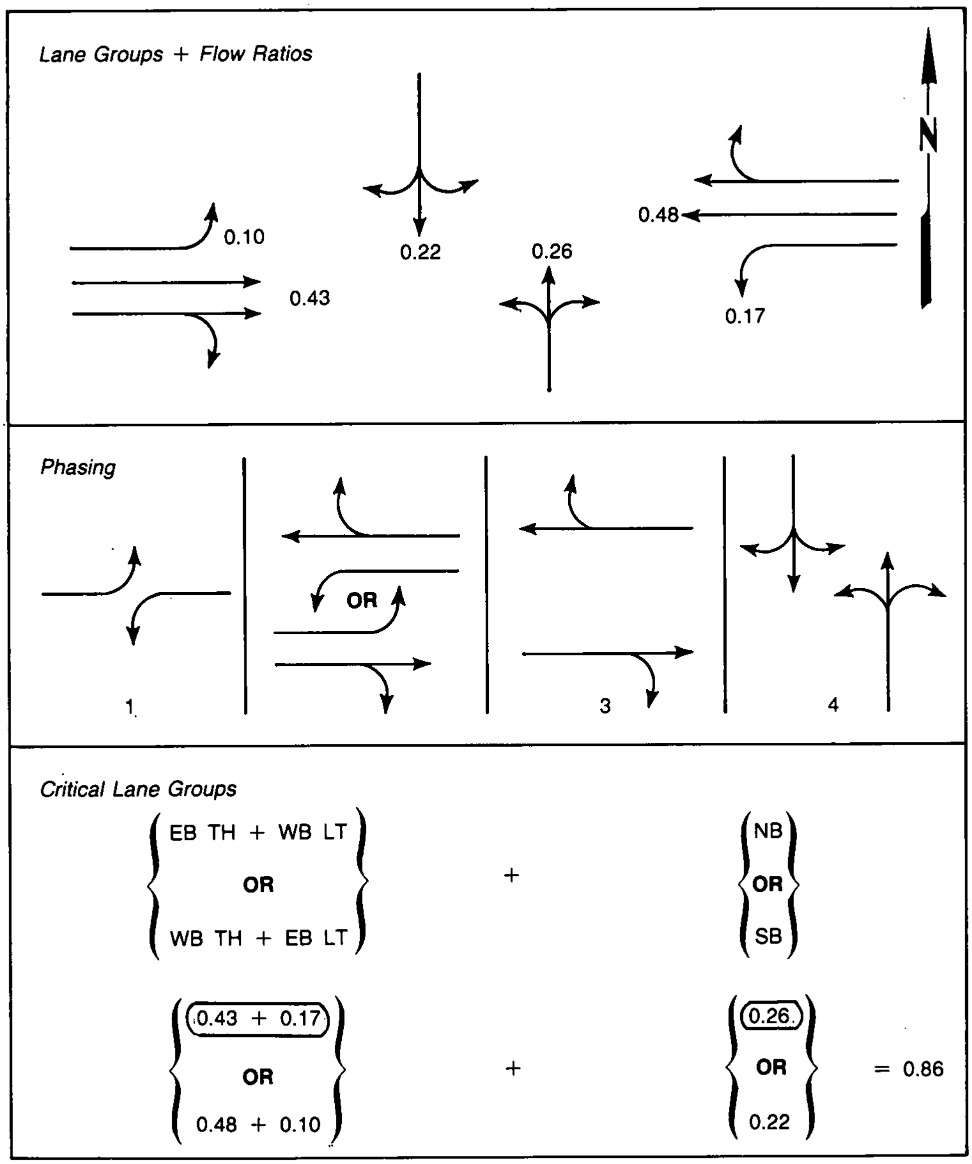


Figure II.9-3. Timing an actuated signal with phase overlaps.

An actuated signal is assumed to be extremely efficient in the use of available green time. Thus, the average cycle length should be estimated using a high critical v/c ratio of approximately X_c = 0.95. Lost time will be taken to be 3 sec/phase, or 9 sec for the cycle. Note that although the change intervals for the two left-turn movements occur at different times in the cycle, they contribute only once to the total cycle lost time. Then:

$$C = L\,X_c/\left[X_c - \sum_i (v/s)_i\right]$$

$$C = 9\ (0.95)/[0.95 - 0.86] = 95 \text{ sec}$$

This value will not be rounded because the signal is actuated, and any cycle length may result from its operation. Green times will be computed based on a constant v/c ratio of 0.95, because it assumed that the actuated signal assigns green time proportionally. Determination of individual phase times must, however, consider the phase overlaps.

The eastbound left-turn is permitted only during phase 1. The green time for phase 1 is, therefore, controlled by the flow ratio for this movement:

$$g_1 \text{ (EB LT)} = 0.10\ (95/0.95) = 10 \text{ sec}$$

The westbound left-turn is allowed during phases 1 and 2. Thus, the total green time for these two phases is controlled by the flow ratio for this movement:

$$g_1 + g_2 \text{ (WB LT)} = 0.17\ (95/0.95) = 17 \text{ sec}$$

The westbound through-movement is allowed during phases 2 and 3. Thus, the total green time for phases 2 and 3 is controlled by the flow ratio for the westbound through-movement:

$$g_2 + g_3 \text{ (WB TH)} = 0.48\ (95/0.95) = 48 \text{ sec}$$

The eastbound through-movement is allowed only during phase 3. Thus, the green time for this phase is controlled by the flow ratio for the eastbound through movement:

$$g_3 \text{ (EB TH)} = 0.43\ (95/0.95) = 43 \text{ sec}$$

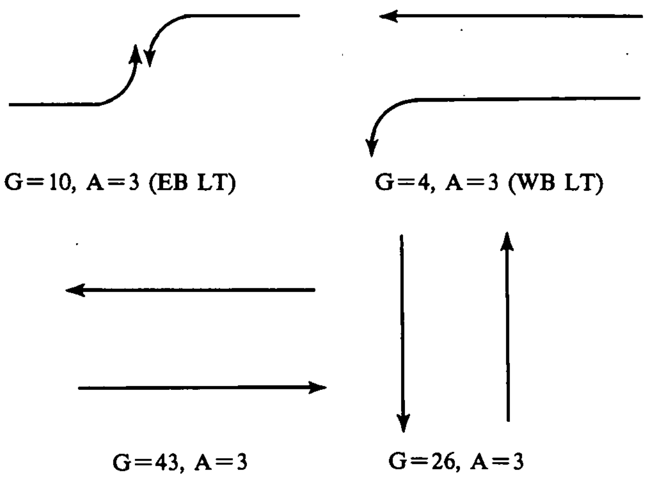
The minor street movement is controlled by the northbound flow ratio, and:

$$g_4 = 26\ (95/0.95) = 26 \text{ sec}$$

The times of all phases have been discretely determined, except for phase 2. If g_1 = 10 sec and $g_1 + g_2$ = 17 sec, then g_2 may be deduced. It is *not,* however, 17 − 10 = 7 sec. Note from the phase diagram of Figure II.9-3 that the westbound left turn moves through the first green phase, the second green phase, *and* the change interval between them. If the change interval is 3 sec, g_2 = 17 − (10 + 3) = 4 sec.

There is another possible computation for phase 2. The green time for phases 2 and 3 is 48 sec, while g_3 = 43 sec. By the same logic as that noted in preceding paragraph, g_2 = 48 − (43 + 3) = 2 sec. As the first requirement is larger, it is taken to control the timing.

The final timing of the signal is as follows:



SUMMARY

This appendix has presented a sampling of information on the design of signals. It is a complex issue that cannot be completely covered in this abbreviated treatment. State and local guidelines, standards, practice, or policy should also be consulted in developing signal designs.

The information presented herein is intended only to assist the analysis in estimating a signal design or timing for use in capacity analysis of signalized intersections.

APPENDIX III

MEASUREMENT OF INTERSECTION DELAY IN THE FIELD

As an alternative to the estimation of average stopped-time delay per vehicle using Eq. 9-18 and the progression factor, delay at existing locations may be measured directly. There are a number of methods for making this measurement, including the use of test-car observations and the recording of arrival and departure volumes on a cycle-by-cycle basis. The method summarized here is based on direct observation of "stopped-vehicle counts" at the intersection, and is the most common approach to direct measurement of intersection delay.

Figure III.9-1 shows a field sheet that may be used for the recording of observations and the computation of average stopped-time delay. The following steps should be observed:

INTERSECTION DELAY WORKSHEET								
	NUMBER OF STOPPED VEHICLES							
MIN. / SEC.								
Totals								

$V_S =$ ______

$\text{Delay} = \dfrac{\Sigma V_S \times I}{V} =$ ________

Volume, $V =$ ______

Figure III.9-1. Worksheet for field observation of intersection delay.

1. Identify the farthest extent of standing queues during the period of interest. This point should be noted and used as the limit of "stopped-vehicle" counts.

2. At regular intervals of between 10 and 20 sec, the number of vehicles stopped within the intersection are counted. Only those vehicles stopped are counted. This is a stopped-vehicle count. The limits of the intersection (for the lane group under observation) are the limit of standing queues identified in step 1 and the exit boundaries of the intersection—i.e., a vehicle stopped within the intersection itself is included as part of the stopped-vehicle count. Stopped-vehicle counts are entered on the field sheet in the appropriate box. Minutes are listed in the leftmost column of the sheet, with second intervals given as column headings.

3. During the entire period of the study, a volume count should be maintained, counting vehicles as they cross the stop-line of the lane group under study. The total volume count is listed in the appropriate box on the worksheet.

4. Sum each column of stopped-vehicle counts; sum the column totals to compute the total of all density observations in the study period.

5. If it is assumed that a vehicle observed stopped in the intersection as part of one of the stopped-vehicle counts is stopped for an average of the interval between stopped-vehicle counts, the average stopped delay per vehicle can be computed as:

$$\text{Delay} = \left(\sum V_s \times I\right)/V$$

where:

$\sum V_s$ = sum of stopped-vehicle counts;
I = interval between stopped-vehicle counts, in sec; and
V = total volume observed during study period.

Consider the following sample data taken during a 10-min study period on a given intersection lane group:

	Seconds						
Minutes	+0		+20		+40		
5:00 PM	2		4		2		
5:01	3		5		0		
5:02	6		3		5		
5:03	4		5		3		
5:04	2		2		4		
5:05	4		4		6		
5:06	5		2		1		
5:07	1		3		2		
5:08	4		4		3		
5:09	2		6		2		
	33	+	38	+	28	=	99

The sum of the stopped-vehicle counts is 99. Observed volume during the study period is 50 vehicles. The average stopped delay per vehicle is then computed as:

$$\text{Delay} = 99 \times 20/50 = 40 \text{ sec/veh}$$

APPENDIX IV

DIRECT MEASUREMENT OF PREVAILING SATURATION FLOW RATES

The base saturation flow rate used in the methodology of this chapter is 1,800 pcphgpl. This value must be adjusted for prevailing traffic conditions such as: lane width, left turns, right turns, heavy vehicles, grades, parking, parking blockage, area type, bus blockage, and left-turn blockage. These computations are made in the saturation flow rate module. As an alternative to these computations, the prevailing saturation flow rate may be measured directly in the field.

Saturation flow rates have been measured and researched by various groups, including the City of Edmonton in conjunction with the University of Alberta, the University of Kentucky, and the Australian Road Research Board. Results of these studies have demonstrated that saturation flow rates have a high degree of variability. A study conducted by JHK&Associates showed that median saturation flow rates for through and turn lanes for fair-to-good geometric and traffic conditions were 1,600 and 1,500 vphgpl, respectively.

Local saturation flow rates may be observed directly, and used directly in operational analysis, as a substitute for the computations of the saturation flow module. Severe weather conditions, unusual traffic mixes, or other critical local conditions can cause values to vary from those estimated using the computations recommended in the methodology. The procedure for observing saturation flow rates is summarized below. A field sheet for recording observations is included as Figure IV.9-1.

FIELD SHEET—SATURATION FLOW STUDY

Location:____________________

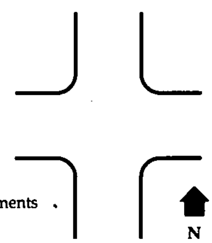
Date:____/____/____ Time:__________ City:__________

______Bound Traffic; Approaching From the__________

Observers:__________ Weather:__________

Movements Allowed
- ☐ Thru
- ☐ Right Turn
- ☐ Left Turn

Identify all Lane Movements & The Lane Studied



Veh. in Queue	Cycle 1			Cycle 2			Cycle 3			Cycle 4			Cycle 5			Cycle 6		
	Time	HV	T	Time	HV	T	Time	HV	T	Time	HV	T	Time	HV	T	Time	HV	T
1																		
2																		
3																		
4																		
5																		
6																		
7																		
8																		
9																		
10																		
11																		
12																		
13																		
14																		
15																		
16																		
17																		
18																		
19																		
20																		
End of Saturation																		
End of Green																		
No. Veh. > 20																		
No. Veh. on Yellow																		

HV = Heavy Vehicles (Vehicles with more than 4 tires)
T = Turning Vehicles (L = Left, R = Right)
Pedestrians and buses which block vehicles should be noted with the time that they block traffic, i.e.,
P12 = pedestrians blocked traffic for 12 sec
B15 = bus blocked traffic for 15 sec

Grade__________ Area Type__________

Figure IV.9-1. Field sheet for direct observation of prevailing saturation flow rate.

A two-person field crew is recommended, one being assigned as a *timer* and the other as an *observer.*

GENERAL TASKS

1. Fill out the data form of Figure IV.9-1 completely.
2. Pick an observation point where the STOP line or crosswalk and the signal observations are clearly visible.
3. Choose a reference point, usually the crosswalk or STOP line. Vehicles should consistently stop *behind* this reference point. When a vehicle crosses this reference point, it has entered the intersection.
4. Perform a study for each cycle.

RECORDER TASKS

1. Note the last vehicle in the stopped queue when signal turns green.
2. Describe the last vehicle to the timer.
3. Note on the form which vehicles are "heavy vehicles" and which vehicles turn left or right.
4. Record the time called out by the timer.

TIMER TASKS

1. Start stop watch at beginning of green and call out to the recorder.
2. Count aloud each vehicle in the queue as it crossed the reference point with its rear axle (e.g., "one," "two," "three," etc.)
3. Call out the time of the fourth, tenth, and last vehicles in the queue. This can be done during the queue departure with a typical stop-watch. Newer and more sophisticated stop-watches with memory permit the timer to call out these times after the queue has dissipated.
4. If queued vehicles are still entering the intersection at the end of the green, call out "saturation through the end of green, last vehicle was number XX."

Note any unusual events that may have influenced the saturation flow rate, such as buses, stalled vehicles, unloading trucks, and so on. Measure and record the area type and width and grade of the lane being studied.

The period defined as saturation flow begins when the rear axle of the fourth vehicle in queue crossed the STOP line or reference point and ends when the rear of the last axle of the last queued vehicle at the beginning of the green crosses the same point. As described in the instructions, measurements are taken by cycle and by lane. To reduce the data, the time recorded for the fourth vehicle is subtracted from the time recorded for the last vehicle in queue. This value is the total headway for $n - 4$ vehicles, where n is total number of vehicles queued at the beginning of the green (or the number of the last vehicle in the queue). The total headway is divided by $n - 4$ to obtain the average headway per vehicle under saturation flow. The saturation flow rate is 3,600 divided by this value.

For example, if the time for the fourth vehicle was observed as 10.2 sec, and the time for the 14th and last vehicle in queue was 36.5 sec, the average saturation headway per vehicle would be:

$$(36.5 - 10.2)/(14 - 4) = 26.3/10 = 2.63 \text{ sec/veh}$$

and the saturation flow rate would be:

$$3{,}600/2.63 = 1{,}369 \text{ vphgpl}$$

APPENDIX V

WORKSHEETS FOR USE IN ANALYSIS

WORKSHEETS *PAGE*

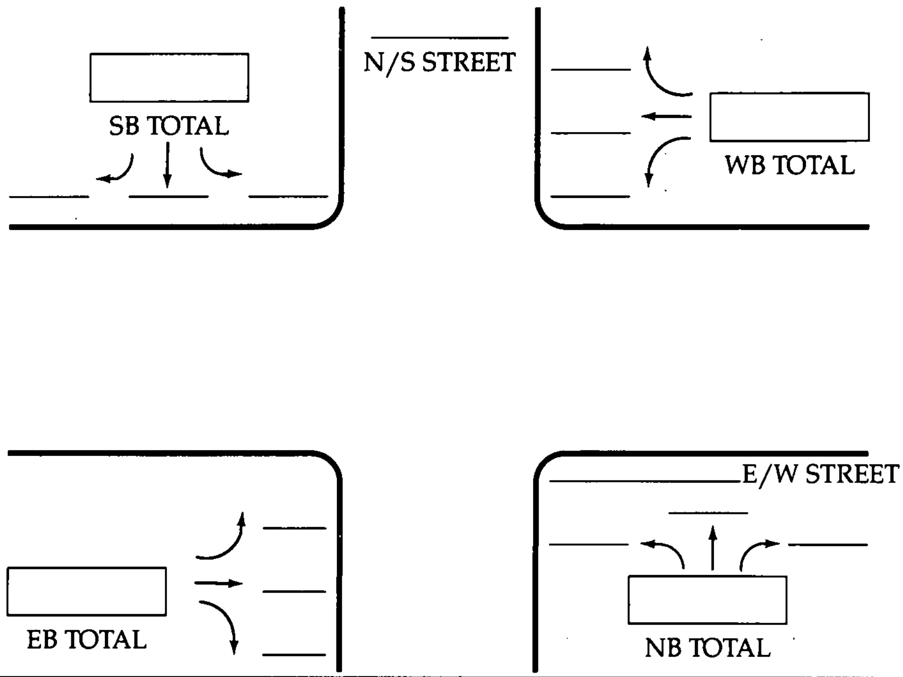
INPUT WORKSHEET

Intersection: ______________________ Date: ______________

Analyst: ______________ Time Period Analyzed: ______________ Area Type: ☐ CBD ☐ Other

Project No.: ______________________ City/State: ______________________

VOLUME AND GEOMETRICS



IDENTIFY IN DIAGRAM:

1. Volumes
2. Lanes, lane widths
3. Movements by lane
4. Parking (PKG) locations
5. Bay storage lengths
6. Islands (physical or painted)
7. Bus stops

TRAFFIC AND ROADWAY CONDITIONS

Approach	Grade (%)	% HV	Adj. Pkg. Lane		Buses (N_B)	PHF	Conf. Peds. (peds./hr)	Pedestrian Button		Arr. Type
			Y or N	N_m				Y or N	Min. Timing	
EB										
WB										
NB										
SB										

Grade: + up, − down
HV: veh. with more than 4 wheels
N_m: pkg. maneuvers/hr

N_B: buses stopping/hr
PHF: peak-hour factor
Conf. Peds: Conflicting peds./hr

Min. Timing: min. green for pedestrian crossing
Arr. Type: Type 1-5

PHASING

DIAGRAM								
Timing	G = Y + R =	G = Y + R =	G = Y + R =	G = Y + R =	G = Y + R =	G = Y + R =	G = Y + R =	G = Y + R =
Pretimed or Actuated								

Protected turns | Permitted turns | ------- Pedestrian | Cycle Length ______ Sec

VOLUME ADJUSTMENT WORKSHEET

① Appr.	② Mvt.	③ Mvt. Volume (vph)	④ Peak Hour Factor PHF	⑤ Flow Rate v_p (vph) ③ ÷ ④	⑥ Lane Group	⑦ Flow rate in Lane Group v_g (vph)	⑧ Number of Lanes N	⑨ Lane Utilization Factor U Table 9-4	⑩ Adj. Flow v (vph) ⑦ × ⑨	⑪ Prop. of LT or RT P_{LT} or P_{RT}
EB	LT									
	TH									
	RT									
WB	LT									
	TH									
	RT									
NB	LT									
	TH									
	RT									
SB	LT									
	TH									
	RT									

SATURATION FLOW ADJUSTMENT WORKSHEET

LANE GROUPS				ADJUSTMENT FACTORS								
① Appr.	② Lane Group Movements	③ Ideal Sat. Flow (pcphgpl)	④ No. of Lanes N	⑤ Lane Width f_w Table 9-5	⑥ Heavy Veh f_{HV} Table 9-6	⑦ Grade f_g Table 9-7	⑧ Pkg. f_p Table 9-8	⑨ Bus Blockage f_{bb} Table 9-9	⑩ Area Type f_a Table 9-10	⑪ Right Turn f_{RT} Table 9-11	⑫ Left Turn f_{LT} Table 9-12	⑬ Adj. Sat. Flow Rate s (vphg)
EB												
WB												
NB												
SB												

SUPPLEMENTAL WORKSHEET FOR LEFT-TURN ADJUSTMENT FACTOR, f_{LT}

INPUT VARIABLES	EB	WB	NB	SB
Cycle Length, C (sec)				
Effective Green, g (sec)				
Number of Lanes, N				
Total Approach Flow Rate, v_a (vph)				
Mainline Flow Rate, v_M (vph)				
Left-Turn Flow Rate, v_{LT} (vph)				
Proportion of LT, P_{LT}				
Opposing Lanes, N_o				
Opposing Flow Rate, v_o (vph)				
Prop. of LT in Opp. Vol., P_{LTO}				

COMPUTATIONS	EB	WB	NB	SB
$S_{op} = \dfrac{1800\,N_o}{1 + P_{LTO}\left[\dfrac{400 + v_M}{1400 - v_M}\right]}$				
$Y_o = v_o / S_{op}$				
$g_u = (g - CY_o) / (1 - Y_o)$				
$f_s = (875 - 0.625\,v_o) / 1000$				
$P_L = P_{LT}\left[1 + \dfrac{(N-1)\,g}{f_s g_u + 4.5}\right]$				
$g_q = g - g_u$				
$P_T = 1 - P_L$				
$g_f = 2\,\dfrac{P_T}{P_L}\left[1 - P_T^{\,0.5\,g_q}\right]$				
$E_L = 1800 / (1400 - v_o)$				
$f_m = \dfrac{g_f}{g} + \dfrac{g_u}{g}\left[\dfrac{1}{1 + P_L(E_L - 1)}\right] + \dfrac{2}{g}(1 + P_L)$				
$f_{LT} = (f_m + N - 1) / N$				

CAPACITY ANALYSIS WORKSHEET

LANE GROUP ① Appr.	LANE GROUP ② Lane Group Movements	③ Adj. Flow Rate v (vph)	④ Adj. Sat. Flow Rate s (vphg)	⑤ Flow Ratio v/s ③ ÷ ④	⑥ Green Ratio g/C	⑦ Lane Group Capacity c (vph) ④×⑥	⑧ v/c Ratio X ③ ÷ ⑦	⑨ Critical ? Lane Group
EB								
WB								
NB								
SB								

Cycle Length, C _______ sec

Lost Time Per Cycle, L _______ sec

$\sum_i (v/s)_{ci} =$ _______

$$X_c = \frac{\sum_i (v/s)_{ci} \times C}{C - L} = \_\_\_\_\_\_\_$$

LEVEL-OF-SERVICE WORKSHEET

Lane Group		First Term Delay				Second Term Delay				Total Delay & LOS		
① Appr.	② Lane Group Move-ments	③ v/c Ratio X	④ Green Ratio g/C	⑤ Cycle Length C (sec)	⑥ Delay d_1 (sec/veh)	⑦ Lane Group Capacity c (vph)	⑧ Delay d_2 (sec/veh)	⑨ Progression Factor PF Table 9-13	⑩ Lane Group Delay (sec/veh) (⑥+⑧) × ⑨	⑪ Lane Group LOS Table 9-1	⑫ Approach Delay (sec/veh)	⑬ Appr. LOS Table 9-1
EB												
WB												
NB												
SB												

Intersection Delay ________ sec/veh

Intersection LOS ________ (Table 9-1)

PLANNING APPLICATION WORKSHEET

Intersection: ______________________ Date: ______________

Analyst: ______________________ Time Period Analyzed: ______________

Project No. ______________________ City/State: ______________

SB TOTAL

N-S STREET

WB TOTAL

EB TOTAL

E-W STREET

NB TOTAL

EB LT = ____	
WB TH = ____	
WB LT = ____	OR
EB TH = ____	

NB LT = ____	
SB TH = ____	
SB LT = ____	OR
NB TH = ____	

MAXIMUM SUM OF CRITICAL VOLUMES	CAPACITY LEVEL
0 TO 1,200	UNDER
1,201 to 1,400	NEAR
> 1,400	OVER

______ + ______ = ______ STATUS? ______

E-W CRITICAL N-S CRITICAL

LANE DISTRIBUTION FOR SHARED LEFT/THRU LANES ON A MULTILANE APPROACH WITH PERMITTED LEFT TURN LANES (OPTIONAL WORKSHEET)

	①	②	③	④	⑤	⑥	⑦	⑧	⑨	⑩	⑪
	V_O Opposing Volume (vph)	PCE_{LT}	V_{LT}	LT Equiv. PCE's	Total Volume (TH+RT)	Total	No. of Lanes on Approach	Equiv. Volume Per Lane	Thru Vehicles in LT+TH Lane	Vol. in LT+TH Lane	Vol. in ea. of the Remaining Lanes
APPR.	0 — 199 = 1.1 200 — 599 = 2.0 600 — 799 = 3.0 800 — 999 = 4.0 ≥ 1,000 = 5.0			②×③		④+⑤		⑥÷⑦	⑧−④	③+⑨	$\frac{(③+⑤)-⑩}{⑦-1.0}$
EB LT											
WB LT											
NB LT											
SB LT											

INTERSECTION DELAY WORKSHEET

	NUMBER OF STOPPED VEHICLES							
SEC. / MIN.								
Totals								

$V_S =$ _______

$\text{Delay} = \dfrac{\Sigma V_S \times I}{V} =$ _________

Volume, $V =$ _______

FIELD SHEET – SATURATION FLOW STUDY

Location:____

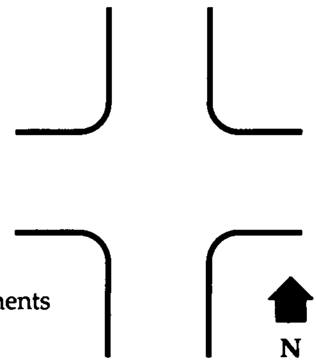
Date:____/____/____ Time:______________ City:______________

______Bound Traffic; Approaching From the____________________

Observers:____________________ Weather:____________________

Movements Allowed
- ☐ Thru
- ☐ Right Turn
- ☐ Left Turn

Identify all Lane Movements & The Lane Studied



Veh. in Queue	Cycle 1			Cycle 2			Cycle 3			Cycle 4			Cycle 5			Cycle 6		
	Time	HV	T	Time	HV	T	Time	HV	T	Time	HV	T	Time	HV	T	Time	HV	T
1																		
2																		
3																		
4																		
5																		
6																		
7																		
8																		
9																		
10																		
11																		
12																		
13																		
14																		
15																		
16																		
17																		
18																		
19																		
20																		
End of Saturation																		
End of Green																		
No. Veh. > 20																		
No. Veh. on Yellow																		

HV = Heavy Vehicles (Vehicles with more than 4 tires)
T = Turning Vehicles (L = Left, R = Right)
Pedestrians and buses which block vehicles should be noted with the time that they block traffic, i.e.,
P12 = pedestrians blocked traffic for 12 sec
B15 = bus blocked traffic for 15 sec

Grade____________ Area Type____________

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.

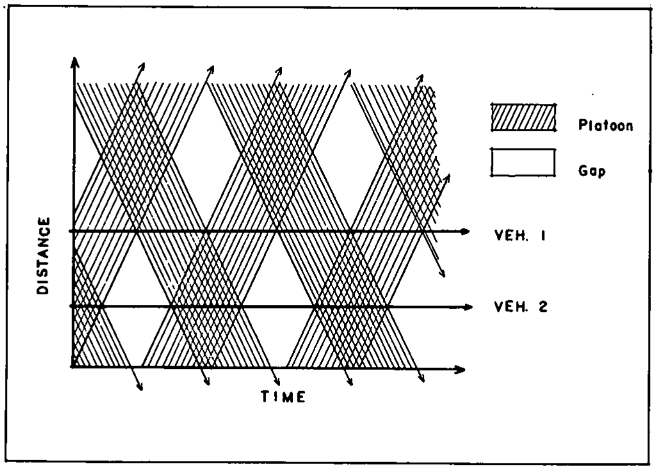


Figure 10-1. Impacts of platoon flow on gap distribution.

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.*

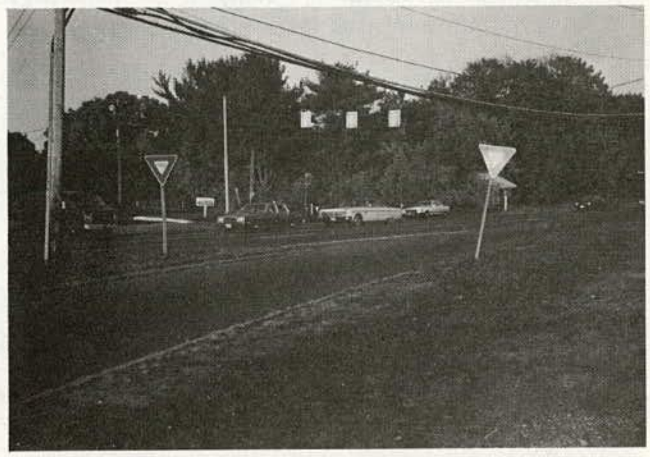


Illustration 10-3. YIELD *signs are used at this right-turn roadway where approach speeds and sight distance are such that vehicles need not come to a full stop to safely select a gap in the major street flow.*

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. pc ph

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.

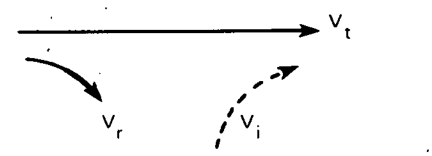
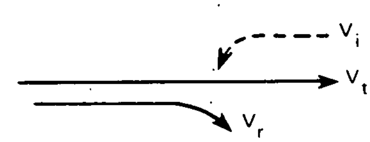
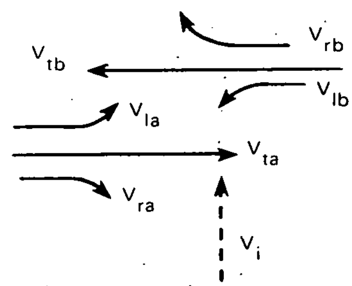
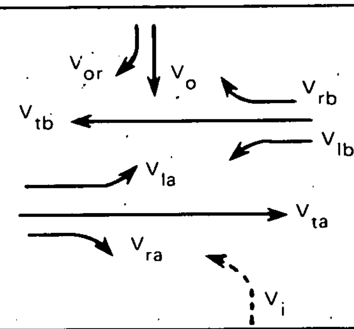
CONFLICTING TRAFFIC

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.	$1/2(V_{ra})^{\overset{***}{**}} + V_{ta} + V_{la}$ $+ V_{rb} + V_{tb} + V_{lb}$	
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 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 in passenger cars per hour 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.

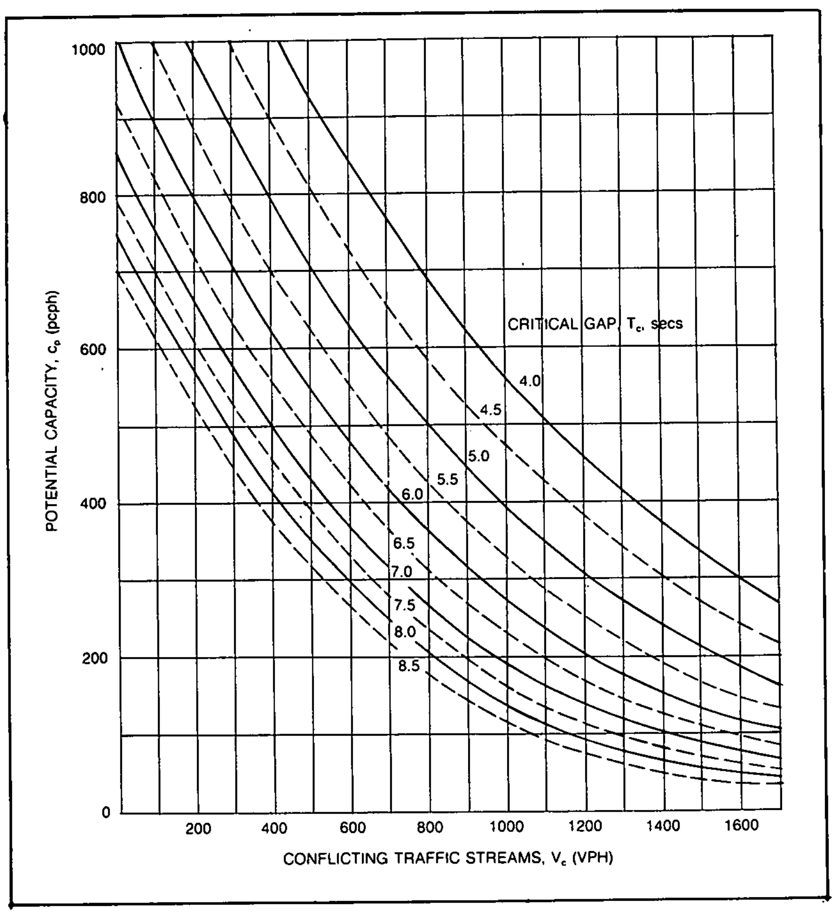


Figure 10-3. Potential capacity based on conflicting traffic volume and critical gap size.

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.

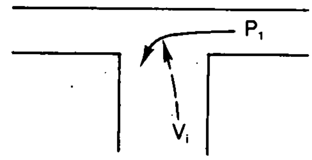
Right turns from the minor street do not generally impede other traffic elements, except for opposing left turns from the minor street where both movements are merging into the same traffic stream. Given the priority of gap usage:

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.

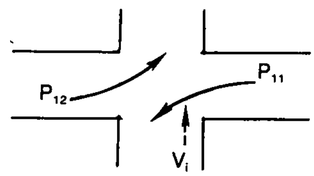
Impedance factors, P_j, are found from Figure 10-5. They are based solely on the percent of potential capacity of the impeding movement used by existing demand. Consider the following example. A left-turn movement from a minor street at a T-intersection is impeded by the left turn from the major street. The latter movement has a potential capacity of 500 pcph and

1. *Left turns* from minor street at a T-intersection.



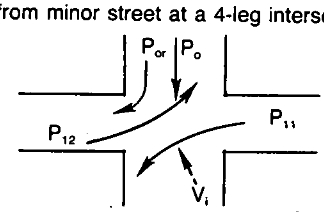
$$c_{mi} = c_{pi} \times P_1$$

2. *Through traffic* from minor street at a 4-leg intersection.



$$c_{mi} = c_{pi} \times P_{11} \times P_{12}$$

3. *Left turns* from minor street at a 4-leg intersection.



$$c_{mi} = c_{pi} \times P_{11} \times P_{12} \times P_o \times P_{or}$$

Figure 10-4. Illustration of impedance computations.

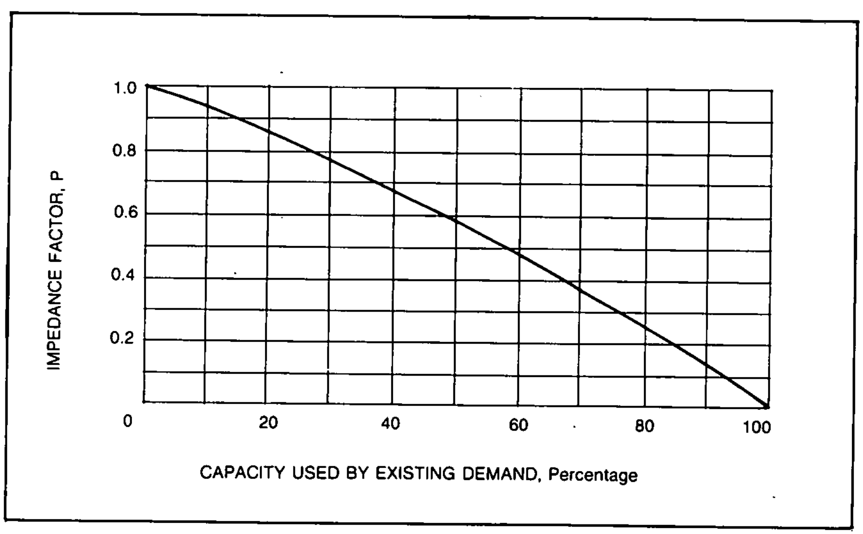


Figure 10-5. Impedance factors as a result of congested movements.

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:

$$c_{SH} = \frac{v_l + v_t + v_r}{[v_l/c_{ml}] + [v_t/c_{mt}] + [v_r/c_{mr}]} \qquad (10\text{-}1)$$

where:

c_{SH} = capacity of the shared lane, in pcph;
v_l = volume or flow rate of left-turn movement in shared lane, in pcph;
v_t = volume or flow rate of through movement in shared lane, in pcph;
v_r = volume or flow rate 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:

$$c_R = c_{SH} - v \qquad (10\text{-}2)$$

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 or flow rate 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
[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.

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 considering 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, corner radii, acceleration lanes, 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.

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.

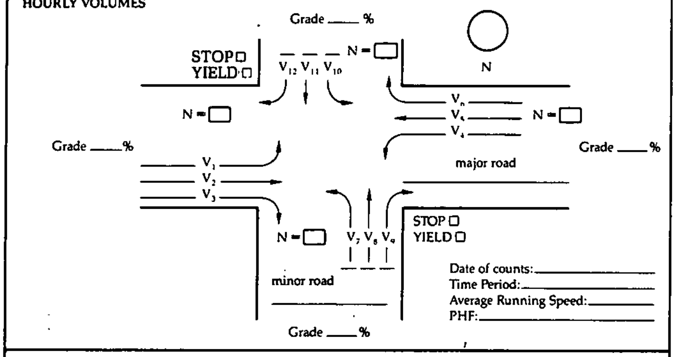
Computation of Movement Capacities

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.

WORKSHEET FOR FOUR-LEG INTERSECTIONS Page 1

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

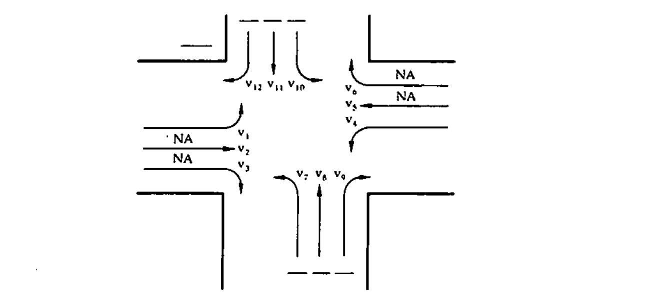


Figure 10-6(a). Worksheet for four-leg intersections (page 1).

WORKSHEET FOR FOUR-LEG INTERSECTIONS Page 2

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)	___ (sec)	___ (sec)
Potential Capacity, c_p (Fig. 10-3)	c_{p9} = ___ pcph	c_{p12} = ___ pcph
Percent of c_p Utilized	$(v_9/c_{p9}) \times 100$ = ___ %	$(v_{12}/c_{p12}) \times 100$ = ___ %
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)	___ (sec)	___ (sec)
Potential Capacity, c_p (Fig. 10-3)	c_{p4} = ___ pcph	c_{p1} = ___ pcph
Percent of c_p Utilized	$(v_4/c_{p4}) \times 100$ = ___ %	$(v_1/c_{p1}) \times 100$ = ___ %
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)	___ (sec)	___ (sec)
Potential Capacity, c_p (Fig. 10-3)	c_{p8} = ___ pcph	c_{p11} = ___ pcph
Percent of c_p Utilized	$(v_8/c_{p8}) \times 100$ = ___ %	$(v_{11}/c_{p11}) \times 100$ = ___ %
Impedance Factor, P (Fig. 10-5)	P_8 = ___	P_{11} = ___
Actual Capacity, c_m	$c_{m8} = c_{p8} \times P_1 \times P_4$ ___ = ___ × ___ × ___ (pcph)	$c_{m11} = c_{p11} \times P_1 \times P_4$ ___ = ___ × ___ × ___ (pcph)
STEP 4: LT From Minor Street	V_7	V_{10}
Conflicting Flows, V_c	V_{c8} (step 3) $+ V_{11} + V_{12} = V_{c7}$ ___ + ___ + ___ = ___ vph	V_{c11} (step 3) $+ V_8 + V_9 = V_{c10}$ ___ + ___ + ___ = ___ vph
Critical Gap, T_c (Tab. 10-2)	___ (sec)	___ (sec)
Potential Capacity, c_p (Fig. 10-3)	c_{p7} = ___ pcph	c_{p10} = ___ pcph
Actual Capacity, c_m	$c_{m7} = c_{p7} \times P_1 \times P_4 \times P_{11} \times P_{12}$ ___ = ___ × ___ × ___ × ___ × ___ (pcph)	$c_{m10} = c_{p10} \times P_4 \times P_1 \times P_8 \times P_9$ ___ = ___ × ___ × ___ × ___ × ___ (pcph)

Figure 10-6(b). Worksheet for four-leg intersections (page 2).

WORKSHEET FOR FOUR-LEG INTERSECTIONS Page 3

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

MINOR STREET APPROACH MOVEMENTS 7, 8, 9

Movement	v(pcph)	c_m(pcph)	c_{SH}(pcph)	$c_R = c_{SH} - v$	LOS
7					
8					
9					

MINOR STREET 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(c). Worksheet for four-leg intersections (page 3).

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.

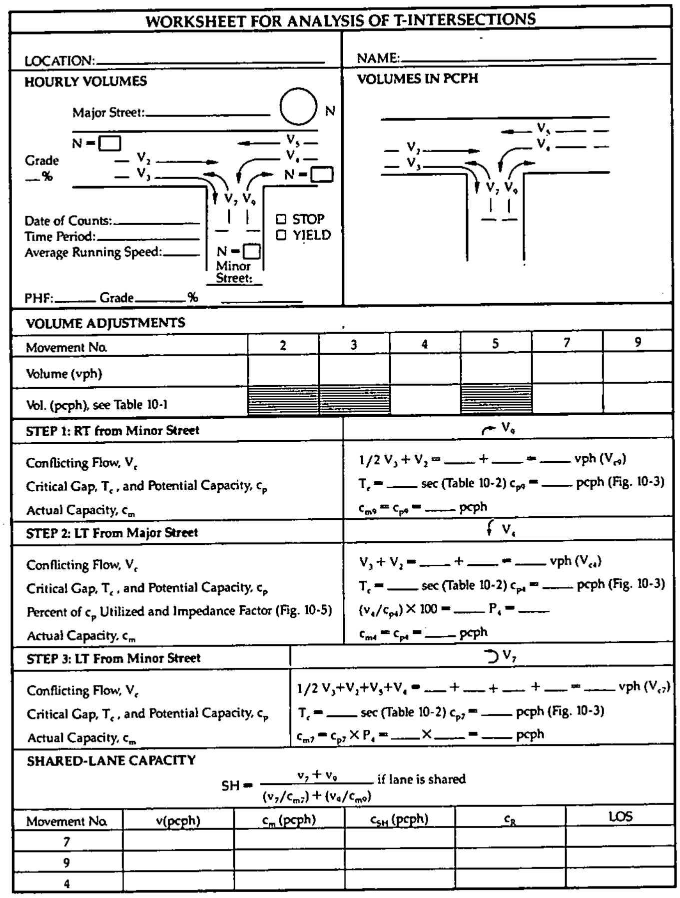
The upper portion of the sheet provides for the summarizing of volume and geometric data, and for the adjustment of volumes, as described for four-leg intersections. Note that there are only six volumes to be considered, and only three of these need be converted to pcph. Again, if the intersection contains 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.


WORKSHEET FOR ANALYSIS OF T-INTERSECTIONS

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	$T_c =$ ___ sec (Table 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	$T_c =$ ___ sec (Table 10-2) $c_{p4} =$ ___ pcph (Fig. 10-3)
Percent of c_p Utilized and Impedance Factor (Fig. 10-5)	$(v_4/c_{p4}) \times 100 =$ ___ $P_4 =$ ___
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	$T_c =$ ___ sec (Table 10-2) $c_{p7} =$ ___ pcph (Fig. 10-3)
Actual Capacity, c_m	$c_{m7} = c_{p7} \times P_4 =$ ___ × ___ = ___ pcph

SHARED-LANE CAPACITY

$$SH = \frac{v_7 + v_9}{(v_7/c_{m7}) + (v_9/c_{m9})} \text{ if lane is shared}$$

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.

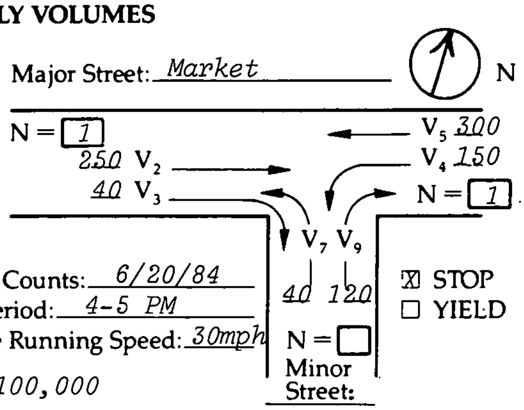
WORKSHEET FOR ANALYSIS OF T-INTERSECTIONS

LOCATION: *Market and Jones* NAME: *Jim Jones*

HOURLY VOLUMES

Major Street: *Market* N

Grade __ %

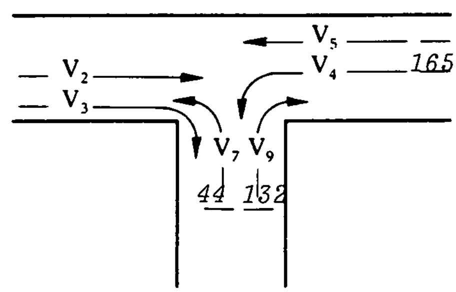


☒ STOP
☐ YIELD

Date of Counts: *6/20/84*
Time Period: *4-5 PM*
Average Running Speed: *30mph*
Pop: 100,000
PHF: ____ Grade ____ %

Minor Street: *Jones*

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 sec (Table 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 sec (Table 10-2) $c_{p4} =$ 900 pcph (Fig. 10-3)
Percent of c_p Utilized and Impedance Factor (Fig. 10-5)	$(v_4/c_{p4}) \times 100 =$ 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 sec (Table 10-2) $c_{p7} =$ 350 pcph (Fig. 10-3)
Actual Capacity, c_m	$c_{m7} = c_{p7} \times P_4 =$ 350 × 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

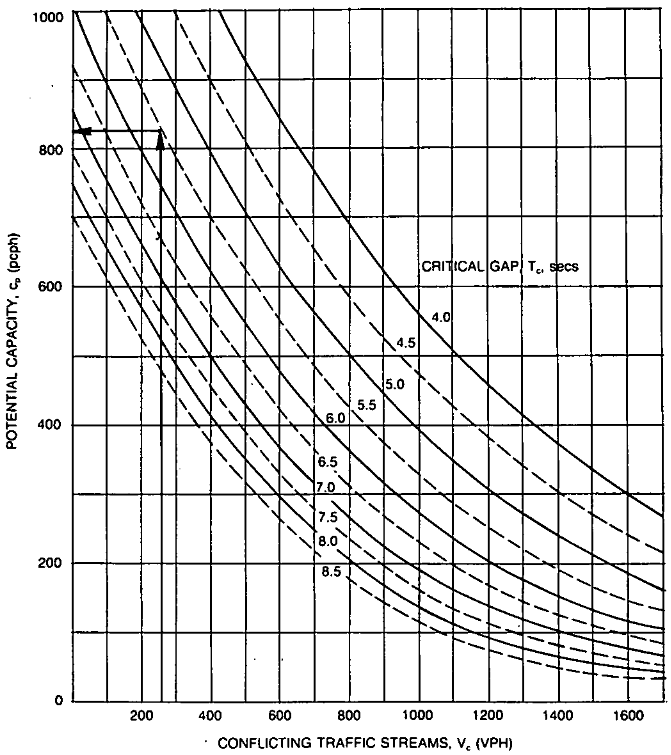
Movement No.	v(pcph)	c_m (pcph)	c_{SH} (pcph)		c_R		LOS	
7	44	308	581	308	405	264	A	C
9	132	825		825		693		A
4	165	900	--		735		A	

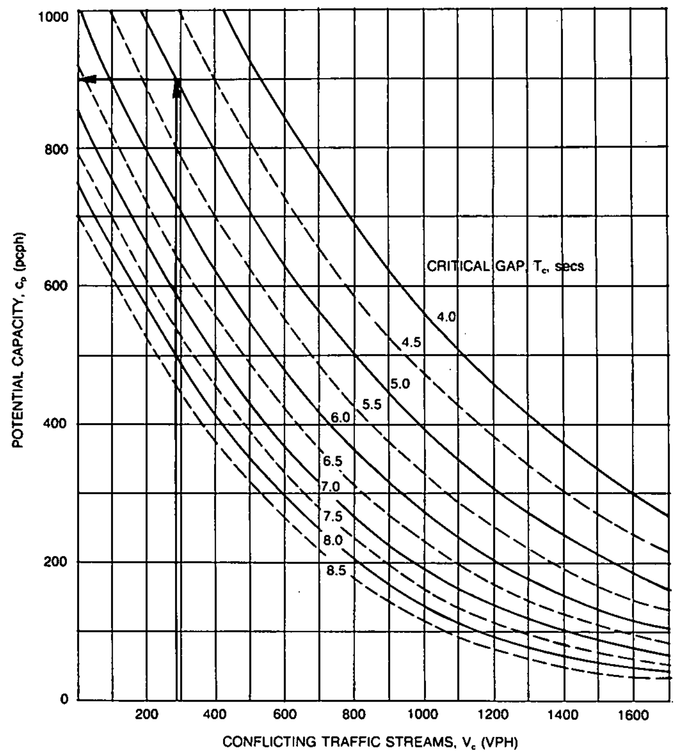
Figure 10-8. Worksheet for Calculation 1.

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.

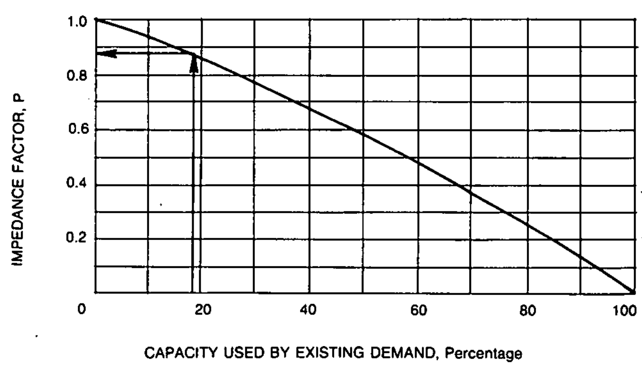
4. The second movement considered is the LT from the major street. The conflicting flow is computed as indicated on the sheet, and is found to be 290 vph. The critical gap is 5.0 sec (from Table 10-2, for major street LT, two lanes on the major street and 30-mph prevailing speed), and potential capacity is found to be 900 from Figure 10-3.



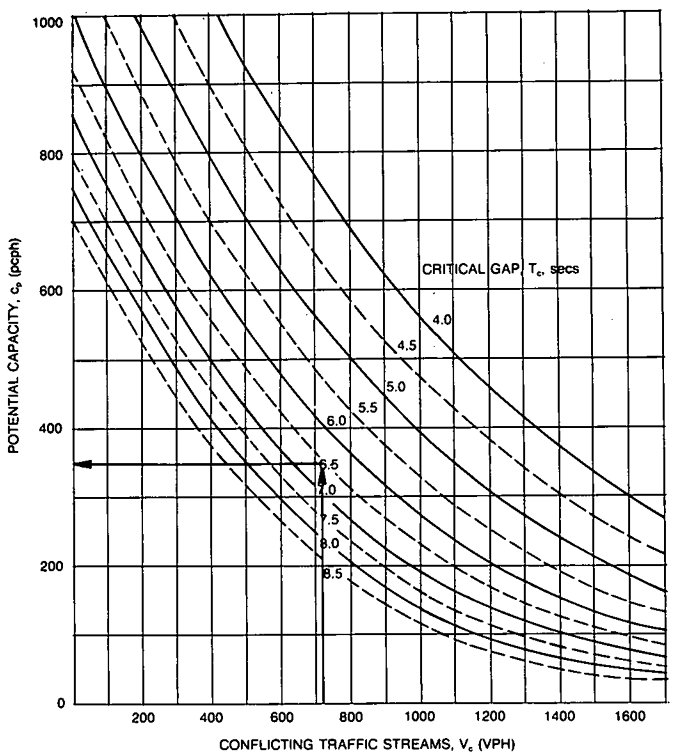


Again, there are no movements which impede the major street LT, and the movement capacity is the same as the potential capacity, 900 pcph.

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 6.5 sec from Table 10-2; and the potential capacity from Figure 10-3, as 350 pcph.



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 a left-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 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.

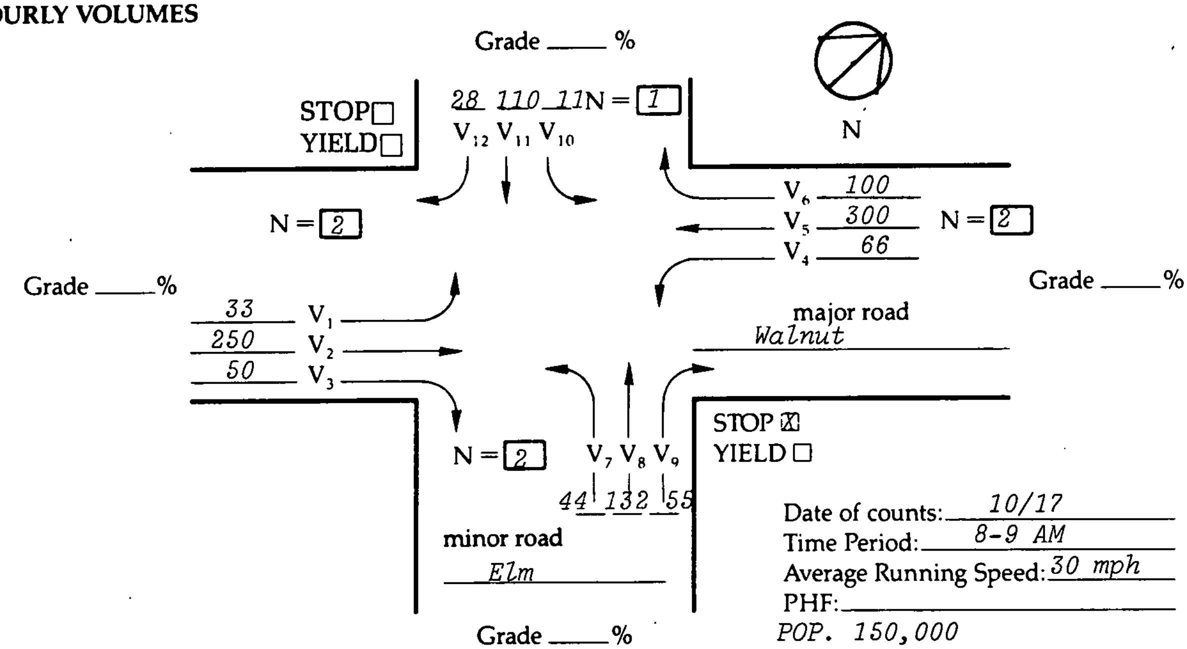
In computing conflicting volumes, note that only one-half of the major street through volumes are included. This is because the major street has two lanes in each direction, and right-turning vehicles merge with a traffic stream consisting of only approximately half these movements. See Figure 10-2 and discussion for a fuller explanation of this effect.

WORKSHEET FOR FOUR-LEG INTERSECTIONS

Page 1

Location: *Walnut and Elm* Name: *Mary Smith*

HOURLY VOLUMES



Date of counts: *10/17*
Time Period: *8-9 AM*
Average Running Speed: *30 mph*
PHF:
POP. 150,000

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

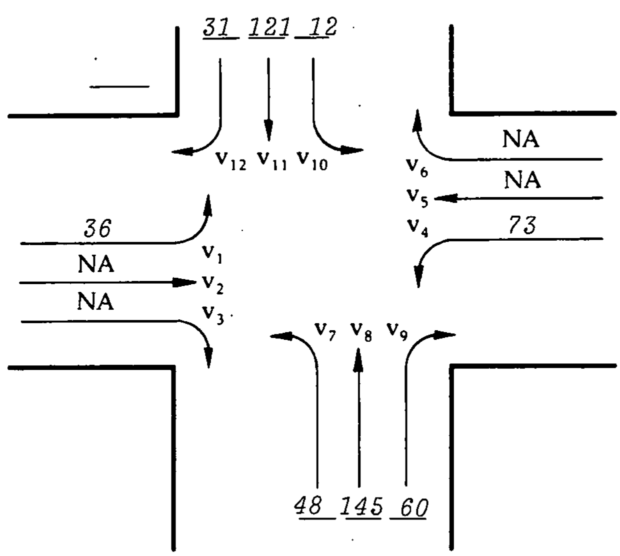


Figure 10-9(a). Worksheet for Calculation 2.

WORKSHEET FOR FOUR-LEG INTERSECTIONS

Page 2

STEP 1: RT From Minor Street	V_9	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 (sec)	5.5 (sec)
Potential Capacity, c_p (Fig. 10-3)	c_{p9} = 940 pcph	c_{p12} = 880 pcph
Percent of c_p Utilized	$(v_9/c_{p9}) \times 100$ = 6.4 %	$(v_{12}/c_{p12}) \times 100$ = 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 (sec)	5.5 (sec)
Potential Capacity, c_p (Fig. 10-3)	c_{p4} = 790 pcph	c_{p1} = 695 pcph
Percent of c_p Utilized	$(v_4/c_{p4}) \times 100$ = 9.2 %	$(v_1/c_{p1}) \times 100$ = 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 (sec)	6.5 (sec)
Potential Capacity, c_p (Fig. 10-3)	c_{p8} = 330 pcph	c_{p11} = 340 pcph
Percent of c_p Utilized	$(v_8/c_{p8}) \times 100$ = 43.9 %	$(v_{11}/c_{p11}) \times 100$ = 35.6 %
Impedance Factor, P (Fig. 10-5)	P_8 = 0.65	P_{11} = 0.72
Actual Capacity, c_m	$c_{m8} = c_{p8} \times P_1 \times P_4$ 298 = 330 × 0.97 × 0.93 (pcph)	$c_{m11} = c_{p11} \times P_1 \times P_4$ 307 = 340 × 0.97 × 0.93 (pcph)
STEP 4: LT From Minor Street	V_7	V_{10}
Conflicting Flows, V_c	V_{c8} (step 3) $+ V_{11} + V_{12} = V_{c7}$ 774 + 110 + 28 = 912 vph	V_{c11} (step 3) $+ V_8 + V_9 = V_{c10}$ 749 + 132 + 55 = 936 vph
Critical Gap, T_c (Tab. 10-2)	7.0 (sec)	7.0 (sec)
Potential Capacity, c_p (Fig. 10-3)	c_{p7} = 230 pcph	c_{p10} = 205 pcph
Actual Capacity, c_m	$c_{m7} = c_{p7} \times P_1 \times P_4 \times P_{11} \times P_{12}$ 146. = 230 × 0.97 × .93 × .72 × .98 (pcph)	$c_{m10} = c_{p10} \times P_4 \times P_1 \times P_8 \times P_9$ 115 = 205 × .93 × .97 × .65 × .96 (pcph)

Figure 10-9(b). Worksheet for Calculation 2 (Continued).

WORKSHEET FOR FOUR-LEG INTERSECTIONS

Page 3

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

MINOR STREET 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	372 298	167 153	D D
9	60	940	940	880	A

MINOR STREET 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

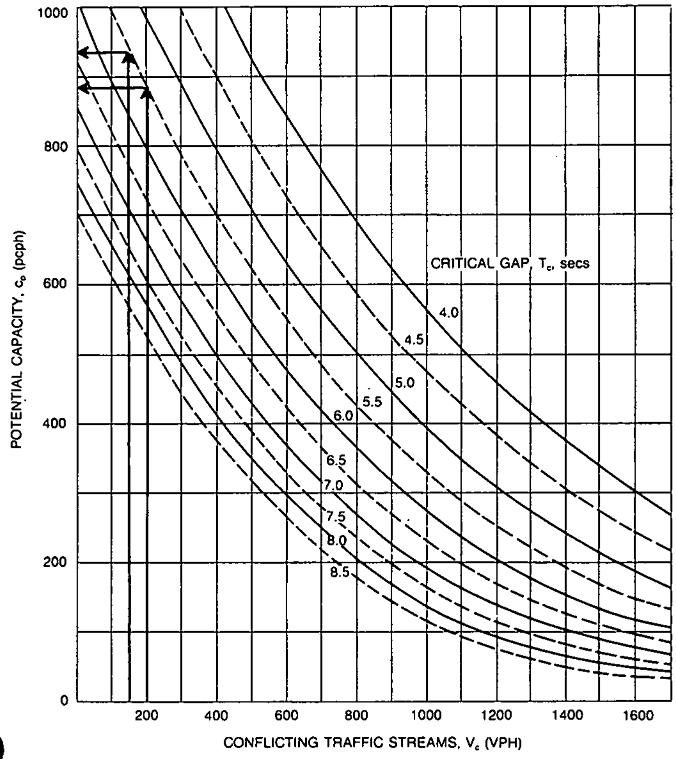
MAJOR STREET LEFT TURNS 1, 4

Movement	v (pcph)	c_m (pcph)	$c_R = c_m - v$	LOS
1	695	36	659	A
4	790	73	717	A

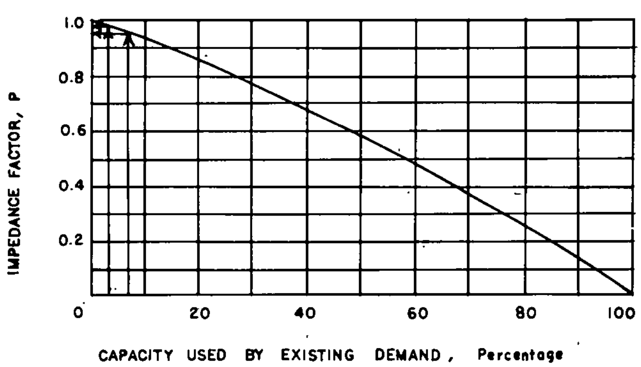
COMMENTS: *RT lane would significantly improve RT operation, but would not have great impact on the rest of the intersection.*

Figure 10-9(c). Worksheet for Calculation 2 (Continued).

The critical gap for the right-turn movements is 5.5 sec from Table 10-2 and the potential capacity for these movements is illustrated. There are no conditions warranting an adjustment in the basic critical gap determination.



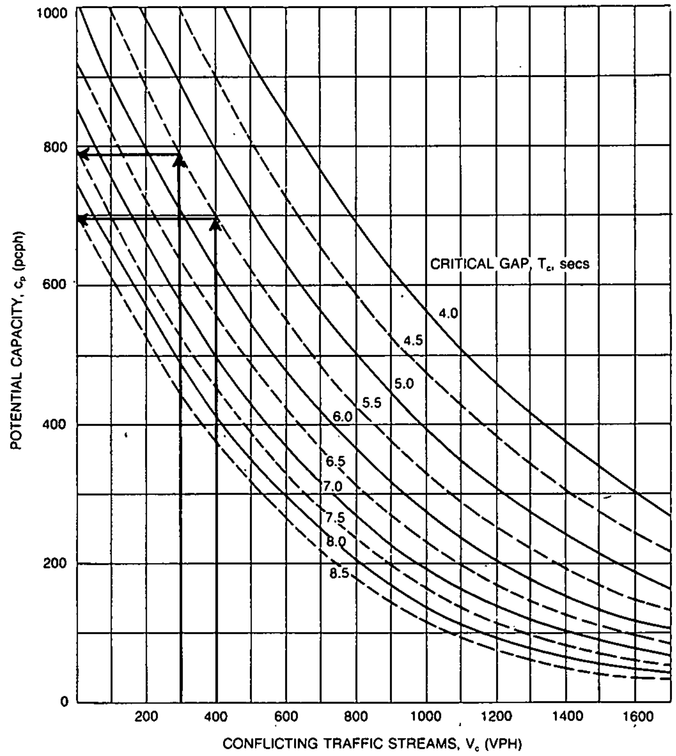
Impedance factors are also computed, as these right turns will impede opposing left turns from the minor street. Movement 9 utilizes 60/940, or 0.064 (6.4 percent), of its potential capacity. Movement 12 utilizes 31/880, or 0.035 (3.5 percent), of its capacity. These values are used to enter Figure 10.5 to determine the respective impedance factors that are listed on the worksheet, 0.96 for movement 9 and 0.98 for movement 12.



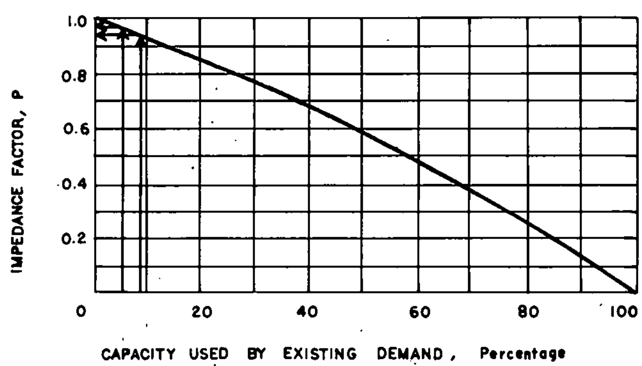
Because the right turns from the minor street are not impeded by other movements, the movement capacities are the same as the potential capacities.

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.

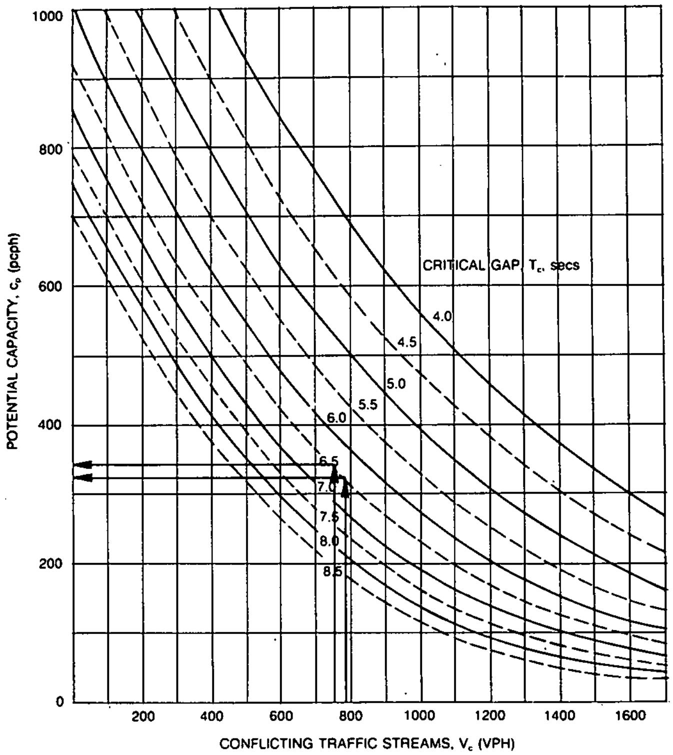
The critical gap is 5.5 sec from Table 10-2, and the potential capacity for these movements is illustrated.

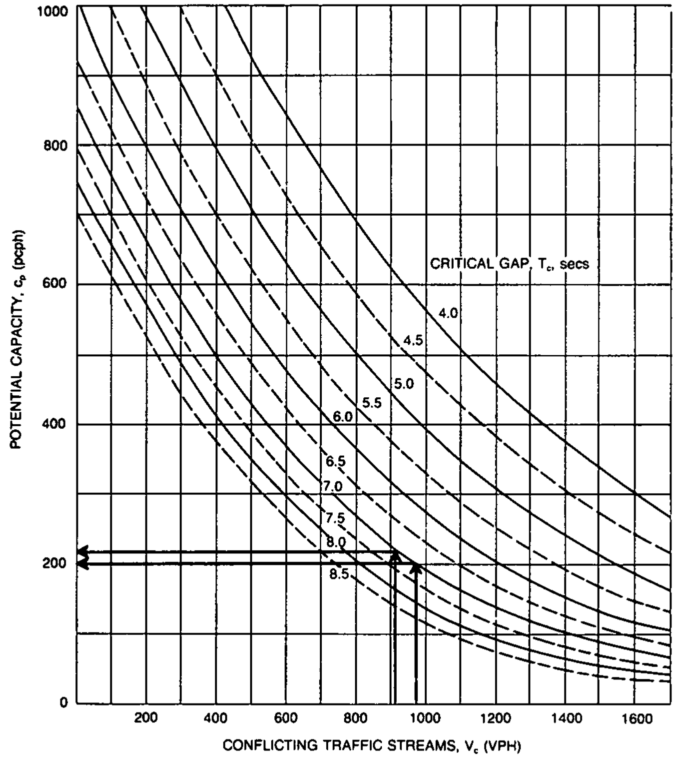


Again, impedance factors will be computed, because the LT from the major street will impede all lower priority movements: Movement 4 utilizes 73/790, or 0.092 (9.2 percent), of its potential capacity; and movement 1 utilizes 36/695, or 0.052 (5.2 percent), of its capacity. These values are used to enter Figure 10-5 to find the impedance factors listed in the worksheet, 0.93 for movement 4 and 0.97 for movement 1.

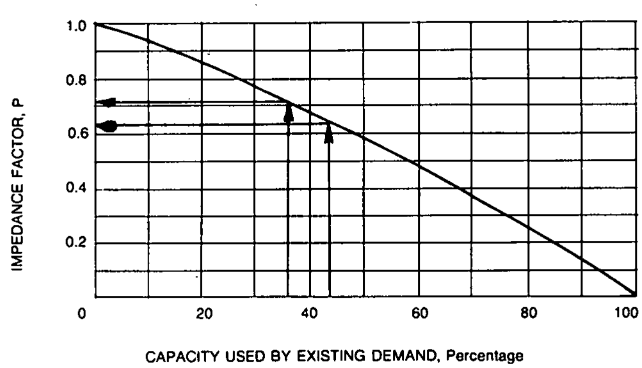


4. The minor street through movements (Nos. 8 and 11) are the next to be considered in the computational process. Again, conflicting traffic volumes are computed as shown, in accordance with the specifications of Figure 10-2. The critical gap for these through movements is found to be 6.5 sec from Table 10-2, and the unadjusted capacities to be 330 pcph and 340 pcph, respectively. These findings are illustrated.





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 from Table 10-2, and the potential capacities are as illustrated.

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 in 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:

Vol. (vph)	*PC*	*SUT*	*CV*	*Vol. (pcph)*
V_1	$60 \times 0.85 \times 1.0 +$	$60 \times 0.12 \times 1.5 +$	$60 \times 0.03 \times 2.0 =$	65
V_4	$40 \times 0.85 \times 1.0 +$	$40 \times 0.12 \times 1.5 +$	$40 \times 0.03 \times 2.0 =$	44
V_7	$20 \times 0.85 \times 1.2 +$	$20 \times 0.12 \times 2.0 +$	$20 \times 0.03 \times 3.0 =$	27
V_8	$40 \times 0.85 \times 1.2 +$	$40 \times 0.12 \times 2.0 +$	$40 \times 0.03 \times 3.0 =$	54
V_9	$10 \times 0.85 \times 1.2 +$	$10 \times 0.12 \times 2.0 +$	$10 \times 0.03 \times 3.0 =$	14
V_{10}	$10 \times 0.85 \times 0.9 +$	$10 \times 0.12 \times 1.2 +$	$10 \times 0.03 \times 1.5 =$	10
V_{11}	$20 \times 0.85 \times 0.9 +$	$20 \times 0.12 \times 1.2 +$	$20 \times 0.03 \times 1.5 =$	19
V_{12}	$120 \times 0.85 \times 0.9 +$	$120 \times 0.12 \times 1.2 +$	$120 \times 0.03 \times 1.5 =$	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.

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:

$$
\begin{aligned}
c_{SH}(8,9) &= [54 + 14]/[(54/608) + (14/1{,}000)] = 661 \text{ pcph} \\
c_{SH}(11,12) &= [19 + 115]/[(19/637) + (115/1{,}000)] = 925 \text{ pcph}
\end{aligned}
$$

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 inter-

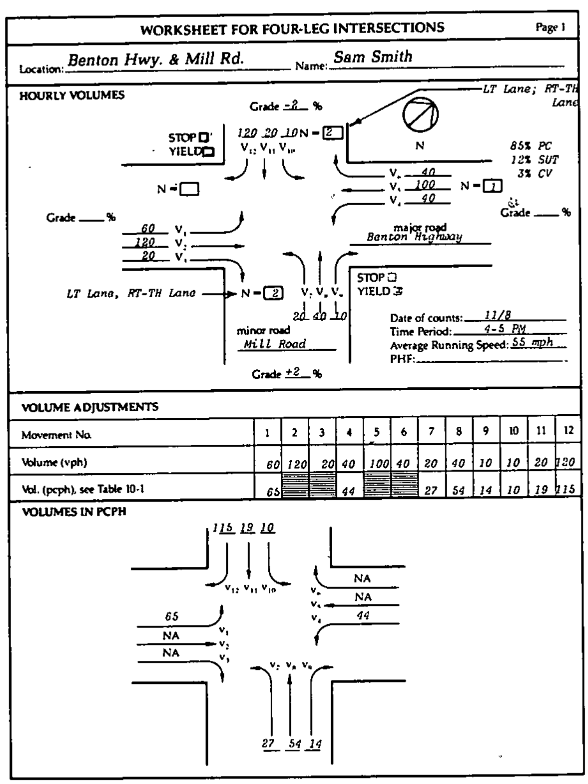


Figure 10-10(a). Worksheet for Calculation 3.

WORKSHEET FOR FOUR-LEG INTERSECTIONS

Page 2

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)	5.0 (sec)	5.0 (sec)
Potential Capacity, c_p (Fig. 10-3)	c_{p9} = 1000 pcph	c_{p12} = 1000 pcph
Percent of c_p Utilized	$(v_9/c_{p9}) \times 100$ = 1.4 %	$(v_{12}/c_{p12}) \times 100$ = 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)	5.0 (sec)	5.0 (sec)
Potential Capacity, c_p (Fig. 10-3)	c_{p4} = 1000 pcph	c_{p1} = 1000 pcph
Percent of c_p Utilized	$(v_4/c_{p4}) \times 100$ = 4.4 %	$(v_1/c_{p1}) \times 100$ = 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)	6.0 (sec)	6.0 (sec)
Potential Capacity, c_p (Fig. 10-3)	c_{p8} = 640 pcph	c_{p11} = 670 pcph
Percent of c_p Utilized	$(v_8/c_{p8}) \times 100$ = 8.4 %	$(v_{11}/c_{p11}) \times 100$ = 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 × .97 × .98 (pcph)	$c_{m11} = c_{p11} \times P_1 \times P_4$ 637 = 670 × .97 × .98 (pcph)
STEP 4: LT From Minor Street	V_7	V_{10}
Conflicting Flows, V_c	V_{c8} (step 3) $+ V_{11} + V_{12} = V_{c7}$ 370 + 20 + 120 = 510 vph	V_{c11} (step 3) $+ V_8 + V_9 = V_{c10}$ 360 + 40 + 10 = 410 vph
Critical Gap, T_c (Tab. 10-2)	6.5 (sec)	6.5 (sec)
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 × .97 × .98 × .99 × .91 (pcph)	$c_{m10} = c_{p10} \times P_4 \times P_1 \times P_8 \times P_9$ 483 = 540 × .98 × .97 × .95 × .99 (pcph)

Figure 10-10(b). Worksheet for Calculation 3 (Continued).

WORKSHEET FOR FOUR-LEG INTERSECTIONS — Page 3

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

MINOR STREET 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

MINOR STREET APPROACH MOVEMENTS 10, 11, 12

Movement	v(pcph)	c_m(pcph)	c_{SH}(pcph)	$c_R = c_{SH} - v$	LOS
10	10	483	483	473	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(c). Worksheet for Calculation 3 (Continued).

section 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.

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

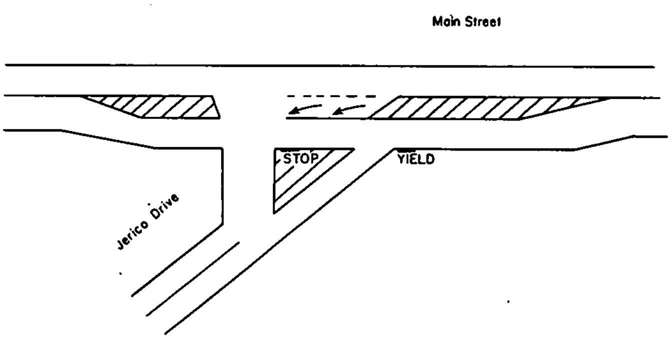


Figure 10-11. Intersection diagram for Calculation 4.

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 Ausgabe 1972.* 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).
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 Uncontrolled 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).

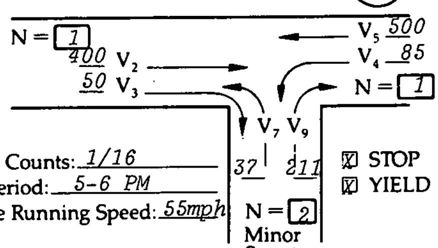
WORKSHEET FOR ANALYSIS OF T-INTERSECTIONS

LOCATION: *Main and Jerico* | NAME: *Bob Mathews*

HOURLY VOLUMES

Major Street: *Main St.* N

Grade __%

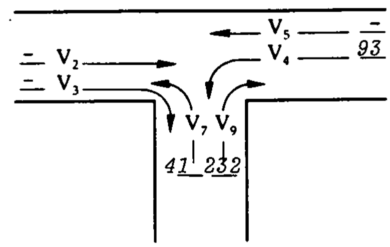


Date of Counts: *1/16*
Time Period: *5-6 PM*
Average Running Speed: *55mph*
Pop: 150,000
PHF: ____ Grade ____ %

☒ STOP
☒ YIELD

Minor Street: *Jerico*

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* sec (Table 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* sec (Table 10-2) $c_{p4} =$ *650* pcph (Fig. 10-3)

Percent 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* sec (Table 10-2) $c_{p7} =$ *145* pcph (Fig. 10-3)

Actual Capacity, c_m: $c_{m7} = c_{p7} \times P_4 =$ *145* $\times$ *0.90* $=$ *126* pcph

SHARED-LANE CAPACITY

$$SH = \frac{v_7 + v_9}{(v_7/c_{m7}) + (v_9/c_{m9})} \quad \text{if lane is shared}$$

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

Figure 10-12. Worksheet for Calculation 4.

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.

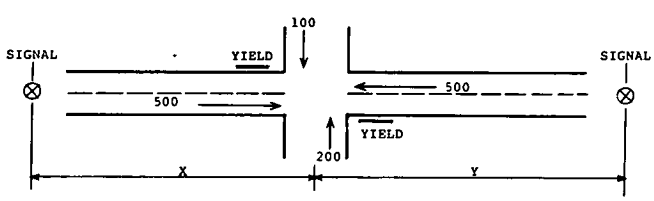


Figure I.10-1. Problem for illustration of platoon flow application.

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.

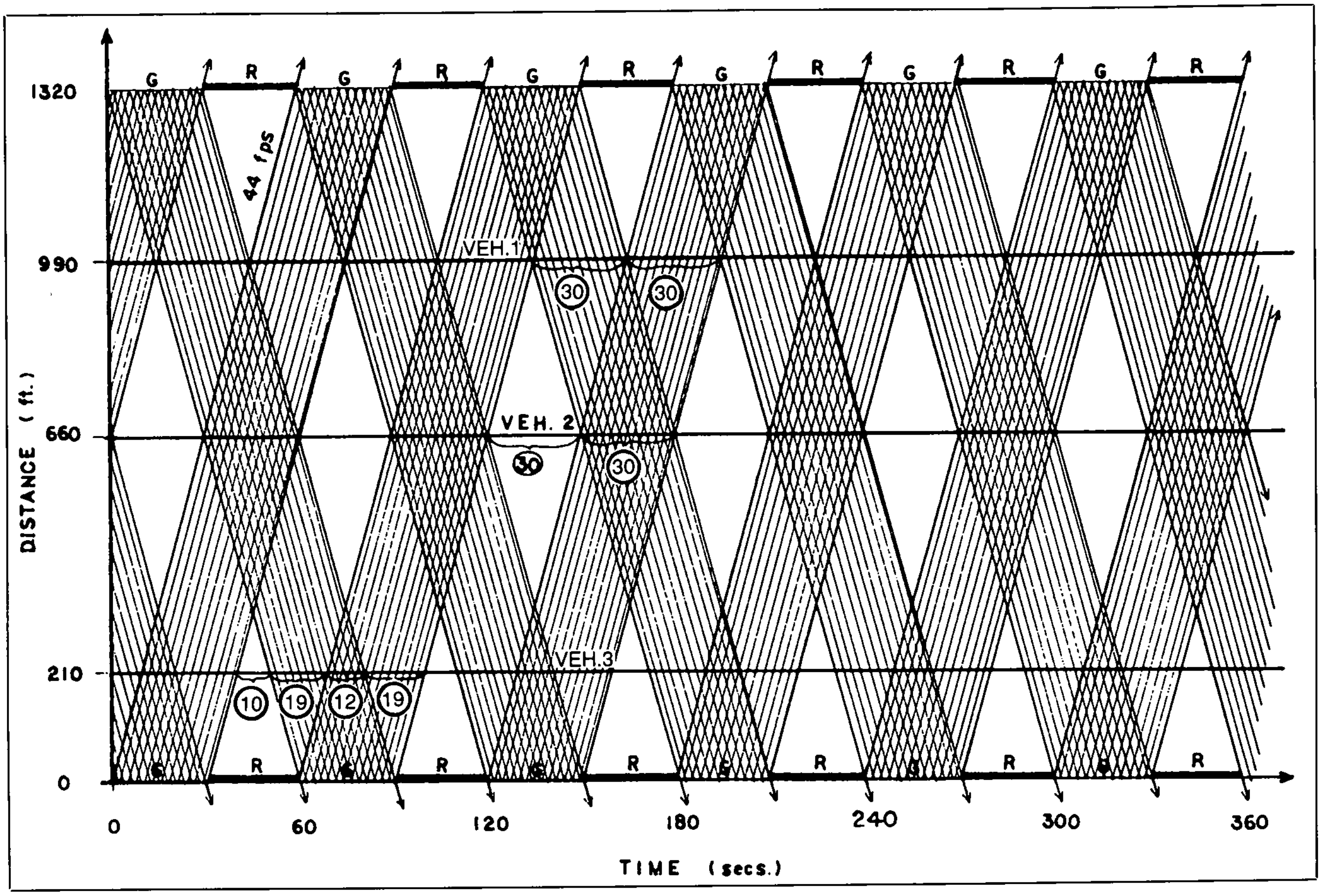


Figure I.10-2. Time-space diagram for illustrative problem.

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.

VEHICLE 1

200 → ; ← 800	800 → ; ← 200
30 of 60 secs.	30 of 60 secs.
V_c = 1000 vph	V_c = 1000 vph
T_c = 5.5 secs	T_c = 5.5 secs
$c_p = c_m$ = 330 vph	$c_p = c_m$ = 330 vph

CAP = 330 (30/60) + 330 (30/60) = 330 vph

c_R (NB) = 330 - 200 = 130 vph (LOS D)

c_R (SB) = 330 - 100 = 230 vph (LOS C)

VEHICLE 2

200 → ; ← 200	800 → ; ← 800
30 of 60 secs.	30 of 60 secs.
V_c = 400 vph	V_c = 1600 vph
T_c = 5.5 secs.	T_c = 5.5 secs.
$c_p = c_m$ = 700 vph	$c_p = c_m$ = 140 vph

CAP = 700 (30/60) + 140 (30/60) = 420 vph

c_R (NB) = 420 - 200 = 220 vph (LOS C)

c_R (SB) = 420 - 100 = 320 vph (LOS B)

Figure I.10-3(a). Capacity computations for sample problem.

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.

VEHICLE 3

200 → ; ← 200	200 → ; ← 800
10 of 60 secs.	19 of 60 secs.
V_c = 400 vph	V_c = 1000 vph
T_c = 5.5 secs	T_c = 5.5 secs
$c_p = c_m$ = 700 vph	$c_p = c_m$ = 330 vph
800 → ; ← 800	800 → ; ← 200
12 of 60 secs.	19 of 60 secs.
V_c = 1600 vph	V_c = 1000 vph
T_c = 5.5 secs	T_c = 5.5 secs
$c_p = c_m$ = 140 vph	$c_p = c_m$ = 330 vph

CAP = 700 (10/60) + 330 (19/60) + 140 (12/60) + 330 (19/60) = 354

c_R (NB) = 354 - 200 = 154 vph (LOS D)

c_R (SB) = 354 - 100 = 254 vph (LOS C)

RANDOM ARRIVALS

V_c = 1000 vph	c_R (NB) = 330 - 200 = 130 vph (LOS D)
T_c = 5.5 secs	c_R (SB) = 330 - 100 = 230 vph (LOS C)
$c_p = c_m$ = 330 secs	

Figure I.10-3(b). Capacity computations for sample problem (Continued).

APPENDIX II

FIGURES AND WORKSHEETS FOR USE IN THE ANALYSIS OF UNSIGNALIZED INTERSECTIONS

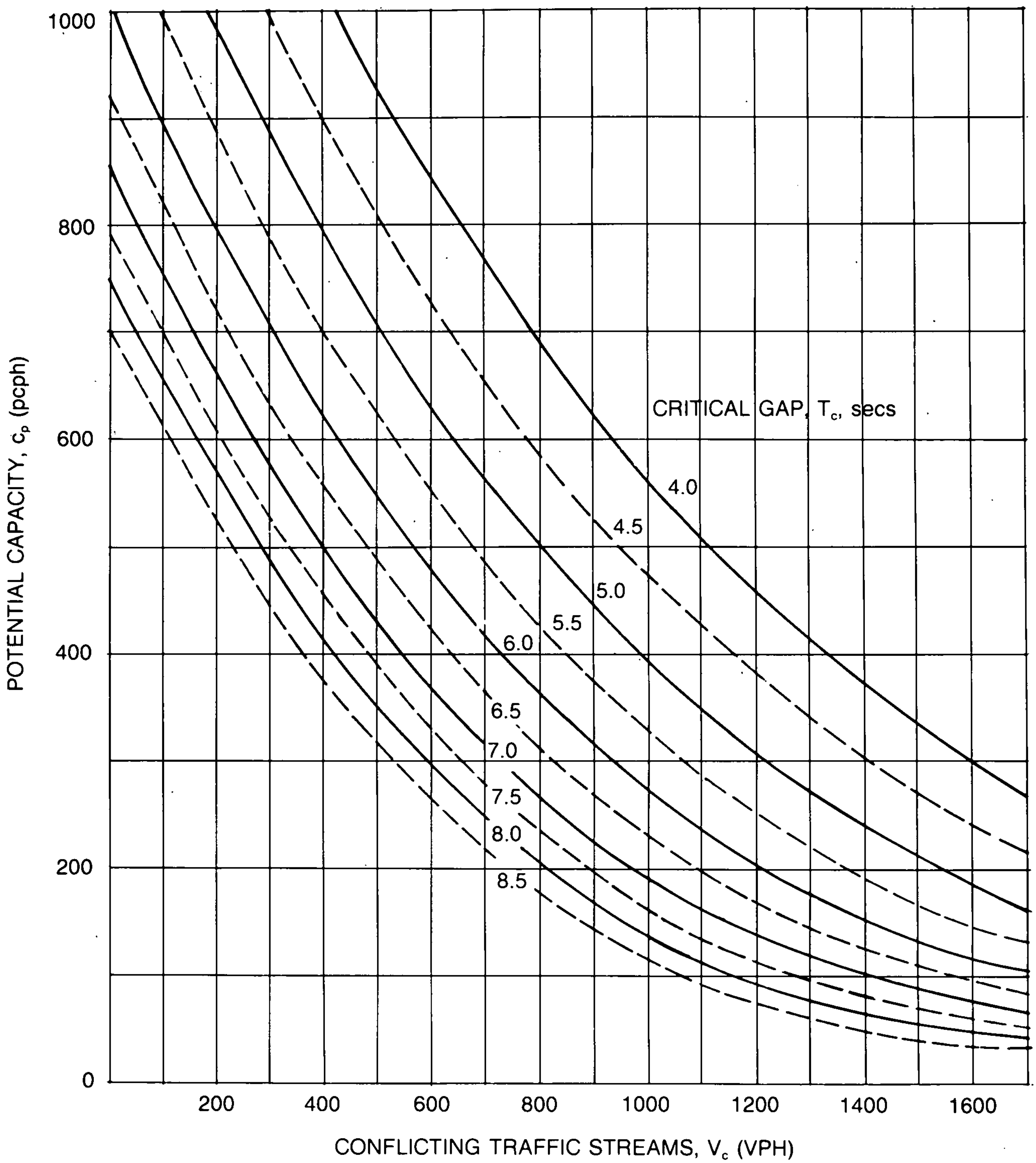


Figure 10-3. Potential capacity based on conflicting traffic volume and critical gap size.

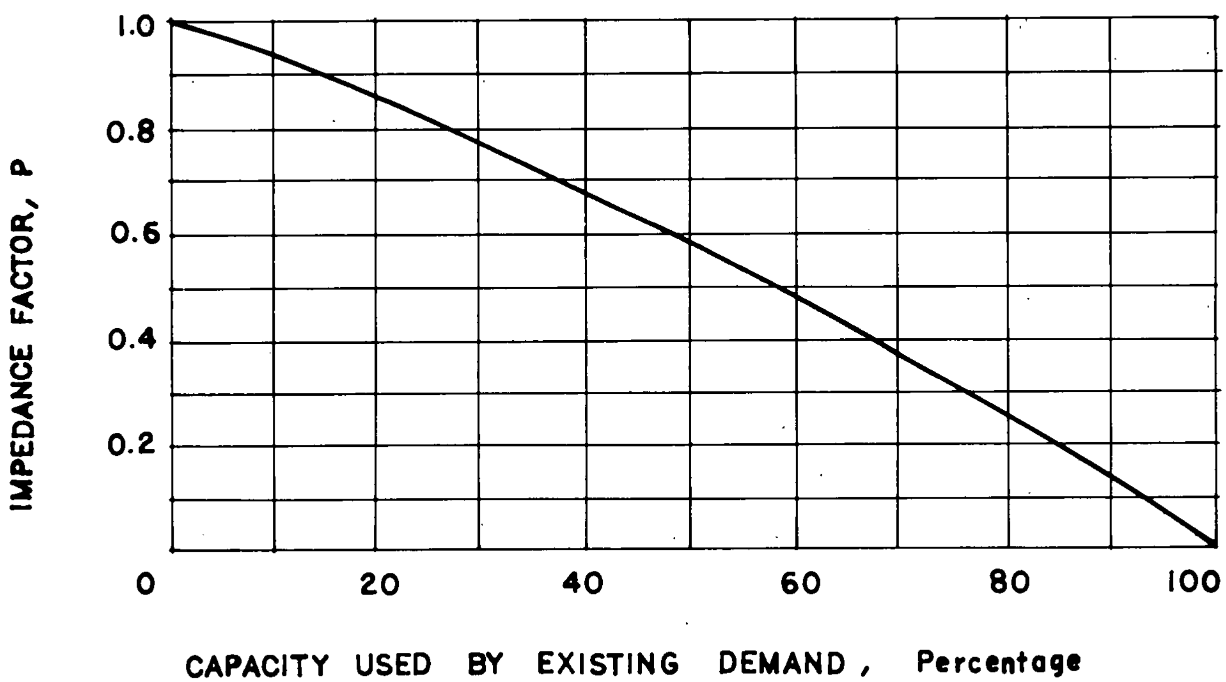


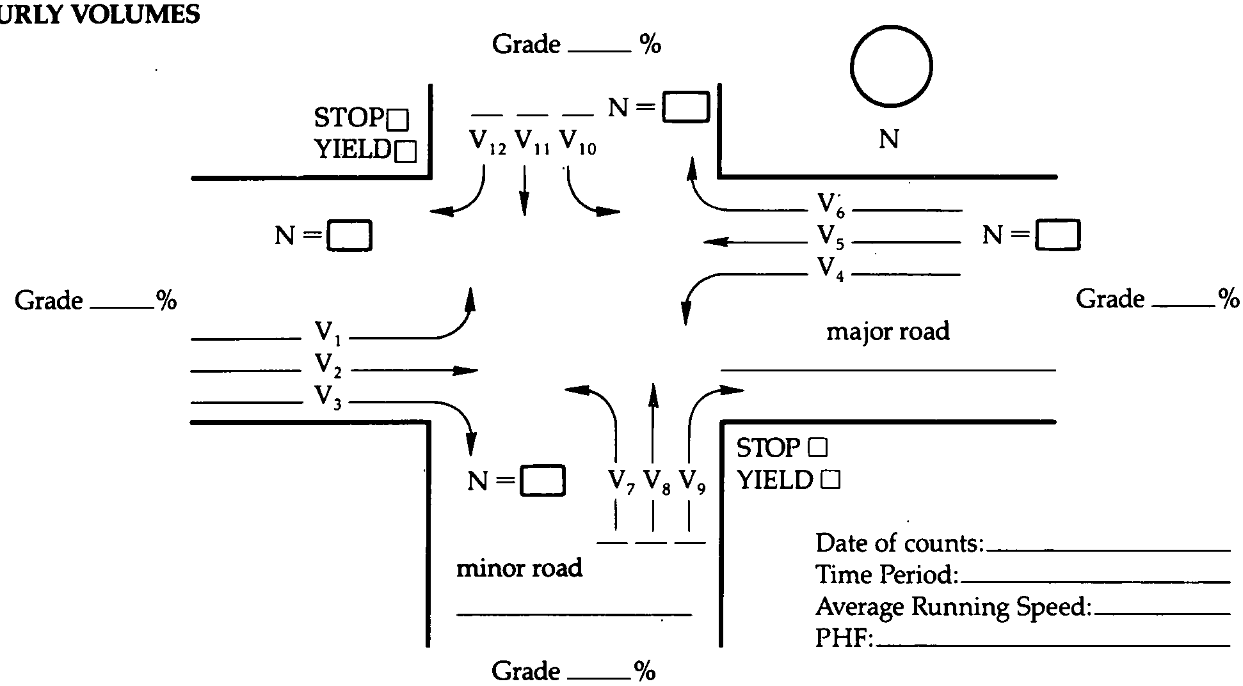
Figure 10-5. Impedance factors as a result of congested movements.

WORKSHEET FOR FOUR-LEG INTERSECTIONS

Page 1

Location:__________ Name:__________

HOURLY VOLUMES



Date of counts:__________

Time Period:__________

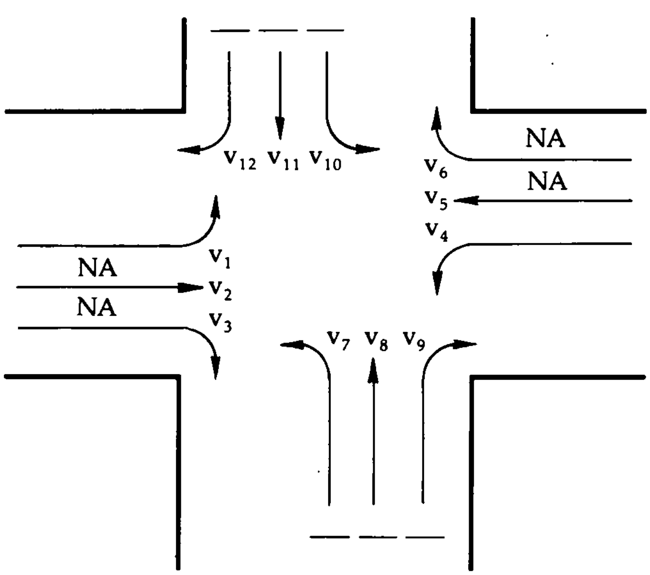
Average Running Speed:__________

PHF:__________

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



WORKSHEET FOR FOUR-LEG INTERSECTIONS

Page 2

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)	______ (sec)	______ (sec)
Potential Capacity, c_p (Fig. 10-3)	c_{p9} = ______ pcph	c_{p12} = ______ pcph
Percent of c_p Utilized	$(v_9/c_{p9}) \times 100$ = ______%	$(v_{12}/c_{p12}) \times 100$ = ______%
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)	______ (sec)	______ (sec)
Potential Capacity, c_p (Fig. 10-3)	c_{p4} = ______ pcph	c_{p1} = ______ pcph
Percent of c_p Utilized	$(v_4/c_{p4}) \times 100$ = ______%	$(v_1/c_{p1}) \times 100$ = ______%
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)	______ (sec)	______ (sec)
Potential Capacity, c_p (Fig. 10-3)	c_{p8} = ______ pcph	c_{p11} = ______ pcph
Percent of c_p Utilized	$(v_8/c_{p8}) \times 100$ = ______%	$(v_{11}/c_{p11}) \times 100$ = ______%
Impedance Factor, P (Fig. 10-5)	P_8 = ______	P_{11} = ______
Actual Capacity, c_m	$c_{m8} = c_{p8} \times P_1 \times P_4$ ______ = ______ × ______ × ______ (pcph)	$c_{m11} = c_{p11} \times P_1 \times P_4$ ______ = ______ × ______ × ______ (pcph)
STEP 4: LT From Minor Street	V_7	V_{10}
Conflicting Flows, V_c	V_{c8} (step 3) $+ V_{11} + V_{12} = V_{c7}$ ___ + ___ + ___ = ___ vph	V_{c11} (step 3) $+ V_8 + V_9 = V_{c10}$ ___ + ___ + ___ = ___ vph
Critical Gap, T_c (Tab. 10-2)	______ (sec)	______ (sec)
Potential Capacity, c_p (Fig. 10-3)	c_{p7} = ______ pcph	c_{p10} = ______ pcph
Actual Capacity, c_m	$c_{m7} = c_{p7} \times P_1 \times P_4 \times P_{11} \times P_{12}$ ______ = ___ × ___ × ___ × ___ × ___ (pcph)	$c_{m10} = c_{p10} \times P_4 \times P_1 \times P_8 \times P_9$ ______ = ___ × ___ × ___ × ___ × ___ (pcph)

WORKSHEET FOR FOUR-LEG INTERSECTIONS

Page 3

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

MINOR STREET APPROACH MOVEMENTS 7, 8, 9

Movement	v(pcph)	c_m(pcph)	c_{SH}(pcph)	$c_R = c_{SH} - v$	LOS
7					
8					
9					

MINOR STREET 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				

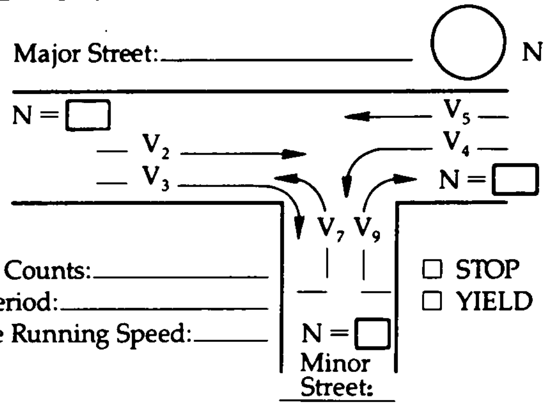
COMMENTS:

WORKSHEET FOR ANALYSIS OF T-INTERSECTIONS

LOCATION: ______________________	NAME: ______________________

HOURLY VOLUMES

Major Street: ______________



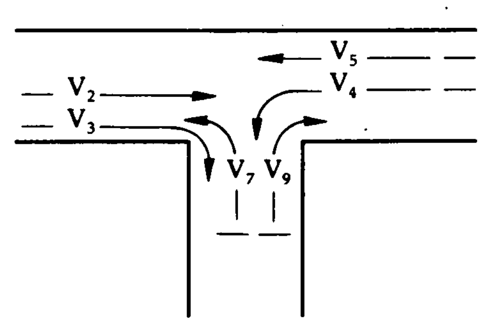
Date of Counts: ______________

Time Period: ______________

Average Running Speed: ______

PHF: ______ Grade ______ %

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	$T_c =$ ____ sec (Table 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	$T_c =$ ____ sec (Table 10-2) $c_{p4} =$ ____ pcph (Fig. 10-3)
Percent of c_p Utilized and Impedance Factor (Fig. 10-5)	$(v_4/c_{p4}) \times 100 =$ ____ $P_4 =$ ____
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	$T_c =$ ____ sec (Table 10-2) $c_{p7} =$ ____ pcph (Fig. 10-3)
Actual Capacity, c_m	$c_{m7} = c_{p7} \times P_4 =$ ____ $\times$ ____ = ____ 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					

CHAPTER 11

URBAN AND SUBURBAN ARTERIALS

CONTENTS

I. INTRODUCTION

Urban and suburban arterials are signalized streets that primarily serve through traffic and provide access to abutting properties as a secondary function. For purposes of this manual, they are defined as facilities with a signalized intersection spacing of 2 mi or less and turning movements at intersections that usually do not exceed 20 percent of total traffic volumes. Roadside development along arterials can be intense, producing frictions to traffic that generally limit a driver's desired speed.

In the system of urban highway transportation facilities, arterial streets are between collector and downtown streets on one side and multilane suburban highways and rural roads on the other side. The difference is mainly determined by their function and the character and intensity of roadside development.

Collector streets provide both land access and traffic circulation service within residential, commercial, and industrial areas. Their access function is more important than that of arterials and, unlike arterials, their operation is not always dominated by traffic signals.

Downtown streets are usually signalized facilities that often resemble arterials. However, their main function is not to move through traffic but to provide access to local business by passenger cars, transit buses, and trucks. Turning movements in downtown intersections are often greater than 20 percent of total traffic because downtown flow involves a substantial amount of circulatory traffic.

Typical of downtown streets are numerous pedestrian conflicts and lane obstructions caused by stopping or standing taxicabs, buses, trucks, and parking and unparking of vehicles which cause turbulences in the traffic flow. Downtown street function may change with the time of the day, and for this reason certain strategically located downtown streets are converted to arterial type operation during the peak traffic hours.

Multilane suburban highways and rural roads differ from suburban arterials in the following features: (1) their roadside development is not as intense, (2) the density of traffic access points is not as high, and (3) signalized intersections are more than 2 mi apart. These conditions result in a smaller number of traffic conflicts, a smoother flow, and the dissipation of the platoon structure associated with arterial traffic.

Urban and suburban arterials include multilane divided arterials, multilane undivided arterials, two-lane, two-way arterials, and one-way arterials. Based on FHWA statistics in the early 1980's, approximately 37 percent of the urban and suburban arterial miles in urbanized areas of more than 100,000 people are multilane divided arterials, 27 percent are multilane undivided arterials, and 33 percent are two-lane, two-way arterials (one travel lane in each direction). The remaining 3 percent of the national distribution of urban arterial miles are one-way arterials.

APPLICATIONS

The methodology contained in this chapter can be used by those concerned with the planning, design, and operation of arterials to evaluate the level of service (LOS) on an existing or proposed facility. The methodology does *not* address arterial capacity: the capacity of an arterial is generally dominated by the capacity of its signalized intersections, which can be addressed by the procedures of Chapter 9. In some cases, there are special midblock restrictions that also limit the capacity. In general, the user can best conduct an arterial capacity analysis by analyzing the point capacity of the signalized intersections and other such points.

The methodology of this chapter is oriented to the evaluation of an existing *operations* situation, or evaluating a specific *design* proposal, by a level-of-service determination. The person doing such design or operations work will be able to investigate the effect of signal spacing, arterial class (as defined herein), and traffic flow on the arterial level of service. The methodology makes use of the signalized intersection procedure of Chapter 9 for the lane group containing the through traffic. By redefining lane arrangement (e.g., presence or absence of left-turn lanes, number of lanes), the analyst may influence which traffic flow is in the "through-traffic" lane group and the capacity of the lane group. This, in turn, influences the arterial LOS determination by changing the intersection evaluation and possibly the arterial classification.

Those interested in *planning* applications may apply the entire arterial methodology in a straightforward but somewhat simplified way by using certain default values when computing the stopped delay by the Chapter 9 procedures. However, knowledge of the intended signal timing and quality of progression is vital. If it is lacking, there can be no meaningful estimation of arterial level of service, even on a "planning" level.

The LOS criteria also can be applied when travel time and delay runs are used to assess the impact of optimizing signal timing or other improvement to the arterial and to periodically evaluate the entire arterial system in an urban area.

The above applications of the methodology *always* require the determination of the LOS and associated measures of effectiveness (i.e., travel time, delay, speed). In certain cases the determination of LOS values is the final objective; in other cases LOS values associated with different alternatives are computed and a decision is made using these values.

CHARACTERISTICS OF ARTERIAL FLOW

The operation of vehicles on arterial streets is influenced by three main factors: (1) the arterial environment, (2) the interaction between vehicles, and (3) the effect of traffic signals. These factors determine the capacity of an arterial street and the level of traffic service offered to its users. They constitute the basic elements of the methodology discussed in Section II of this chapter.

The *arterial environment* includes the geometric characteristics of the facility and adjacent land uses. The number of lanes and lane width, type of median, access point density, and spacing between signalized intersections are among the environmental factors, as well as the existence of parking, level of pedestrian activity, speed limit, and population of the city.

The arterial environment affects a driver's notion of safe speed. Even if the effect of the other factors is negligible, the environment restricts driver's desired speed; that is to say, the maximum speed at which a driver would like to travel under a given set of environmental conditions. The average desired speed of all drivers on an arterial segment or section is referred to in this chapter as *free flow* speed.

The *interaction between vehicles* is determined by traffic density, the proportion of trucks and buses, and turning movements. This interaction affects the operation of vehicles at intersections and, to a lesser extent, between signals.

Very seldom can a driver travel at the desired speed. Most of the time, the presence of other vehicles restricts the speed of a vehicle in motion because of differences in desired speeds among drivers, or because downstream vehicles are accelerating from a stop and have not yet reached their driver's desired speeds. Therefore, the average speed of a vehicle in motion over a certain length of road, or running speed, is usually lower than the desired speed of its driver because of the effect of vehicle interactions. Likewise, the *average running speed* of all vehicles on an arterial segment is usually lower than the free flow speed of the segment.

Traffic signals force vehicles to stop and to remain stopped for a certain time, and then release vehicles in platoons. The delays and speed changes caused by traffic signal operation considerably reduce the capacity of an urban arterial and lower the quality of traffic flow.

The duration of the average stop per vehicle, or average

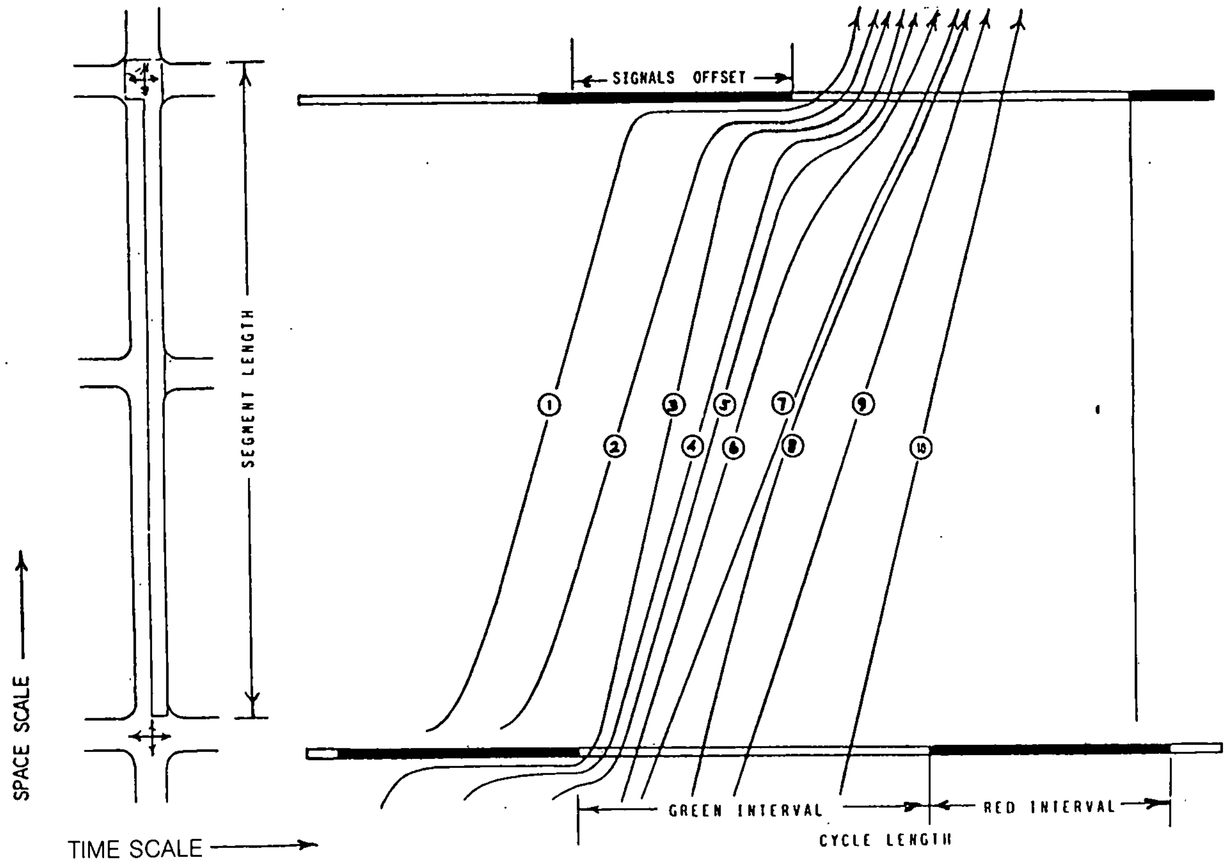


Figure 11-1. Typical time-space trajectories of vehicles on a one-lane arterial segment.

stopped delay, depends mainly on the proportion of red time displayed to the arterial segment, the proportion of vehicles arriving on green (or quality of traffic signal progression), and the traffic volume. The travel speed (which includes time lost due to intersection effects, including stops and all associated approach delay) over an arterial segment is generally lower than the corresponding running speed. Similarly, the *average travel speed* of all vehicles on the segment is lower than their average running speed unless no vehicles stop.

Figure 11-1 shows simplified time-space trajectories of representative vehicles along one lane of an arterial. Vehicles 1 and 2 turned onto the arterial from side streets and the rest were discharged from the upstream signal. Vehicles 1, 2, and 3 arrived at the downstream signal approach during the red interval and had to stop. Vehicle 4 could have arrived at the stop line on green but had to stop because vehicle 3 was not yet in motion and blocked vehicle 4. Vehicles 5, 6, and 7 did not stop but had to reduce their speeds because they were still affected by the stoppages caused by the signal. Vehicle 8 was delayed because its driver's desired speed was higher than that of vehicle 7's driver. Vehicles 9 and 10 traveled at their driver's desired speeds.

The travel speeds of vehicles 1, 2, 3, and 4 were lower than their respective running speeds, which, in turn, were lower than the desired speeds of their drivers. The travel speeds of vehicles 5, 6, 7, and 8 were equal to their corresponding running speeds, but both of the speeds were lower than their driver's desired speed. Finally, for vehicles 9 and 10, whose drivers were traveling at their desired speeds, the three types of speeds have the same values.

ARTERIAL LEVEL OF SERVICE

The arterial level of service is based on the *average travel speed* for the segment, section, or entire arterial under consideration. This is the basic measure of effectiveness (MOE) for this chapter. The average travel speed is computed from the running time on the arterial segment(s) and the intersection approach delay.

The arterial levels of service are defined precisely in a table within the methodology. However, some broad descriptions of the various levels are useful.

Arterial level of service is defined in terms of *average travel speed* of all through-vehicles on the arterial. It is strongly influenced by the number of signals per mile and the average intersection delay. On a given facility, such factors as inappropriate signal timing, poor progression, and increasing traffic flow can substantially degrade the arterial LOS. Arterials with high signal densities are even more susceptible to these factors. Arterial LOS D will probably be observed even before substantial intersection problems, but both such problems and even poorer arterial LOS values are not far behind arterial LOS D.

TABLE 11-1. ARTERIAL LEVELS OF SERVICE

ARTERIAL CLASS	I	II	III
Range of Free Flow Speeds (mph)	45 to 35	35 to 30	35 to 25
Typical Free Flow Speed (mph)	40 mph	33 mph	27 mph
LEVEL OF SERVICE	AVERAGE TRAVEL SPEED (MPH)		
A	≥ 35	≥ 30	≥ 25
B	≥ 28	≥ 24	≥ 19
C	≥ 22	≥ 18	≥ 13
D	≥ 17	≥ 14	≥ 9
E	≥ 13	≥ 10	≥ 7
F	< 13	< 10	< 7

The following general statements may be made regarding arterial level of service.

Level-of-service A describes primarily free flow-operations at average travel speeds usually about 90 percent of the free flow speed for the arterial class. Vehicles are completely unimpeded in their ability to maneuver within the traffic stream. Stopped delay at signalized intersections is minimal.

Level-of-service B represents reasonably unimpeded operations at average travel speeds usually about 70 percent of the free flow speed for the arterial class. The ability to maneuver within the traffic stream is only slightly restricted and stopped delays are not bothersome. Drivers are not generally subjected to appreciable tension.

Level-of-service C represents stable operations. However, ability to maneuver and change lanes in midblock locations may be more restricted than in LOS B, and longer queues and/or adverse signal coordination may contribute to lower average travel speeds of about 50 percent of the average free flow speed for the arterial class. Motorists will experience an appreciable tension while driving.

Level-of-service D borders on a range on which small increases in flow may cause substantial increases in approach delay and, hence, decreases in arterial speed. This may be due to adverse signal progression, inappropriate signal timing, high volumes, or some combination of these. Average travel speeds are about 40 percent of free flow speed.

Level-of-service E is characterized by significant approach delays and average travel speeds of one-third the free flow speed or lower. Such operations are caused by some combination or adverse progression, high signal density, extensive queuing at critical intersections, and inappropriate signal timing.

Level-of-service F characterizes arterial flow at extremely low speeds below one-third to one-quarter of the free flow speed. Intersection congestion is likely at critical signalized locations, with high approach delays resulting. Adverse progression is frequently a contributor to this condition.

Table 11-1 contains the arterial level-of-service definitions, which are based on average travel speed over the segment being considered (up to and including the entire facility). The "arterial class" concept is defined as part of the Methodology to follow.

II. METHODOLOGY AND PROCEDURES FOR APPLICATION

This methodology provides the *framework* for arterial evaluation. If field data are available, this framework can be used to determine the level of service of a given arterial without reference to running time and intersection delay estimates of this chapter. Rather than considering field evaluation as a lesser method, the transportation specialist should consider this as a better and more accurate alternative.

Note that field data on free flow speed will help in determining the arterial class and also in estimating the running time per mile. In cases where the specific arterial does not yet exist, data on free flow speed at comparable facilities would be most useful.

The procedure to determine arterial level of service has seven steps, as shown in Figure 11-2:

1. Establish the *location and length* of arterial to be considered.
2. Determine the *arterial class,* using the classification scheme presented herein in conjunction with the measurement of free flow speed.
3. Divide the arterial into *sections* for the purpose of the evaluation, where each section contains one or more arterial *segments.*
4. Compute the arterial *running time* for each segment, and aggregate for the sections (depending on whether or not sections larger than the individual segments were defined).
5. Tabulate the necessary *information on each intersection,* and compute the *approach delay* at each intersection taking into account:
   a. Intersection parameters for the through movement

      C, the cycle length;
      g/C ratio;

X, the v/c ratio; and
c, the capacity of the through-lane group.

b. Quality of the signal progression.
c. Relation between approach delay and stopped delay.

6. Compute *average travel speed*:
a. By section to prepare a speed profile.
b. Over the entire facility.

7. Assess the *level of service* (LOS) by referring to the table that defines the LOS ranges.

On two-way arterials, the methodology must be applied *twice* (i.e., once in each direction).

Steps 4 through 6 can be superseded by field data measurements of the average speed by doing travel time and delay studies along the arterial. Appendix I presents the field data collection procedures needed to provide the necessary data.

The remaining sections of this "Methodology" address each of the foregoing steps.

STEP 1—ESTABLISH THE ARTERIAL TO BE CONSIDERED

As a preliminary to subsequent steps, it is useful to precisely define the location and length of the arterial to be considered, including the assembly of all relevant physical, signalization, and traffic data.

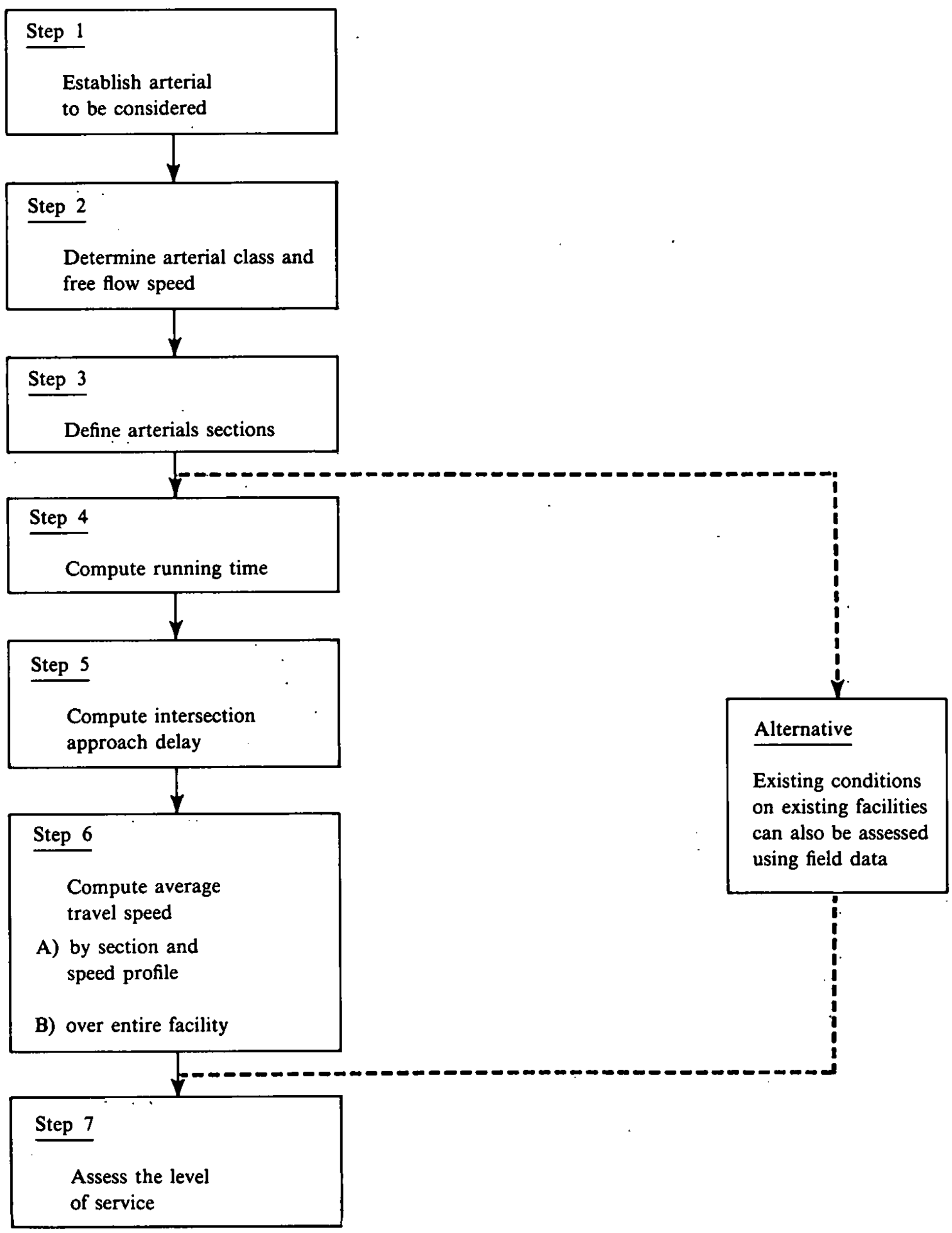


Figure 11-2. Arterial level of service methodology.

Consideration should be given to whether the extent of the arterial being analyzed is sufficient, or whether additional sections should be considered.

STEP 2—DETERMINE THE ARTERIAL CLASS AND FREE FLOW SPEED

There are three arterial classes defined in this chapter, based on the arterial's function and design. Within each class, there is also a range of free flow speeds to consider.

In all cases, the arterial must be classified first by *functional category,* and then by *design category.* In some cases, the measurement of the free flow speed will be a valuable aid in determining the proper arterial class because of ambiguities in the classification.

Both free flow speed and actual average travel speed can be obtained by arterial travel time studies. Thus, the application of this chapter can be based entirely on actual field measurements. Appendix I presents the necessary field procedures.

Free flow speed is the average speed of motorists over those portions of arterial sections that are not close to signalized intersections, as observed during very low traffic volume conditions while drivers are not constrained by other vehicles or by traffic signals. The average free flow speed should approximate the *desired* speeds of the motorists for the given facility and its use. Free flow speeds may be measured by test cars or by spot speed observations away from the intersections.

The *functional category* must be identified first: is the facility a principal arterial or a minor arterial?

A *principal arterial* serves major through movements between important centers of activities in a metropolitan area and a substantial portion of trips entering and leaving the area. It also connects freeways with major traffic generators. In small cities (under 50,000), its importance is derived from the service provided to traffic passing through the urban area. Service to abutting land is very subordinate to the function of moving through traffic.

A *minor arterial* is a facility that connects and augments the principal arterial system. Although its main function is still traffic mobility, it performs this function at a somewhat lower level and places more emphasis on land access than on the principal arterial.

A system of minor arterials serves trips of moderate length and distributes travel to geographical areas smaller than those served by the principal arterial.

Within the functional classification, the arterial is further classified by its *design category.*

Typical suburban design represents an arterial with partial to almost full control of access with separate left-turn lanes and no parking. It may be multilane divided or undivided, or a two-lane facility with shoulders. Signals are spaced for good progressive movement (one to four signals per mile) or at even greater distances. Roadside development is of low density and the speed limits are usually 40 to 45 mph.

Intermediate design represents an arterial with partial control of access. It may be a multilane divided or an undivided one-way or a two-lane facility. It may have some separate or continuous left-turn lanes and some portions with parking permitted. It has a higher density of roadside development than the typical suburban design. It usually has 4 to 8 signals per mile. Speed limits are normally 30 to 40 mph.

Typical urban design represents an arterial with little or no control of access from driveways. It is an undivided one-way or two-way facility with two or more lanes. Parking is usually permitted. Generally, there are no separate left-turn lanes and some pedestrian interference is present. It commonly has 8 to 12 signals per mile. Roadside development is dense with residential and/or commercial strip development. Speed limits range from 25 to 35 mph.

Refer to Figure 11-3 for illustrations.

Table 11-2 should be used as an aid in the determination of the functional and design categories, in addition to the above definitions. Once the functional and design categories are established, the arterial class may be established by referring to Table 11-3.

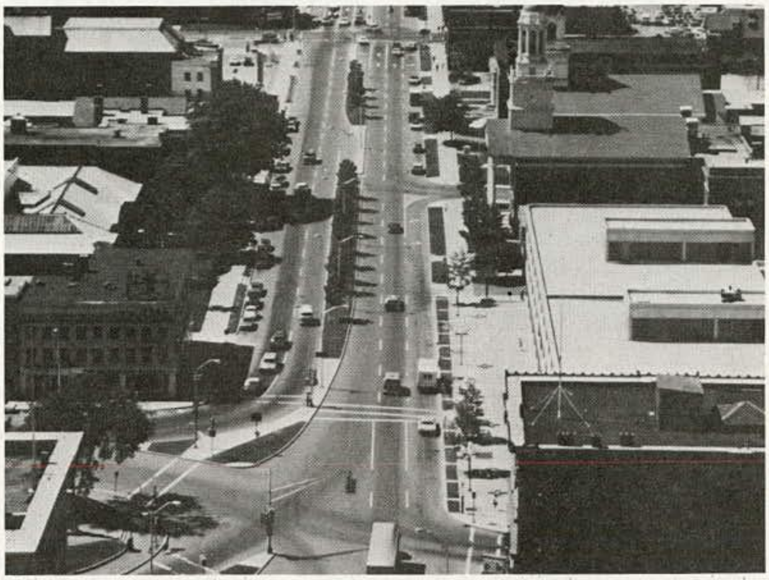
As a practical matter, there are sometimes ambiguities in determining the proper categories. The measurement or estimation of the *free flow speed* is a great aid in this determination, because each arterial class has a characteristic range of free flow speeds. As will be used in this chapter, note the following:

Arterial Class	Range of Free Flow Speeds, (mph)
I	35 → 45
II	30 → 35
III	25 → 35

Free flow speed alone cannot be used to determine the arterial class, but can be used as an effective check on the classification scheme.

The information on arterial class is used in Steps 4 and 7 of the methodology.

(a) Typical Suburban Design



(b) Intermediate Design

(c) Typical Urban Design

Figure 11-3. Illustration of design categories.

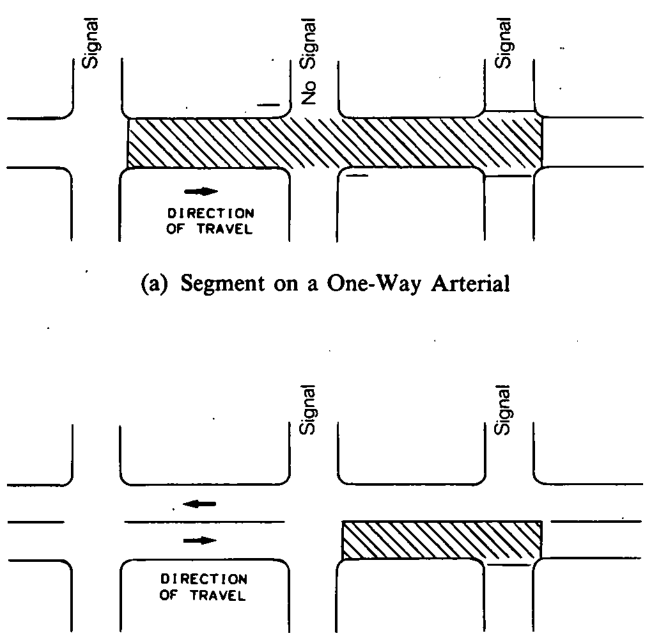
TABLE 11-2. AN AID IN ESTABLISHING ARTERIAL CLASSIFICATION

CRITERIA	FUNCTIONAL CATEGORY: PRINCIPAL ARTERIALS	FUNCTIONAL CATEGORY: MINOR ARTERIALS
Mobility function	Very important	Important
Access function	Very minor	Substantial
Points connected	Freeways, important activity centers, major traffic generators	Principal arterials
Predominant trips served	Relatively long trips between above points and through trips entering, leaving, and going through the city	Trips of moderate lengths within relatively small geographical areas

CRITERIA	DESIGN CATEGORY: SUBURBAN DESIGN	DESIGN CATEGORY: INTERMEDIATE	DESIGN CATEGORY: URBAN DESIGN
Control of access	Partial to almost full	Partial	Little or no control
Arterial type	Multilane divided; undivided or two-lane with shoulders	Multilane divided or undivided; one-way; two-lane	Undivided one-way; two-way, two or more lanes
Parking	No parking	Some parking	Parking permitted
Separate left-turn lanes	Yes	Some	No
Signals per mile	1 to 4	4 to 8	8 to 12
Speed limits	40 to 45 mph	30 to 40 mph	25 to 35 mph
Pedestrian interference	None	None	Some
Roadside development	Low density	Moderate	High density

TABLE 11-3. ARTERIAL CLASSES ACCORDING TO THEIR FUNCTION AND DESIGN CATEGORY

DESIGN CATEGORY	FUNCTIONAL CATEGORY: PRINCIPAL ARTERIAL	FUNCTIONAL CATEGORY: MINOR ARTERIAL
Typical Suburban Design and Control	I	II
Intermediate Design	II	III
Typical Urban Design	III	III



(a) Segment on a One-Way Arterial

(b) Segment on a Two-Way Arterial

Figure 11-4. Illustration of segments.

STEP 3—DIVIDE THE ARTERIAL INTO SECTIONS

The basic unit of the arterial is the *segment,* which is the one-directional distance from one signalized intersection to the next. Figure 11-4 illustrates the concept of "segments" on one- and two-way arterials.

If two or more consecutive segments are comparable in arterial class, segment length, speed limit, and general land use and activity, the user may wish to aggregate these into a *section.* All results would then focus on the section rather than on the component segments, because of the judgment that they can be aggregated.

In cases in which the length of the consecutive segments differs by 20 percent or more, segments should *not* be aggregated into the same section. Different sections should be defined. When a section is defined, it is the *average* segment length that should be used in finding the running time per mile in the next step.

STEP 4—COMPUTE THE ARTERIAL RUNNING TIME

There are two principal components to the total time a vehicle spends in a section, and on the arterial: arterial running time and intersection approach delay. This step is focused on computing the first of these terms so that it may be used in the denominator of the equation

$$\text{ART SPD} = [(3{,}600) \times (\text{Length})] / [(\text{Running Time Per Mile}) \times (\text{Length}) + (\text{Total Intersection Approach Delay})] \qquad (11\text{-}1)$$

where

ART SPD = arterial or section average travel speed, in mph;

Length = arterial or section length, in miles;

Running Time Per Mile = total of the running time per mile on all segments in the arterial or section, in sec;

Total Intersection Approach Delay = Total of the approach delay at all intersections within the defined arterial or sections, in sec: This is easily related to the stopped delay of Chapter 9.

The 3,600 is a conversion factor to compute ART SPD in mph.

In special cases, there may be unusual midblock delays due to pedestrian crosswalks at which vehicles must regularly stop. There may be other such factors. Such delays may be added as a third term in the denominator of Eq. 11-1.

To compute the running time in a segment, the user must know

- The arterial class.
- The segment length, in miles.
- The free flow speed, in mph.

The segment running time may then be found by looking up Table 11-4.

If a section has been defined that encompasses several segments, it is the *average* segment length that should be used in finding the running time per mile from Table 11-4. However, it is then multiplied by the section length.

Within each arterial class, there are a number of factors that can influence the actual free flow speed and the running speed per mile. Table 11-4 shows the effect of length directly; this has been synthesized from arterial research conducted by FHWA and other sources. In addition, there are such factors as the presence of parking, opportunities for side frictions, and the local development and street use. In this chapter, these factors are taken to influence the free flow speed, so that observation of free flow speed is a proxy for these unstated factors. Once free flow speed is estimated, the running speed used also reflects these unstated factors; Table 11-4 contains higher running times for the lower free flow speeds within each class.

If for some reason it is not possible to observe the free flow speed on the actual facility or on comparable existing facilities, Table 11-4 contains a note on which "default" values to use.

TABLE 11-4. SEGMENT RUNNING TIME PER MILE

ARTERIAL CLASS	I			II		III		
Free Flow Speed (mph)	45	40	35	35	30	35	30	25
Average Segment Length (miles)	RUNNING TIME PER MILE (SEC/MI)							
0.05							227	265
0.10				145	155	165	180	220
0.15				135	141	140	150	180
0.20	109	115	125	128	134	130	140	165
0.25	104	110	119	120	127	122	132	153
0.30	99	102	110					
0.40	94	96	105					
0.50	88	93	103					
1.00	80	90	103					

NOTES:

1. It is best to have an estimate of free flow speed. If one is lacking, however, then use the above table assuming the following default values:

For Class	*Free Flow Speed (mph)*
I	40
II	35
III	30

2. For very long segment lengths on Class I arterials (1 mi or longer), free flow speeds may be used to compute running time per mile. These times are shown in the entries for a 1.0-mile segment length.
3. If a Class I arterial has a segment length lower than 0.20 mi, the user should (a) reevaluate the classification and if it should remain as a distinct segment, then (b) use the values for 0.20 mi.
4. Likewise, Class II and Class III arterials with segment lengths greater than 0.25 mi should first be reevaluated (i.e., is the classification correct?). If necessary, the above values can be extrapolated.
5. Although this table does not show segment running time dependent on traffic flow rate, it is logical that there is such a dependence. However, the dependence of intersection delay on traffic flow rate is much stronger and thus dominates in the computation of arterial travel speed.

However, there should be a local history of free flow speeds on different arterial types.

Example—What is the running time on a segment that is 0.20 mi long and has a free flow speed of 40 mph? The arterial is a principal arterial, suburban design.

Solution—Note that the arterial is Class I, based on Table 11-3. Referring to Table 11-4, the running time per mile is estimated at 115 sec per mile, so that the segment running time is 115 × 0.20 = 23 sec.

Example—Consider the above case, but with an average 30-sec midblock delay due to a pedestrian crosswalk. What should be done?

Solution—The analysis should be done as above, but the 30 sec should be added as a third term in the denominator of Eq. 11-1 when the computations are done.

Example—There are three consecutive segments on a north-south two-lane two-way facility (i.e., one lane in each direction), of lengths 0.15, 0.17, and 0.13 mi respectively, all with a free flow speed of 30 mph. It is a principal arterial. What is the northbound running time in the section? The arterial is Class III.

Solution—Note that it *is* reasonable to define a single section if all necessary conditions are met, including all lengths within 20 percent of the average segment (see Step 3 of the Methodology). From Table 11-4, the running time per mile for a Class III arterial with 30 mph free flow speed is 150 sec per mile for a 0.15-mi segment. This is the average length of the three segments within this section. The actual running time is computed by

$$(150) \times (0.15 + 0.17 + 0.13) = 67.5 \text{ sec}$$

in the section.

Example—What is the southbound running time in the same section?

Solution—The southbound running time is found in the same way, and the answer is therefore the same. However, this is a useful reminder that two-way arterials must be evaluated in each direction; the answers will generally be different because of the influence of intersection delay (the effect of different signal progression quality in the two directions will contribute to this).

As noted in Table 11-4, it is logical that there is a dependence of segment running time on traffic flow rate. However, arterial research conducted for FHWA in the early 1980's did not establish a quantitative relation for such a dependence. It logically exists, but is not strong. Certainly it is not as strong as the effect of segment length on segment running time. Nor is it as strong as the substantial variation of intersection approach delay with traffic flow rate.

As a practical matter, computations of *arterial travel speed* for different traffic flow rates would be dominated by the changes in intersection approach delay, whether or not the segment running time dependence were clearly identified. Thus the absence of such an explicit factor does not affect the practical result, namely the computation of arterial travel speed.

STEP 5—TABULATE INTERSECTION INFORMATION AND COMPUTE DELAY

In order to compute the arterial or section speed, the individual intersection delays are needed. Because the arterial function is to serve through traffic, the dominant lane group in which the through traffic is included is to be used for characterizing the arterial.

The correct delay to use in the arterial evaluation is the total *approach* delay, which can be related to the intersection stopped delay as follows:

(Total Approach Delay = 1.3 × (Intersection Stopped Delay) + (Second Order Term)

The "second order term" is a refinement on the effect of the signal progression on the approach delay, as distinct from the effect on the stopped delay. Given the precision of the various estimates in the stopped delay itself, this refinement can be neglected. Thus the *intersection approach delay* can be computed by

$$D = 1.3\, d \qquad (11\text{-}2)$$

where:

D = intersection approach delay, in sec/veh; and
d = intersection stopped delay, in sec/veh

and where the intersection stopped delay is computed in accord with Chapter 9. In general, the user will have the necessary information available because the intersections would have had to have been evaluated individually as part of the overall effort.

The random intersection stopped delay equation is

$$d = 0.38C \frac{[1 - g/C]^2}{[1 - (g/C)(X)]} + 173\, X^2 \left[(X - 1) + \sqrt{(X - 1)^2 + (16X/c)}\right] \qquad (11\text{-}3)$$

where:

d = average stopped delay per vehicle for the subject lane group, in sec/veh;
C = cycle length, in sec;
g/C = green ratio for the subject lane group; the ratio of effective green time to cycle length;
X = v/c ratio for the subject lane group; and
c = capacity of the through lane group.

The progression factor, PF, must be applied to this to yield the stopped delay.

Equation 11-3 predicts the average stopped delay per vehicle for an assumed random arrival pattern for approaching vehicles. The first term of the equation accounts for *uniform delay*, i.e., the delay that occurs if arrival demand in the subject lane group is uniformly distributed over time. The second term of the equation accounts for *incremental delay* of random arrivals over uniform arrivals, and for the additional delay due to occasional cycle failures. The equation yields reasonable results for values of X between 0.0 and 1.0. Where oversaturation occurs for long periods (> 15 min), it is difficult to accurately estimate delay, as spillbacks may extend to adjacent intersections. The equation may be used with caution for values of X up to 1.2, but delay estimates for higher values are not recommended. Oversaturation, i.e., $X > 1.0$, is an undesirable condition that should be ameliorated if possible.

The information needed to compute the intersection stopped delay is almost certainly available from computations done using Chapter 9.

If for any reason the capacity is not readily available or if the *adjusted demand flow rate* (denoted v, with units of vph) is desired, recall that the v/c ratio X is defined by $X = v/c$. The "adjusted demand flow rate" is computed by correcting for the peak-hour factor and the lane utilization factor, as done in Chapter 9:

$$v = (V/\text{PHF}) \times U \qquad (11\text{-}4)$$

where:

v = adjusted demand flow rate for the lane group, in vph;
V = demand volume for the lane group, in vph;
PHF = peak-hour factor; and
U = lane utilization factor.

The lane utilization factor is shown in Table 11-5, which is taken from Chapter 9.

In certain applications in which approximations are needed or desired (such as a *planning application* of the methodology), it may also be useful to recall a default relation for the capacity of the lane group:

$$c = 1{,}600 \times N \times (g/C) \qquad (11\text{-}5)$$

where N is the number of lanes in the lane group and both C and g/C have been defined above. When Eq. 11-5 is used to compute a capacity value (rather than using the multiple correction factors of Chapter 9), the evaluation becomes highly approximate. This may be used in "planning" applications of the arterial methodology. Note that some detailed information on signal timing and quality of progression is needed in *all* applications of the arterial methodology.

The *quality of the progression* on the segment that includes the intersection has a significant impact on the intersection delay. There are five "arrival types" defined in Chapter 9:

1. *Type 1*—This condition is defined as a dense platoon arriving at the intersection at the beginning of the red phase. This is the worst platoon condition.
2. *Type 2*—This condition may be a dense platoon arriving during the middle of the red phase, or a dispersed platoon arriving throughout the red phase. Better than Type 1, this is still an unfavorable platoon condition.
3. *Type 3*—This condition represents totally random arrivals. This occurs when arrivals are widely dispersed throughout the red and green phases, and/or where the approach is totally uncoordinated with other signals—either because it is at an isolated location or because nearby signals operate on different cycle lengths. This is an average condition.
4. *Type 4*—This condition is defined as a dense platoon arriving during the middle of the green phase, or a dispersed platoon arriving throughout the green phase. This is a moderately favorable platoon condition.
5. *Type 5*—This condition is defined as a dense platoon arriving at the beginning of the green phase. *It is the most favorable platoon condition.*

TABLE 11-5. LANE UTILIZATION FACTORS

NO. OF THROUGH LANES IN GROUP (EXCLUDING LANES USED BY LEFT-TURNING VEHICLES)	LANE UTILIZATION FACTOR U
1	1.00
2	1.05
≥3	1.10

The arrival type is best observed in the field, but could be approximated by examining time-space diagrams for the arterial or street in question, using the platoon ratio R, as explained in Chapter 9. As noted in Chapter 9, the arrival type should be determined as accurately as possible, because it will have a significant impact on delay estimates and level-of-service determination.

As noted, *the stopped delay estimate* obtained from Eq. 11-3 *is for an assumed random arrival condition.* In most cases, arrivals are not random, but are platooned as a result of signal progression and other factors. As part of the input data for an operational analysis, five arrival types were defined, and one would be specified for each lane group. The delay obtained from Eq. 11-3 is multiplied by the platoon adjustment factor, given in Table 11-6.

When the signal progression is favorable to the through-vehicle lane group, delay will be considerably less than that for random arrivals. Similarly, when signal progression is unfavorable, delay can be considerably higher than that for random arrivals. The variation of delay with progression quality decreases as the v/c ratio, X, approaches 1.00, and is greater for pretimed signals than for other types of signalization. Left-turn movement delays are generally unaffected by progression: protected left-turn phases are rarely progressed, and permissive left-turn delay is most dependent on opposing traffic.

Delay is a complicated variable that is sensitive to a variety of local and environmental conditions. These procedures provide reasonable estimates for delays expected for average conditions. They are most useful when used to compare operational conditions for various geometric or signalization designs. When evaluating existing conditions, it is advisable to measure delay in the field. Appendix III of Chapter 9 contains guidelines for intersection delay measurements using lane occupancy and volume counts.

Example—Consider an arterial segment with a through-lane group with $N = 2$ lanes, a demand volume of 1,500 vph, and a PHF = 0.91. Further, there is a pretimed signal with a cycle length of 90 sec and a g/C ratio of 0.60. Vehicles arrive as a dense platoon at the beginning of the green. What is the estimated intersection approach delay?

Solution—To use Eq. 11-3 to compute intersection stopped delay, it is necessary to know C, g/C, X, and c. The last two terms must be computed because they are not given.

From Eq. 11-4, the adjusted demand flow rate is

$$v = (1{,}500/0.91) \times (1.05) = 1{,}731 \text{ vph}$$

where the lane utilization factor is read from Table 11-5.

There is no source for capacity information other than the default relation of Eq. 11-5:

TABLE 11-6. PROGRESSION ADJUSTMENT FACTOR, PF

TYPE OF SIGNAL	LANE GROUP TYPES	v/c RATIO, X	ARRIVAL TYPE 1	2	3	4	5
Pretimed	TH, RT	≤ 0.6	1.85	1.35	1.00	0.72	0.53
		0.8	1.50	1.22	1.00	0.82	0.67
		1.0	1.40	1.18	1.00	0.90	0.82
Actuated	TH, RT	≤ 0.6	1.54	1.08	0.85	0.62	0.40
		0.8	1.25	0.98	0.85	0.71	0.50
		1.0	1.16	0.94	0.85	0.78	0.61
Semiactuated[a]	Main St. TH, RT	≤ 0.6	1.85	1.35	1.00	0.72	0.42
		0.8	1.50	1.22	1.00	0.82	0.53
		1.0	1.40	1.18	1.00	0.90	0.65

[a] Semiactuated signals are typically timed to give all extra green time to the main street. This effect should be taken into account in the allocation of green times.

ARTERIAL SUMMARY OF INTERSECTION DELAY ESTIMATES

Arterial: ______________________ ______________________-bound

File or Case No.: ______________________ Date: ______________________

Prepared by: ______________________

Seg-ment	Cycle Length C	g/C	v/c Ratio X	Lane Group Capacity c	Random Arrival Delay[a] (sec)	Arrival Type	Progression Factor PF[b]	Estimated Stopped Delay[c]	Intersec. LOS	Estimated Approach Delay[d]
1										
2										
3										
4										
5										
6										
7										
8										
9										
10										
11										
12										
13										
14										
15										

[a] Equation 11-3
[b] Table 11-6
[c] Multiply (Random Arrival Delay) times (Progression Factor PF)
[d] Multiply Stopped Delay by 1.3 as in Equation 11-2

NOTES:
1. Adjusted demand flow rate v may be computed from Equation 11-4: v = (V/PHF) X U.
2. If lane group capacity c is not known, it may be computed from Chapter 9 or estimated from the default Equation 11-5: c = 1,600 × N × (g/C). This is highly approximate.
3. Round delay estimates to one place after the decimal.

Figure 11-5. Arterial summary of intersection delay estimates worksheet.

$$c = 1{,}600 \times 2 \times (0.60) = 1{,}920 \text{ vph}$$

Thus the remaining computations are approximate, and are suitable primarily for planning estimates.

The v/c ratio is computed from $X = v/c$, where $X = 1{,}731/1{,}920 = 0.90$. This may be used in the delay computations.

Using Eq. 11-3 with

$$C = 90 \text{ sec}$$
$$g/C = 0.60$$
$$X = 0.90$$
$$c = 1{,}920 \text{ vph}$$

the intersection random stopped delay is computed as $d = 16.4$ sec/veh.

By virtue of the description of the arriving vehicles, the "arrival type" is Type 5. Given a pretimed signal and a v/c ratio of 0.90, consult Table 11-6 to find the platoon factor PF = 0.67 for a v/c ratio of 0.80 and PF = 0.82 for a v/c ratio of 1.00. Interpolating, the v/c ratio of 0.90 would have PF = 0.75. Thus the estimated stopped delay is $0.75 \times 16.4 = 12.3$ sec/veh.

The *approach* delay is related to the stopped delay by a factor of 1.3 as cited in Eq. 11-2, so that the approach delay is $1.3 \times 12.3 = 16.0$ sec/veh.

The computations must be done for each signalized intersection, or obtained from the results of Chapter 9 evaluations. Figure 11-5 is a summary worksheet for the intersection delay computations. An additional blank worksheet is contained at the end of the chapter in Appendix II.

STEP 6—COMPUTE AVERAGE TRAVEL SPEED

The average speed is to be computed by section and over the entire arterial. It is recommended that the user also prepare a speed profile of the facility, and supplement the LOS assessment with insights gained from the speed profile and the levels of service of the individual intersections.

Figure 11-6 shows some illustrative data filled in on a worksheet which is provided to ease the task of assembling the information. A blank worksheet is contained at the end of the chapter in Appendix II.

COMPUTATION OF ARTERIAL LEVEL OF SERVICE WORKSHEET

Arterial: *Fourth Avenue* *North*-bound

File or Case #: *001* Date: *10/20/84*

Fourth Ave. is a principal arterial, intermediate design.

Prepared by: --

ART SPD = $\frac{3600 \text{ (Sum of Length)}}{\text{Sum of Time}}$

Segment	Length (mi)	Arterial Class	Free Flow (mph)	Section	Running Time[a] (sec)	Intersec. Approach Delay[b]	Other Delay (sec)	Sum of Time by Section	Sum of Length by Section	Arterial SPD[c] (mph)	Arterial LOS by Section
1	0.08		35			10.0					
2	0.12		35	1		13.0					
3	0.10		35			11.0					
4	0.20		35			5.0					
5	0.20		35	2		7.0					
6	0.20		35			10.0					
7	0.10		35	3		8.0					
8	0.15		35	4		4.0					
9	0.05		35	5		6.0					
10											
11											
12											
13											
14											
15											

Grand Sum of Time = ☐

Grand Sum of Length = ☐

3,600 × ☐ / ☐ = ☐ MPH

[a] Use Table 11-4 and multiply by segment length

[b] From summary on Arterial Summary of Intersection Delay Estimates Worksheet

[c] See upper right corner of this Table for the Equation

Note: Round delay estimates to one place after decimal

Figure 11-6. Computation of arterial level of service worksheet.

Equation 11-1 is used in each section and on the overall facility to compute the arterial speed in the section or on the facility:

ART SPD = [(3,600) × (Length)]/[(Running Time Per Mile) × (Length) + Total Intersection Approach Delay)]

where the terms have already been defined.

Figure 11-7 shows the worksheet (Fig. 11-6) with the computations done and entered. Doing such computations for each section and for the total, the speed profile illustrated in Figure 11-8 may be constructed. For segments 1 and 9, the running time per mile for a segment length of 0.10 mi is used, but is multiplied by the actual segment lengths.

Sample Computation. It is given that Fourth Avenue is a principal arterial, intermediate design, with a 35-mph free flow speed. From Table 11-3, it is arterial class II. In section 2 of the arterial, the average segment length is 0.20 mi. From Table 11-4, the running time per mile is 128 sec per mile for a class II arterial with a 35-mph free flow speed and this segment length. The total running time in the section is given by

$$128 \times (0.20 + 0.20 + 0.20) = 76.8 \text{ sec}$$

The total intersection *approach* delay is given in Figure 11-6 as (5.0 + 7.0 + 10.0) = 22.0 sec, so that the total time is 76.8 + 22.0 = 98.8 sec.

The arterial speed in the section is 3,600 × 0.60/98.8 = 21.9 mph.

COMPUTATION OF ARTERIAL LEVEL OF SERVICE WORKSHEET

Arterial: *Fourth Avenue* *North* -bound

File or Case #: *001* Date: *10/20/84*

Prepared by:

ART SPD = 3600 (Sum of Length) / Sum of Time

Segment	Length (mi)	Arterial Class	Free Flow (mph)	Section	Running Time[a] (sec)	Intersec. Approach Delay[b]	Other Delay (sec)	Sum of Time by Section	Sum of Length by Section	Arterial SPD[c] (mph)	Arterial LOS by Section
1	0.08	II	35		11.6	10.0	-				
2	0.12	II	35	1	17.4	13.0	-	77.5	0.30	13.9	
3	0.10	II	35		14.5	11.0	-				
4	0.20	II	35		25.6	5.0	-				
5	0.20	II	35	2	25.6	7.0	-	98.8	0.60	21.9	
6	0.20	II	35		25.6	10.0	-				
7	0.10	II	35	3	14.5	8.0	-	22.5	0.10	16.0	
8	0.15	II	35	4	20.2	4.0	-	24.2	0.15	22.3	
9	0.05	II	35	5	7.2	6.0	-	13.2	0.05	13.6	
10											
11											
12											
13											
14											
15											

Note: Given: Principal Arterial, Intermediate Design. From Table 11-4. Class II.

[a] Use Table 11-4 and multiply by segment length

[b] From summary on Arterial Summary of Intersection Delay Estimates Worksheet

[c] See upper right corner of this Table for the Equation

Note: Round delay estimates to one place after decimal

Grand Sum of Time = $\boxed{236.2}$

Grand Sum of Length = $\boxed{1.20}$

3,600 × $\boxed{1.20}$ = $\boxed{18.3}$ MPH

$\boxed{236.2}$

Figure 11-7. Computation of arterial level of service worksheet.

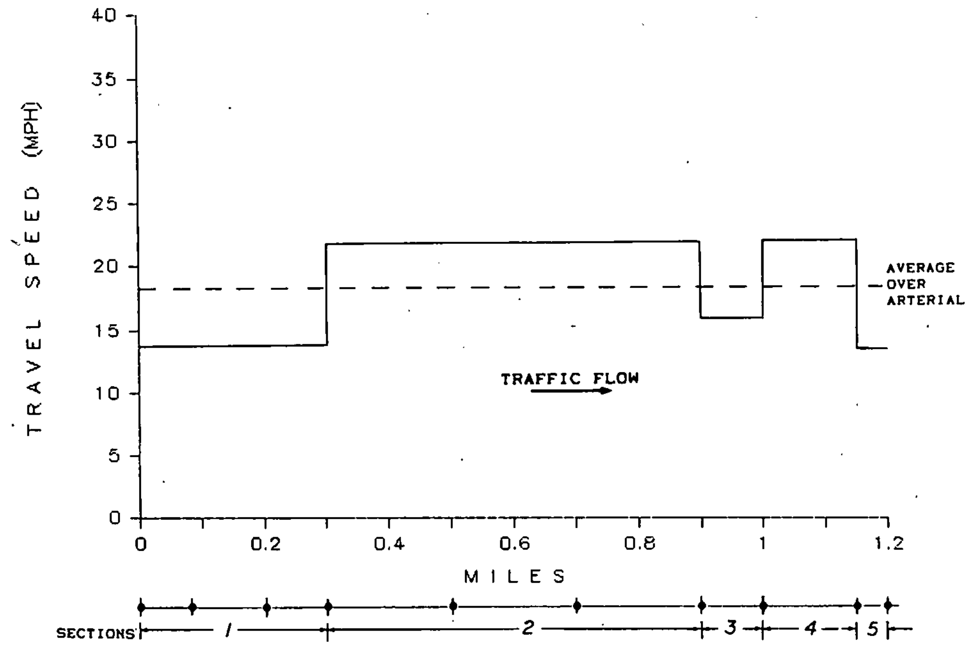


Figure 11-8. Speed profile by arterial section.

STEP 7—ASSESS THE LEVEL OF SERVICE

There is a distinct set of arterial level-of-service values established for each *arterial class.* These are based on the differing expectations drivers are judged to have for the different classes of arterials.

In defining the levels of service, both the free flow speed of the class and the intersection LOS definitions were taken into account. In general, the arterial levels of service are based on the smooth and efficient movement of the through traffic along an entire arterial. Therefore, it is necessary to expect *less* delay per segment than the corresponding intersection level of service.

Table 11-1 gives the arterial level-of-service definitions for each of the three arterial classes. The level-of-service definitions vary with the arterial class: the lesser the arterial class (i.e., the higher the class number), the lower the driver's expectation while driving on that facility, and the lower is the speed associated with a given level of service. Thus, a Class III arterial provides LOS B at a lower speed than does a Class I arterial.

The user must be aware of this aspect in explaining before-after assessments of arterials when upgrading is involved: if reconstruction results in upgrading a facility from Class II to Class I, it is possible that the LOS will not change (or may even technically degrade) despite average speed and other improvements, because expectations would be higher.

Note that the concept of an overall arterial level of service is generally only meaningful when all segments on the arterial are of the same class. If there are different arterial classes represented, the LOS criteria are different.

The intersection levels of service used in Chapter 9 are given in Table 11-7. The arterial levels of service are also shown in this table. The arterial LOS definitions were made with the understanding that a smooth, good quality service to the through traffic is the prime concern. Thus, the arterial LOS definitions generally expect that there will be *less* delay per intersection than the corresponding intersection levels of service.

TABLE 11-7. LEVEL-OF-SERVICE CRITERIA FOR INTERSECTIONS AND ARTERIALS.

INTERSECTION LOS	
LEVEL OF SERVICE	STOPPED TIME DELAY PER VEHICLE (SEC)
A	≤ 5.0
B	5.1 to 15.0
C	15.1 to 25.0
D	26.1 to 40.0
E	40.1 to 60.0
F	> 60.0

ARTERIAL LOS			
ARTERIAL CLASS	I	II	III
Range of Free Flow Speeds (mph)	45 to 35	35 to 30	35 to 25
Typical Free Flow Speed (mph)	40 mph	33 mph	27 mph
LEVEL OF SERVICE	AVERAGE TRAVEL SPEED (MPH)		
A	≥ 35	≥ 30	≥ 25
B	≥ 28	≥ 24	≥ 19
C	≥ 22	≥ 18	≥ 13
D	≥ 17	≥ 14	≥ 9
E	≥ 13	≥ 10	≥ 7
F	< 13	< 10	< 7

III. SAMPLE CALCULATIONS

CALCULATION 1—ARTERIAL CLASS AND CLASSIFICATION

1. *Description*—An arterial with three lanes in each direction and signal spacing of 0.15 mi passes through an area with moderate roadside development. It is undivided. Virtually all of the traffic passes through the area; there is very little pedestrian activity. Identify the arterial class.

2. *Solution*—To determine the arterial class, it is necessary to decide the design and functional categories of the arterial, and then to use Table 11-3 to specify the arterial class.

The simple statement that "virtually all of the traffic passes through the area" defines the *functional category:* it is a principal arterial.

Table 11-2 can be used to assist in determining the design category: note that there are approximately 7 signals per mile (based on a 0.15-mi spacing); there is moderate roadside development; there is very little pedestrian activity; it is a multilane undivided facility. Thus the *design category* is found to be "intermediate."

Referring to Table 11-3, one concludes that *the arterial is Class II.* This information is used in determining the level-of-service definitions to be used in evaluating the arterial. Further, lacking more specific information, one can expect a free flow speed in the order of 33 mph (refer to the top of Table 11-1), with a range being 30 to 35 mph.

CALCULATION 2—COMPUTATION OF ARTERIAL LEVEL OF SERVICE

1. *Description*—Given a multilane divided arterial that functions as a principal arterial. There is significant control of access, no parking, and a signal spacing of approximately 0.30 mi between signals that are pretimed. There is little roadside development, and there are two lanes in each direction and a measured free flow speed of 39 mph.

Detailed information on the intersection parameters and the arterial segments for the southbound flow is contained in Figures 11-9 and 11-10. The progression is excellent in the southbound direction.

ARTERIAL SUMMARY OF INTERSECTION DELAY ESTIMATES

Arterial: *Sample Calculation 2* — *South* -bound

File or Case No.: -

Date: -

Prepared by: -

Segment	Cycle Length C	g/C	v/c Ratio X	Lane Group Capacity c	Random Arrival Delay[a] (sec)	Arrival Type	Progression Factor PF[b]	Estimated Stopped Delay[c]	Intersec. LOS	Estimated Approach Delay[d]
1	70	0.60	0.583	1800		5				
2	70	0.60	0.611	1800		5				
3	70	0.60	0.611	1800		5				
4	70	0.60	0.611	1800		5				
5	70	0.60	0.597	1800		5				
6	70	0.60	0.593	1800		5				
7	70	0.60	0.593	1800		5				
8										
9										
10										
11										
12										
13										
14										
15										

[a] Equation 11-3
[b] Table 11-6
[c] Multiply (Random Arrival Delay) times (Progression Factor PF)
[d] Multiply Stopped Delay by 1.3 as in Equation 11-2

NOTES:
1. Adjusted demand flow rate v may be computed from Equation 11-4: v = (V/PHF) X U.
2. If lane group capacity c is not known, it may be computed from Chapter 9 or estimated from the default Equation 11-5: c = 1,600 × N × (g/C). This is highly approximate.
3. Round delay estimates to one place after the decimal.

Figure 11-9. Sample calculation 2—description—using arterial summary of intersection delay estimates worksheet.

Determine the arterial level of service, by segment and for the entire facility. Do not aggregate the segments.

2. *Solution*—This solution will proceed according to the steps outlined in Figure 11-2. In some applications, it may not be necessary to do all steps, or it may be easier to do certain steps before others. For instance, if the intersection evaluations had been done previously (or if the summary information is available), that information may be entered on the appropriate worksheet (Fig. 11-6) by computing approach delay before the arterial running times are computed.

Step 1—Establish Arterial to be Considered

This has been done in the above statement.

Step 2—Determine Arterial Class

The *functional* category is given as that of principal arterial. The *design* category may be established by referring to Table 11-2 as an aid, and noting

- Multilane divided
- Significant control of access
- No parking
- Little roadside development
- Seven signals in 2.1 mi $\simeq$ 3 signals per mi

The facility is clearly a *suburban design.*

On the basis of a functional category of "principal arterial" and a design category of "suburban," the facility would be found to be a Class I arterial by Table 11-3.

Step 3—Define Arterial Sections

This step may be skipped, because the specification was given "do not aggregate the segments."

Nonetheless, it is relevant to note that there could have been some aggregation, based on average segment lengths and volume pattern. For instance, the following aggregations could have been done:

COMPUTATION OF ARTERIAL LEVEL OF SERVICE WORKSHEET

Arterial: *Sample Calculation 2* *South* -bound

File or Case #: -   Date: -

Prepared by:

ART SPD = $\frac{3600\ (\text{Sum of Length})}{\text{Sum of Time}}$

Traffic Flow

Segment	Length (mi)	Arterial Class	Free Flow (mph)	Section	Running Time[a] (sec)	Intersec. Approach Delay[b]	Other Delay (sec)	Sum of Time by Section	Sum of Length by Section	Arterial SPD[c] (mph)	Arterial LOS by Section
1	0.20		39	1							
2	0.20		39	2							
3	0.30		39	3							
4	0.30		39	4							
5	0.30		39	5							
6	0.40		39	6							
7	0.40		39	7							
8											
9											
10											
11											
12											
13											
14											
15											

[a] Use Table 11-4 and multiply by segment length
[b] From summary on Arterial Summary of Intersection Delay Estimates Worksheet
[c] See upper right corner of this Table for the Equation
Note: Round delay estimates to one place after decimal

Grand Sum of Time = ☐

Grand Sum of Length = ☐   ☐

3,600 × ☐ = ☐ MPH

☐

Figure 11-10. Sample calculation 2—description—using computation of arterial level of service worksheet.

Segment	Section
1	1
2	1
3	2
4	2
5	2
6	3
7	3

If the volume differences made the user uncomfortable with this aggregation, it could have been checked after the intersection delay was estimated.

Step 4—Compute Running Time

The arterial is Class I with a free flow speed of 39 mph, which establishes the relation to be used for the running time computation. Refer to Table 11-4.

Consider segment 1. For a Class I arterial, a segment length of 0.20 mi and a free flow speed of 40 mph, Table 11-4 indicates a running time per mile of 115 sec/mi; and for 35 mph, a running time per mile of 125 sec/mi. Interpolating, for 39 mph, use 117 sec/mi. The running time in the 0.20-mi segment is 117 × 0.20 = 23.4 sec. This is entered on the arterial level-of-service worksheet for summary of information (Fig. 11-5). The *given* information is shown on the worksheet in Figure 11-10; the completed worksheet is shown in Figure 11-12.

Step 5—Compute Intersection Delay

Figure 11-9 is the "Arterial Summary of Intersection Delay Estimates" for this sample calculation. Note that the information must be for the lane group containing the principal part of the through movement, because it is an *arterial* evaluation. This information is generally available for the desired lane group based on the Chapter 9 evaluations of individual intersections, as in the present case.

Equation 11-3 is used to compute the "random arrival delay," which can then be entered on the summary form.

The selection of the "arrival type" for the approaching vehicles is a special consideration. In this case, it is straightforward because of the given information that "the progression is excellent in the southbound direction." Matching this to the arrival

ARTERIAL SUMMARY OF INTERSECTION DELAY ESTIMATES

Arterial: *Sample Calculation 2* — *South* -bound

File or Case No.: - — Date: -

Prepared by: -

Traffic Flow

Segment	Cycle Length C	g/C	v/c Ratio X	Lane Group Capacity c	Random Arrival Delay[a] (sec)	Arrival Type	Progression Factor PF[b]	Estimated Stopped Delay[c]	Intersec. LOS	Estimated Approach Delay[d]
1	70	0.60	0.583		6.9	5	0.53	3.7	A	4.8
2	70	0.60	0.611		7.2	5	0.53	3.8	A	4.9
3	70	0.60	0.611		7.2	5	0.53	3.8	A	4.9
4	70	0.60	0.611		7.2	5	0.53	3.8	A	4.9
5	70	0.60	0.547		7.0	5	0.53	3.7	A	4.8
6	70	0.60	0.583		6.9	5	0.53	3.7	A	4.8
7	70	0.60	0.583		6.9	5	0.53	3.7	A	4.8
8										
9										
10										
11										
12										
13										
14										
15										

[a] Equation 11-3
[b] Table 11-6
[c] Multiply (Random Arrival Delay) times (Progression Factor PF)
[d] Multiply Stopped Delay by 1.3 as in Equation 11-2

NOTES:
1. Adjusted demand flow rate v may be computed from Equation 11-4: v = (V/PHF) X U.
2. If lane group capacity c is not known, it may be computed from Chapter 9 or estimated from the default Equation 11-5: c = 1,600 × N × (g/C). This is highly approximate.
3. Round delay estimates to one place after the decimal.

Figure 11-11. Sample calculation 2—solution—using arterial summary of intersection delay estimates worksheet.

type definitions, Type 5 is selected because it is defined as ". . . .a dense platoon arriving at the beginning of the green phase. It is the most favorable platoon condition."

Table 11-6 shows the progression factors PF for the given pretimed signals and arrival type 5 as follows:

v/c Ratio, X	Progression Factor, PF
$\leq$ 0.60	0.53
0.80	0.67
1.00	0.82

As shown in Figure 11-9, all of the intersections have v/c ratios near 0.60, so that PF = 0.53 is used for all of them.

The results of the intersection computations are shown in Figure 11-11, and are transferred to the arterial worksheet in Figure 11-12.

Step 6—Compute Average Travel Speed

Given the running time for Step 4 and the intersection delay time from Step 5, the computations may be done using the arterial level-of-service worksheet for summary of information. The completed worksheet is shown in Figure 11-12, with the calculation for each section (in this case, each segment) identical in form to that shown on the bottom of the worksheet for the entire arterial.

Figure 11-13 contains the *speed profile* for the arterial. It is a valuable depiction of the operation, and should be constructed as part of each evaluation.

Step 7—Assess the Level of Service

With all of the preliminary work done, the final determination of the level-of-service (LOS) values is straightforward. Referring to Table 11-1, the speeds computed in the arterial level-of-service worksheet for summary of information can be compared to the definitions for the appropriate arterial class (in this case, Class I, as established in Step 2). These are entered in the summaries of Figure 11-12 and the speed profile of Figure 11-13, together with the intersection levels of service determined previously.

As cited at the end of the "Introduction," the intersection

COMPUTATION OF ARTERIAL LEVEL OF SERVICE WORKSHEET

Arterial: *Sample Calculation 2* *South*-bound

File or Case #: - Date: -

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ART SPD = $\frac{3600 \text{ (Sum of Length)}}{\text{Sum of Time}}$

See Table 11-1

Traffic Flow

Segment	Length (mi)	Arterial Class	Free Flow (mph)	Section	Running Time[a] (sec)	Intersec. Approach Delay[b]	Other Delay (sec)	Sum of Time by Section	Sum of Length by Section	Arterial SPD[c] (mph)	Arterial LOS by Section
1	0.20	I	39	1	23.4	4.8	-	28.2	0.20	25.5	C
2	0.20	I	39	2	23.4	4.9	-	28.3	0.20	25.4	C
3	0.30	I	39	3	31.1	4.9	-	36.0	0.30	30.0	B
4	0.30	I	39	4	31.1	4.9	-	36.0	0.30	30.0	B
5	0.30	I	39	5	31.1	4.8	-	35.9	0.30	30.1	B
6	0.40	I	39	6	39.5	4.8	-	44.3	0.40	32.5	B
7	0.40	I	39	7	39.5	4.8	-	44.3	0.40	32.5	B
8											
9											
10											
11											
12											
13											
14											
15											

Grand Sum of Time = 253.1

Grand Sum of Length = 2.10 B

$\frac{3{,}600 \times 2.10}{253.1} = 29.9$ MPH

[a] Use Table 11-4 and multiply by segment length
[b] From summary on Arterial Summary of Intersection Delay Estimates Worksheet
[c] See upper right corner of this Table for the Equation
Note: Round delay estimates to one place after decimal

Figure 11-12. Sample calculation 2—solution—using computation of arterial level of service worksheet.

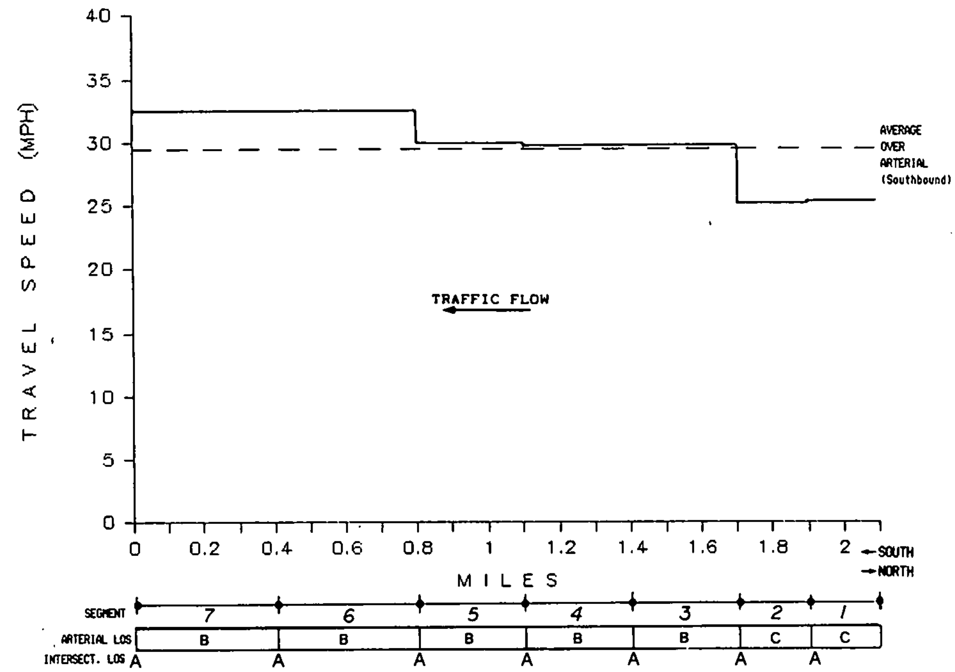


Figure 11-13. Speed profile for sample calculation 2 southbound traffic.

ARTERIAL SUMMARY OF INTERSECTION DELAY ESTIMATES

Arterial: *Sample Calculation 3* — *North* -bound

File or Case No.: - Date: -

Prepared by: -

Segment	Cycle Length C	g/C	v/c Ratio X	Lane Group Capacity c	Random Arrival Delay[a] (sec)	Arrival Type	Progression Factor PF[b]	Estimated Stopped Delay[c]	Intersec. LOS	Estimated Approach Delay[d]
1	70	0.60	0.417	1800						
2	70	0.60	0.417	1800						
3	70	0.60	0.417	1800						
4	70	0.60	0.361	1800						
5	70	0.60	0.361	1800						
6	70	0.60	0.306	1800						
7	70	0.60	0.306	1800						
8										
9										
10										
11										
12										
13										
14										
15										

a Equation 11-3
b Table 11-6
c Multiply (Random Arrival Delay) times (Progression Factor PF)
d Multiply Stopped Delay by 1.3 as in Equation 11-2

NOTES:

1. Adjusted demand flow rate v may be computed from Equation 11-4: $v = (V/PHF) \times U$.
2. If lane group capacity c is not known, it may be computed from Chapter 9 or estimated from the default Equation 11-5: $c = 1{,}600 \times N \times (g/C)$. This is highly approximate.
3. Round delay estimates to one place after the decimal.

Figure 11-14. Sample calculation 3—description—using arterial summary of intersection delay estimates worksheet.

LOS values are generally better than the arterial LOS values. This is logical, for an intersection with 3 to 4 sec of delay per vehicle is certainly LOS A, whereas an arterial that can have a speed of 39 mph but has one of 25 to 30 mph is certainly not LOS A.

CALCULATION 3—COMPUTATION OF ARTERIAL LEVEL OF SERVICE

1. *Description*—Consider the northbound side of the arterial cited in sample calculation 2, which has the intersection traffic as shown in Figure 11-14 and has a very poor progression, with virtually the entire northbound platoon arriving in the middle of the red at each intersection.

Determine the arterial level of service, by segment and for the entire facility. Do not aggregate the segments.

2. *Solution*—The calculations for this solution are indentical in form and sequence to those of sample calculation 2, and will not be repeated. However, there are certain key points that must be highlighted:

- The evaluation of an arterial is *by direction,* and a two-way arterial requires two evaluations, one for each direction, just as required in sample calculation 2.
- The arrival types in the two directions will generally be different, because the progression of the signal timing is often set to favor one direction over the other. This will have a major impact on the intersection delay estimates.
- It is useful to continue a complementary segment or link numbering sequence (as illustrated in Figures 11-13 and 11-17), so as not to confuse the final presentation. It is also useful to clearly mark the direction of travel.
- The intersections are those within the individual segments, and at the terminal end of the segment (i.e., the output end).

The results of the computations are shown in Figures 11-15 and 11-16, with the speed profile contained in Figure 11-17. For comparative purposes, the southbound speed profile is also contained on this figure. As shown in this figure, the intersection and arterial levels of service for *both* directions are also shown.

In the entire solution, only one additional point stands out: the selection of the arrival type so that the correct progression factors PF may be selected. The sample calculation description

ARTERIAL SUMMARY OF INTERSECTION DELAY ESTIMATES

Arterial: *Sample Calculation 3* — *North* -bound

File or Case No.: - Date: -

Prepared by: -

Segment	Cycle Length C	g/C	v/c Ratio X	Lane Group Capacity c	Random Arrival Delay[a] (sec)	Arrival Type	Progression Factor PF[b]	Estimated Stopped Delay[c]	Intersec. LOS	Estimated Approach Delay[d]
1	70	0.60	0.417	1800	5.8	2	1.35	7.8	B	10.1
2	70	0.60	0.417	1800	5.8	2	1.35	7.8	B	10.1
3	70	0.60	0.417	1800	5.8	2	1.35	7.8	B	10.1
4	70	0.60	0.361	1800	5.5	2	1.35	7.4	B	9.6
5	70	0.60	0.361	1800	5.5	2	1.35	7.4	B	9.6
6	70	0.60	0.306	1800	5.2	2	1.35	7.0	B	9.1
7	70	0.60	0.306	1800	5.2	2	1.35	7.0	B	9.1
8										
9										
10										
11										
12										
13										
14										
15										

a Equation 11-3
b Table 11-6
c Multiply (Random Arrival Delay) times (Progression Factor PF)
d Multiply Stopped Delay by 1.3 as in Equation 11-2

NOTES:
1. Adjusted demand flow rate v may be computed from Equation 11-4: v = (V/PHF) X U.
2. If lane group capacity c is not known, it may be computed from Chapter 9 or estimated from the default Equation 11-5: c = 1,600 × N × (g/C). This is highly approximate.
3. Round delay estimates to one place after the decimal.

Figure 11-15. Sample calculation 3—solution—using arterial summary of intersection delay estimates worksheet.

COMPUTATION OF ARTERIAL LEVEL OF SERVICE WORKSHEET

Arterial: *Sample Calculation 3* *North*-bound

File or Case #: - Date: -

Prepared by:

$$\text{ART SPD} = \frac{3600\ (\text{Sum of Length})}{\text{Sum of Time}}$$

Segment	Length (mi)	Arterial Class	Free Flow (mph)	Section	Running Time[a] (sec)	Intersec. Approach Delay[b]	Other Delay (sec)	Sum of Time by Section	Sum of Length by Section	Arterial SPD[c] (mph)	Arterial LOS by Section
1	0.20	I	39	1	23.4	10.1	-	33.5	0.20	21.5	D
2	0.20	I	39	2	23.4	10.1	-	33.5	0.20	21.5	D
3	0.30	I	39	3	31.1	10.1	-	41.2	0.30	26.2	C
4	0.30	I	39	4	31.1	9.6	-	40.7	0.30	26.5	C
5	0.30	I	39	5	31.1	9.6	-	40.7	0.30	26.5	C
6	0.40	I	39	6	39.5	9.1	-	48.6	0.40	29.6	B
7	0.40	I	39	7	39.5	9.1	-	48.6	0.40	29.6	B
8											
9											
10											
11											
12											
13											
14											
15											

Traffic Flow ↑

[a] Use Table 11-4 and multiply by segment length
[b] From summary on Arterial Summary of Intersection Delay Estimates Worksheet
[c] See upper right corner of this Table for the Equation

Note: Round delay estimates to one place after decimal

Grand Sum of Time = 286.8

Grand Sum of Length = 2.10 C

$$\frac{3{,}600 \times 2.10}{286.8} = 26.4 \text{ MPH}$$

Figure 11-16. Sample calculation 3—solution—using computation of arterial level of service worksheet.

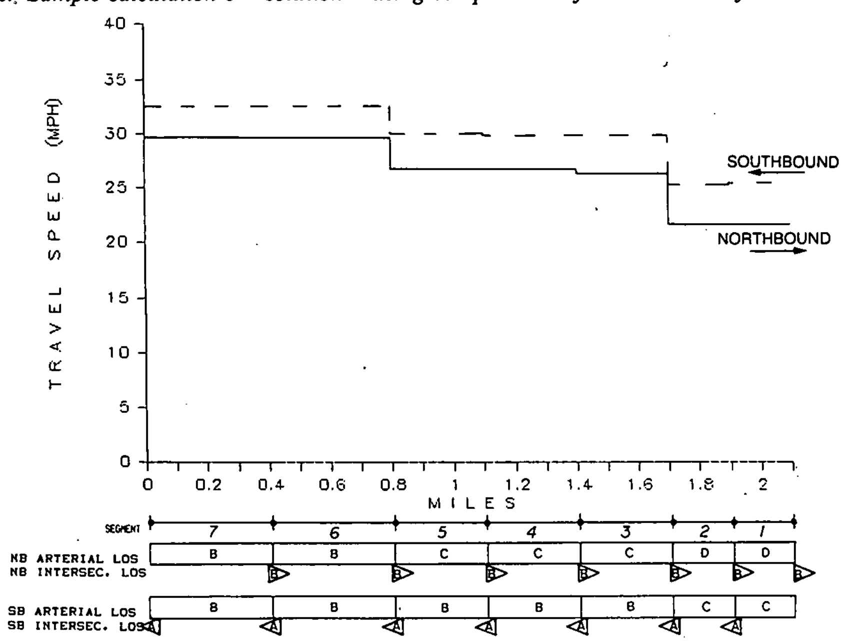


Figure 11-17. Speed profile for sample calculation 3 northbound traffic.

states that there is "a very poor progression, with virtually the entire northbound platoon arriving in the middle of the red at each intersection." It is important to note that this is *not* the worst condition: a careful reading of the arrival type descriptions makes it clear that it is Type 2 which covers the present case, with the worst case—Type 1—reserved for "a dense platoon arriving at the *beginning* of the red phase" (emphasis added).

CALCULATION 4—EFFECT OF TRAFFIC FLOW RATE ON ARTERIAL LEVEL OF SERVICE

1. *Description*—Given an arterial with two lanes in each direction and a 35-mph free flow speed, which has been found to be a Class II arterial. There are ten signals with a spacing of 0.20 mi between signals.

The intersections all have pretimed signals with a 60-sec cycle length and $g/C = 0.50$. The progression is excellent.

For a range of adjusted traffic demand from a flow rate of 600 vph to 1,600 vph, plot the arterial segment speed and find the arterial level of service, as well as the intersection levels of service.

2. *Solution*—The relations shown in this chapter for arterial running time do not depend explicitly on arterial volume or flow rate. See note 5 of Table 11-4.

The arterial speed is sensitive to traffic volume because the intersection delay is dependent on that volume. Recall that the basic relation is

$$\text{ART SPD} = [(3{,}600) \times (\text{Length})]/[(\text{Running Time Per Mile}) \times (\text{Length}) + (\text{Total Intersection Approach Delay})]$$

as specified in Eq. 11-1.

For the stated situation, the segment running time per mile is found from Table 11-4 as 128 sec/mi for a segment length of 0.20 mi. The running time in the segment is therefore $128 \times 0.20 = 25.6$ sec.

The intersection stopped delay is based on Eq. 11-3 and the application of the progression factor. Two parameters in Eq. 11-2 are given ($C = 60$ sec and $g/C = 0.50$). The other two, namely arterial lane group capacity, c, and *v/c ratio, X,* are not directly given.

Lacking specific information on the lane group capacity, it is both possible and necessary to use Eq. 11-5 to compute $c = 1{,}600 \times 2 \times 0.50 = 1{,}600$ vph, for all segments. If the g/C differed from segment to segment, the computed value would also differ. When using this relation for a specific site, the evaluation becomes highly approximate. However, this sample calculation is for a "typical" or representative arterial.

To compute the v/c ratio X, compute the adjusted demand flow rate as shown in Eq. 11-4. In the given information, this varies from $v = 600$ vph to $v = 1{,}600$ vph. For each value of v, the corresponding value of $X = v/1{,}600$, where $c = 1{,}600$ vph was just computed above.

The arrival type is Type 5, because "the progression is excellent." The progression factor PF is selected from Table 11-6 for arrival type 5 and pretimed signalization, with interpolation used as appropriate.

The results of the computations are given in Table 11-8, where the intersection approach delay is 1.3 times the stopped delay. The stopped delay is the "random arrival delay" of Eq. 11-3, multiplied by the progression factor PF.

The levels of service are identified by referring to Table 11-1 for a Class II arterial and to Table 11-7 for the intersections. Note that the intersection LOS is based on *stopped* delay, and is shown for the lane group containing the through traffic.

Figure 11-18 contains a plot of the arterial/segment speed as a function of the arterial volume, for the stated condition of a 0.20 segment length. Note that the intersection approach delay ranges from 16 percent to 55 percent of the total time spent on the segment, depending on the traffic flow rate.

TABLE 11-8. COMPUTATIONS FOR SAMPLE CALCULATION 4

							DIST = 0.20 MILES						
							INTERSECTION						
CYCLE LENGTH	g/c	FLOW (VPH)	CAPACITY (VPH)	X, THE v/c RATIO	RANDOM DELAY	PF	STOPPED DELAY	LOS	APPROACH DELAY	SEGMENT RUNNING TIME	SUM TIME (SEC)	AVERAGE TRAVEL SPEED (MPH)	ARTERIAL LOS
60	0.50	600	1,600	0.38	7.1	0.53	3.8	A	4.9	25.6	30.5	23.6	C
60	0.50	700	1,600	0.44	7.4	0.53	3.9	A	5.1	25.6	30.7	23.4	C
60	0.50	800	1,600	0.50	7.8	0.53	4.1	A	5.4	25.6	31.0	23.2	C
60	0.50	900	1,600	0.56	8.3	0.53	4.4	A	5.7	25.6	31.3	23.0	C
60	0.50	1,000	1,600	0.63	8.8	0.55	4.9	A	6.3	25.6	31.9	22.6	C
60	0.50	1,100	1,600	0.69	9.6	0.59	5.6	B	7.3	25.6	32.9	21.9	C
60	0.50	1,200	1,600	0.75	10.5	0.64	6.7	B	8.8	25.6	34.4	21.0	C
60	0.50	1,300	1,600	0.81	12.0	0.68	8.1	B	10.6	25.6	36.2	19.9	C
60	0.50	1,400	1,600	0.88	14.3	0.73	10.4	B	13.5	25.6	39.1	18.4	C
60	0.50	1,500	1,600	0.94	18.8	0.78	14.6	B	19.0	25.6	44.6	16.1	D
60	0.50	1,600	1,600	1.00	28.7	0.82	23.5	C	30.6	25.6	56.2	12.8	E

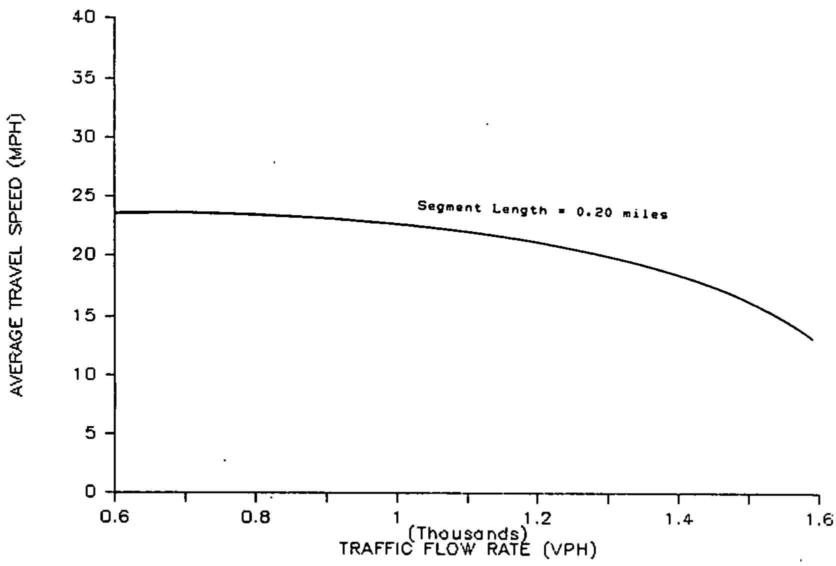


Figure 11-18. Sample calculation 4 speed as a function of arterial flow rate.

CALCULATION 5—EFFECT OF TRAFFIC FLOW RATE AND LENGTH ON ARTERIAL LEVEL OF SERVICE

1. *Description*—Reevaluate sample calculation 4, given the signal spacing is 0.10 mi between signals, and all other given information is the same as in calculation 4 including the arterial class.

2. *Solution*—Numerically, the computations are the same as in sample calculation 4, and all the introductory remarks are the same. The results of the computations are given in Table 11-9.

The levels of service are again identified by referring to Table 11-1 for a Class II arterial and to Table 11-7 for the intersections.

Figure 11-19 contains a plot of the arterial/segment average travel speed as a function of the arterial flow rate, for the stated condition of a 0.10 segment length. For comparative purposes, the plot for a 0.20-mi segment length is also shown.

The fact that the speeds are much lower and that the arterial level of service is now significantly lower than the intersection LOS deserves attention.

First, it is necessary to observe that the intersection delay per mile has increased (relative to sample calculation 4) by the simple fact that there are now more intersections per mile: at 0.20-mi spacings, there were 5 intersections per mile, whereas at 0.10-mi spacings, there are now 10 intersections per mile. Thus a delay of 8.0 sec/veh per intersection now contributes 10 × (8.0) = 80 sec/mi to the *arterial* travel time, whereas it was 5 × (8.0) = 40 sec/mi in the previous computation. Thus two radically different arterials are being compared.

The driver's expectation on an arterial is more demanding, as compared to an individual intersection. With 10 signals per mile, there must be very little approach delay per intersection in order to achieve a good quality of flow for the through-traffic.

TABLE 11-9. COMPUTATIONS FOR SAMPLE CALCULATION 5

							DIST = 0.10 MILES						
							INTERSECTION						
CYCLE LENGTH	g/c	FLOW (VPH)	CAPACITY (VPH)	X, THE v/c RATIO	RANDOM DELAY	PF	STOPPED DELAY	LOS	APPROACH DELAY	SEGMENT RUNNING TIME	SUM TIME (SEC)	AVERAGE TRAVEL SPEED (MPH)	ARTERIAL LOS
60	0.50	600	1,600	0.38	7.1	0.53	3.8	A	4.9	14.5	19.4	18.6	C
60	0.50	700	1,600	0.44	7.4	0.53	3.9	A	5.1	14.5	19.6	18.4	C
60	0.50	800	1,600	0.50	7.8	0.53	4.1	A	5.4	14.5	19.9	18.1	C
60	0.50	900	1,600	0.56	8.3	0.53	4.4	A	5.7	14.5	20.2	17.8	D
60	0.50	1,000	1,600	0.63	8.8	0.55	4.9	A	6.3	14.5	20.8	17.3	D
60	0.50	1,100	1,600	0.69	9.6	0.59	5.6	B	7.3	14.5	21.8	16.5	D
60	0.50	1,200	1,600	0.75	10.5	0.64	6.7	B	8.8	14.5	23.3	15.5	D
60	0.50	1,300	1,600	0.81	12.0	0.68	8.1	B	10.6	14.5	25.1	14.4	D
60	0.50	1,400	1,600	0.88	14.3	0.73	10.4	B	13.5	14.5	28.0	12.8	E
60	0.50	1,500	1,600	0.94	18.8	0.78	14.6	B	19.0	14.5	33.5	10.7	E
60	0.50	1,600	1,600	1.00	28.7	0.82	23.5	C	30.6	14.5	45.1	8.0	F

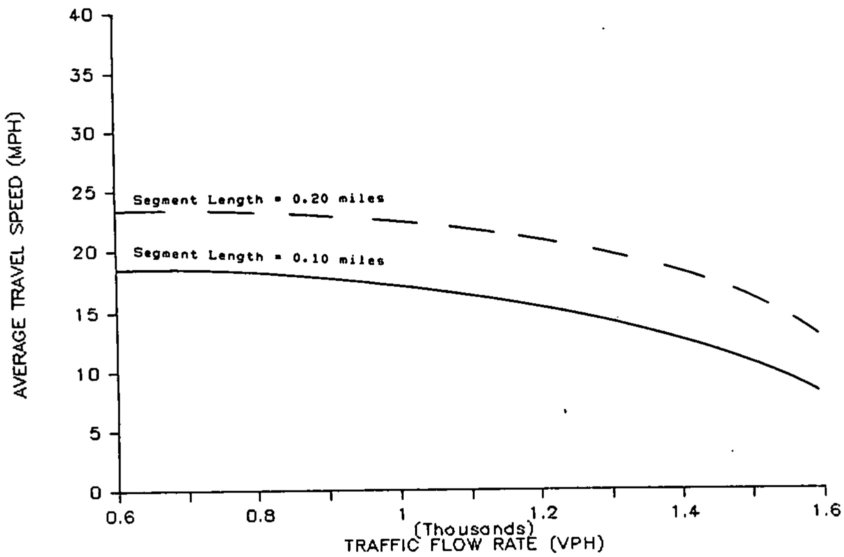


Figure 11-19. Sample calculation 5 speed as a function of arterial flow rate on two different segment lengths.

However, any intersection with less than 5.0 sec of stopped delay is operating rather well (i.e., intersection LOS A is a realistic statement for such an intersection).

Table 11-9 illustrates this point: because of the close signal spacing and the total delay per unit length, it is possible for the arterial level of service to be two or even three levels worse than a typical intersection. (As shown in sample calculation 7, it is also possible for the arterial LOS to be better than the intersection LOS, when the segment is very long).

Note that in this computation, the intersection delay ranges from 25 percent to 68 percent of the total time spent on the segment, depending on the traffic flow rate. In sample calculation 4, the range was 16 percent to 54 percent.

CALCULATION 6—EVALUATION BASED ON FIELD DATA

1. *Description*—On a given multilane two-way divided arterial with left-turn bays and good access control, the free flow speed is measured along its length as 45.0 mph. The following data are collected along its eight eastbound segments, using the field data procedures of Appendix I:

Segment	Length (mi)	Average Travel Time (sec)	Average Stopped Delay (sec/veh)
1	0.20	28.3	3.4
2	0.15	19.2	1.7
3	0.15	21.8	3.6
4	0.20	29.4	5.3
5	0.25	49.7	17.6
6	0.25	40.6	10.5
7	0.25	35.2	6.2
8	0.20	28.1	3.2

These data are based on an appropriate number of travel time runs that include both the running time and the intersection stopped delay.

Find the arterial level of service, by segment and for the entire facility, and the intersection levels of service.

2. *Solution*—To determine the arterial class, consult Tables 11-2 and 11-3 and note that

- The facility is multilane divided.
- Access control is good.
- There are 8 signals in 1.65 mi, or about 5 signals per mile.

It is likely that the design category is "suburban," based on Table 11-2. Given that it is a principal arterial, Table 11-3 leads one to the determination of arterial Class I.

As reflected in the range of free flow speeds in Table 11-4, a *measured* free flow speed of 45.0 mph clearly indicates an arterial Class I. Thus, if there were any uncertainty in the classification, the field information has settled it.

The field data can also be used to compute the arterial speed by segment and for the entire facility. There is no need to use Table 11-4.

The computations for the arterial speed are shown on the completed arterial level of service worksheet for summary of calculations in Figure 11-20. The speed calculations are straightforward, based on

$$\text{ART SPD} = \frac{3{,}600 \text{ (Segment Length)}}{\text{(Segment Travel Time)}}$$

For instance, for segment 1, ART SPD $= 3{,}600 \times 0.20/28.3 = 25.4$ mph.

The level-of-service determination is done by referring to Table 11-1 for arterial Class I and simply applying the definitions. For instance, segment 1 with a computed speed of 25.4 mph is level-of-service B.

Figure 11-21 shows the speed profile of the arterial, and graphically demonstrates where the problem occurs on the arterial. Note that the overall level of service does not highlight the problem as well as the speed profile or the set of segment levels of service.

COMPUTATION OF ARTERIAL LEVEL OF SERVICE WORKSHEET

Arterial: *Sample Calculation 6* ________-bound

File or Case #: - Date: -

Prepared by: ________

$$\text{ART SPD} = \frac{3600\ (\text{Sum of Length})}{\text{Sum of Time}}$$

Table 11-1

Segment	Length (mi)	Arterial Class	Free Flow (mph)	Section	Running Time[a] (sec)	Intersec. Approach Delay[b]	Other Delay (sec)	Sum of Time by Section	Sum of Length by Section	Arterial SPD[c] (mph)	Arterial LOS by Section
1	0.20	I	45					28.3	0.20	25.4	C
2	0.15	I	45					19.2	0.15	28.1	B
3	0.15	I	45					21.8	0.15	24.8	C
4	0.20	I	45					29.4	0.20	24.5	C
5	0.25	I	45					49.7	0.25	18.1	D
6	0.25	I	45					40.6	0.25	22.2	C
7	0.25	I	45					35.2	0.25	25.6	C
8	0.20	I	45					28.1	0.20	25.6	C
9											
10											
11											
12											
13											
14											
15											

Class I, based upon 42 mph free flow speed

Grand Sum of Time = $\boxed{252.3}$

Grand Sum of Length = $\boxed{1.65}$ $\boxed{C}$

$$\frac{3{,}600 \times \boxed{1.65}}{\boxed{252.3}} = \boxed{23.5}\ \text{MPH}$$

[a]Use Table 11-4 and multiply by segment length
[b]From summary on Arterial Summary of Intersection Delay Estimates Worksheet
[c]See upper right corner of this Table for the Equation
Note: Round delay estimates to one place after decimal

Figure 11-20. Solution to sample calculation 6 worksheet: computation of arterial level of service.

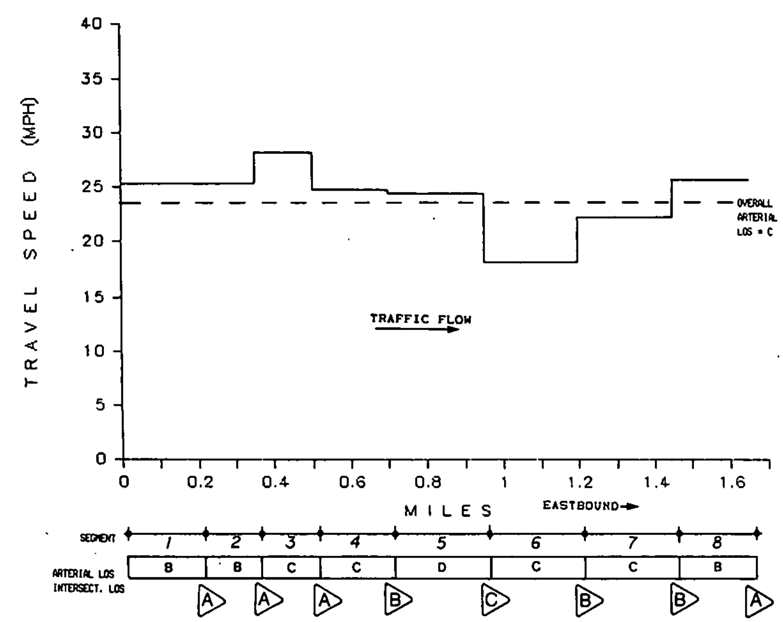


Figure 11-21. Speed profile for sample calculation 6.

The field data also allow a direct determination of intersection level of service, based on measured stopped delay. Referring to the LOS definitions of Table 11-7, the determination is straightforward:

Segment	Intersection LOS	Based on Measured Stop Delay (sec/veh)
1	A	3.4
2	A	1.7
3	A	3.6
4	B	5.3
5	C	17.6
6	B	10.5
7	B	6.2
8	A	3.2

These are also shown in Figure 11-21.

CALCULATION 7—ARTERIAL WITH LARGE SIGNAL SPACINGS

1. *Description*—Route 25 is a suburban arterial with a free flow speed of 51 mph, based on field studies. It is an undivided facility, with two lanes in each direction, has left-turn bays, and is dominated by its signals. A pretimed set of signals is used on the portion of the facility that is of interest. The following information is available for the westbound traffic, for the period of interest:

Segment	Length (mi)	C (sec)	g/C	X	c (vph)
1	0.7	70	0.60	0.89	1,800
2	0.6	70	0.57	0.97	1,710
3	0.7	70	0.60	0.94	1,800
4	0.7	70	0.60	0.94	1,800
5	0.7	70	0.60	0.94	1,800

The signal progression is good, with less than 10 percent of the through-traffic stopping.

Determine the arterial level of service, by segment and for the entire facility.

2. *Solution*—Based on the free flow speed, the facility is arterial Class I. Refer to Table 11-1 or Table 11-7.

The intersection delay may be computed using Eqs. 11-2 and 11-3, with the computations summarized on the worksheet for arterial summary of intersection delay estimates, as shown in Figure 11-22.

ARTERIAL SUMMARY OF INTERSECTION DELAY ESTIMATES

Arterial: Route 25 — West -bound

File or Case No.: Sample Calculation 7 — Date: 02/18/84

Prepared by:

Segment	Cycle Length C	g/C	v/c Ratio X	Lane Group Capacity c	Random Arrival Delay[a] (sec)	Arrival Type	Progression Factor PF[b]	Estimated Stopped Delay[c]	Intersec. LOS	Estimated Approach Delay[d]
1	70	0.60	0.89	1800	13.4	5	0.74	9.9	B	12.9
2	70	0.57	0.97	1710	21.6	5	0.79	17.1	C	22.2
3	70	0.60	0.94	1800	17.7	5	0.78	13.8	B	17.9
4	70	0.60	0.94	1800	17.7	5	0.78	13.8	B	17.9
5	70	0.60	0.94	1800	17.7	5	0.78	13.8	B	17.9
6										
7										
8										
9										
10										
11										
12										
13										
14										
15										

[a] Equation 11-3
[b] Table 11-6
[c] Multiply (Random Arrival Delay) times (Progression Factor PF)
[d] Multiply Stopped Delay by 1.3 as in Equation 11-2

NOTES:

1. Adjusted demand flow rate v may be computed from Equation 11-4: v = (V/PHF) X U.
2. If lane group capacity c is not known, it may be computed from Chapter 9 or estimated from the default Equation 11-5: c = 1,600 × N × (g/C). This is highly approximate.
3. Round delay estimates to one place after the decimal.

Figure 11-22. Sample calculation 7—solution—using arterial summary of intersection delay estimates worksheet.

Based on the progression factor descriptions in this chapter, the judgment is made that the arrival type is Type 5:—a dense platoon arriving at the beginning of the green phase. It is the most favorable platoon condition. This judgment is based on the given condition that the signal progression is good, with less than 10 percent of the through-traffic stopping.

Referring to Table 11-6 for pretimed control and arrival Type 5, note

v/c Ratio, X	Progression Factor, PF
≤ 0.60	0.53
0.80	0.67
1.00	0.82

From these values, progression factors may be interpolated:

0.80	0.67
0.89	0.74
0.94	0.78
0.97	0.79
1.00	0.82

These values are shown in the computations of Figure 11-22.

Given the free flow speed of 51 mph and the fact that this is well outside the range of Table 11-4 or Table 11-5, the 51 mph is used as the arterial speed in computing the running time:

(Segment Running Time)
$$= 3{,}600 \times (\text{Segment Length})/(\text{ART SPD})$$

For instance, in segment 1,

(Segment Running Time)
$$= 3{,}600 \times (0.70)/(51 \text{ mph}) = 49.4 \text{ sec}$$

To this computed running time is added the intersection delay time in the usual way, as illustrated in Figure 11-23.

If Table 11-4 is inspected very carefully, a "more precise"

COMPUTATION OF ARTERIAL LEVEL OF SERVICE WORKSHEET

Arterial: Route 25 West-bound

File or Case #: Sample Calculation 7 Date: 02/18/84

Prepared by: -

$$\text{ART SPD} = \frac{3600\ (\text{Sum of Length})}{\text{Sum of Time}}$$

Seg-ment	Length (mi)	Arterial Class	Free Flow (mph)	Section	Running Time[a] (sec)	Intersec. Approach Delay[b]	Other Delay (sec)	Sum of Time by Section	Sum of Length by Section	Arterial SPD[c] (mph)	Arterial LOS by Section
1	0.70	I	51	1	49.4	12.9	-	62.3	0.70	40.4	A
2	0.60	I	51	2	42.3	22.2	-	64.5	0.60	33.5	B
3	0.70	I	51	3	49.4	17.9	-	67.3	0.70	37.4	A
4	0.70	I	51	4	49.4	17.9	-	67.3	0.70	37.4	A
5	0.70	I	51	5	49.4	17.9	-	67.3	0.70	37.4	A
6											
7											
8											
9											
10											
11											
12											
13											
14											
15											

Class I based upon 51 mph free flow.

[a] Use Table 11-4 and multiply by segment length

[b] From summary on Arterial Summary of Intersection Delay Estimates Worksheet

[c] See upper right corner of this Table for the Equation

Note: Round delay estimates to one place after decimal

Grand Sum of Time = $\boxed{328.7}$

Grand Sum of Length = $\boxed{3.40}$ $\boxed{A}$

$$\frac{3{,}600 \times \boxed{3.40}}{\boxed{328.7}} = \boxed{37.2}\ \text{MPH}$$

Figure 11-23. Sample calculation 7—solution—computation of arterial level of service worksheet.

estimate of the computed running time can be generated: for instance, for a segment length of 0.50 mi, the segment running time is (88/80) = 1.10 higher than the value for a 1.0-mi segment, which is based on the free flow speed. Thus, more precise estimates for such segment lengths as 0.50, 0.60, and 0.70 mi could be generated for a free flow speed of 51 mph by similar logic. However, the better and more accurate approach would be to rely on field data for such an arterial.

Figure 11-23 also contains the level of service for each arterial segment, based on the fact that the arterial is Class I, and referring to the level of service boundaries in Table 11-1.

Figure 11-24 contains the speed profile for the arterial, and also indicates the arterial and intersection levels of service, based on the average travel speed and stopped delay values respectively.

Note that for large signal spacings on such an arterial, one can expect that the intersections will provide poorer levels of service to the driver than the arterial, based simply on the definitions of LOS: on even a Class I arterial, LOS A can be achieved with a speed of 35 mph or greater. However, more than 5.0 sec of stopped delay per vehicle removes an *intersection* from LOS A (refer to Table 11-7).

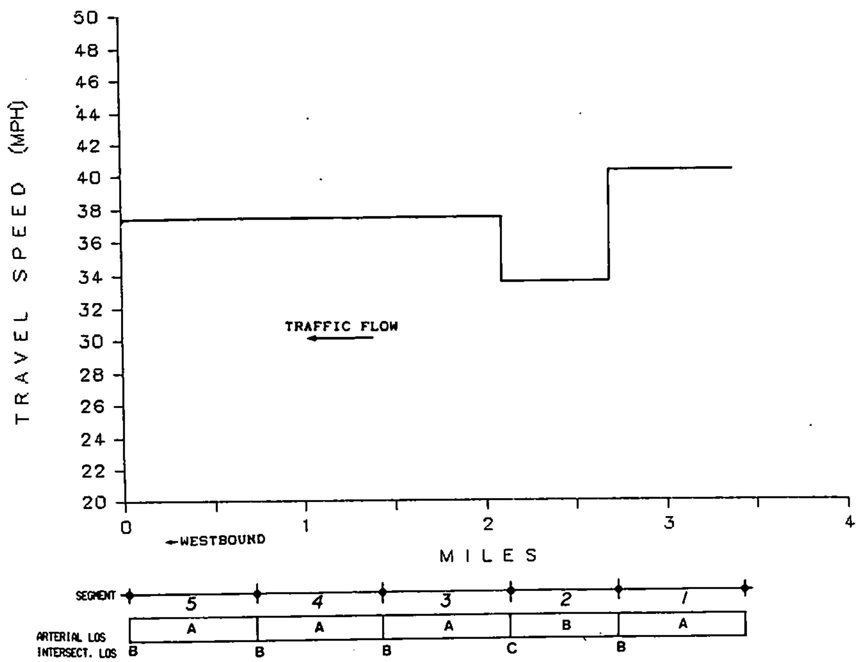


Figure 11-24. Speed profile for sample calculation 7.

APPENDIX I

TEST-CAR METHOD FOR EXISTING ARTERIALS

The following steps are used when applying the test-car method for determining levels of service for existing urban and suburban arterials.

1. Identify and inventory the geometrics and the access control of each arterial segment, the segment lengths, and existing signal timing, and the 15-min flow rates for selected times of the day (such as the peak AM period, the peak PM period, and a representative off-peak period, by direction of flow).

2. Determine the appropriate free flow speed for the arterial section being evaluated. For existing arterials, this may be done by making runs with a test car equipped with a calibrated speedometer at times of low volumes. An observer should read the speedometer at midblock locations where the vehicle is not impeded by other vehicles. Record readings for each segment in an arterial. These observations may be supplemented by spot speed studies made at typical midblock locations during low-volume conditions. Other data, such as design type, access points, roadside development, and speed limit can be considered also.

3. Use Tables 11-2 and 11-3, along with the physical information and free flow speed cited above, to determine the arterial class.

4. Make test-car travel-time runs over the arterial section during the selected times of the day.

   a. The observer should use appropriate measurement equipment to obtain the information in Table I.11-1 the "Travel Time Field Worksheet." That equipment may be a computer-based collection system or a pair of stopwatches.
   b. Record travel times between centers of signalized intersections, and the location, cause and duration of each stop.
   c. In starting test-car runs, begin the runs at different time points in the signal cycle, so as to avoid having all trips be a "first in platoon" placement.
   d. Also record some midblock speedometer readings as a check on unimpeded travel speeds, and see how they relate to free flow speed.
   e. Summarize data to provide for each segment and each time period, the average travel time, the average stopped time for the signal, other stops and events (such as four-way stops, parking disruptions, etc).
   f. The minimum number of test-car runs will depend on the variance in the data and the accuracy desired. Six to twelve runs for each traffic-volume condition may be adequate. (See HRB Proc. 1952, pp. 864–866.)
   g. An instrumented test-car should be used if available, to reduce labor requirements and to facilitate recording and analysis. Computer-produced summaries of test-car runs are now common, with all data recorded and analyzed by computer.

5. Determine the average travel speed for each segment for each time period, utilizing travel times and segment lengths. Also determine average travel speed for the arterial section.

6. Use Table 11-1 to obtain a level-of-service value for each arterial segment and for the overall arterial, for each time period and direction of flow. This is done by comparing the average travel speed obtained in Step 5 above with the speed values given in Table 11-1 for the appropriate arterial class.

7. The test-car data can be modified to permit evaluation of different signal timing plans. As shown in Table 11-6, adjustment factors can be applied to stopped delays to evaluate effects on stopped delay of changes in offsets. It then is possible to evaluate effects of these changes on average travel speeds and levels of service.

TABLE I.11-1

TRAVEL TIME FIELD WORKSHEET

Arterial________ Date________

Driver________ Recorder________ Direction________

SIGNAL LOCATION	DIST (mi)	Run No.___ Time___ CUM TT (sec)	STOP TIME (sec)	Run No.___ Time___ CUM TT (sec)	STOP TIME (sec)	Run No.___ Time___ CUM TT (sec)	STOP TIME (sec)

S—Signal (lower box)
LT—Left Turn (upper box)
P—Pedestrian (upper box)
PK—Parking (upper box)
4W—4-Way Stop (upper box)

APPENDIX II

WORKSHEETS FOR USE IN ANALYSIS

WORKSHEETS *PAGE*

ARTERIAL SUMMARY OF INTERSECTION DELAY ESTIMATES

Arterial: ______________________ ______________________ -bound

File or Case No.: ______________________ Date: ______________________

Prepared by: ______________________

Segment	Cycle Length C	g/C	v/c Ratio X	Lane Group Capacity c	Random Arrival Delay[a] (sec)	Arrival Type	Progression Factor PF[b]	Estimated Stopped Delay[c]	Intersec. LOS	Estimated Approach Delay[d]
1										
2										
3										
4										
5										
6										
7										
8										
9										
10										
11										
12										
13										
14										
15										

[a] Equation 11-3
[b] Table 11-6
[c] Multiply (Random Arrival Delay) times (Progression Factor PF)
[d] Multiply Stopped Delay by 1.3 as in Equation 11-2

NOTES:

1. Adjusted demand flow rate v may be computed from Equation 11-4: $v = (V/PHF) \times U$.
2. If lane group capacity c is not known, it may be computed from Chapter 9 or estimated from the default Equation 11-5: $c = 1{,}600 \times N \times (g/C)$. This is highly approximate.
3. Round delay estimates to one place after the decimal.

COMPUTATION OF ARTERIAL LEVEL OF SERVICE WORKSHEET

Arterial: ________________ ________-bound

File or Case #: ________________ Date: ________

Prepared by: ________________

$$\text{ART SPD} = \frac{3600\ (\text{Sum of Length})}{\text{Sum of Time}}$$

Segment	Length (mi)	Arterial Class	Free Flow (mph)	Section	Running Time[a] (sec)	Intersec. Approach Delay[b]	Other Delay (sec)	Sum of Time by Section	Sum of Length by Section	Arterial SPD[c] (mph)	Arterial LOS by Section
1											
2											
3											
4											
5											
6											
7											
8											
9											
10											
11											
12											
13											
14											
15											

Grand Sum of Time = $\boxed{}$

Grand Sum of Length = $\boxed{}$ $\boxed{}$

$$\frac{3{,}600 \times \boxed{}}{\boxed{}} = \boxed{}\ \text{MPH}$$

[a] Use Table 11-4 and multiply by segment length
[b] From summary on Arterial Summary of Intersection Delay Estimates Worksheet
[c] See upper right corner of this Table for the Equation

Note: Round delay estimates to one place after decimal

TRAVEL TIME FIELD WORKSHEET

Arterial____________________ Date______________

Driver______________ Recorder______________ Direction______________

SIGNAL LOCATION	DIST (mi)	Run No.____ Time CUM TT (sec)	STOP TIME (sec)	Run No.____ Time CUM TT (sec)	STOP TIME (sec)	Run No.____ Time CUM TT (sec)	STOP TIME (sec)

S—Signal (lower box)
LT—Left Turn (upper box)
P—Pedestrian (upper box)
PK—Parking (upper box)
4W—4-Way Stop (upper box)

CHAPTER 12

TRANSIT CAPACITY

CONTENTS

I. INTRODUCTION

This chapter contains guidelines and procedures for estimating bus and rail transit capacities. It defines basic capacity concepts and principles; summarizes previous studies and current experience; develops analytical relationships; presents capacity guidelines; and sets forth illustrative applications.

The guidelines and procedures may be used to estimate:

1. The effects of bus flows on freeway and signalized intersection capacity.
2. Total passenger or person flow based on roadway operating conditions, and the prevailing mix of cars and buses (or rail vehicles).
3. Generalized ranges of bus capacities for arterial streets, downtown streets, and busways.
4. Bus berth (stop) requirements at terminals and along downtown busways, bus-only streets, and other city streets.
5. Passenger flows on rail transit lines for varying car sizes, train lengths, service frequencies, and loading conditions—for both light rail and rapid transit lines.

The chapter also provides ways to address various questions normally encountered in transit service planning and operations. For instance:

1. How many passengers can be carried per unit of time?
2. How many transit vehicles are needed to provide a specific rate of passenger flow?
3. How many passengers can be carried with a given vehicle fleet?

It emphasizes bus transit capacities because buses operate over the street and highway systems in most urban areas. However, it also presents salient rail transit characteristics and capacities. It builds upon and extends previous transit capacity analysis.

The initial research on transit capacity relative to streets and highways was developed in 1961 by the Transit Subcommittee of the Committee on Highway Capacity and Quality of Service. It summarized operating experience in the United States and contained broad guidelines for passenger dwell times and vehicle occupanices.

More detailed analyses of bus capacity were contained in NCHRP Reports 143 and 155 on Bus Use of Highways (*2,4*). These reports summarized, synthesized, and interpreted available information on bus flows, passengers, and service times. They also analyzed bus berth capacity. The findings on bus transit capacity were summarized in "Bus Capacity Analysis," Transportation Research Record 546 (*5*).

Rail transit capacity has a long history of actual operating experience and analysis. The Board of Supervising Engineers for Chicago Traction, for example, analyzed street railway capacity in 1912, and passenger dwell times by door width in 1916. Lang and Soberman derived formulas for rapid transit track capacity in 1964 (*40*). More recent studies by Homberger, Pushkarev, and Vuchic further addressed rail transit capacity theory and practice (*7,8,9*). Relevant materials from the more recent references are incorporated into this chapter.

CONTEXT

Transport system management solutions to urban transport problems have increased the interest in the *person-capacity* characteristics of transportation facilities in addition to their *vehicle-capacity* characteristics. The underlying rationale is that although buses and rail transit cars require more street space per vehicle than private automobiles, they carry many more passengers per vehicle than automobiles, especially during peak hours. Thus, public transportation emerges as an important way to increase the number of people carried by urban transportation systems.

Transit vehicles carry a substantial number and proportion of peak-hour person trips to and from the downtown areas, and along many urban freeways, arterials, and downtown streets.

Table 12-1 indicates the peak period use of public transport, bus and rail combined, by persons entering the central business districts of selected cities in Canada and United States (*1*). Transit carries more than two-thirds of all peak-hour travelers to or from the New York, Chicago, Philadelphia, and Toronto downtown areas, and more than a third of all peak-hour travelers entering or leaving most other CBD's. The variations in transit use reflect differences in population, central business district employment, extent of bus and rail transit services, and geographic characteristics.

Buses carry over 85 percent of all peak-hour person-trips through the Lincoln Tunnel in the City of New York, account for about half of all peak-hour travelers on the Shirley Highway

(I-95), Virginia, and the Long Island and Gowanus Expressways (New York City), and for more than a quarter of all passengers on radial freeways approaching or leaving other large-city CBD's.

Buses carry an even higher proportion of peak-hour travelers on many city streets. More than 80 percent of all peak-hour passengers on Hillside Avenue and Madison Avenue in New York City, Market Street in Philadelphia, and Main Street in Dallas are carried by buses. Buses accommodate more than half of all peak-hour person-trips on downtown streets in many other cities(*2*).

These observations do not necessarily represent maximum possible bus volumes or total traffic volumes. They do, however, clearly indicate that while buses account for a relatively small proportion of the vehicles in a traffic stream, they carry a sizable part of the total person flow.

Rail transit, operating mainly off-street, becomes important in serving large, intensively developed city centers where it accounts for more than half of all people entering or leaving in the peak hour.

TABLE 12-1. PEAK-HOUR USE OF PUBLIC TRANSPORT BY PERSONS ENTERING OR LEAVING THE CENTRAL BUSINESS DISTRICT

URBAN AREA	YEAR	PERCENT BY PUBLIC TRANSPORT IN PEAK DIRECTION
New York, New York	1982	89[a]
Chicago, Illinois	1974	82[a]
Toronto, Ontario	1970	68[a]
Boston, Massachusetts	1974	49[a]
Cleveland, Ohio	1970	44[a]
Ottawa, Ontario	1974	40
Vancouver, British Columbia	1970	40
Los Angeles, California	1974	37
Washington, D.C.	1979	36[b]
Detroit, Michigan	1974	35
Baltimore, Maryland	1982	33
Denver, Colorado	1977	30
Dallas, Texas	1971	28
Milwaukee, Wisconsin	1974	25
Providence, Rhode Island	1977	21
New Haven, Connecticut	1982	20
Minneapolis, Minnesota	1965	20
Houston, Texas	1971	14

[a] With rail transit.
[b] Includes Pentagon area; data for 6.30–9.30 AM
SOURCE: Cordon Counts for each city, mainly compiled in Ref. *1*.

CONCEPTS

Transit capacity is more complex and less precise than highway capacity: it deals with the movement of *both* people and vehicles; depends on the size of the transit vehicles and how often they operate; and reflects the interaction between passenger traffic concentrations and vehicle flow. It depends on the operating policy of the transit agency, which normally specifies service frequencies and allowable passenger loadings. Accordingly the traditional concepts applied to highway capacity must be adapted and broadened.

Table 12-2 defines the important terms that relate to transit capacity.

TABLE 12-2. IMPORTANT TERMS IN TRANSIT CAPACITY

- *Clearance Time*—All time losses at a stop other than passenger dwell times, in seconds. It can be viewed as the minimum time, in seconds, between one transit vehicle leaving a stop and the following vehicle entering, i.e., the clearance time between successive buses should not be less than 15 sec.
- *Crush Capacity*—The maximum number of passengers that can be physically accommodated on a transit vehicle. It is also defined as level-of-service F. It can be viewed as an "offered" capacity, since it cannot be achieved on all vehicles for any sustained period of time.
- *Dwell Time*—The time, in seconds, that a transit vehicle is stopped for the purpose of serving passengers. It includes the total passenger service time plus the time needed to open and close doors.
- *Interrupted Flow*—Transit vehicles moving along a roadway or track and having to make service stops at regular intervals.
- *Maximum Load Point*—The point, actually section, along a transit route at which the greatest number of passengers is being carried.
- *Passenger Service Time*—The time, in seconds, that is required for a passenger to board or alight from a transit vehicle.
- *Person-Capacity*—The maximum number of persons that can be carried past a given location during a given time period under specified operating conditions without unreasonable delay, hazard, or restriction. Usually measured in terms of persons per hour.
- *Person Level of Service*—The quality of service offered the passenger within a transit vehicle, as determined by the available space per passenger.
- *Productive Capacity*—A measure of efficiency or performance. The product of passenger capacity along a transit line and speed.
- *Seat Capacity*—The number of passenger seats on a transit vehicle.
- *Standees*—The number of standing passengers on a transit vehicle. The ratio of total passengers carried to the number of seats during a specified time period is called the *load factor*. The *percent standees* represents the number of standing passengers expressed as a percentage of the number of seats. A transit vehicle with 40 seats and 60 passengers has a load factor of 1.5 and 50 percent standees.
- *Uninterrupted Flow*—Transit vehicles moving along a roadway or track without stopping. This term is most applicable to transit service on freeways or on its own right-of-way.

Person Movement

Each roadway or transit facility should be analyzed in terms of the number of people it carries in a specific time period. This calls for knowing both the number and occupancies of each type of vehicle.

For example, an urban freeway lane carrying 1,800 passenger cars per lane per hour with an average occupancy of 1.5 persons would have a person movement of 2,700 people per hour. Likewise, an arterial street carrying 600 automobiles per hour and 50 buses per hour, with occupancies of 1.5 and 40, respectively, would have a total person movement of 2,900 persons per hour of which approximately 70 percent would be carried by public transport.

Person-Capacity

The person-capacity or passenger-carrying capability for any given transport route can be defined as "the maximum number of *people* that can be carried past a given location during a given time period under specified operating conditions without unreasonable delay, hazard, or restriction, and with reasonable certainty."

This definition is less absolute than definitions for vehicle-capacity, because it recognizes that when dealing with transit, additional considerations enter the picture. More specifically, person-capacity depends on the mix in the traffic stream, including the number and occupancy of each type of vehicle that can reasonably be expected to pass a point on a roadway. It is a function of vehicle size, type, occupancy, and headway.

The number of transit vehicles should be based on a specified flow. The number of cars should reflect the auto capacity of the facility after deducting the passenger car equivalents of the buses. The total person-capacity then represents the number of people that can be carried by the specified number of buses and the remaining passenger car capacity.

The person-capacity of a freeway lane with bus and car traffic under prevailing conditions of flow can be estimated as follows:

$$c_p = f' O_1 + [(1{,}800 - 1.5 f') O_2] \qquad (12\text{-}1)$$

where:

f' = number of buses per hour;
O_1 = bus occupancy;
O_2 = car occupancy; and
c_p = person-capacity, people per hour.

The number of persons that can be carried in buses depends on the number of buses scheduled. This may be below the maximum capacity of a street to accommodate buses. It is certainly the case for most urban freeways, as illustrated by the following example.

Figure 12-1 shows the person-capacities for an urban freeway lane, with various numbers of buses in the traffic stream. This example assumes a maximum freeway capacity of 1,800 vph without buses, a bus-passenger car equivalency of 1.5, and occupancies of 1.5 and 50 for cars and buses respectively. As the number of buses on the freeway increase to 300, the total person-capacity increases from 2,700 to nearly 17,000, while the vehicle-capacity drops from 1,800 to 1,620. (Note that this figure only refers to capacity, not demand or actual use.) If each car carried five passengers, then with 1,320 cars and 300 buses, the total person-capacity would be 21,600.

A slightly different approach should be used for downtown streets. The person-capacity of the bus or street car lane (assuming only transit use) can be estimated by the procedures outlined in this chapter (Sec. II and III). The vehicular capacities of the general traffic lanes can be estimated based on the procedures outlined in Chapter 9 and weighted by their passenger occupancies. The total person-capacity equals the sum, and may be higher than figures based entirely on actual usage. Note that this approach is different from that for freeways, where it is usually unrealistic to preempt an entire lane for buses; however it could be applied where dedicated freeway bus lanes are considered, by taking into account the limits on bus capacity resulting from approaches to and from the freeway, as well as stops along it.

Basic Factors and Equations

The passenger capacity of a transit line is the product of the number of vehicles per hour (usually past the busiest stop) and the number of passengers that each vehicle can carry. Four basic factors determine the maximum passenger capacity:

1. The maximum number of vehicles per transit unit (bus, car, train).
2. The passenger capacity of the individual transit vehicles.
3. The minimum possible headway or time spacing between individual vehicles or trains.
4. The number of movement channels or loading positions.

The many variables that influence these factors and transit capacities are given in Table 12-3. Some affect the number of passengers per unit, while others affect the number of units that can pass a given location within a specified time period.

The capacity of a transit line varies along the route. Limitations may occur (1) between stops (i.e., way capacity) (2) at stops or stations (i.e., station capacity), (3) at major intersections with cross traffic, or (4) at terminals (station capacity). *In most cases station capacity rather than way capacity* is the critical constraint.

Capacities are generally governed by the critical stops where major passenger boarding or alighting takes place, or where vehicles terminate or turn around. This is similar to estimating arterial street system capacity based on critical intersections along a route. Sometimes, however, outlying rail transit terminals limit system capacity due to heavy passenger boardings, and track configurations or operating practices that limit train turnarounds.

The actual mix of automobiles and transit vehicles in a traffic stream results from the choice of travel mode by the traveler and from the number of transit vehicles scheduled over the facility. The number of persons that can be carried by a given bus or rail line, therefore, reflects the operating policy of the transit property with respect to minimum service frequency and passenger loading conditions (i.e., number of standees).

The following considerations are important:

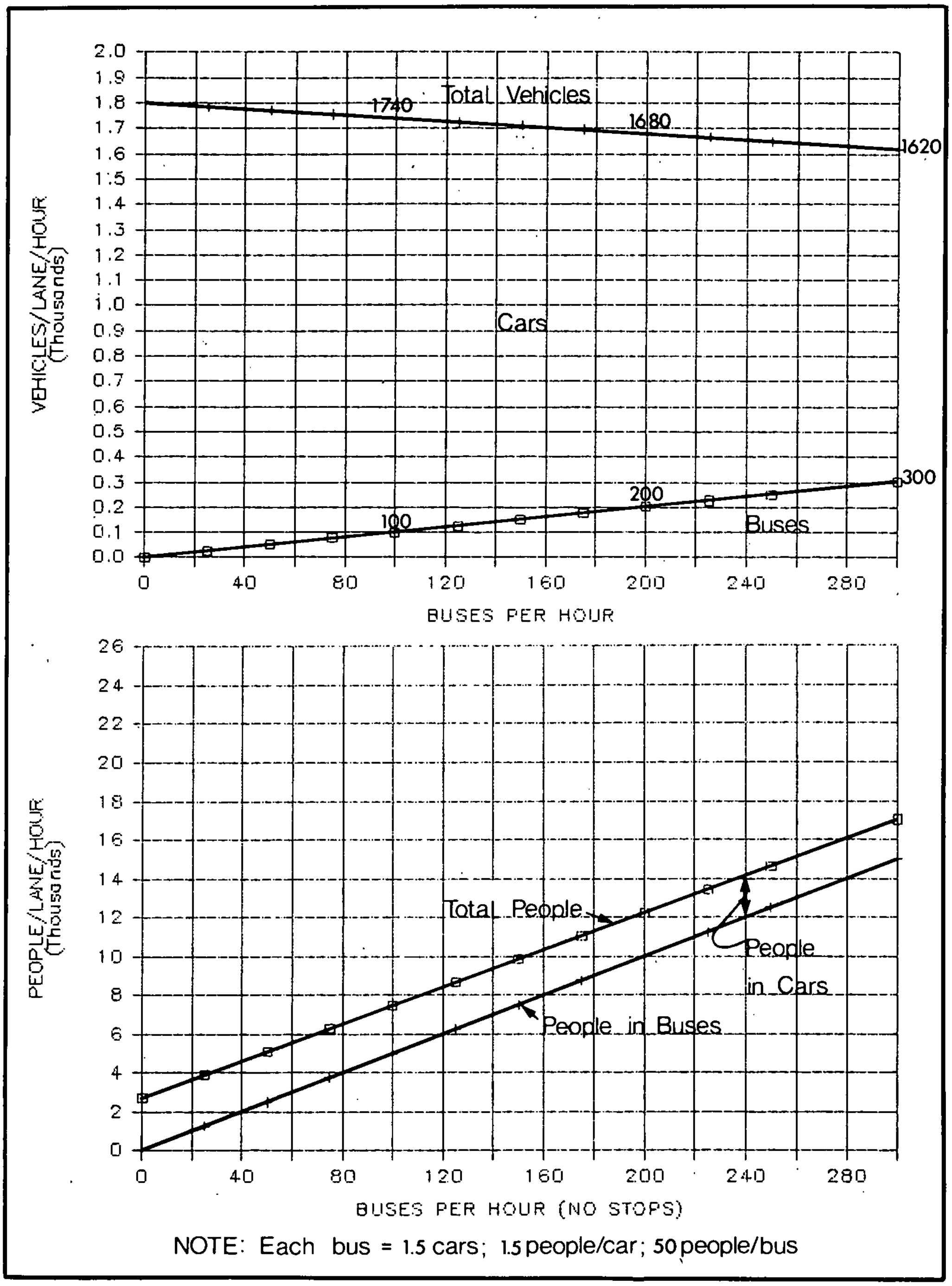


Figure 12-1. Example of freeway person-capacity.

1. A transit line with a relatively uniform distribution of boarding passengers among stops will usually have a higher capacity than one where passenger boarding is concentrated at a single stop.

2. Short-term fluctuations in ridership demand must be considered to avoid unacceptable passenger queuing or overcrowding. Variations in arrival patterns and dwell times at stops will tend to reduce capacity.

3. The *maximum* rate of passenger flow is usually constrained by such factors as acceptable levels of passenger comfort, the presence of other traffic sharing the same right-of-way, and safety considerations. Therefore, transit operators generally are more concerned with the realistic rates of flow that can be achieved by different modes, rather than with physical capacity in the theoretical engineering sense.

4. Operations at "capacity" tend to strain transit systems,

TABLE 12-3. FACTORS THAT INFLUENCE TRANSIT CAPACITY

1.	*Vehicle Characteristics* • Allowable number of vehicles per transit unit (i.e., single unit bus, or several units-cars per train) • Vehicle dimensions • Seating configuration and capacity • Number, location, width of doors • Number and height of steps • Maximum speed • Acceleration and deceleration rates • Type of door actuation control
2.	*Right-of-Way Characteristics* • Cross-section design (i.e., number of lanes or tracks) • Degree of separation from other traffic • Intersection design (at grade or grade separated, type of traffic controls) • Horizontal and vertical alignment
3.	*Stop Characteristics* • Spacing (frequency) and duration • Design (on-line or off-line) • Platform height (high level or low level loading) • Number and length of loading positions • Method of fare collection (prepayments, pay when entering vehicle; pay when leaving vehicle) • Type of fare (single-coin, penny, exact) • Common or separate areas for passenger boarding and alighting • Passenger accessibility to stops
4.	*Operating Characteristics* • Intercity versus suburban operations at terminals • Layover and schedule adjustment practices • Time losses to obtain clock headways or provide driver relief • Regularity of arrivals at a given stop
5.	*Passenger Traffic Characteristics* • Passenger concentrations and distribution at major stops • Peaking of ridership (i.e., peak-hour factor)
6.	*Street Traffic Characteristics* • Volume and nature of other traffic (on shared right-of-way) • Cross traffic at intersections if at grade
7.	*Method of Headway Control* • Automatic or by driver/trainman • Policy spacing between vehicles

SOURCE: Adapted from *Canadian Transit Handbook* (Ref. *12*)

and do not represent desirable operating conditions. Moreover, most U.S. transit systems operate at capacity for a relatively short period of time, if at all.

5. Capacity relates closely to system performance and service quality in terms of speed, comfort, and service reliability. A single fixed number often can be misleading. The concept of "productive capacity," the product of passenger flow and speed, provides an important index of system efficiency (*9*).

6. Capacities obtained by analytical methods must be cross-checked against actual operating experience for reasonableness.

The capacity of a transit line can be estimated from the following equations:

$$c_v = \frac{3{,}600\ R}{h} = \frac{3{,}600\ R}{D + t_c} \tag{12-2a}$$

$$c_p = nSc_v = \frac{3{,}600\ nSR}{D + t_c} \tag{12-2b}$$

where:

c_v = vehicles per hour per channel or berth (maximum);
c_p = people per hour per channel or berth (maximum);
h = headway between successive units, in sec;
t_c = clearance between successive vehicles, in sec;
D = dwell time at major stop under consideration, in sec;
S = passengers per vehicle;
n = vehicles per unit ($n = 1$ for buses; $n = 1$ to $n = 11$ for rail vehicles); and
R = reductive factor to compensate for dwell time and arrival variations.

The factor R reduces the capacity to account for variations in bus arrival patterns and in dwell times at stop. It may approach 1.00 for a rail transit system on private right-of-way with wayside cab signal control or with automatic train operations. For bus operations, especially on city streets, it is always less than 1—a value of 0.833 is suggested for maximum capacity. Using this factor, the term (3,600 R) in Eqs. 12-2a and 12-2b becomes 3,000 for maximum capacity. In effect, it increases headways by 20 percent.

These equations, with further adjustments for the reductive effects of traffic signals, form the basis for all transit capacity computations. In this case the basic equation becomes:

$$c_p = \frac{(g/C)\ 3{,}600\ n\ SR}{(g/C)\ D + t_c} \tag{12-2c}$$

where: g = green time, in seconds, and C = cycle length in seconds.

Equations 12-2b and 12-2c may be used to estimate passenger capacity when the number of effective loading positions is taken into account. They provide a realistic estimate where loading patterns and/or door configurations enable vehicles to fill up as they reach the maximum load point. Where this condition is not likely, as along many bus routes, more detailed analyses are needed. In such cases, the detailed methods described in Section III should be applied.

Level of Service

The concept of level of service (LOS) for transit is far more complex than for highways. It includes such factors as coverage of major residential and activity areas, comfort, speed, and reliability (i.e., on-time performance). Convenient schedules, comfortable vehicles, and frequent, fast, and reliable service contribute to LOS. Speed is influenced not only by the number of riders using a transit line, but, to an even greater extent, by stop frequency and dwell times, traffic interferences, and right-of-way design.

Productive capacity, the product of passenger capacity and speed, is an important measure of transport system efficiency. It is important in that it distinguishes between equal passenger throughputs achieved at different speeds. Thus, express bus service normally has a higher productive capacity than local bus service; similarly, commuter rail line operating at 40 mph is twice as "productive" as an urban rail transit line carrying the same number of people at 20 mph. In general, "productive capacity" will be influenced by the type of technology (rail versus bus), the method of operation (private right-of-way versus shared), and the spacing of stops (*9*).

Two aspects of level of service are important from a *capacity perspective:* the number of passengers per vehicle, and the number of vehicles per hour. Capacity-related level-of-service criteria should reflect both. Figure 12-2 illustrates this two-dimensional nature of urban transit capacity.

It can be seen that it is possible to operate many transit vehicles, each carrying few passengers. From a roadway capacity perspective, the number of vehicles could be at or near capacity, even if they run nearly empty.

A few vehicles could operate, each overcrowded. This represents a poor level of service from a passenger comfort (user) perspective. Long waiting times would also detract from user convenience.

Finally the domain of peak-period operation commonly involves a large number of vehicles each heavily loaded.

Vehicle Capacities and Loading Criteria

Typical transit vehicle types, dimensions, and passenger capacities are given in Table 12-4. The total passengers carried varies depending on bus or rail car capacity and the tradeoff between seated capacity and standees. The largest number of seats and lowest number of standees should occur on longer suburban bus routes or on commuter rail routes where higher levels of comfort are essential.

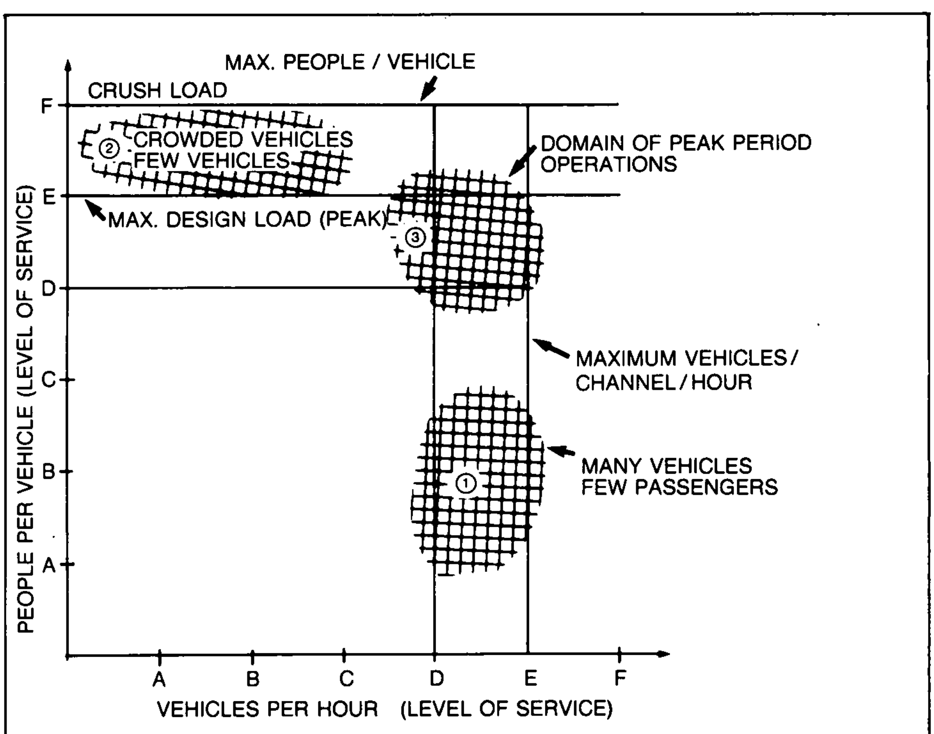


Figure 12-2. The two-dimensional nature of transit level of service as related to transit capacity.

TABLE 12-4. CHARACTERISTICS OF TYPICAL TRANSIT VEHICLES—UNITED STATES AND CANADA

TYPE OF VEHICLE OR TRAIN	LENGTH (FT)	WIDTH (FT)	TYPICAL CAPACITY[a]			REMARKS
			SEATS	STANDEES[b]	TOTAL	
Minibus-short haul	18–25	6.5–8.0	15–25	0–15	15–40	
Transit bus	30.0	8.0	36	19	55	Example: General
	35.0	8.0	45	25	80	Motors, RTS II,
	40.0	8.5	83	32	85	1978
Articulated transit bus	55.0	8.5	66	34	100	Chicago-AM General-MAN
	59.7	8.5	73	37	110	AM General-MAN
Street car	46.7	9.0	59	40–80	99–139	P.C.C.[c]
Light rail car train	151.2	8.7	128	248–272	376–400	San Diego—6-axle car, 2-car train (DU-WAG)
	142.0	8.8	104	250–356	354–460	Boston—6-axle car, 2-car train (Boeing Vertol)
Rail rapid transit train	605.0	10.0	500	1,300–1,700	1,800–2,200	10-car train, IND New York
	600.0	10.0	576	1,224–1,664	1,800–2,240	8-car train, R-46 cars, New York
	448.6	10.3	504	876–1,356	1,380–1,860	8-car train, Toronto
Commuter rail train	85.0	10.5	1,100	200–1,200	1,300–2,300	Regular car, 10-car train

[a] In any transit vehicle the total passenger capacity can be increased by removing seats and by making more standing room available, and vice-versa.
[b] Higher figures denote crush capacity; lower figures, schedule-design capacity.
[c] Presidents' Conference Committee Cars.
SOURCE: Refs. *8* and *34*.

TABLE 12-5. PASSENGER LOADING STANDARDS AND LEVELS OF SERVICE FOR BUS TRANSIT VEHICLES (5-SEAT, 340-SQ FT BUS)

PEAK-HOUR LEVEL OF SERVICE	PASSENGERS	APPROX. SQ FT/PASS.	PASS./SEAT (APPROX.)
A	0 to 26	13.1 or more	0.00 to 0.50
B	27 to 40	13.0 to 8.5	0.51 to 0.75
C	41 to 53	8.4 to 6.4	0.76 to 1.00
D	54 to 66	6.3 to 5.2	1.01 to 1.25
E (Max. scheduled load)	67 to 80	5.1 to 4.3	1.26 to 1.50
F (Crush load)	81 to 85	< 4.3	1.51 to 1.60

SOURCE: Ref. *34*.

A typical 40-ft urban transit bus can normally seat 53 passengers and can carry up to 32 additional standees. Similarly, a 60-ft articulated bus can carry 69 passengers and 41 standees.

An 8-car train of 75-ft rail transit cars normally can seat about 500 and carry a "crush load" of over 2,000 people.

Doorways on buses range from 22 to 30 in. each, while doors on rail vehicles typically average 50 in. each.

Table 12-5 gives suggested "passenger" levels-of-service for a conventional 40-ft bus, based on 53 passengers per bus and 340 gross square feet per vehicle. These approximate comfort-related levels of service are from the perspective of passengers on the vehicle rather than the number of vehicles in a given channel. They are based on local bus operations where short trips at relatively slow speed allow standees. Express bus service on expressways and busways should not allow standees; hence, their scheduling should be guided by level-of-service C.

Suggested passenger levels of service for urban rail transit vehicles are given in Table 12-6. LOS D, which allows up to 2 persons per seat and a minimum 5.0 sq ft per person provides a reasonable balance between operating economy and passenger comfort. It is consistent with the use of 5.4 sq ft per passenger suggested by Pushkarev and Zupan as a realistic passenger capacity for rapid transit lines (*7*).

Level-of-service E is synonymous with "capacity" assuming a reasonable number of standees. It represents the upper limit for scheduling purposes. These maximum scheduled loads are normally 65 to 75 percent of the crush loads.

Level-of-service F defines "crush load" conditions in which standees and other passengers are subject to unreasonable discomfort. Such loads are unacceptable to passengers. Although LOS F represents the theoretically *offered* capacity it cannot be sustained on every vehicle for any given period, and it exceeds the maximum *utilized* capacity. Moreover, it is not reasonable to assume that passengers will be equally distributed among all cars of all trains. *Therefore, level F should not be used for transit capacity calculations.* Note, however, that when the maximum schedule loads are used, some transit units will operate at LOS F.

TABLE 12-6. PASSENGER LOADING STANDARDS AND LEVELS OF SERVICE FOR URBAN RAIL TRANSIT VEHICLES

PEAK-HOUR LEVEL OF SERVICE	APPROX. SQ FT/PASS.	APPROX. PASS./SEAT
A	15.4 or more	0.00 to 0.65
B	15.2 to 10.0	0.66 to 1.00
C	9.9 to 7.5	1.01 to 1.50
D	6.6 to 5.0	1.51 to 2.00
E-1	4.9 to 4.0	2.01 to 2.50
E-2 (Maximum scheduled load)	3.9 to 3.3	2.51 to 3.00
F (Crush load)	3.2 to 2.6[a]	3.01 to 3.80

[a] The maximum crush load can be realized in a single car, but not in every car on the train.

NOTE: Fifty percent standees reflects a load factor of 1.5 passengers per seat.

SOURCES: H.S. Levinson and W.R. Reilly as reported in Ref. *34*.

TABLE 12-7. TYPICAL SPACE REQUIREMENTS FOR SEATED AND STANDING PASSENGERS

	SQ FT PER PASS. (NET)[a]
Seated Passenger	
Typical commuter rail	4 to 6
Typical urban rail transit	3 to 5
Typical urban bus transit	3 to 4
Standing Passenger	
Spacing of persons in unconstrained condition	4 to 9
Minimum space requirement to avoid contact (maximum schedule load LOS E)	2.4 to 2.8
DuWag Standard—commonly used in German LRT systems	2.7
NYCTA—maximum "practical" capacity (crush loads)	1.8

[a] Excludes nonusable space. For seated passengers includes space consumed by seat plus space between seats for legs. For standing passengers, based on clear floor area per standee.

SOURCE: Ref. *37*.

The gross passenger loading criteria provide a reasonable approximation of passengers' levels of service. However, because such loading criteria do not reflect specific space criteria for seated and standing passengers, more refined computations sometimes may be desirable. Table 12-7 gives suggested *net* space requirements for various types of transit that can be applied to specific vehicle sizes and seating considerations. The standing passenger criteria reflect LOS E, schedule capacity.

The precise passenger capacity of a transit vehicle can be estimated by the following relationship:

$$S_i = s_n + \frac{A_n}{L_i} \qquad (12\text{-}2d)$$

where:

- s_n = seats per vehicle;
- A_n = net area for standees;
- L_i = net sq ft/standee for service level i; and
- S_i = passengers/vehicle or passenger spaces/vehicle, for service level i.

L_i should equal 2.6 for maximum schedule loads (level-of-service E) and 2.0 for crush load conditions.

An 8.5 ft by 40 ft 53-passenger bus would have the following capacities under maximum load schedule (LOS E) conditions:

340 Gross sq ft		
245 Net sq ft		
53 Seats at 3.3 sq ft/seat	=	175 sq ft
Net area for standees 245 − 175	=	70 sq ft
Standees at 2.6 sq ft/person	=	27 standees
Total capacity		80
Maximum capacity Table 12-5 based on gross floor area:		80

Car dimensions, seats, and schedule and crush capacities for specific U.S. and Canadian rail transit lines are contained in Appendix II. More detailed information on specific transit vehicle characteristics and capacities is contained in Ref. *39*.

The data in Tables 12-4 through 12-7 may be used to estimate transit vehicle requirements for specified passenger demands at the maximum load points. They also can be used to assess the "level of service" from the passengers' standpoint.

II. TRANSIT CAPACITY EXPERIENCE

This section presents bus and rail transit operating experiences. It identifies service frequencies, passengers carried, and passenger car equivalents; and it indicates the ranges in capacity based on this experience.

BUS CAPACITY EXPERIENCE

The number of buses that can operate past any point in a given period of time varies according to specific roadway conditions and operating practices. Results of both theoretical studies and actual operating experience are summarized as follows.

Bus Flow and Equivalency Studies

Several studies have analyzed the effects of buses on the capacity of mixed-traffic roadways and have estimated the capacity of a bus lane.

1. *Theoretical capacities*—Simulation analysis and field observations of passenger car equivalents have shown that capacities of 1,400 or more buses per lane per hour can be achieved on exclusive bus roadways with uninterrupted flow and no stops for passengers. They compare with some 700 to 750 buses per hour moving through the Lincoln Tunnel—the highest bus flows found in the United States. (*4, 10, 11*)

Theoretical simulation studies based on buses with 30-sec dwell times that operate in platoons of six between stations 0.3 mi apart result in capacities ranging from 350 to 400 buses per hour on an exclusive grade-separated busway (*14*). These results have not been verified, because reported bus volumes of this magnitude occur only under express operations without stops. Maximum hourly bus flows in a single lane on city streets in the United States rarely exceed 100.

2. *Bus headways and passenger car equivalents—freeways*—Field studies of bus-car equivalency factors conducted by the Port of New York Authority in the Lincoln Tunnel found an equivalent of 1.5 cars per bus (*15*). A nationwide study of mixed traffic flows on seven expressways conducted by the Bureau of Public Roads found an equivalency factor of 1.6 (*13*). The effects of grades on bus flows are summarized in other chapters of the Manual.

The similarity of these findings indicates that when buses are in motion either in exclusively bus traffic or in mixed traffic, under uninterrupted flow conditions over a broad range of levels of service, their equivalency factor will be *approximately 1.5 passenger cars.*

3. *Capacity of freeway bus lane (no stops)*—The capacity or service volume of an exclusive bus lane with uninterrupted flow can be computed by applying the 1.5 car equivalency factor to the computed capacity or corresponding service volume in passenger cars per hour. For example, a roadway lane having a capacity of 1,500 passenger cars per hour would have an equivalency of 940 buses per hour. Corresponding uninterrupted bus-flow capacities for various freeway levels of service are as follows, assuming 70-mi per hour design speeds:

LOS	Passenger Cars/ Lane/Hour	Buses/Lane/Hour
A	700	467
B	1,100	733
C	1,550	1,033
D	1,850	1,233
E	2,000	1,333

These uninterrupted bus flow volumes require that bus stops be located off of the travel lane and that adequate acceleration and deceleration lanes be provided.

4. *Arterial streets*—A bus capacity demonstration study on Hotel Street in Honolulu found a capacity of 95 to 100 buses per hour one-way. Bus dwell times averaged 19 sec (ranging from 9 to 32 at individual stops), and bus speeds averaged 2 to 3 mph. However, about 200 other vehicles also used the 36-ft wide, signal-controlled street each way (*32*).

Effects of Buses on Vehicular Capacity

The reductive effect of buses on vehicular capacity varies according to the method of operation. The time available for other vehicles generally will be reduced by the time preempted by buses. This time loss depends on the number of buses in the traffic flow and their service time requirements at stops.

Consequently, for uninterrupted flow, buses are the equivalent of 1.5 passenger car units in the lane where they operate. At bus stops buses have a greater reductive effect because of the time involved in discharging and receiving passengers. The equivalency factors for these conditions depend on the specific duration of the bus stop and its reductive effect on arterial street green time.

The reductive effects of local transit buses on other vehicles in an arterial street lane can be estimated as follows:

1. Where the buses stop in a lane that is not used by moving traffic (for example in a curb parking lane), the time loss to other vehicles is approximately 3 to 4 sec per bus. For this case, buses would either accelerate or decelerate across the intersection, thereby reducing the impeditive effects to other traffic.

2. Where buses stop in a normal traffic lane, the time loss involves the dwell time for buses plus a time loss for stopping and starting, and the associated queuing effects on other traffic. The time loss can be estimated from the following equation *for the lane* in which the buses operate.

$$T_L = (g/C) \times N \times (D + \mathrm{L}) \qquad (12\text{-}3)$$

where:

T_L = time loss, in sec per hr;
g/C = green time/cycle time ratio;
N = buses per hour that stop;
D = average dwell time, in sec; and
L = additional time loss due to stopping, starting, and queuing, in sec (L = 6 to 8 sec, assuming average conditions)

Equivalent passenger car units derived from this equation for various rates of vehicle flow, dwell times, g/C ratios, and bus volumes are given in Table 12-8. Alternatively, the (effective) green for the lane in which the buses operate can be obtained by deducting the time loss. The data are precise for near side bus stops and a reasonable approximation for far side stops.

Observed Bus Flows—Streets and Highways

Observed bus volumes on urban freeways, city streets, and bus-only streets clearly show the reductive effects of bus stops on bus capacity. The highest bus volumes, 735 buses per hour through the Lincoln Tunnel and on the Port Authority Midtown Bus Terminal access ramps are achieved on an exclusive right-of-way where buses make no stops. Where bus stops or layovers are involved, reported bus volumes are much less.

Stopping a bus to receive or discharge passengers limits the capacity of a bus lane. Time must be allowed for acceleration, deceleration, and stop clearance, as well as for the time when the doors are open.

When intermediate stops are made bus volumes rarely exceed 120 buses per hour. However, volumes of 180 to 200 buses per hour are formed where buses may use two or more lanes to allow bus passing, especially where stops are short. An example is Hillside Avenue, New York City. Two parallel bus lanes in the same direction, as found along Madison Avenue, New York, and Portland's Fifth and Sixth Street Transit Mall also achieve this flow rate. Chicago's State Street Mall moves up to 45 buses one-way in a single lane in 15 min (180/hour); however, this is achieved by advance marshalling of buses into 3-bus platoons, and by auxiliary rear-door fare collection during the evening peak hours to expedite passenger loading.

Several downtown streets carry bus volumes of 80 to 100 buses per hour, where there are two or three boarding positions per stop, and where passenger boarding is not concentrated at a single stop. (This frequency corresponds to about 5,000 to 7,500 passengers per hour, depending on load factors.)

These bus volumes provide initial capacity ranges that are suitable for general planning purposes. They compare with maximum streetcar volumes on city streets some 50 years ago approaching 150 cars per track per hour, under conditions of extensive queuing and platoon loading at heavy stops (*16*). However, the street cars had two-person operations, and large rear platforms where boarding passengers could assemble.

TABLE 12-8. PASSENGER CAR EQUIVALENCY OF URBAN BUSES AT SIGNALIZED INTERSECTIONS (APPLIES WHERE BUSES BLOCK CARS)

DURATION OF STOP (SEC)	PERCENT GREEN TIME ON STREET WITH BUSES			
	30%	40%	50%	60%
5	2	2	3	3
10	2	3	4	5
15	3	4	5	6
20	4	5	7	8
25	5	6	8	9
30	5	7	9	11
45	8	10	13	15
60	10	13	19	20

NOTE: Computations are based on the following relationship Pass. car equivalent per bus $= \frac{g}{C} \times \frac{(D+6)}{h}$

where:

h = 2 sec per car;
g/C = green time/cycle ratio;
6 = additional time loss due to starting, stopping, and queuing, sec; and
D = dwell time per bus, sec.

SOURCE: Computed.

Observed Bus Flows—Terminals

Peak-hour bus flows at 13 major bus terminals in the United States and Canada range from 2.5 buses per berth at the George Washington Bridge Terminal to 19 at the Eglinton Station, Toronto. The mean is 8.1; median, 8.0; and standard deviation, 4.2.

The high berth productivity in Toronto reflects the special design of the terminal (with multiple positions in each berthing area), the wide doors on the trolley buses using the terminal, and the free transfer between bus and subway. The relatively low productivity at the New York terminals reflects the substantial number of intercity buses that use the terminals and the single-entrance doors available on many suburban buses.

This current experience suggests about 8 to 10 buses per berth for commuter operations. Intercity berths can accommodate 1 to 2 buses per hour.

Passenger Service Times and Bus Headways

The passenger service times and dwell times at bus stops are necessary for estimating bus and passenger capacities, and the capacity increases that would result from changes in equipment or operating practices. More specifically, they provide the key parameters for capacity calculations.

The minimum headway of buses at a stop consists of (1) actual dwell time when the bus doors are open for boarding and alighting, plus (2) clearance times between buses. The time lost in opening and closing doors may be added to the dwell times, or incorporated in the clearance intervals.

Dwell times may be governed by boarding demand (e.g., in the PM peak when relatively empty buses arrive at a heavily used stop), alighting demand (e.g., in the AM peak at the same location), or total interchanging passenger demand (e.g., at a major transfer point on the system). In all cases, dwell times are proportional to boarding and/or alighting volumes times the service time per passenger.

1. *Clearance times*—Field observations of bus clearance times are limited. A British study (*17*) reported "dead time" (the time spent standing at a stop with the doors closed) of 2 to 5 sec. On-bus analysis of time spent at stops in New Haven and Boston suggests a dead time of about 4 to 6 sec, for opening and closing doors (*18*).

Scheel and Foote (*10*) found that bus start-up times also range from 2 to 5 sec. The time for a bus to travel its own length after starting ranges from 5 to 10 sec, depending on acceleration and traffic conditions. Accordingly, the following ranges are reasonable for normal operations:

- Start-up time: 2 to 5 sec
- Clearance: 5 to 10 sec
- Lag time (before passengers board): 2 to 5 sec

Thus, a reasonable range for clearance time appears to be between 9 and 20 sec for conventional buses, depending on traffic conditions and driver behavior.

2. *Passenger service times*—The amount of time required by each boarding or alighting passenger depends on many factors (*19, 20*). These include:

- Number and widths of doors used.
- Number and height of steps.
- Type of door actuation control.
- Fare collection system.
- Amount of baggage or parcels carried by passengers.
- Procedures and time required to serve wheelchair passengers.

TABLE 12-9. PASSENGER BOARDING AND ALIGHTING TIMES RELATED TO SERVICE CONDITIONS

CONDITIONS	TIME SECONDS PER PASSENGER
UNLOADING (ALIGHTING)	
Very little hand baggage and parcels; few transfers	1.5 to 2.5
Moderate amount hand baggage or many transfers	2.5 to 4.0
Considerable baggage from racks, (intercity runs)	4.0 to 6.0
LOADING[a] (BOARDING)	
Prepayment before entering bus or pay when leaving bus	1.5 to 2.5
Single coin or token with fare box	2.0 to 3.0
Odd-penny multiple-coin cash fares, paid on vehicle	3.0 to 4.0
Zone fares prepaid and registered on bus	4.0 to 6.0
Multiple zone fares; cash; including registration on bus	6.0 to 8.0

[a] Add 1 sec where fare receipts are involved.
NOTE: Assumes single chanel loading
SOURCE: Adapted from Ref. *4*

- Seating configuration.
- Aisle width.
- The mix of alighting vs. boarding.
- The condition and configuration of the pavement, curb, and stop area.

Time at stops is reduced and there are less boarding accidents when the vehicle floor is flush with the station platform. This is commonly achieved in rapid transit, but is not currently used in bus operation for safety reasons.

Research on passenger service times found the following (*19, 20*):

- There is no difference between front and rear door alighting times.
- Using both doors to alight requires more than one-half the time than it does to alight from one door. Time reductions of 27 to 80 percent were observed.
- For alighting passengers, double stream doors require 27 to 46 percent less time than single stream doors.
- Rear door boarding times for double stream doors were observed to be 0.4 sec per passenger faster than for double stream front doors, a reduction of 30 percent.
- The use of boarding through both doors requires less time than for one door, but the time requirements for two doors is more than half that required for one door.
- Reducing double seats on each side of the vehicle to a single seat on one side of the vehicle may reduce passenger service time during periods of peak flow.
- Boarding service time requirements exceed those for alighting.
- Alighting times are greater when boarding passengers are present.
- Fewer delays to alighting and boarding passengers occur when boarding queues are organized and orderly.
- The presence of standees increases passenger service time. Observations of bus operations on Bloomfield Avenue in Newark, New Jersey, indicated an increase of 20 percent in boarding and alighting times when standees were present. It was observed that standees did not always interfere with the boarding and alighting of passengers.

Observed ranges in passenger service times for various bus operating and fare collection procedures in U.S. cities are summarized in Table 12-9. Boarding times are greater than alighting times. American experience with single-door buses shows passenger boarding times ranging from 2 sec (single-coin) to over 8 sec for multiple-zone fares. Alighting times vary from about 1½ to 2½ sec for typical urban conditions to 6 sec or more where baggage is involved.

The importance of fare collection procedures to bus berth capacities is apparent. A simplified method can substantially reduce service times per boarding passenger. Zone fare collection schemes, which require monitoring of access and egress points, are the most time consuming.

Ranges in bus service time in relation to door width, methods of operation, and fare collection practices are given in Table 12-10. These suggested operating service times (seconds per passenger) based on current experience provide a basis for estimating bus dwell times at stop and, in turn, bus and person capacity.

Suggested service times for typical operating conditions—single door loading, pay on bus—are:

Boarding
2.6 sec single coin
3.0 sec exact fare
3.5 sec exact fare—standees on bus

Alighting
1.7 to 2.0 sec

Passenger service times decrease as the number of door channels available to passengers increases. The time values in Table 12-10 reflect inefficiencies in using additional doorway capacity. For example, one passenger may occupy a double door; moreover, passengers do not distribute themselves uniformly among doorway openings. The values do not, however, reflect doorway and aisle turbulence at points of heavy simultaneous boarding and alighting (See Refs. *19* and *20* for more details).

The values assume that prepayment would reduce passenger service time, a reasonable assumption for downtown busways and bus terminals. However, many of the values for multichannel doors and multidoor loading are derived, because relatively little operating experience is available in the United States.

3. *Dwell times at bus stops*—The amount of time that buses spend at specific stops reflects the time of day, location of stop, surrounding land uses, and the number of interchanging transit lines. Stops during the PM rush hour generally average less than 15 to 20 sec; however buses may spend 30 to 60 sec at major transfer points, terminals, or rail-bus interchange locations. Within the central business district, dwell times average 50 to 60 sec at busy locations, although individual stops may be as long as 2 min (*21*).

4. *Queuing at stops*—Studies of bus flow found that queues of 2 to 4 buses develop approximately 20 percent of the time when bus volumes exceed 100 per hour at various locations along Michigan Avenue, Chicago (*22*).

General Capacity Ranges

The observed peak-hour bus movements along freeways, city

TABLE 12-10. TYPICAL BUS PASSENGER BOARDING AND ALIGHTING SERVICE TIMES FOR SELECTED BUS TYPES AND DOOR CONFIGURATIONS (SECONDS PER PASSENGER)

BUS TYPE	AVAILABLE DOORS OR CHANNELS NUMBER	LOCATION	TYPICAL BOARDING SERVICE TIMES[a] PREPAYMENT[b]	SINGLE COIN FARE[c]	TYPICAL ALIGHTING SERVICE TIMES
Conventional	1	Front	2.0 sec	2.6 to 3.0 sec	1.7 to 2.0 sec
	1	Rear	2.0 sec	N.A.	1.7 to 2.0 sec
	2	Front	1.2 sec	1.8 to 2.0 sec	1.0 to 1.2 sec
	2	Rear	1.2 sec	N.A.	1.0 to 1.2 sec
	2	Front, Rear[d]	1.2 sec	N.A.	0.9 sec
	4	Front, Rear[f]	0.7 sec	N.A.	0.6 sec
Articulated	3	Front, Rear, Center	0.9 sec[f]	N.A.	0.8 sec
	2	Rear	1.2 sec[g]	N.A.	—
	2	Front, Center[d]	—	—	0.6 sec
	6	Front, Rear, Center[e]	0.5 sec	N.A.	0.4 sec
Special Single Unit	6	3 Double Doors[h]	0.5 sec	N.A.	0.4 sec

[a] Typical interval in seconds between successive boarding and alighting passengers. Does not allow for clearance times between successive buses or dead time at stop.
[b] Also applies to pay-on-leave or free transfer situations.
[c] Not applicable with rear-door boarding. Higher end of range is for exact fare.
[d] One each.
[e] Two double doors each position.
[f] Less use of separated doors for simultaneous loading and unloading.
[g] Double door rear loading with single exits, typical European design. Provides one-way flow within vehicle, reducing internal congestion. Desirable for line-haul, especially if 2-person operation is feasible. May not be best configuration for busway operation.
[h] Examples: Neoplan TR-40 Mobile Lounge designed by Trepal Systems, Inc., for airport apron use.
SOURCE: Refs. *4, 17, 31, 36*

streets, and to or from bus terminals provide guidelines for estimating the capacity of similar facilities. They also provide means of checking or verifying more detailed capacity calculations. General guidelines for planning purposes follow:

1. *Bus capacity*—Suggested arterial street bus capacity ranges based on actual operating experience are given in Table 12-11. This table gives representative service volumes for downtown streets and arterial streets leading to the city center for each

TABLE 12-11. SUGGESTED BUS FLOW SERVICE VOLUMES FOR PLANNING PURPOSES (FLOW RATES FOR EXCLUSIVE OR NEAR-EXCLUSIVE LANE)

LEVEL OF SERVICE	DESCRIPTION	BUSES/LANE/HOUR	MIDVALUE
	ARTERIAL STREETS		
A	Free Flow	25 or less	15
B	Stable Flow, Unconstrained	26 to 45	35
C	Stable Flow, Interference	46 to 75	60
D	Stable Flow, Some Platooning	76 to 105	90
E	Unstable Flow, Queuing	106 to 135	120
F	Forced Flow, Poor Operation	over 135[a]	150[a]
	MAIN CBD STREET		
A	Free Flow	20 or less	15
B	Stable Flow, Unconstrained	21 to 40	30
C	Stable Flow, Interference	41 to 60	50
D	Stable Flow, Some Platooning	61 to 80	70
E	Unstable Flow, Queuing	81 to 100	90
F	Forced Flow, Poor Operation	over 100[a]	110[a]

[a] Results in more than one-lane operation.
SOURCE: Adapted from Refs. *5* and *34*

level of service. Where stops are not heavily patronized, as along outlying arterial streets, volumes could be increased by about 25 percent.

These service volumes may be used for planning purposes. More precise values for operations and design purposes should be computed from the capacity relationships and procedures set forth in the following sections.

THE VALUES FOR LOS F, FORCED FLOW CONDITIONS, SHOULD NOT BE USED FOR PLANNING OR DESIGN. They are merely given for comparative purposes.

2. *Person capacity*—The people per hour that can be served by varying bus flow rates and passenger load factors are given in Table 12-12. This table provides a broad person-capacity planning guide *assuming* that key boarding points are sufficiently dispersed to achieve these bus loads. It suggests *maximum* person-flow rates of about 7,500 people per hour per lane on downtown streets and 10,000 people per hour per lane on arterial streets. Corresponding maximum values for *seated* passenger flow rates are 5,000 and 6,750 people respectively. Exclusive use of articulated buses would increase these values 15 to 20 percent.

3. *Peak-hour factor*—These person-flow rates indicate the number of people that can be carried, assuming uniform flow during the peak-hour. Accordingly, appropriate peak-hour factors should be used in discounting these values to reflect flow-variations within the 15-min peak hour. Preferably, these flows should be compared directly with the observed 15-min volume multiplied by four.

The peak-hour factor (PHF) is defined as the *hourly volume divided by four times the highest 15-min volume occurring within the hour.* The actual hourly volume can be calculated by:

$$HV = (\text{Peak 15-min volume})\ (4)\ (\text{PHF}) \qquad (12\text{-}4)$$

Typical peak-hour factors range from 0.60 to 0.95 for transit lines. The Los Angeles SCRTD reports peak-hour factors of 0.66 for commuter buses and 0.74 for local buses on Route 83, Wilshire Boulevard. The *Transportation and Traffic Engineering Handbook* suggests peak-hour factors of 0.70 to 0.95. A peak-hour factor close to 1.0 may well indicate system overloading (underservicing) and reveal the potential for more service.

A peak-hour factor of 0.80 would result in a maximum one-way *hourly* passenger volume of about 6,000 persons on downtown streets and 8,000 persons on arterial streets.

RAIL TRANSIT CAPACITY EXPERIENCE

This section briefly overviews peak-hour rail transit ridership in the United States and Canada, and its passenger capacity implications. More detailed information on rail transit ridership and capacity is set forth in a variety of references (*3,7,8, 9,12,24,25,26*).

The rail transit capacities have been included to provide a complete picture of urban transit capacities and to indicate the passenger volume ranges for which rail transit may be appropriate. Thus, they provide an important input for modal planning decisions. In addition, light rail transit operates on city streets and affects street operations.

Rail transit encompasses a variety of modes—each with distinctive service and performance characteristics. It includes commuter rail lines (both electric and diesel); urban rapid transit (both city and suburban systems), street car and light-rail transit with both on- and off-street running. All U.S. and Canadian systems, except for Montreal's rubber-tire Metro, operate on conventional steel rails.

All types of rail transit except street car and light rail lines operate totally off-street. Light rail transit vehicles (LRV's) operate singly or in trains (1) on streets in mixed traffic or within reserved areas or (2) off-street in exclusive rights-of-way.

Differences among rail transit modes also exist in station spacing and design, fare structure and collection methods, train length and propulsion, degree of access control and markets served. Sometimes, however, such differences may be difficult to discern.

TABLE 12-12. SUGGESTED BUS PASSENGER SERVICE VOLUMES FOR PLANNING PURPOSES (HOURLY FLOW RATES BASED ON 50 SEATS PER BUS)

LEVEL OF SERVICE (STREET)	BUSES/ PASSENGERS/SEAT	LEVEL OF SERVICE (PASSENGERS) A 0.00–0.50	B 0.51–0.75	C 0.76–1.00	D 1.01–1.25	E 1.26–1.50
ARTERIAL STREETS						
A	25 or less	625	940	1,250	1,560	1,875
B	26 to 45	1,125	1,690	2,250	2,810	3,375
C	46 to 80	2,000	3,000	4,000	5,000	6,000
D	81 to 105	2,625	3,940	5,250	6,560	7,875
E	106 to 135	3,375	5,060	6,750	8,440	10,125
CBD STREETS						
A	20 or less	500	750	1,000	1,250	1,500
B	21 to 40	1,000	1,500	2,000	2,500	3,000
C	41 to 60	1,500	2,250	3,000	3,750	4,500
D	61 to 80	2,000	3,000	4,000	5,000	6,000
E	81 to 100	2,500	3,750	5,000	6,250	7,500

NOTE: Ratio shown for level of service (passengers) is "passengers per seat" on average bus. Thus 1.00 means 50 passengers for the assumed 50 seats.
Values would be 6 percent higher for a 53-seat bus.
Values for articulated buses would be 15 to 20 percent greater.

SOURCE: Computed

TABLE 12-13. OBSERVED PEAK-HOUR PASSENGER VOLUMES ON U.S. AND CANADIAN RAPID TRANSIT SYSTEMS (IN PEAK DIRECTIONS)

CITY AND YEAR		LINE/LOCATION	TRAINS PER HOUR	CARS PER HOUR	HEADWAY SECONDS	APPROX. CAR LENGTH FT (ROUNDED)	PERSONS/ HOUR IN PEAK DIRECTION (MAX. LOAD SECTION)	PASSENGERS PER TRAIN (ROUNDED)
New York City	1982	IND E, F, 53rd St. Tunnel	26	208	128	75	54,500	2,100
		IND A, D, 8th Ave Express	21	210	159	60, 75	43,500	2,070
		IRT 4, 5, Lexington Ave. Exp.	25	250	157	50	38,100	1,520
		PATH-World Trade Center[a]	38	266	98	50	25,500	670
	1960	IND E, F, 53rd St. Tunnel	32	320	112	60	61,400	1,920
		IND A, D, 8th Ave. Express	30	300	120	60	62,000	2,070
		IND 4, 5, Lexington Ave. Exp.	31	310	116	50	44,500	1,430
		IND 2, 3, 7th Ave. Express	24	240	150	50	36,800	1,530
Toronto	1978	Yonge St.	30	210	120	75	32,000	1,060
	1974	Yonge St.	28	168	129	75	36,000	1,290
	1960	Yonge St.	28	224	129	57	32,200	1,260
Montreal	1976	N Line	23	207	157	56	28,200	940
Chicago	1984	Milwaukee	17	136	212	50	12,400	730
		Lake-Ryan	19	152	189	50	12,300	647
		North-South	15	120	240	50	11,400	760
	1978	Lake-Ryan	21	168	111	50	16,500	790
		North-South	20	160	180	50	14,000	700
Philadelphia	1976	North Broad (2 tracks)	23	126	157	67	10,600	460
Boston	1977–78	Red Line	17	68	212	70	13,000	460
		Orange Line	13	52	277	55	8,400	650
San Francisco	1977	BART-Transbay	11	98	327	75	8,000	730
		BART-Mission	10	85	360	75	6,500	650
Washington	1980	Blue-Orange	20	120	180	75	13,000	650
Atlanta	1980	East Line	6	36	600	75	4,250	710
Cleveland	1976	West Side	14	52	258	50, 70	5,400	390
	1960	West Side	20	80	180	50	6,200	360

[a] Multiple track terminal.
SOURCE: Refs. *1, 7, 8, 9,* New York Metropolitan Transportation Council, Chicago Transit Authority.

For rail systems other than street cars, travel times between stations are relatively unaffected by increased passenger volumes or service frequency.

Observed Train and Passenger Flows

The operating experience for typical rail rapid transit and light-rail transit lines are given in Tables 12-13 and 12-14, respectively. These tables give typical peak-hour peak-direction passenger volumes, service frequencies, and train lengths for principal U.S. and Canadian rail transit lines.

These figures mainly reflect current (1976–1984) experience. However, since many of the lines carried substantially higher passenger flows in peak years, 1946–1960 data are shown for comparative purposes. Thus, the observed number of peak-hour passengers at the maximum load point usually reflects demand rather than capacity. Peak 15- to 20-min volumes expressed as hourly flow rates are about 15 percent higher.

1. *Rapid transit*—There is a wide range of peak-hour passengers carried by the various rapid transit lines. This range reflects factors such as the number, length, and frequency of the trains operated. Especially important are the peak-period trains assigned for scheduling purposes, the demands in the specific corridors, and the configuration or constraints of principal stations and switching points.

There are generally less than 30 trains per track during the peak hour in the United States and Canada, although during portions of this period slightly shorter headways are operated. In general, the 90-sec headway that is possible with modern signaling systems is not realized on an hourly basis in the United States and Canada. The single exception is the PATH system, which operates 38 trains per hour on a single track under the Hudson River from a multitrack World Trade Center Terminal in New York City; in this case signals and interlocking points limit capacities.

Before the State Street subway was opened, Chicago's elevated Loop carried 78 trains/track with 438 cars in a single hour.

TABLE 12-14. OBSERVED PEAK-HOUR PASSENGER VOLUMES ON STREET CAR AND LIGHT RAIL SYSTEMS IN UNITED STATES AND CANADA (PEAK DIRECTION)

CITY	LOCATION	YEAR	TRAINS PER HOUR	CARS PER HOUR	HEADWAY SECONDS	LENGTH OF CAR OR TRAIN	PASSENGER/HOUR IN PEAK DIRECTION	PASSENGER/CAR OR TRAIN	EQUIPMENT
ON STREET									
Pittsburgh	Smithfield St. Bridge	1949	120	120	30	46.5	9,000[a]	75[a]	PCC
Pittsburgh	Smithfield St.	1976	51	51	71	46.5	3,800	74	PCC
San Francisco	Market Street (before subway)	1977	68	68	53	46.0	4,900	72	PCC
Toronto	Queen St. East	1978	66	66	55	46.5	4,200	64	PCC
IN TUNNEL OR OFF STREET									
Philadelphia	Market St.	1956	133	133	27	46.0	9,000	67	PCC
Boston	Green Line (Boylston St.)	1976	36	88	100	46.5	6,900	192	PCC
Philadelphia	Market Street	1978	73	73	180	46.0	3,700	151	PCC
San Francisco	Market Street	1983	NA	62	NA	70.0	6,340	109	Boeing LRV
Cleveland	Shaker Hts.	1976	30[a]	60[a]	120[a]	50.0	4,400	143	PCC
Boston	Green Line (Lechmere)	1978	16	48	225	46.5	1,500	94	PCC
Newark	City Subway	1978	30	30	120	46.5	1,500	50	PCC
Edmonton	LRT Line	1978	12	24	300	77.0	2,100	87	DUWAG
San Diego	LRT	1981	3	6	1,200	151	600	200	DUWAG

[a] Estimated.
SOURCE: Refs. *7, 8, 9.*

This high movement of trains resulted from manual train control and platoon loading of trains at stations. The Loop under cab signal control has carried up to 35 trains per track in peak hours.

Train lengths of 4 to 10 cars are commonly operated. *Maximum train lengths* range up to 8 cars in Chicago, San Francisco and Toronto, and 10 cars in New York City. The IRT Flushing Line in New York City is the only line that operates 11 cars per train.

Rail car lengths range from about 50 ft in Chicago and New York City (IRT, PATH) to 75 ft in Washington, San Francisco, and New York (new cars). Maximum train lengths are 600 ft.

Peak-hour passengers carried per track past the maximum load point range upward from 5,400 in Cleveland to 36,000 in Toronto and over 50,000 in New York City, as of 1974–1983. The highest volumes carried are found on the Queens-Manhattan trains passing through the 53rd Street Tunnel: 53,000 persons per hour per track in one direction. This line carried more than 60,000 passengers per hour, one-way, in previous years.

Lines in cities such as Moscow, Tokyo, and São Paulo carry peak-hour flows of 50,000 to 60,000 persons per track per hour.

2. *Light rail transit*—Operating characteristics of U.S. and Canadian LRT and streetcar lines are given in Table 12-14.

Post-World War II streetcars operated at 30-sec headways both on-street (Pittsburgh) and in tunnel (Philadelphia). Peak-hour passenger flows approximated 9,000 persons per hour.

Current operating experience shows up to 75 single cars per track carrying 5,000 passenger per hour. San Francisco's Market street surface routes carried 4,900 peak-hour one-way passengers per hour before they were placed underground. Approximately 4,000 passengers per hour are carried by Toronto's Queen Street line, and Pittsburgh's Smithfield Street Bridge lines.

Both 50-ft and 70-ft to 75-ft cars operate in two and three car trains. Up to 40 trains per track with 90 cars are operated.

Off-street passenger volumes range from 1,500 people/hour on the Newark City subway and Boston's Lechmere line to over 6,000 persons per hour on Boston's Boylston Street subway and San Francisco's Market Street subway.

The observed volumes generally reflect passenger demands and scheduling policies rather than maximum possible flows. Flows as high as 15,000 persons per hour have been observed in the past. Moreover several European systems report peak flows ranging up to 18,000 persons per hour. Philadelphia's Market Street subway has carried 140 cars per hour with a minimum headway of 23 sec. It has carried as many as 12,000 people per hour. Boston's Tremont-Boylston LRT subways traditionally scheduled 60 trains and 250 cars per hour. The MBTA estimated the capacity of 15,000 persons per hour in 1971 when inbound peak flows approximated 12,000 persons (*27*).

General Capacity Ranges

The capacity of a rail line is determined by station capacity or way capacity, whichever is smaller. In most cases, station (or stop) capacity governs.

Capacity depends on: (1) car size and train-station length, (2) allowable standees as determined by scheduling policy, and (3) the minimum spacing (headway) between trains. This minimum headway is a function not only of dwell times at major stations, but also train length, acceleration and deceleration rates (including deceleration), and train control systems.

Time-space diagrams can be used to estimate the "safe separation" or minimum headway between trains. Theoretical approaches to estimating the minimum spacing are sometimes used; examples are given in Appendix II. A more common practice is to obtain the minimum spacing between trains based on actual experience, station dwell times, and signal control systems.

Passenger capacity in the peak direction during the peak hour can be estimated from the following equations:

$$\text{Passengers/hour} = \frac{\text{Trains}}{\text{Hour}} \times \frac{\text{Cars}}{\text{Train}} \times \frac{\text{Seats}}{\text{Car}} \times \frac{\text{Passengers}}{\text{Seat}} \tag{12-5a}$$

or,

$$\text{Passengers/hour} = \frac{\text{Cars}}{\text{Hour}} \times \frac{\text{Seats}}{\text{Car}} \times \frac{\text{Passengers}}{\text{Seat}} \tag{12-5b}$$

An alternative formulation, based on allowable levels of pedestrian space, is as follows:

$$\text{Passengers/hour} = \frac{\text{Trains}}{\text{Hour}} \times \frac{\text{Cars}}{\text{Train}} \times \frac{\text{Ft}^2}{\text{Car}} \Big/ \frac{\text{Ft}^2}{\text{Passenger}} \tag{12-6}$$

This latter formulation derives a passenger capacity that is independent of the seating configuration and that directly relates to the area of each car. Cars that maximize total passenger capacity generally minimize the number of seats.

The precise values for these equations will vary among individual transit properties depending on the type of equipment used and operating policy.

1. *Rapid transit*—Typical ranges in rail rapid transit capacities are summarized in Table 12-15 for U.S. and Canadian operating experiences, based on 30 trains per track per hour. Ranges reflect varying car lengths (50 ft and 75 ft) and train sizes (6, 8, or 10 cars) and passenger load factors. These capacities can be adjusted upward or downward based on specific operating policies. Detailed car dimensions, seated passengers, schedule-loads, and crush loads are contained in Appendix II.

Levels of service are also shown in Table 12-15 for various load factors, i.e., percent standees. These are keyed to the levels of service given in Table 12-6.

- The "crush load" capacities are shown for comparative purposes. They should not be used in determining capacity.
- LOS D (5.0 sq ft per passenger) represents a realistic value for use in transit operations and planning. It results in capacities ranging from 18,000 to 30,000 persons per hour for train lengths up to 600 ft. These figures compare with 20,000 to 34,000 persons per hour per track derived by Pushkarev as a comfortable peak-hour capacity (*7*).

Where signal controls, station dwell times, and operating policies allow closer than 2-min headways, capacities can be increased accordingly.

In estimating rail transit capacities and levels of service for overcrowding, it is essential to analyze the peak 15-min period. For example, a "scheduled load" of 200 percent standees (3.3 sq ft per passenger) would relate to the peak 15-min period. Similarly, if an hourly capacity of 27,000 people is provided by 6-car trains with 200 percent standees, this implies that the peak 15 min would carry 27,000/4 = 6,750 people. If half of the peak-hour passengers moved in 15 min, the effective hourly capacity would be 13,500. In this case, the peak-hour factor would be 0.5; therefore, the hourly service volume would be 0.5 × 27,000 or 13,500. These peaking characteristics further explain the differences between *observed passengers* and *theoretical capacities* (i.e., utilized and offered capacities).

2. *Light rail transit*—The passenger carrying capacity of light rail transit (LRT) depends on vehicle size, train length, and headway. However, the realizable LRT capacities also depend on design and policy considerations that reflect specific local constraints of station design, at-grade operations, and type of right-of-way.

Where trains operate on-street, capacity estimates can be derived by adapting the equations for bus transit (Section III) to account for differing vehicle sizes, train lengths, and clearance requirements. Capacity estimates for off-street operations may be derived from the approaches set forth for rail transit.

LRT trains usually are limited to a maximum of three cars,

TABLE 12-15. TYPICAL RAIL TRANSIT CAPACITIES—30 TRAINS PER TRACK PER HOUR, 2-MIN HEADWAY (FLOW RATE)

CARS/TRAIN	CARS/HOUR	CAR/LENGTH (FT)	APPROX. SEATS/TRAIN	PASSENGERS PER HOUR: 0%[a] STANDEES	50% STANDEES	100% STANDEES	150% STANDEES	200% STANDEES	250% STANDEES
			SEAT LOAD =	(1.00)[b]	(1.50)[b]	(2.00)[b]	(2.50)[a]	(3.00)[b]	(3.50)[b]
6	180	50	300	9,000	13,500	18,000	22,500	27,000	40,500
		75	450	13,500	20,250	27,000	33,750	40,500	60,750
8	240	50	400	12,000	18,000	24,000	30,000	36,000	54,000
		75	600	18,000	27,000	36,000	45,000	54,000	81,000
10	300	50	500	15,000	22,500	30,000	37,500	45,000	67,50C
		75[c]	750	22,500	33,750	45,000	56,250	67,500	101,250
FT²/PASSENGER:				10.0	6.7	5.0	4.0	3.3	2.6
PASSENGER LEVEL OF SERVICE (U.S. & CANADA CONDITIONS)				B	C	D	E-1	E-2	F
COMMENTS:								Maximum schedule loads	Not attainable on a train basis

[a] Approximate.
[b] Passengers per seat.
[c] This condition does not exist in the United States.
SOURCE: Ref. *34*

where on-street operation is involved: (1) longer trains could not operate on city streets without simultaneously occupying more than the space between adjacent cross streets when traversing short blocks; (2) long trains cannot clear at-grade intersections rapidly; and (3) they need long platform lengths at stations.

Minimum headways for light-rail systems will depend on train length, platform design (high versus low), fare collection methods (prepayment versus pay on train), and headway controls (manual versus block signals). Under manual operations, 80 to 100 single-unit cars per track per hour could be accommodated. When trains run under block signal controls, as is common with rapid transit systems, 120-sec headways are common, although shorter headways could be realized.

At 120-sec headways, a high-speed LRT system operating on mainly reserved right-of-way with three-unit Boeing vehicle trains would have a line capacity slightly in excess of 6,000 seated and 19,000 total passengers per hour (thirty 3-car units at 211 persons/car). Under single-vehicle manual operation at lower speeds, closer headways are feasible. At 60-sec headways, single Boeing LRT units have a capacity of 4,000 seated and 13,000 total passengers per hour (schedule load) (*29*). However, in practice these capacities are not realized because of limited ridership demands, route convergence limitations, and terminal constraints.

Typical ranges in capacities are as follows:

		Pass. Level of Service	
		D	E
	Units Per Hour	5.0 Sq Ft Per Person	Max. Schedule Loads 3.3 sq ft Per Person
Street cars (single 46-50 ft unit on street)	90	7,500	12,000
LRT—Off street (three 75-ft car units)	30	11,000	17,500
	35	13,000	20,000

Current operating experience in the United States and Canada suggests maximum realizable capacities of 12,000 to 15,000 persons per track per hour. However, the European experience shows up to 20,000 persons per hour (Appendix II).

III. METHODS AND PROCEDURES—ON-STREET TRANSIT

This section sets forth detailed methods and procedures for estimating on-street transit capacity. It contains approaches for:

1. Estimating bus berth and system capacity.
2. Planning bus priority treatments.

Although the methods are keyed to bus transit, many can be applied to on-street light rail transit operations. Illustrative applications show how the methods can be used.

BUS BERTH AND SYSTEM CAPACITY

The section describes detailed methods for estimating the capacity of a bus berth, bus stop, or bus route. These approaches should be used for *operations* and *design* purposes. Capacity values assume that the bus lane or stop area would be exclusively for bus use. Where other traffic shares a lane with transit vehicles, the time needed for this traffic should be deducted. The net time should then be used for transit capacity analysis.

General Considerations

The capacity of a bus system is determined by the capacity of the most heavily used stop or the capacity of the line, whichever is less. Consequently, theoretical capacities for uninterrupted flow have little practical application for other than express nonstop runs.

The capacities of bus routes, terminals and busways, in persons carried, are generally limited by the ability of stops or loading areas to pick up and discharge passengers. Just as the critical signalized intersection usually determines arterial street capacity, bus route capacity is determined by the passenger service times at major loading and unloading points.

One of the basic considerations in analyzing bus capacity, therefore, is the bus berth or bus stop, and its ability to process buses and passengers. Capacity is influenced by many factors, including the type and number of berths, number of boarding and alighting passengers at major stops, design of the vehicle, method of fare collection, location of the berth, bus layover practices at terminals, other operating policies, and traffic signal controls.

Each bus requires a certain amount of service time at stops that varies with the number of boarding and alighting passengers, door configuration of buses, and methods of fare collection. The minimum safe spacing between buses in motion and the number of loading positions available at any stop also influence the total number of buses and people that a given stream can carry. Bus volume may be increased where vehicles can overtake or pass each other in entering or leaving loading positions.

The most common form of berth is the linear bus stop at a street curb. The length of such a berth may be adjusted to simultaneously handle multiple buses within reason, and buses not stopping may pass stopped buses in other lanes where street width permits. The same type of stop may be provided under two conditions on busways:

1. In the travel lane (on-line), in which case following buses may *not* pass the stopped bus.
2. Out of the travel lane (off-line), in which case following buses *can* pass stopped vehicles.

Berths in bus terminals may be linear, or they may take various other forms. Angle berths are limited to one bus per berth, and they require buses to back out. Drive-thru angle berths are also feasible, and may accommodate multiple vehicles. Shallow "sawtooth" berths are popular in urban bus-rail interchange terminals, and are designed to permit independent movements into and out of each berth.

Equations and Guidelines

The following equations show how the capacity of a busway, bus terminal, or city street can be estimated in terms of (1) buses per hour, and (2) people per hour. They establish ranges in typical time requirements for each of the operations at a bus berth, and they identify relationships between bus passenger line-haul capacity, boarding and alighting volumes, and types of bus equipment. They should be applied to the peak 15 min in each rush hour since this period, when the maximum boarding and alighting volumes normally occur.

The number of buses that can be handled at stops without developing unacceptably long queues (and associated waiting lines) varies principally with the service time per bus and, to a lesser degree, with the number of loading positions. Additional loading spaces (or additional length of bus zones) increase the capacity, but at a decreasing rate as the number of spaces increases.

1. *Capacity of a bus berth (vehicles)*—The number of buses that can use a bus berth (or stop) varies inversely with passenger service (dwell) times, D, and bus clearance times, t_c. The passenger service times depend on the number of boarding, alighting, or interchanging passengers, fare collection practices, and door configurations. The clearance time should include door opening and closing times, when they are not incorporated into the dwell times directly. It includes all time losses associated with a bus entering and leaving a stop, other than passenger loading.

a. Uninterrupted flow, no delays caused by traffic signals—The number of buses per berth per hour can be estimated from the following set of equations.

$$f' = \frac{3{,}600\ R}{h'} = \frac{3{,}600\ R}{D + t_c} \qquad (12\text{-}7)$$

where:

h' = minimum headway at the bus berth or stop;
D = dwell times at bus berth or stop;
t_c = clearance time between successive buses;
R = reductive factor to compensate for dwell time and arrival variations; and
f' = maximum buses per berth per hour.

In estimating the total time that buses spend at a stop, the time spent opening and closing doors should be taken into account. This normally approximates 4 to 5 sec. Thus, if 10 passengers board a bus at 3-sec headways, the total dwell time could be 35 sec. *As formulated in the various capacity analyses herein, this door opening and closing time is included in the clearance time between buses.* Whichever approach is used, the door opening and closing time must be considered.

The reductive factor R is 0.833 for *maximum* bus capacity. This occurs when one-third of all buses are waiting in approach queues, reducing the capacity of the berth to about three-quarters of the ideal value. Thus, *3,000 normally replaces, 3,600*R *in the equation.*

The minimum headway, h', can be obtained as follows:

Boarding only; one way flow

$$h' = bB + t_c \qquad (12\text{-}8a)$$

Alighting only; one way flow

$$h' = aA + t_c \qquad (12\text{-}8b)$$

Two-way flow through door

$$h' = [aA + bB + t_c] \qquad (12\text{-}8c)$$

where:

A = alighting passengers per bus, in peak 15 min;
a = alighting service time, in seconds per passenger;
B = boarding passengers per bus, in peak 15 min; and
b = boarding service time, in seconds per passenger.

Where passengers enter via the front door, and exit via the rear door, the *greater* result from Eqs. 12-8a and 12-8b determines the minimum headways. For heavy two-way flow through a single door, the headways in Eq. 12-8c could be increased by 20 percent.

Substituting the appropriate values of h' in Eq. 12-7 produces the following equations for the maximum number of buses per berth per hour (R is assumed as 0.833):

Boarding only; one way flow

$$f' = \frac{3{,}600\ R}{h'} = \frac{3{,}600\ (0.833)}{bB + t_c} = \frac{3{,}000}{bB + t_c} \qquad (12\text{-}9a)$$

Alighting only; one-way flow

$$f' = \frac{3{,}600\ R}{aA + t_c} = \frac{3{,}000}{aA + t_c} \qquad (12\text{-}9b)$$

Two-way flow through door

$$f' = \frac{3{,}600\ R}{aA + bB + t_c} = \frac{3{,}000}{aA + bB + t_c} \qquad (12\text{-}9c)$$

Equations 12-7 and 12-8 are *precise* where there are no delays due to traffic signals, as along a busway or at a terminal. For city street operations at signalized intersections, they provide an upper limit of berth (stop) capacity. Several downward adjustments are necessary, especially for dwell times less than 60 sec per stop.

b. Bus flow interrupted by traffic signals—The number of buses that can stop for passengers and then pass through a signalized intersection can be estimated from Eq. 12-10a and

TABLE 12-16. ESTIMATED MAXIMUM CAPACITY OF BUS STOPS—BUSES PER HOUR (FLOW RATE)

DWELL TIME (SEC)	CLEARANCE TIME (SEC) t_c 10, g/C 0.5	10, g/C 1.0	15, g/C 0.5	15, g/C 1.0
15	86	120	67	100
30	60	75	50	67
45	46	54	40	50
60[a]	38	42	33	40
75	32	35	28	33
90	28	30	25	28
105	24	26	22	25
120	22	23	20	22
150	18	18	17	18
180[b]	15	16	14	16

[a] Typical CBD Stop-PM peak.
[b] Maximum CBD Stop-PM peak.
SOURCE: Computed from Eq. 12-10. Assumes $R = 0.833$.

12-10b. These equations assume that the time spent loading and/or discharging passengers on the green, g, and red, r, phases are proportional to the g/C and r/C ratios, respectively. The yellow time is assumed as part of the green time. The equations are precise for near side stops and provide a reasonable approximation for far side stops.

$$f'_c = \frac{g}{t_c + D\,(g/C)} \tag{12-10a}$$

$$f' = (g/C)\,\frac{3{,}600\,R}{t_c + D\,(g/C)} = g/C\,\frac{3{,}000}{t_c + D\,(g/C)} \tag{12-10b}$$

where:

g = green (plus yellow) time per cycle, in sec;
C = cycle length, in sec;
f'_c = buses per cycle;
D = dwell time per bus resulting from loading and/or unloading passengers (i.e., bB, aA, $[aA + bB]$, *or* if inflow and outflow both heavy, use $1.2\,[aA + bB]$);
t_c = clearance time per bus; and
f' = buses per hour.

Note that as g/C approaches one, Eqs. 12-7 and 12-10b become identical. Both equations assume that there is no other traffic in the bus lane and that buses do not pass and overtake each other.

c. Berth capacity values—The number of buses per hour are given in Table 12-16 for g/C ratios of 0.5 and 1.0. Values are tabulated for clearance times of 10 and 15 sec, and dwell times ranging from 15 to 180 sec. This table can be used to estimate the number of buses per hour that can be served by a single berth. Values for g/C times between 0.5 and 1.0 can be interpolated; values for g/C times less than 0.5 and for other dwell times can be computed directly from Eq. 12-10b.

The 10-sec clearance time represents the absolute minimum time spacing possible at a stop for conventional buses. However, for most situations the 15-sec clearance values should be used.

A 60-sec dwell time per bus with a 15-sec clearance time would result in a capacity of 33 buses per berth per hour for a g/C of 0.5 and 40 buses per hour for a g/C of 1.0. These values are based on 3,000 "effective" sec per hour. Corresponding values assuming perfect schedule reliability and a uniform distribution of dwell times during the peak 15 min would be 40 and 48 buses per hour (i.e., $R = 1.00$).

The number of buses that are stopped at a traffic signal must fit within the available block length. For short blocks, the block spacing may limit capacity. (This is even more true where LRT trains run on-street.)

Where passenger boardings are dispensed along the transit line, the passenger capacity can be estimated by multiplying the number of buses per berth obtained from Table 12-16, or from Eq. 12-10b by (1) the number of effective loading positions and (2) the specific loading standard, i.e., persons per vehicle.

d. Levels of service—Suggested levels of service for the number of buses per berth (i.e., per stop) are given in Table 12-17. The levels of service are keyed to the *approximate* probability or likelihood of queues forming behind the bus stop.

The number of buses per berth that can be accommodated at any level of service can be estimated as follows:

$$c_{V(i)} = (g/C)\,\frac{3{,}000\ (\text{LOS Factor})_i}{t_c + D\,(g/C)} \tag{12-11}$$

where: $c_{V(i)}$ is the capacity at service level i and (LOS Factor)$_i$ are the index values given in column 4 of Table 12-17.

For example, assuming a g/C of 0.5, a dwell time, D, of 60 sec, and a clearance of 15 sec, the number of buses per berth at service C would be

$$c_{v(C)} = \frac{0.5(3{,}000)\,(0.80)}{15 + 60\,(0.50)} = \frac{1{,}500\,(0.80)}{45} = 26.7,\ \text{Say } 27$$

Finally, if the peak-hour factor, PHF, were 0.67, the service volume at LOS C would be 0.67 (27) or 18 buses per berth.

Typical (rounded) values for a 60-sec dwell time, 15-sec clearance, and g/C ratios of 0.5 and 1.0 are given in Table 12-18.

e. Berth use efficiency—Each loading position at a multiple-berth stop does not have the same capacity as a single berth stop. This is because it is not likely that the loading positions at a multiberth stop will be equally used, or that passengers will distribute equally among loading positions. Moreover, where stops are designated for specific routes, bus schedules may not permit an even distribution of buses among loading positions. Buses also may be delayed in entering or leaving a berth by buses in adjacent loading positions.

The *actual* efficiency of a system of loading positions will also vary with the type of design. Consequently, the design of the bus loading areas influence capacities.

Suggested "berth efficiency factors" are given in Table 12-19 for "on-line" and "off-line" stops. These factors are based on experience at the Port Authority of New York and New Jersey's Midtown Bus Terminal. The table suggests that four or five, "on-line" positions could have a maximum efficiency of 2.5 berths. Five "off-line" positions would have an efficiency of about 3.75 berths.

Note that to provide for two "effective" berths, *three* physical berths would have to be provided (partial berths are never built). Thus, N_b is not the number of berths which must necessarily

TABLE 12-17. LEVELS OF SERVICE FOR BUS STOPS

1 LEVEL OF SERVICE (LOS)	2 R VALUE	3 EFFECTIVE SEC/HOUR (3,600 R)	4 INDEX LOS E = 1.00	5 APPROX. PROBABILITY OF QUEUES FORMING BEHIND BUS STOP
A	0.400	1,200	0.40	1
B	0.500	1,800	0.60	2.5
C	0.667	2,400	0.80	10
D	0.750	2,700	0.90	20
E (Capacity)	0.833	3,000	1.00	30
Capacity E- Perfect Conditions	1.000	3,600		50

NOTE: For use in this equation:

$$C_v = \frac{3{,}600\ R}{D + t_c} \quad \text{or} \quad \frac{(g/C)\ 3{,}600\ R}{t_c + (g/C)\ D}$$

be built. Table 12-19 may be entered with knowledge of N_b to find the number that must be provided.

Also note that Table 12-19 applies only to *linear berths.* All other types of multiple berths are *fully* effective.

f. Guidelines—Estimated capacities of *on-line* bus berths are given in Table 12-20. This table shows the number of buses per hour for varying clearance times, dwell times, g/C ratios, and loading positions. The maximum capacities attainable are 2.5 times those for a single berth.

Thus, for a 60-sec dwell time per bus, and a 15-sec clearance time, a 5-berth stop would result in capacities of 82 and 100 buses per hour at g/C ratios of 0.5 and 1.0 respectively.

Figure 12-3 provides a further planning guide for estimating bus berth capacity. It shows the number of buses per hour for selected dwell times and g/C ratios based on a 15-sec clearance time. *Increasing the number of loading positions has a much smaller effect on changes in capacity than reducing dwell times.* Note that for dwell times more than 60 sec, the differences between a g/C of 0.5 and 1.0 are small.

The application of Table 12-20 and Figure 12-3 calls for estimates of the approximate dwell times at the major stops. These can be obtained from field observations or from counts of the number of people boarding each bus and their associated passenger service times. Where such data are lacking or cannot be obtained easily, the following representative values can be used: 60 sec per CBD stop, 30 sec per major outlying stop, and 15 sec per typical outlying stop.

Table 12-20 and Figure 12-3 provide a means of estimating the number of buses per hour that can pass through the busiest stop. The number of people that these buses can carry depends on seated and standing passengers per bus—assuming that these "places" are filled as the bus reaches its maximum load point.

TABLE 12-18. TYPICAL SERVICE LEVELS, SINGLE STOP, NO PASSING (15-SEC BUS CLEARANCE TIME, 60-SEC STOP)

LEVEL OF SERVICE	BUSES PER HOUR (FLOW RATE) $g/C = 0.5$	$g/C = 1.0$
A	13	16
B	20	24
C	26	32
D	30	36
E	33	40

SOURCE: Computed from Tables 12-16 and 12-17.

TABLE 12-19. EFFICIENCY OF MULTIPLE LINEAR BUS BERTHS

BERTH NO.	ON-LINE STATIONS EFFICIENCY %	ON-LINE STATIONS NO. OF CUMULATIVE EFFECTIVE BERTHS	OFF-LINE STATIONS EFFICIENCY %	OFF-LINE STATIONS NO. OF CUMULATIVE EFFECTIVE BERTHS
1	100	1.00	100	1.00
2	75	1.75	85	1.85
3	50	2.25	75	2.60
4	20	2.45	65	3.25
5	5	2.50	50	3.75

NOTE: On-line station figures assume that buses do not overtake each other. In Ref. *3,* efficiency values were (1) 100 percent, (2) 73 percent, (3) 41 percent, (4) 27 percent, and (5) 18 percent. The resulting capacity factors (cumulative) were 1.00, 1.73, 2.14, 2.41, and 2.54.

SOURCE: Refs. *3* and *4*

TABLE 12-20. ESTIMATED CAPACITY OF ON-LINE BUS STOPS BY NUMBER OF BERTHS (BUSES PER HOUR)

	NO. OF BERTHS									
	1		2		3		4		5	
	NO. OF EFFECTIVE BERTHS									
	1.00		1.75		2.25		2.45		2.50	
10-sec Clearance	g/C		g/C		g/C		g/C		g/C	
Dwell Time/Stop	0.50	1.00	0.50	1.00	0.50	1.00	0.50	1.00	0.50	1.00
30 sec	60	75	105	131	135	169	147	184	150	188
60 sec[a]	38	42	66	74	86	94	93	103	95	105
90 sec	28	30	49	52	63	68	69	74	70	75
120 sec	22	23	38	40	50	54	54	56	55	58
15-sec Clearance Dwell Time/Stop										
30 sec	50	67	88	117	112	151	122	164	125	168
60 sec[a]	33	40	58	70	74	90	81	98	82	100
90 sec	25	28	44	49	56	63	61	69	62	70
120 sec	20	22	35	38	45	50	49	54	50	55

[a] Typical bus stop (PM peak)

NOTE: Assumes $R = 0.833$
SOURCE: Computed

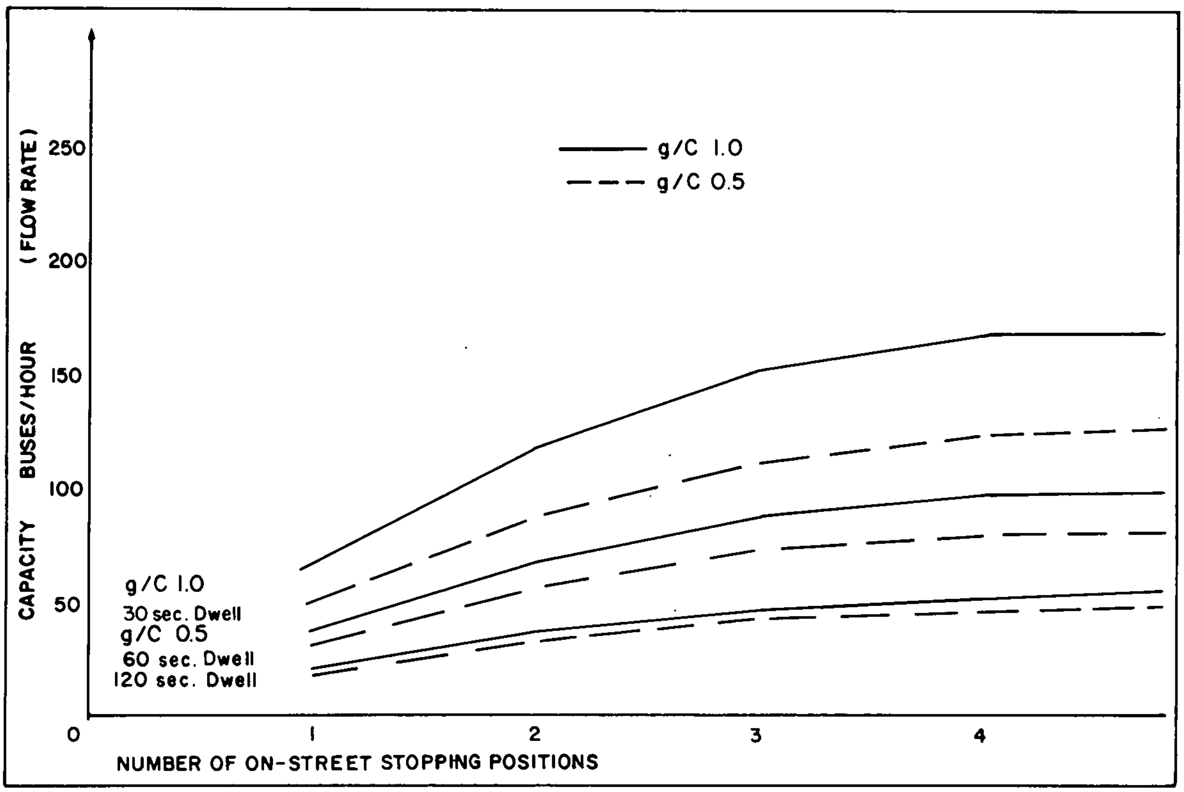


Figure 12-3. Bus stop capacity related to dwell times and loading positions (15-sec clearance between buses).

Thus, the number of people per hour that a bus route can carry depends not only on the dwell times at the busiest stop, but also on the distribution of boarding passengers along the line.

The number of buses per hour that can pass the heaviest boarding point does not in itself determine the number of people per hour along the route. This can be illustrated as follows:

> A dwell time of 60 sec per stop and 15-sec clearance time results in 33 buses per hour for a single loading position. This corresponds to 2,000 people per hour on the bus route assuming 60 people per bus (g/C of 0.5).
>
> The 60-sec dwell time enables 20 people to board each bus assuming a service time of 3 sec per passenger. This translates into 1,200 people per berth per hour.
>
> Consequently, the 2,000 people/bus/hour can be achieved *only* if another 800 people board buses before the maximum load point is reached.

2. *Passenger capacity of a bus berth*—The capacity of a bus berth in *persons* served per hour can be estimated from the following equations. They assume that loading conditions govern and that there are no traffic signal interruptions. Similar equations can be derived based on passenger interchange or alighting. For alighting, K replaces J in these equations.

Maximum passengers per berth per hour, Q

$$Q = f' B = R\ (3{,}600\ B/(bB + t_c) = \frac{3{,}000\ B}{bB + t_c} \qquad (12\text{-}12)$$

Effective berths required, N_b, to serve J passengers per hour

$$N_b = J/Q = \frac{J\ (bB + t_c)}{R\ (3{,}600)\ B} = \frac{bB + t_c}{Rh'} \qquad (12\text{-}13)$$

where R = 0.833

Where traffic signals control bus movements, the following equations should be used:

$$Q = f'\mathrm{B} = (g/C)\,\frac{3{,}600\ BR}{(t_c + Bb\ (g/C))} \qquad (12\text{-}14a)$$

$$N_b = J/Q = \frac{\mathrm{J}\ [t_c + Bb\ (g/C)]}{(g/C)\ 3{,}600\ BR} \qquad (12\text{-}14b)$$

Since R equals 0.833, 3,000 replaces (3,600 R), and Q equals maximum passenger per berth per hour; J equals number of passengers boarding at heaviest stop, per hour (peak-hour flow rate); K equals number of passengers alighting at the heaviest stop per hour, and other symbols are as described before.

Table 12-21 contains illustrative calculations based on these equations for a single berth, assuming that loading conditions govern:

a. Uninterrupted flow conditions ($g/C = 1.00$) would require 2 *effective* berths in the example shown. To achieve 2.0 effective berths, 3 physical berths would have to be provided, with a capacity of 2.25 effective berths.

b. A 0.50 green/cycle would require 2.7 effective berths. Providing 5 berths would result in 2.5 effective berths. With online stations in practice, 5 berths would be provided. To provide sufficient capacity, loading times should be reduced by prepayment, rear-door loading and/or changes in stopping patterns, and signal timing adjustments should be made.

3. *Passenger capacity of a bus route*—The capacity of any busway, bus terminal-approach system, downtown bus street or bus lane will be governed by the number of passengers (a) boarding and/or alighting at the heaviest stop or (b) traveling past the maximum point (between stops), *whichever is less.* The sequence of analysis is as follows when the approach volumes of buses and passengers are specified, and it is desired to estimate the required number of berth positions:

- The maximum load point demand establishes bus frequency requirements in the corridor.
- Bus service frequency and boarding volumes determine the minimum headway per berth. (For planned systems, where no boarding counts are available, the estimated percentage of passengers boarding at the heaviest stop is a key parameter of total passenger capacity.)
- The maximum bus frequency per berth depends on this minimum headway.
- Berth needs or stopping positions are derived from the required bus frequency at the maximum load point and the maximum bus frequency that can load at the heaviest berth.

The following equations show how the maximum load point and heaviest transit stops interrelate. They assume that loading conditions govern. A similar set of equations would apply where passengers alighting (or passenger interchange) dominate and determine capacities.

The capacity of a *bus route* at the maximum (peak) load point is given by the following expression:

$$P = f \times S \qquad (12\text{-}15)$$

where:

P = capacity of bus route past peak load point, in persons/hour;
f = frequency of buses past the peak load point during peak hour; and
S = passengers/bus

Normally, P is derived based on the peak 15-min values for f and S, and adjusted downward to an hourly basis by means of a PHF.

a. <u>Uninterrupted flow, busway or bus terminal</u>—The passengers P at the maximum load point (maximum service volume) can be obtained as follows:

- As a function of boarding passengers per bus at the busiest stop:

$$P = \frac{3{,}600\ R\ N_b S}{(bB + t_c)} = \frac{3{,}000\ N_b S}{bB + t_c} \qquad (12\text{-}16)$$

- As a function of the proportion of the total passengers boarding at the busiest stop: The proportion of passengers boarding at the heaviest stop, X, equals B/S. Thus Eq. 12-16 becomes:

$$P = \frac{3{,}600\ N_b R}{Xb + t_c/S} = \frac{3{,}000\ N_b}{Xb + t_c/S} \qquad (12\text{-}17)$$

- As a function of the passenger capacity per berth:

TABLE 12-21. BUS BERTH PASSENGER CAPACITY EQUATIONS AND ILLUSTRATIVE EXAMPLES—BOARDING CONDITIONS GOVERN (R = 0.833)

CASE 1 UNINTERRUPTED FLOW $g/C = 1.00$

VARIABLES	EQUATION (HOURLY RATES)	EXAMPLES
		Let: t_c = 15sec b = 3 sec/pass. B = 10 pass./bus J = 1,340 boarding pass./hr R = 0.833
Minimum headway at stop	$h' = Bb + t_c$	$h' = 10(3) + 15 = 45$ sec
Maximum buses per berth per hour	$f' = R\ 3{,}600/h' = \dfrac{3{,}000}{Bb + t_c}$	$f' = 3{,}000/45 = 67$ buses/berth/hr
Maximum passengers per berth per hour	$Q = f'B = \dfrac{3{,}000B}{Bb + t_c}$	$Q = 67(10) = 670$ pass./berth/hr
Effective berths required to serve J passengers per hour	$N_b = J/Q = \dfrac{J(\text{Bb} + \text{t}_c)}{3{,}000B}$	$N_b = 1{,}340/670 = 2$ berths
Bus frequency required to serve J passengers per hour	$f = f'N_b = J/B$	$f = 2(67) = 134$ buses

CASE 2 $g/C = 0.50$

VARIABLES	EQUATION (HOURLY RATES)	EXAMPLES
		Same assumptions as Case 1
Minimum headway at stop		$h' = 10(3) + 15 = 45$ sec
Maximum buses per berth per hour	$f' = \left(\dfrac{g}{C}\right)\left(\dfrac{3{,}600\ R}{t_c + Bb\dfrac{g}{C}}\right)$	$f' = (0.5)\left(\dfrac{3{,}600\ R}{15 + 10(3)(0.5)}\right) = 50$
Maximum passenger per berth per hour	$Q = f'B = B\left(\dfrac{g}{C}\right)\left(\dfrac{3{,}000}{t_c + Bb\dfrac{g}{C}}\right)$	$Q = 50(10) = 500$ pass./berth/hr
Effective berths required to serve J passengers per hour	$N_b = J/Q = \dfrac{J\,[t_c + Bb\,(g/C)]}{(g/C)\ 3{,}600\ BR}$	$N_b = \dfrac{1{,}340}{500} = 2.7$ berths
Bus-frequency required to serve J passengers per hour	$f = f'N_b = J/B$	$f = 50(2.7) = 135$ buses

SOURCE: Adapted from Refs. *4* and *34*

$$P = \frac{N_b\,Q}{X} = f \times S \tag{12-18}$$

• Number of berths at the busiest stop as a function of service volume at maximum load point: The number of effective berths at the busiest stop, N_b, to serve P passengers per hour is:

$$N_b = \frac{P\,(Xb + t_c/S)}{3{,}600\ R} = \left(\frac{P}{S}\right)\frac{bXS + t_c}{3{,}600\ R} = \left(\frac{P}{S}\right)\frac{bXS + t_c}{3{,}000} \tag{12-19}$$

Note that R = 0.833.

These equations indicate that the number of bus berths required at the heaviest stop or bus terminal varies directly with the total passengers to be served at that stop, the boarding and alighting service times required per passenger, and the clearance times between buses.

The following *example* shows how the equations can be applied. It is desired to find the total passengers that can be carried past the maximum load point in an hour, based on the peak 15-min flow rate. There is a 20-sec clearance between buses (t_c = 20), and 50 passenger buses with a load factor of 1.00 (S = 50). There is prepayment of fares and the ability to load buses at a rate of 2.0 sec per passenger (b = 2.0). System design anticipates that 50 percent of the total passengers will board at the maximum load point (X = 0.5).

Four off-line berths are provided, i.e., 3.25 effective berths. The busway is grade separated in the central area, and there are no traffic signal interruptions.

The number of people that can be carried on the system can be estimated by applying Eq. 12-17.

$$P = \frac{(3{,}600)\ N_b\ (0.833)}{Xb + t_c} = \frac{(3{,}000)\ (3.25)}{(0.5)(2) + (20/50)}$$
$$= 6{,}964 = 7{,}000 \text{ persons/hr}$$

The actual hourly volume will be less since the capacity represents four times the peak 15-min flow rate. To calculate the hourly volume, a peak-hour factor, PHF, is used. In this example, if the PHF where 0.75, the hourly passenger volume carried would be about 5,250 persons (i.e., 7,000 × 0.75).

This represents level-of-service E, insofar as the movement of buses is concerned. The number of people that can be carried at LOS D would be 0.90 (5,250) or about 4,700 persons per hour.

b. Bus flow interrupted by traffic signals, arterial streets—The preceding equations should be modified as follows to account for the reductive effect of traffic signals.

- As a function of boarding passengers per bus at the busiest stop:

$$P = \frac{3{,}600\ N_b SgR}{C\ [Bb\ (g/C) + t_c]} = \frac{3{,}000\ N_b Sg}{C\ [Bb\ (g/C) + t_c]} \qquad (12\text{-}20)$$

- As a function of the proportion of the total volume boarding at the heaviest stop:

$$P = \frac{3{,}600\ N_b SgR}{C\ [XbS\ (g/C) + t_c]} = \frac{3{,}000\ (N_b Sg)}{C\ [XbS\ (g/C) + t_c]} \qquad (12\text{-}21a)$$

or, by rearranging,

$$P = \frac{3{,}600\ N_b\ (g/C)\ R}{[Xb\ (g/C) + t_c/S]} \qquad (12\text{-}21b)$$

- Number of berths at the busiest stop:

$$N_b = \frac{PC\ [Xb\ (g/C) + t_c/S]}{(g)\ (3{,}600)\ R}$$

$$N_b = \frac{PC\ [(g/C)\ bXS + t_c]}{(g)\ (3{,}600\ S)\ R} \qquad (12\text{-}22)$$

In the following *example,* assume that the busway in the preceding example has signalized intersections in the central area with a g/C ratio of 0.5, then the number of people passing the maximum load point would drop from 7,000 to 5,400 persons per hour. This result reflects the effects of traffic signal operations and is obtained by applying Eq. 12-21b.

$$P = \frac{(0.833)\ (3{,}600)\ N_b\ (g/C)}{[(g/C)\ Xb + t_c/S]}$$

$$P = \frac{3{,}000\ (3.25)\ (0.5)}{(0.5)(0.5)(2) + (20/50)} = 5{,}400$$

Applying a peak-hour factor of 0.75 would result in about 4,100 persons per hour.

c. Guidelines—The general expression for the maximum load point passengers that can be carried for *each* effective berth at the busiest stop provides a simple means of estimating system capacity. This equation is as follows:

$$P_b = \frac{3{,}600\ (g/C)\ R}{[Xb\ (g/C) + t_c/S]} = \frac{3{,}000\ g/C}{[Xb\ (g/C) + t_c/S]} \qquad (12\text{-}23)$$

This equation is similar to Eq. 12-20 except that P_b is keyed to a single berth at the busiest stop, rather than to N_b effective berths.

Typical values for key parameters that should be used with this equation are as follows:

- Busway—prepayment of fares:

  2 sec per passenger
  $t_c/S = 0.50$ max
  Peak-hour factor = 0.67 to 0.85

- Arterial street—pay on entering bus:

  3 sec per passenger
  $t_c/S = 0.30$ to 0.40
  Peak-hour factor = 0.67 to 0.85

Table 12-22 gives values for P_b for uninterrupted flow conditions ($g/C = 1.0$); It should be used for grade-separated busway conditions. Table 12-23 gives values for interrupted flow conditions ($g/C = 0.50$) along city streets.

TABLE 12-22. MAXIMUM LOAD POINT HOURLY PASSENGERS PER EFFECTIVE BERTH AT THE BUSIEST STATION—UNINTERRUPTED FLOW CONDITIONS $g/C = 1.00$ ($R = 0.833$)

RATIO: (CLEARANCE BETWEEN BUSES)/(PASSENGERS PER BUS)	PROPORTION OF PASSENGERS BOARDING AT BUSIEST STOP			
	0.25	0.50	0.75	1.00
	A. 2 SEC/BOARDING PASSENGER			
0.1	5,000	2,720	1,870	1,590
0.2	4,210	2,500	1,770	1,370
0.3	3,750	2.310	1,670	1,300
0.4	3,330	2,140	1,580	1,250
0.5	2,600	2,000	1,500	1,200
	B. 3 SEC/BOARDING PASSENGER			
0.1	3,530	1,870	1,280	970
0.2	3,160	1,770	1,230	930
0.3	2,860	1,670	1,180	910
0.4	2,610	1,580	1,130	880
0.5	2,400	1,500	1,030	860

SOURCE: Computed from Eq. 12-23

TABLE 12-23. MAXIMUM LOAD POINT HOURLY PASSENGERS PER EFFECTIVE BERTH AT BUSIEST STATION—INTERRUPTED FLOW CONDITIONS $g/C = 0.50$ ($R = 0.833$)

RATIO: (CLEARANCE BETWEEN BUSES)/ (PASSENGERS PER BUS)	PROPORTION OF PASSENGERS BOARDING AT BUSIEST STOP			
	0.25	0.50	0.75	1.00
	A. 2 SEC/BOARDING PASSENGER			
0.1	4,280	2,500	1,770	1,370
0.2	3,330	2,140	1,580	1,250
0.3	2,720	1,870	1,420	1,150
0.4	2,310	1,670	1,310	1,075
0.5	2,000	1,500	1,200	1,000
	B. 3 SEC/BOARDING PASSENGER			
0.1	3,160	1,770	1,220	930
0.2	2,610	1,580	1,130	880
0.3	2,220	1,420	1,050	830
0.4	1,930	1,320	980	790
0.5	1,720	1,200	920	750

SOURCE: Computed from Eq. 12-23

Applications

The following sections apply the methodology to busways, arterial streets, and bus terminals. They present key parameters and set forth ranges in capacities that are useful for planning, operations, and design.

1. *CBD busways*—CBD busway capacity can be computed from the preceding equations utilizing appropriate assumptions regarding type of bus used, maximum allowable bus loading, distribution of ridership among CBD stops, peak-hour factor, and type of berth.

a. Bus use—The number of people per bus will depend on (1) size of vehicles (about 50 seats/regular bus to 60 seats/articulated bus), and (2) operating policies with regard to standees. To provide an acceptable level of comfort for express bus commuters with a minimum nonstop run of 3 to 5 mi, the passenger load factor in the peak 15-min period should not exceed 1.00—i.e., there should be a seat available for each passenger. Higher load factors are acceptable on local bus services.

b. Passenger distribution among CBD stops—A reasonable design assumption is that 50 percent of the maximum load point volume is served at the heaviest CBD busway stop—assuming a minimum of three stops in the downtown area. (The Washington-State Street subway station in Chicago accounts for about half of all boarding passengers at the three downtown stops on the State Street subway line.)

c. Peak-hour factor—Peak-hour factors of 0.67 to 0.75 are reasonable, depending on the location and type of operation.

d. Capacity guidelines—Illustrative busway capacity guidelines for central areas are given in Table 12-24 for a variety of bus types and service conditions. The key assumptions are:

(1) The peak load point volume is limited to 50 passengers/bus for standard vehicles and 60 passengers/bus for articulated vehicles; this corresponds to a load factor of approximately 1.00, or level-of-service C, and provides a seat for all passengers. For other load factors, multiply the values cited by the load factor (the number of passengers/seat).

(2) Clearance time is 15 sec.

(3) Fifty percent of the peak load point passengers board at the heaviest stop.

(4) Three loading berths are provided at the heaviest stop with loading and unloading areas separated.

(5) An adjustment factor of 0.75 is applied to all results to allow for on-vehicle turbulence and schedule irregularity and variations in dwell times at major bus stops. This R value corresponds to LOS D as shown in Table 12-17.

(6) For D, a peak-hour factor of 0.67 is used to adjust from peak 15-min flow rates to full-hour volumes.

(7) Fares are prepaid (no fares collected on bus in the CBD); the boarding time is 2 sec per passenger.

Table 12-25 gives the resulting average hourly *passenger service volumes* at the maximum load point for two types of stations and four types of bus operations. (Note that this reflects service level D).

Figure 12-4 shows how the door configuration and number of berths increase the maximum load point capacity. The lower horizontal scale applies to through-station operations where 50 percent of all passengers board at the heaviest stop. The upper scale applies to a single-station situation where all riders board at the major stop. This figure can be used to estimate the number of passengers per hour that can be accommodated by various numbers and types of loading berths.

2. *Bus operations on city streets*—The most common operating condition for bus services is along downtown and arterial streets. Transit capacity estimates are complex in this setting, because:

a. Buses must share the roadway with other vehicles, and are subject to interference from other elements of the traffic stream, such as traffic signals.

b. The number and percentage of buses stopping at each bus stop depend on demand and operational factors such as "bunching" of buses in platoons.

c. Buses are subject to a variety of conflicts at intersections with pedestrians and turning vehicles, which add delay to the transit system.

d. Passenger loading and unloading take place both on the

TABLE 12-24. ILLUSTRATIVE BUS CAPACITY GUIDELINES FOR CBD BUSWAYS

	LOADING CONDITION							
	A		B		C		D	
STATION	ON-LINE	OFF-LINE	ON-LINE	OFF-LINE	ON-LINE	OFF-LINE	ON-LINE	OFF-LINE
Passengers boarding at heaviest stop								
Number of passengers per bus	25	25	25	25	25	25	30	30
Boarding time per passenger, in seconds	2.0	2.0	1.2	1.2	0.7	0.7	0.5	0.5
Total boarding time, in seconds[a]	65	65	45	45	32.5	32.5	30	30
Berth use, in buses per hour								
Maximum buses per hour per berth[b]	55	55	80	80	111	111	120	120
Use factor, for three berths	2.25	2.60	2.25	2.60	2.25	2.60	2.25	2.60
Total for all berths	124	143	180	208	250	289	270	312
Adjusted total for all berths[c]	93	107	135	156	188	217	200	234
Passengers per hour—maximum load point[d]								
Peak[d]—flow rate (15 min × 4)	4,650	5,350	6,750	7,800	9,400	10,850	12,000	14,040
Average[e]—peak hour	3,115	3,570	4,520	5,200	6,300	7,320	8,040	9,360

Loading condition A: Single door conventional bus, simultaneous loading and unloading
Loading condition B: Two-door conventional bus, both doors loading or double-stream doors simultaneously loading and unloading
Loading condition C: Four-door conventional bus, all double-stream doors loading
Loading condition D: Six-door articulated bus, all doors loading

[a] Includes 15-sec bus clearance interval.
[b] Computed based on 3,600 effective sec per hour; ($R = 1$).
[c] Adjusted by a factor of 0.75 to account for turbulence, schedule irregularities, and the like.
[d] Assumes that 50 percent of all passengers board at heaviest stop.
[e] Adjusted by a factor of 0.67 from peak flow rate.
SOURCE: Adapted from Ref. *5*, p. 39, Table 8.

green and red signal, and hence capacities are less than for uninterrupted flow conditions.

Aside from bus berth capacity considerations, the capacity of an arterial street for buses is influenced by the total capacity of the street and the amount of other traffic present.

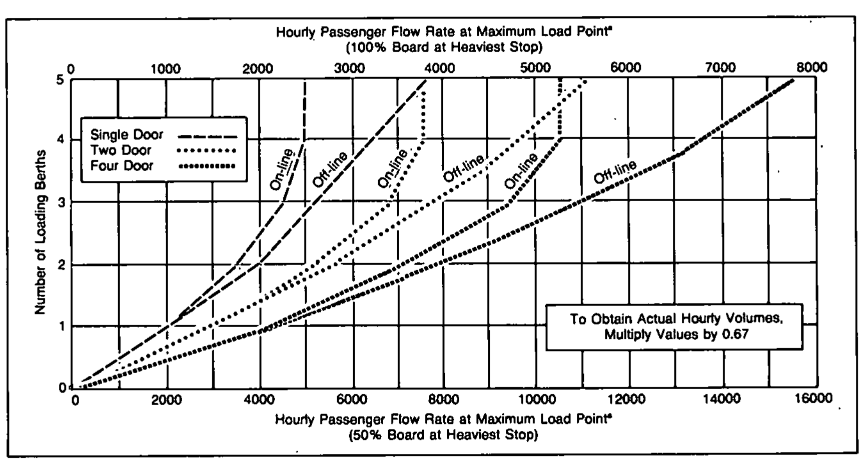
Arterial street bus passenger capacity at maximum load points should be estimated by using the general factors given in Table 12-23. Alternatively, the dwell times given in Table 12-16 can be used, by making assumptions regarding load factors and passenger distribution. In both cases, a maximum of 2.5 effective berths should be used because on-line stopping conditions prevail. Table 12-17 then can be used to adjust for specific levels of service as desired.

TABLE 12-25. BUSWAY SERVICE VOLUMES AT MAXIMUM LOAD POINTS (PASSENGERS PER HOUR)

TYPE OF OPERATION	ON-LINE STATIONS[e]	OFF-LINE STATIONS[e]
Conventional bus[a]		
One door available	3,100	3,550
Two doors available[b]	4,500	5,200
Four doors available[c]	6,300	7,250
Articulated		
60-passenger bus[d]	8,050	9,350

[a] Single door for loading.
[b] Double-door entrance or front and rear single doors with separate or negligible alighting.
[c] Wide double-doors front and rear with separate or negligible alighting.
[d] Six-door channels and separate or negligible alighting.
[e] Three loading positions.

NOTE: Peak 15-minute flow rates would be 50 percent higher, assuming a typical load factor of 0.67.
SOURCE: Summarized from Table 12-24



*Hourly Flow Based on Peak 15 minute Volumes

Figure 12-4. Typical CBD busway line-haul passenger volumes (flow rates).
(Source: Adapted from Ref. *5*, p. 39)

TABLE 12-26. TYPICAL ARTERIAL STREET BUS SERVICE VOLUMES AT MAXIMUM LOAD POINT (SERVICE LEVEL E)

		SEATED PASSENGERS (50 PERSONS/BUS)			50 PERCENT STANDEES (75 PERSONS/BUS)	
CONDITION	APPROX. DWELL TIME AT BUSIEST STOP (SEC)	FLOW RATE	HOURLY VOLUME (PHF = 0.80)	APPROX. DWELL TIME AT BUSIEST STOP (SEC)	FLOW RATE	HOURLY VOLUME (PHF = 0.80)
20 percent board at busiest stop	30	6,250	5,000	45	7,500	6,000
25 percent board at busiest stop	38	5,560	4,450	56	6,270	5,020
30 percent board at busiest stop	45	5,000	4,000	68	5,770	4,620
40 percent board at busiest stop	60	4,170	3,340	90	4,690	3,750
50 percent board at busiest stop	75	3,560	2,850	112	3,950	3,160
Level-of-service E[a] (based on current operating experience) Table 12-12 CBD streets		5,000	4,000		7,500	6,000

Assumptions:

1. 15 Sec clearance between buses.
2. Clearance time (sec)/(pass./bus) = 0.3 for seated and 0.2 for standees.
3. 3-Sec service time per passenger.
4. g/C Ratio of 0.5.
5. All buses stop at busiest stop.
6. 2.5 Effective berths.
7. R factor = 0.833

[a] These values are given for comparison purposes.

Ranges in passenger service volumes are given in Table 12-26 for 50-passenger transit buses. The table gives hourly flow rates and likely hourly passenger volumes for seated loads and for 50 percent standees under varying assumptions regarding passenger distribution among stops and dwell times at the busiest stop. The salient figures, based on a PHF of 0.8 for service-level E, are:

Maximum service volume for dispersed loading conditions	5,000 to 6,000 persons/hour
Maximum service volume for typical CBD conditions (45 sec to 60 sec/stop)	3,300 to 5,000 persons/hour
Maximum service volume for concentrated stop—CBD (75 sec or more/stop)	2,500-3,300 persons/hour
Maximum service volume for CBD conditions—planning method (Table 12-12)	4,000-6,000 person/hour

The data given in Table 12-26 provide a realistic set of parameters for estimating service volumes of arterial streets. Because of the many variables involved, it becomes difficult to select a single "number" for capacity.

These volumes could be adjusted downward to reflect specific service levels by the factors given in Table 12-4, i.e.,

A	0.40
B	0.60
C	0.80
D	0.90
E	1.00

3. *Bus stops*—The number of bus berths at outlying stops should reflect: (a) the number of buses that each stop should accommodate simultaneously during the peak 15-min period; (b) maneuvering requirements of buses to enter and leave the stop; (c) minimum clearance times between buses; (d) the type of stop, and (e) allowable queues.

Equation 12-10b can be used for any given berth configuration and dwell time. Alternatively, maximum capacities can be obtained directly from Table 12-20. In both cases suitable reductions must be made to avoid unacceptably long queues. Accordingly, the following guidelines are suggested.

a. CBD stops—Levels-of-service C and D are acceptable. They result in probabilities of 10 to 20 percent, respectively, that queues will develop beyond the bus stop (Table 12-17).

b. Outlying stops—Level-of-service B should be provided wherever possible, especially when buses must pull into stops from the traveled lane. This results in queues beyond bus stops only 2.5 percent of the time. Level-of-service C is, however, acceptable.

The level-of-service B criteria result in the following equation for estimating bus berth requirements along arterial streets *outside* of the city center:

$$f_d = \frac{1{,}800\ g/C}{h'} = \frac{1{,}800\ g/C}{(g/C)\ D + t_c} \qquad (12\text{-}24)$$

where:

f_d = maximum buses per hour per berth (for service level B);
h' = minimum headway at stop;
D = dwell time = passenger loading time;
t_c = clearance time between buses;
g = green time; and
C = cycle length.

For example, a 30-sec headway between buses (i.e., 20 sec stop, 10 sec clearance), would result in 1,800/30 or 60 buses per hour at a nonsignalized location.

An alternate approach to bus berth requirements in outlying areas is to assume that buses arrive at random. Table 12-27 gives the number of bus berths that should be provided based on the Poisson distribution, and allowing only a 5 percent chance that the berths will overload. Thus, it is a reasonable approximation of level-of-service C. Emergent criteria for arterial (non CBD) bus stop capacity are as follows:

Passenger service times of 20 sec or less: one bus berth per 60 peak-hour buses (this is the typical radial arterial street condition).

Passenger service times of 30 to 40 sec: one bus berth per 30 peak-hour buses.

c. Bus pullouts on exclusive roadways—Bus loading zones on an exclusive roadway (pullout or turnout) within a freeway right-of-way have capacities generally similar to those for curbside loading zones. Here again, the length of the stop and the ability of buses to overtake other buses are important. Given similar loading facilities, differences reflect the length and capacity of the roadway lane leading into and away from the stop. Uninterrupted flow conditions should be used to estimate stop capacities, that is, the g/C ratio should be 1.0 in Eq. 12-24. However, clearance times should be adequate to assure reentry into the main freeway lanes.

Criteria for the spacing, location, and geometric design of bus stops are given in several references (*4, 8, 33*). Such criteria must be carefully applied to assure both good traffic and transit operations.

4. *Bus terminals*—The design of a bus terminal or "transit center" involves not only estimates of passenger service times of buses that will use the center, but also a clear understanding of how each bus route will operate. Therefore, such factors as schedule recovery times, driver relief times, and layovers to meet scheduled departure times become the key factors in establishing berth requirements and sizing the facility. In addition, good operating practice suggests that each bus route, or geographically compatible groups of routes, should have a separate loading position; this is essential to provide clarity for the passengers.

Berth space requirements should recognize the specific type of carrier operations, fare collection practices, bus door configurations, passenger arrival patterns, amount of baggage, driver layover-recovery times, terminal design, and berth configuration. They should reflect both scheduled and actual peak period bus arrivals and departures, since intercity bus services regularly run "extras" during the busiest seasonal travel periods.

Bus route and service patterns also influence berth requirements. Good operating practice calls for a maximum of two distinct routes (i.e., "services") per loading position.

Berth space requirements at major bus terminals can be computed by the preceding equations. However, because passenger service times represent only a small portion of the total time that buses spend at a terminal, the equations will seriously overstate berth capacity unless the other key factors also are considered. It is essential to add the time needed for entering and leaving bus berths, schedule recovery, and driver relief. Bus service times also may be increased to enable buses to meet scheduled departure times. Consequently, it may be necessary to add 5 min or more to computed clearance and dwell times for urban services.

Typical urban transit and commuter bus capacities, based on operating experiences, suggest 8 to 10 buses per berth per hour.

TABLE 12-27. BERTH REQUIREMENTS AT BUS STOPS

PEAK-HOUR BUS FLOW (BUS/HR)	HEADWAY (MIN)	NO. OF BERTHS WHEN SERVICE TIME AT STOP IS 10 SEC	20 SEC	30 SEC	40 SEC	50 SEC	60 SEC
15	4	1	1	1	1	1	1
30	2	1	1	1	1	1	2
45		1	1	2	2	2	2
60	1	1	1	2	2	2	2
75		1	2	2	3	3	3
90		1	2	3	3	4	4
105		1	2	3	3	4	4
120	½	1	2	3	3	5	5
150		2	3	3	4	5	5
180	⅓	2	3	4	5	6	6

Note: 95 percent probability that number of berths will not be overloaded. Assumes a Poisson distribution of bus arrivals

SOURCE: Ref. *4*

Intercity berth capacities are lower, in the range of 1 to 2 buses per hour.

5. *Increasing capacities*—The person capacity of a busway, bus lane or terminal depends heavily on the number of doors per bus, methods of fare collection, and concentrations of passengers at major stops. Consequently, bus system and bus stop capacities can be increased by (a) increasing the number of major downtown (or "terminal") stations on a busway, or bus route, thereby decreasing the number of boarding and alighting passengers at the heaviest stop; (b) reducing the loading and unloading times for passengers through multiple doors on buses, prepayment, and/or selective separation of loading-unloading; and (c) using larger buses (or where feasible higher load factors) to reduce the clearance interval time losses between successive vehicles.

a. Spreading stops—Where the number of buses to be accommodated along a street exceeds the capacity of the busiest stops, routes may be separated into two groups of approximately equal bus volumes. Separated stops can be provided for each group of routes. This requires buses to be able to pass each other, and land use patterns that make the dispersal of stops practical from a passenger standpoint. In such cases, the total number of buses that can be accommodated represents the sum of the capacities for the stops in each group.

b. Reducing dwell times—Dwell times can be reduced by (1) prepayment of fares, (2) use of auxiliary personnel to allow rear-door fare collection and entry, (3) pay-as-you-leave fare collection on outbound trips, (4) removal of sidewalk obstructions at bus stops, (5) dispersal of downtown boarding points where possible, and (6) platooning of buses. The Chicago Transit Authority has been able to handle 45 to 50 buses in 15 min on the State St. Mall by operating buses in 3-bus platoons, and providing auxiliary rear door loading and fare collection in the evening peak hours.

In extreme cases, buses (or trains) cannot be unloaded or loaded at certain stops as rapidly as passengers accumulate (or before the next unit arrives). Thus, the headway that theoretically would be adequate for the demand volume as measured at the maximum load point cannot be delivered as line throughput. Such situations can be alleviated by changing vehicle or stop configuration, using collectors to load rear doors, or having prepaid areas.

BUS PRIORITY TREATMENTS

Over the past decade, much attention has been paid to expediting transit flow by providing various forms of priority treatment. Such treatments are aimed at improving schedule adherence and reducing travel times and delays for transit users. They may attract new riders, increase transit capacity, and/or improve the transit level of service.

A growing number of cities have established exclusive bus lanes and other bus priority measures to improve person-flow over city streets and highways. Bus priority measures are an essential part of transportation system management (TSM) programs that attempt to maximize transport system efficiency consistent with social, economic, and environmental objectives.

Because buses may stop within priority lanes to pick up and discharge passengers, the ability of these lanes to carry people will be affected by loading and unloading time requirements set forth earlier. Guidelines presented in the previous section can be used to estimate capacities. The following section summarizes the pertinent features, planning guidelines, and potential benefits associated with various bus and high-occupancy vehicle priority measures.

Operational Overview

Table 12-28 summarizes the state of the art of bus priority treatments as of January 1985. It groups treatments by type of facility (freeways, arterial streets, and terminals), and within each group it further classifies treatments by type of operation:

1. *Freeways* busways, reserved lanes and ramps.
2. *Arterial streets,* reserved lanes, bus streets, signal preference, and turn permissions.
3. *Terminals,* central and outlying areas.

Most bus priority measures take the form of reserved bus lanes on city streets, usually in the same direction as the general traffic flow. However, the number of bus-only streets—such as State Street in Chicago, Nicollet Mall in Minneapolis, and Chestnut Street in Philadelphia—is increasing.

Busways and reserved lanes on freeways are mainly found or are being proposed in larger American cities, usually with a large downtown employment and heavy peak-hour bus ridership. In the early 1980's, a few medium-sized cities, such as Miami and Portland, installed normal flow freeway bus and car pool lanes, but this tendency has subsided.

Effective distribution of buses in central areas remains an important challenge, and communities are giving this item increased attention. Freeway-related treatments generally provide good access to the CBD perimeter, but do not substantially improve service within the downtown core. Terminals are not always located near major employment concentrations and may require secondary distribution. Because curb bus lanes are not always effective, there have been several efforts to install contraflow bus lanes in downtown areas.

Many bus priority measures have produced important passenger benefits, especially those relating to freeways. Some have achieved time savings of 5 to 30 min—savings that compare favorably with those resulting from rail transit extensions or new systems.

Successful priority treatments are usually characterized by: (1) an intensively developed downtown area with limited street capacity and high all day parking costs, (2) a long-term reliance on public transport, (3) highway capacity limitations on approaches to downtown, (4) major water barriers that limit road access to the CBD and channel bus flows, (5) fast nonstop bus runs for considerable distances, (6) bus priorities on approaches to or across water barriers, (7) special bus distribution within the CBD (often off-street terminals), and (8) active traffic management, maintenance, operations, and enforcement programs (*4*).

Planning Considerations

Planning and implementing bus priority measures requires: (1) a reasonable concentration of bus services, (2) a high degree of bus and car congestion, (3) suitable street and road geometry, and (4) community willingness to support public transport and

TABLE 12-28. SIGNIFICANT EXAMPLES OF BUS PRIORITY TREATMENTS—UNITED STATES AND CANADA (1984–1985)

TYPE OF TREATMENT	SIGNIFICANT EXAMPLES
1. *Freeways*	
A. Busways	
1. Busway on special right-of-way	• Ottawa • South PAT way, Pittsburgh
2. Busway in freeway median or right-of-way	• Shirley Busway (I-95), Washington, D.C. area[a] • San Bernardino Busway, Los Angeles[a] • Gulf Freeway, Houston[b]
B. Reserved Lanes and Ramps	
1. Bus preemption of freeway lanes (peak-hours)	• Ottawa River Pkwy, Ottawa
2. Bus lanes on freeways, normal flow	• U.S. 101, Marin County, California[a] • 9th St. Expressway, Washington, D.C. • I-95, Miami[a] • I-280, San Francisco[a] • Moanalua Freeway, Hawaii[a] • Banfield Freeway, Portland, Ore.[a]
3. Bus lanes on freeways, contraflow	• I-495, New Jersey • Long Island Expressway, N.Y. City • Gowanus Expressway, N.Y. City • U.S. 101, Marin County • North Freeway, Houston
4. Bus lane bypass of toil plaza	• San Francisco Oakland Bay Bridge[a]
5. Exclusive bus access to nonreserved freeway (or arterial) lanes	• I-5 Seattle, Blue Street Express Bus Service & Ramp • Braddock Ave., Pittsburgh • O'Hare Field Connection to Kennedy Expressway, Chicago
6. Metered freeway ramps with bus bypass lanes	• South Capitol St. Bridge Washington, D.C. • Various Freeways, Los Angeles, San Diego • I-35 W, Minneapolis
7. Bus stops along freeway	• Hollywood Freeway, Los Angeles
2. *Arterial Streets*	
A. Reserved Lanes and Streets	
1. Bus tunnels	• Harvard Sq., Cambridge • Providence, Rhode Island
2. Bus streets	• Fifth & Sixth Streets, Portland, Ore. • 10th Street, Washington, D.C. • Nicollet Mall, Minneapolis • State Street, Chicago • State Street, Madison • Chestnut Street, Philadelphia • Granville Street, Vancouver • Halsted and 63rd. Streets, Chicago • Fulton Street, Brooklyn, N.Y.
3. CBD bus lanes, normal flow[c]	• Washington, D.C. • Baltimore, Md. • New York City, N.Y. • San Francisco, California • Rochester, N.Y. (Main Street) • Ottawa, Ont.
4. Dual CBD bus lanes, normal flow	• Madison Ave., N.Y. City
5. Arterial curb bus lanes, normal flow[c]	• Hillside Ave., Queens, N.Y. City • Connecticut Ave., Washington D.C. • Lincoln Ave., Denver • Post. Sutter, Geary, O'Farrel St., San Francisco • Eglinton Ave., Toronto
6. CBD median bus lanes	• Canal St., New Orleans
7. Arterial median bus lanes	• Broadway, Denver • Barbour Blvd., Portland[d] • S. Dixie Highway, Miami[a]

CONTINUED

TABLE 12-28. SIGNIFICANT EXAMPLES OF BUS PRIORITY TREATMENTS—UNITED STATES AND CANADA (1984–1985) CONTINUED

TYPE OF TREATMENT	SIGNIFICANT EXAMPLES
8. CBD curb bus lane, contraflow	• Spring St., Los Angeles • Alamo Plaza, San Antonio • Market St., Harrisburg • Marquette, 2nd. Aves., Minneapolis • Fifth Ave., Pittsburgh • Madison, Washington, • Adams, Jackson Streets, Chicago
9. Arterial curb bus lanes, contraflow	• Ponce de Leon, Fernandez Juncos, San Juan • College Ave., Indianapolis • Kakanianole, Honolulu
B. Miscellaneous	
1. Bus signal preemption	• Barbour Blvd., Portland, Ore. • Kent, Ohio
2. Special signal phases	• Cermak Rd. At Kenton, Chicago • Washington, D.C.
3. Special turn permission[c]	• Los Angeles • Washington, D.C.
3. *Terminals*	
A. Central Area Terminals[c]	• Midtown Terminal, N.Y. City • Transbay Terminal, San Francisco
B. Outlying Transfer Terminals[c]	• Eglinton, Toronto • 95th, Dan Ryan Bus Bridge, Chicago • River Road, Chicago • Pentagon, Washington, D.C. • Wilson, Toronto
C. Outlying Park-and-Ride Terminals[c]	• Route 3 on Lincoln Tunnel Approach at I-495 Contraflow Bus Lane, New Jersey

[a]Includes priority use by car-pools.
[b]Under construction.
[c]Selected examples.
[d]Reversible lane.

SOURCE: Updated from Ref. *33*

to enforce regulations. There is little value in providing bus priority measures where service is poor, costly, or nonexistent; where there are neither buses nor congestion; or where the community has no desire to maintain and improve bus services or to enforce bus lanes.

1. *Objectives*—Planning calls for a realistic assessment of demands, costs, benefits, and impacts. The objective is to apply measures that (a) alleviate existing bus service deficiencies, (b) achieve attractive and reliable bus service, (c) serve demonstrated existing demands, (d) provide reserve capacity for future growths in bus trips, (e) attract auto drivers, and (f) relate to long-range transit improvement and downtown development programs, and (g) have reasonable operating costs.

2. *Factors*—Key factors include: (a) the intensity and growth prospects of the city center; (b) the historic and potential future reliance on public transport; (c) street width, configuration, continuity, and congestion; (d) the suitability of existing streets (and expressways) for express bus service; (e) bus operating speeds and service reliability in the city center; (f) availability of alternate routes for displaced auto traffic; (g) locations of major employment centers in relation to bus routes; (h) goods and service vehicle loading requirements; (i) express and local bus routing patterns; (j) bus passenger loading requirements along curbs; and (k) community attitudes and resources.

Bus priority measures must fit real-world street systems. They must be reasonable, not only in how they improve bus service, but how they impact other traffic as well. Community acceptance and support are essential, especially over the long run. Effective enforcement and maintenance are also necessary elements in priority treatments.

Buses must be able to enter and leave priority lanes easily and safely, and alternative routings must be available for potentially displaced automobile traffic. New problems should not be created, nor should existing problems merely be transferred from one location to another.

Before any treatment is placed into effect, an *a priori* assessment should be made of its benefits and effects. This is important to provide a rational basis for implementing the treatment and to ensure good operations. A commitment also should be obtained from appropriate government agencies regarding enforcement and maintenance. Unless enforcement is strict, frequent violations may occur, undermining the benefits of the priority operations.

Traffic management and bus priority studies of urban freeways are, in reality, freeway operations studies. Demands, queues, and densities, as well as speeds and volumes, should be clearly identified. Various computer models may be used to investigate lane and ramp control strategies.

3. *Principles*—The following principles underly bus priority planning:

a. Bus priorities should be developed as an integrated system of treatments that improve bus speeds and schedule dependability.

b. Bus priority treatments should maximize person-flow and minimize person-delay over the long run. There should be a net saving in the average travel time per person.

c. Priority measures should expedite bus service without adversely impacting general traffic flow.

d. Costs should be reasonable in relation to existing and potential demands and benefits.

e. The benefits resulting from priority measures generally should be proportional to the amount of congestion before the measure was installed.

Guidelines for Specific Treatments

Specific criteria for introducing bus priority measures will vary among cities. The illustrative planning and installation guidelines given in Table 12-29 are based on NCHRP studies (*4*) as updated by more recent research. They are expressed in terms of peak-hour buses and passengers, but they also identify other relevant planning factors. Bus and passenger volumes should be based on future "design year" demands to allow for generated traffic. However base-year (existing) conditions should meet approximately 75 percent of the volume requirements.

TABLE 12-29. SUMMARY OF ILLUSTRATIVE PLANNING GUIDELINES FOR BUS PRIORITY TREATMENTS

TYPE OF TREATMENT	GENERAL APPLICABILITY TO: LOCAL BUS SERVICE	LIMITED-EXPRESS BUS SERVICE	PLANNING PERIOD IN YEARS	DESIGN-YEAR CONDITIONS: RANGE IN MINIMUM ONE-WAY PEAK-HOUR BUS VOLUMES	RANGE IN MINIMUM ONE-WAY PEAK-HOUR BUS PASSENGER VOLUMES	RELATED LAND-USE AND TRANSPORTATION FACTORS
Freeway-Related						
Busways on special right-of-way	x	x	10 to 20	40 to 60	1,600 to 2,400	Urban population, 750,000; CBD employment, 50,000; 20-million sq ft CBD floor space congestion in corridor; save buses 1 min/mi or more.
Busways within freeway right-of-way		x	10 to 20	40 to 60	1,600 to 2,400	Freeways in corridor congestion in peak hour; save 1 min/mi or more.
Busways on railroad right-of-way	x	x	5 to 10	40 to 60	1,600 to 2,400	Not well located in relation to service area. Stations required.
Freeway bus lanes, normal flow		x	5	60 to 90	2,400 to 3,600	Applicable upstream from lane-drop. Bus passenger time saving should exceed other road user delays. Normally acheived by adding a lane. Save buses 1 min/mi or more.
Freeway bus lanes, contraflow		x	5	40 to 60	1,600 to 2,400	Freeways six or more lanes. Imbalance in traffic volumes permits level-of-service D in off-peak travel directions. Save buses 1 min/mi.
Bus lane bypass at toll plaza		x	5	20 to 30	800 to 1,200	Adequate reservoir on approach to toll station.
Exclusive bus access ramp to nonreserved freeway or arterial lane	x	x	5	10 to 15	400 to 600	
Bus bypass lane at metered freeway ramp		x	5	10 to 15	400 to 600	Alternate surface street route available for metered traffic. Express buses leave freeways to make intermediate stops.
Bus stops along freeway		x	5	5 to 10	50 to 100[a]	Generally provided at surface street level in conjunction with metered ramp.
Arterial-Related						
Bus streets	x	x	5 to 10	20 to 30	800 to 1,200	Commercially oriented frontage.
CBD curb bus lanes, main street	x		5	20 to 30	800 to 1,200	Commercially oriented frontage.

CONTINUED

TABLE 12-29. SUMMARY OF ILLUSTRATIVE PLANNING GUIDELINES FOR BUS PRIORITY TREATMENTS CONTINUED

TYPE OF TREATMENT	GENERAL APPLICABILITY TO: LOCAL BUS SERVICE	LIMITED-EXPRESS BUS SERVICE	PLANNING PERIOD IN YEARS	DESIGN-YEAR CONDITIONS: RANGE IN MINIMUM ONE-WAY PEAK-HOUR BUS VOLUMES	DESIGN-YEAR CONDITIONS: RANGE IN MINIMUM ONE-WAY PEAK-HOUR BUS PASSENGER VOLUMES	RELATED LAND-USE AND TRANSPORTATION FACTORS
Curb bus lanes, normal flow	x		5	30 to 40	1,200 to 1,600	At least 2 lanes available for other traffic in same direction.
Median bus lanes	x	x	5	60 to 90	2,400 to 3,600	At least 2 lanes available for other traffic in same direction; ability to separate vehicular turn conflicts from buses.
Contraflow bus lanes, short segments	x		5	20 to 30	800 to 1,200	Allow buses to proceed on normal route, turnaround, or bypass congestion on bridge approach.
Contraflow bus lanes, extended	x	x	5	40 to 60	1,000 to 2,400	At least 2 lanes available for other traffic in opposite direction. Signal spacing greater than 500-ft intervals.
Bus turnouts	x		5	10 to 15	400 to 600	Points of major passenger loadings on streets with more than 500 peak-hour autos using curb lane.
Bus preemption of traffic signals	x		1 to 5	10 to 15	400 to 600	Wherever not constrained by pedestrian clearance or signal network constraints.
Special bus signals and signal phase, bus-actuated	x		1 to 5	5 to 10	200 to 400	At access points to bus lanes busways; or terminals; or where special bus turning movements must be accommodated.
Special bus turn provisions	x		1 to 5	5 to 10	200 to 900	Wherever vehicular turn prohibitions are located along bus routes.

[a] Boarding or alighting passengers in peak hour.

SOURCE: Ref. *4*, p.28

IV. APPLICATIONS AND SAMPLE PROBLEMS

Transportation engineers and planners encounter many problems that involve transit operations and capacities. This section contains sample problems that illustrate the use of the various charts, tables, equations, and procedures. It presents each problem in step-by-step detail, and it fully discusses the results. In practice, many solutions would be shorter and less detailed.

GENERAL APPROACH

Transit capacity estimates require many assumptions regarding passenger distribution, service and dwell times, vehicle clearance and method of operation. It is essential to make reasonable assumptions regarding these factors because they have important effects on transit system capacity.

Capacity of a transit stop or lane depends on the size and loading standards of vehicles, the minimum clearance time between buses or trains at stops, and passenger service times. Passenger service times, in turn, depend on method of fare collection, and door size and configuration. It is important to recognize that these factors are largely determined by transit system operating policy, and may vary from system to system.

It is necessary to identify the controlling bottleneck along any transit route, and to estimate the maximum frequency of service at this point. The passengers per vehicle can be established from field observations, projections, or system policy. Peak-hour load factors should be estimated to relate peak 15-min periods to hourly flows. Berth efficiency factors or, alternatively "unequal loading factors" should be used to discount for the unequal use of a group of buses or trains of rail cars, as appropriate.

Table 12-30 gives the various equations to be used, and shows where each applies. Table 12-31 defines the basic capacity variables used. Table 12-32 identifies the application of each figure and table; and Table 12-33 sets forth suggested planning parameters for use where local experience is unavailable.

1. Equations 12-25a and 12-25b (renumbered Eqs. 12-2a and 12-2b) identify the basic relationships from which other equations flow. The number of *transit vehicles* per hour per channel or stop that can move past a critical point, assuming no signal interruptions is expressed as

$$c_V = \frac{3{,}600\ R}{D + t_c} \qquad (12\text{-}25a)$$

R is used to adjust for irregularities in dwell times, arrival rates, or for varying levels of service (LOS). Additional adjustments are made to compensate for the reductive effect of signal timing. R is assumed as 0.833; thus 3,000 replaces the (3,600 R) in the equation.

2. The number of vehicles that can pass through the heaviest boarding point is limited by the passengers that board and alight there. *If* these vehicles are able to be filled to their *maximum* seated and/or standing loads, Eq. 12-25b, that is, applies directly, for each effective loading position.

$$c_p = nSc_v = \frac{3{,}600\,nSR}{D + t_c} \tag{12-25b}$$

3. Where traffic signals are involved, the dwell time D is reduced by g/C and the entire expression is then reduced by g/C. In this case, Eq. 12-25c applies:

$$c_p = \frac{3{,}600\,nSR\,(g/C)}{(g/C)\,D + t_c} \tag{12-25c}$$

4. Where the distribution of passengers along a bus (or rail) route limits the numbers of vehicle that can get on at other points along the line, then it is necessary to apply Eqs. 12-16; or some derivative of it, i.e., Eq. 12-20 or Eq. 12-21.

5. The number of *effective* bus berths at a stop can be estimated from Eq. 12-13, or Eq. 12-14. Factors then can be applied to estimate the *actual* number of berths that should be provided.

6. Along arterial streets, where the curb lane is used by parked cars, it is essential that bus stops are long enough to prevent buses from backing out into the traffic lane. For this, and for other arterial street design purposes, Eq. 12-24 is used. In effect, the value 1,800 replaces the term (3,600 R) in Eq. 12-9a or Eq. 12-10b.

7. On-street rail transit operation is similar to bus operations except for differing car lengths, seating configurations, and door arrangements. Estimates of passenger dwell times at a stop must recognize the unequal loading among doors. Clearance times should consider train length.

TABLE 12-30. SUMMARY AND APPLICATIONS OF TRANSIT CAPACITY EQUATIONS

EQ. NO.	EQUATION		APPLICATION
12-1	$c_p = f'\,o_1 + [(1{,}800 - 1.5\,f_1)\,o_2]$		Person capacity of a freeway lane.
12-2a	$c_v = \frac{3{,}600\,R}{h} = \frac{3{,}600\,R}{D + t_c}$		General equation—number of vehicles past a critical point, per channel or berth, uninterrupted flow
12-2b	$c_p = nSc_v = \frac{3{,}600\,nS}{D + t_c}R$		General equation—number of people past a critical point, per channel or berth, uninterrupted flow
12-2c	$c_p = nSc_v = \frac{(g/C)\,3{,}600\,nSR}{(g/C)\,D + t_c}$		General equation—number of people past a critical point, per channel or berth, flow interrupted by traffic signals
12-2d	$S_i = s_n + \frac{A_n}{L_i}$		Passengers per vehicle based on number of seats and square feet per standee
12-3	$T_L = (g/C)\,N(D + L)$		Time loss, seconds per hour, resulting to queues in same lane as buses stopping for passengers
12-4	HV = (Peak 15-min volume) (4) (PHF)		Determining hourly service volume
12-5a	$P = \frac{\text{Trains}}{\text{Hour}} \times \frac{\text{Cars}}{\text{Train}} \times \frac{\text{Seats}}{\text{Car}} \times \frac{\text{Pass.}}{\text{Seat}}$		Rail transit capacity, passengers per hour
OR,			
12-5b	$P = \frac{\text{Cars}}{\text{Hour}} \times \frac{\text{Seats}}{\text{Car}} \times \frac{\text{Pass.}}{\text{Seat}}$		
12-6	$P = \frac{\text{Trains}}{\text{Hour}} \times \frac{\text{Cars}}{\text{Train}} \times \frac{\text{Ft}^2}{\text{Car}} \Big/ \frac{\text{Ft}^2}{\text{Pass.}}$		
12-7	$f' = \frac{3{,}600\,R}{h'} = \frac{3{,}600\,R}{D + t_c}$		Buses per hour at critical stop (no interruptions) general equation
12-8a	$h' = bB + t_c$	Boarding	Minimum headway at a bus stop
12-8b	$h' = aA + t_c$	Alighting	
12-8c	$h' = aA + bB + t_c$	Two-way flow	

CONTINUED

TABLE 12-30. SUMMARY AND APPLICATIONS OF TRANSIT CAPACITY EQUATIONS CONTINUED

EQ. NO.	EQUATION		APPLICATION
Note: R is assumed as 0.833 in formulas that follow:			
12-9a	$f' = \frac{3{,}600R}{bB + t_c} = \frac{3{,}000}{bB + t_c}$ Boarding		Maximum buses per berth per hour, uninterrupted flow; busway, terminal
12-9b	$f' = \frac{3{,}600R}{aA + t_c} = \frac{3{,}000}{aA + t_c}$ Alighting		
12-9c	$f' = \frac{3{,}600R}{aA + bB + t_c} = \frac{3{,}000}{aA + bB + t_c}$ Two-way flow		
12-10a	$f'_c = \frac{g}{t_c + D(g/C)}$ per cycle		Maximum buses per berth, signal interrupted per cycle (12-10a) per hour (12-10b)
12-10b	$f' = \frac{(g/C)3{,}600R}{t_c + D(g/C)} = \frac{(g/C)3{,}000}{t_c + D(g/C)}$ per hour		
12-11	$c_{vi} = \frac{(g/C)3{,}000(\text{LOS Factor})_i}{t_c + D(g/C)}$		City street per hour at level-of-service i
12-12	$Q = \frac{3{,}600RB}{bB + t_c}$		Max. boarding pass. per berth per hour, uninterrupted flow, busway, terminal
12-13	$N_b = \frac{J(bB + t_c)}{(3{,}600)R(B)} = \frac{bB + t_c}{h'R} = \frac{bB + t_c}{0.833h'}$		Number of effective berths to serve a given passenger flow(s), uninterrupted flow, busway, terminal
12-14a	$Q = (g/C)\frac{3{,}600RB}{t_c + Bb(g/C)}$		Max. pass. per berth per hour with traffic signal interruptions, city street
12-14b	$N_b = \frac{J[t_c + Bb(g/C)]}{(g/C)\ 3{,}600RB}$		Number of effective berths to serve a given passenger flow with traffic signal interruptions, city street
12-15	$P = f \times S$		Max load point pass./hour based on bus frequency and load factor
12-16	$P = \frac{3{,}600RN_bS}{bB + t_c}$	As a function of number of boarding passengers at busiest stop	Passenger capacity at max. load point, uninterrupted flow, busway
12-17	$P = \frac{3{,}600RN_b}{Xb + (t_c/S)}$	As a function of proportion of passengers boarding at busiest stop	Psssenger capacity at max. load point, uninterrupted flow, busway
12-18	$P = \frac{N_bQ}{X}$	As a function of passenger capacity per berth	Passenger capacity at max. load point, uninterrupted flow, busway
12-19	$N_b = \frac{P(Xb + t_c/S)}{3{,}600R} =$		Number of effective berths at busiest stop, uninterrupted flow
	$N_b = (P/S)\frac{bXS + t_c}{3{,}600R}$		Keyed to pass volume at max. load point, busway, or terminal
12-20	$P = \frac{3{,}600Rg\ N_bS}{C[Bb(g/C) + t_c]}$	Function of no. of boarding passengers at busiest stop	Passenger capacity at max. load point, signals interrupt flow (City street)
12-21a	$P = \frac{3{,}600N_bSgR}{C[XbS(g/C) + t_c]}$	Function of proportion of passengers boarding at busiest stop	Passenger capacity at max. load point, signals interrupt flow (City street)
12-21b	$P = \frac{3{,}600R\ N_b(g/C)}{[Xb(g/C) + t_c/S]}$		
12-22	$N_b = \frac{PC[Xb(g/C) + t_c/S]}{(g)(3{,}600)R}$		Number of effective berths at busiest stop, signals interrupt flow, keyed to pass volume at max. load point
12-23	$P_b = \frac{3{,}600R(g/C)}{[Xb(g/C) + t_c/S]}$		Line-haul passenger capacity at maximum load point per effective berth, all applications/general equation
12-24	$f_d = \frac{1{,}800(g/C)}{(g/C)D + t_c}$		Design capacity of a bus stop, service-level B; stops along outlying arterial route, best applications

TABLE 12-31. BASIC TRANSIT CAPACITY VARIABLES

SYMBOL	DESCRIPTION
A	Alighting passengers per bus measured in peak 15 min
A_n	Net area available on a transit vehicle for standees
a	Alighting service time per passenger, in seconds
B	Boarding passengers per bus measured in peak 15 min
b	Boarding service time per passenger, in seconds
C	Cycle length, in seconds
c'_b	Design capacity of a bus stop in buses per hour
c_v	Buses per hour per channel
$c_{v(i)}$	Buses per hour at level-of-service i
c_p	People per hour per channel
D	Bus dwell time at bus stop, in seconds (time when doors open and bus is stopped)
f	Bus frequency, in buses per hour (all routes using the facility), at maximum load point (if all buses stop at all stations, $f = (N)f'$)
f'	Maximum peak bus frequency at a berth, in buses per berth per hour
f'_c	Bus frequency at a Berth, in buses per cycle
f'_d	Design bus frequency, in buses per berth per hour
g	Green + yellow time per cycle
H	Alighting passenger capacity per berth per hour
HV	Hourly volume, vehicles or passengers in an hour
h	Bus headway on a facility, in seconds, at maximum load point; for cars, h is the headway between successive vehicles, in seconds
h'	Minimum bus headway at a berth, in seconds ($h' = 3{,}600/f'$)
J	Passengers boarding at heaviest stop, per hour
K	Passengers alighting at heaviest stop, per hour
L	Additional time loss due to stopping, starting, and queuing, in seconds
L_i	Net square feet per standee for level-of-service i
N	Buses per hour that stop at given location
N_b	Number of effective berths at a bus station or stop ($N = N' \times u$)
N'_b	Number of berth spaces provided in a multiberth station
n	Number of vehicles per unit, i.e., cars per train
O_1	Bus occupancy (in peak 15-min) along freeway (passengers per hour)
O_2	Car occupancy (in peak 15-min) along freeway (passengers per car)
P	Linehaul capacity of a bus facility, in persons per hour, past the maximum load point (hourly flow rate on maximum 15 min)
P_b	Unit linehaul capacity of a bus facility in persons per hour, at the maximum load point, based on a single berth at the busiest stop (hourly flow rate based on busiest 15 min)
PHF	Peak-hour factor
Q	Boarding passenger capacity per berth per hour
R	Reductive factor to compensate for variations in dwell time or bus arrivals, also can be used to obtain levels of service
S	Passengers on bus or rail car (varies with design and policy, may include seated passengers and standees)
S_i	Passengers/vehicle or passenger spaces/vehicle, for service level i
s_n	Seats per transit vehicle.
T	Total time at a stop = dwell time plus clearance time
T_L	Time loss, seconds per hour, resulting from buses blocking cars at a stop
t_c	Clearance time between successive buses, in seconds (time between closing of doors on first bus and opening of doors on second bus)
u	Berth utilization factor (an efficiency factor applied to the total number of berths to estimate realistic capacity of multiberth stations ($u = N_b/N_b'$)
X	Proportion of maximum load point passengers that board at heaviest stop ($X = J/P = B/S$)
Y	Proportion of maximum load point passengers that alight at heaviest stop ($Y = K/P$)

SOURCE: Adapted from Ref. *4*, p.41.

TABLE 12-32. SUMMARY AND APPLICATION OF TRANSIT CAPACITY FIGURES AND TABLES

EXHIBIT NUMBER	DESCRIPTION	APPLICATION
Table 12-1	Peak-hour use of public transit by persons entering or leaving the central business district	Informational
Table 12-2	Important terms in transit capacity	Informational
Table 12-3	Factors that influence transit capacity	Informational
Figure 12-1	Example of freeway person capacity	Informational
Figure 12-2	The two-dimensional nature of transit level of service	Informational
Table 12-4	Characteristics of transit vehicles	Informational
Table 12-5	Levels of service for bus transit vehicles	Informational
Table 12-6	Levels of service for rail transit vehicles	Informational
Table 12-7	Typical space requirements for seated and standing passengers	Estimating the passengers on vehicles for varying seating configurations
Table 12-8	Passenger equivalency of urban buses at signalized intersections	Adjustments in intersection capacity
Table 12-9	Passenger boarding and alighting times related to service conditions	Estimates of boarding and alighting coefficients
Table 12-10	Typical bus passenger boarding and alighting service times for selected bus types and door configurations	Estimates of boarding and alighting coefficients
Table 12-11	Suggested bus flow service volumes for planning purposes	Planning estimates of bus service volumes on city streets
Table 12-12	Suggested bus passenger service volumes for planning purposes	Planning estimates of bus passenger service volumes
Table 12-13	Reported rail rapid transit peak-hour passenger volumes	Informational; analogy comparisons
Table 12-14	Reported light rail (street car) peak-hour passenger volumes (in peak direction)	Informational; analogy comparisons
Table 12-15	Typical rail transit capacities	Estimate rail transit capacities and passenger service volumes
Table 12-16	Estimated maximum capacity of bus stops	Bus berth capacity and berth requirements
Table 12-17	Suggested levels of service for bus stops	All level-of-service computations for design purposes
Table 12-18	Typical service levels, single stop	Detailed capacity data for 15-sec bus clearance, 60-sec dwell time
Table 12-19	Efficiency of multiple linear berths	Capacity provided by more than one berth
Table 12-20	Estimated capacity of on-line bus stops	Detailed bus stop by number of berths capacity. Data for 10 and 15-sec clearance and 30,60, 120-sec dwell times
Figure 12-3	Bus stop capacity related to dwell times	Detailed bus stop and loading positions capacity. Data for 15-sec clearance and 30,60, 120-sec dwell times
Table 12-21	Bus berth passenger capacity equations and illustrative examples	Informational
Table 12-22	Maximum load point hourly passengers per effective berth at the busiest station, uninterrupted flow conditions	Estimate number of berths for a given flow at max. load point. Also, estimate flow at max. load point for a given number of berths
Table 12-23	Maximum load point hourly passengers per effective berth at the busiest station	
Table 12-24	Illustrative bus capacity guidelines for CBD busways	Informational
Table 12-25	Busway service volumes at maximum load point	Estimate passsenger service volume at max. load point for various types of operation
Figure 12-4	Typical CBD busway linehaul passenger volumes flow rates	Estimate berth requirements for given busway passenger flow and conversely
Table 12-26	Typical arterial street service volumes at maximum load point	Design and operations— estimate maximum passenger capacities and service volumes
Table 12-27	Berth requirements at bus stops (outlying locations)	Alt. approach to design of bus berths at outlying locations
Table 12-28	Significant examples of bus priority treatments— U.S. and Canada	Informational
Table 12-29	Summary of illustrative planning guidelines for bus priority treatments	Informational for planning decisions

TABLE 12-33. GUIDELINES FOR APPLICATION

1. *Boarding Times per Passenger—Pay Fare on Vehicle (Single Vehicle)*

   Low-level platform—single door:

   2.6 sec single coin fare
   3.0 sec exact fare (general values)
   3.5 sec exact fare (standees on bus)

2. *Alighting Times Per Passenger—Low Level Platform*

   1.7 to 2.0 sec (use 2 sec)

3. *Boarding and Alighting—Heavy Two-Way Passenger Flows*

   Through a single door:

   $1.2\,(2\,A + 3\,B)$

   where: A = alighting passengers/bus
   B = boarding passengers/bus

4. *Clearance Between Successive Buses*

   15 sec—desirable minimum
   20 sec—minimum for operations on high-speed roadway

5. *Levels of Service—Buses*

LOS	R	$3{,}600R$	Proportion of E (LOS Index)	Approximate Probability of Queue Forming Behind Bus Stop
A	0.400	1,200	0.40	< 1
B	0.500	1,800	0.60	2.5
C	0.667	2,400	0.80	10
D	0.750	2,700	0.90	20
E-Capacity	0.833	3,000	1.00	30
E-Capacity (Perfect conditions)	1.000	3,600		50

6. *High-Level Platform*

   Boarding-prepayment 1.0 to 1.5 sec
   Alighting-prepayment 1.0 to 1.5 sec

TYPES OF PROBLEMS

Many kinds of problems can be addressed by the transit capacity analysis procedures. A common problem from the perspective of the transit agency is to determine how many vehicles are needed to carry a given number of riders and to see if these vehicles can be accommodated at the major boarding points. The solution is simple if only one transit line is involved, but it may become more complex where several routes converge. Solution of this problem calls for establishing load factor criteria (i.e., persons per vehicle) and identifying dwell times, berth requirements, fare collection practices, and bus stopping patterns in the central terminal area.

Typical problems that can be solved by the procedures in this chapter include the following:

1. *Person-flow*—Using car and bus occupancies, estimate the total person-flow for an arterial street or freeway.

2. *Person-capacity*—Using observed car and bus occupancies, and the present mix of transit vehicles in the traffic flow, estimate the total person-capacity.

3. *Effect of buses on highway capacity*—

a. Freeway—For a freeway traffic lane carrying mixed traffic (automobiles, buses), estimate the capacity reduction resulting from buses and the passenger car equivalent (PCE) volume.

b. Arterial street—For a lane carrying mixed traffic along an urban arterial, estimate the losses occurring to auto traffic and the resulting PCE values corresponding to the operations of buses making stops, using berths located either in a through lane (on-line) or in a separated area (off-line).

4. *Passenger service times*—Estimate the passenger service times (a) at a stop and (b) along a bus route for various boarding and alighting characteristics, fare collection methods, and bus door configurations.

5. *Arterial street bus capacities and service levels planning applications*—Estimate the level of service for a specified bus passenger volume along an arterial street.

6. *Bus berth capacity*—Estimate the capacity of a bus berth for given passenger loading and unloading characteristics; alternatively estimate the number of berths needed for a given passenger volume.

7. *Bus terminal capacity*—Estimate the number of loading positions needed to accommodate given passenger and bus volumes consistent with operating criteria.

8. *Bus system (route) capacity*—Estimate the capacity of an arterial street or busway in passengers per hour. Alternatively, estimate berth requirements at major stops to serve a specified transit flow.

9. *Design capacity, arterial street bus stops*—Estimate the number of berths needed to serve a given bus flow and dwell time for design purposes.

10. *Rail transit capacity*—Estimate the number of people per hour that can be carried past the maximum load point for a specified train length and level of service (i.e., private right-of-way).

11. *Light rail transit*—Estimate the number of people per hour that can be carried past the maximum load point, with on-street operations and traffic signal control.

These problems are mainly defined in terms of bus transit. However, many can also apply to light rail transit. The problems cited are illustrated in the sample calculations that follow.

SAMPLE CALCULATIONS

Calculation 1—Person-Flow

1. *Description*—A given urban freeway carries 4,500 cars and 50 buses in the peak hour. Sample vehicle occupancy counts show 1.3 for cars and 50 for buses, respectively. Find the person-flow.

2. *Solution*—The total flow represents the sum of the number of people carried by each type of vehicle. Data can be tabulated as follows:

	Veh/hr	People/veh	People/hr
Cars	4,500	1.3	5,850
Buses	50	50.0	2,500
Total	4,550		8,350
Percent Bus	1.1		29.9

The total person flow is 8,350. Buses represent 1.1 percent of the total traffic and account for 29.9 percent of the total person flow.

Calculation 2—Person-Capacity

1. *Description*—A four-lane urban freeway (two lanes in each direction) has a capacity of 1,800 passenger car equivalents per lane per hour. Car occupancy averages 1.5 people per car. It is planned to initiate express bus service with 100 buses per hour, and each bus is estimated to carry 50 people. The buses will be restricted to one lane. It is desired to find the one-way, peak-hour, person capacity of the freeway. Each bus is assumed as 1.5 equivalent passenger cars.

2. *Solution*—The person capacity of the freeway lane where no buses will operate is 1,800 × 1.5 or 2,700 people.

The person capacity of the lane with bus and car traffic can be estimated by using Eq. 12-1:

$$\text{Person-capacity} = [f_1 \times O_1] + [(1{,}800 - 1.5 f_1) \times O_2]$$

where

f_1 = number of buses/hour;
O_1 = bus occupancy, 50 people/bus; and
O_2 = car occupancy, 1.5 people/car.

Thus, the person capacity of the shared lane is:

$$[100 \times 50] + [1.5 \times (1{,}800 - (1.5)(100))] = 5{,}000 + 2{,}475 = 7{,}475 \text{ people}$$

The person-capacity of the two lanes is 2,700 + 7,475, or 10,175 people.

The effects of various bus volumes on the person capacity of the shared freeway lane are given in Table 12-34.

3. *Comment*—In some situations, such as a downtown street, with a bus lane, the person capacity of the bus lane should be estimated and added to that of the other lanes. Note that this represents the maximum potential person capacity, while the example computed the person-capacity under *prevailing* or likely conditions of flow.

Calculation 3—Effect of Buses on Freeway Capacity

1. *Description*—Ninety buses operate in the peak direction of a four-lane freeway during the peak hour. The freeway also carries 3,400 passenger cars in this direction. Average occupancies are 40 persons/bus and 1.4 persons per car.

It is desired to find: (a) the equivalent peak hour, peak direction passenger car volume; (b) level of service, assuming 12-ft lanes, no lateral obstructions, and 70-mph design speed; and (c) the total person-volume.

2. *Solution*—It is reasonable to assume that each bus is the equivalent of 1.5 passenger vehicles. Therefore, 90 buses are the equivalent of 135 cars (90 × 1.5 = 135). The equivalent passenger car volume is 3,400 plus 135, or 3,535. Service volumes for LOS E range from 3,100 × PHF to 3,700 × PHF. If PHF = 0.90, the volumes are 2,790 vph to 3,330 vph (see Chapter 3). This indicates that the freeway is operating at LOS E.

The total person-volume is calculated as follows:

TABLE 12-34. PERSON-CAPACITY OF A FREEWAY LANE FOR VARYING BUS VOLUMES

CONDITION AND VEHICLE CAP. BEFORE BUSES		BUSES (VEH)	BUSES (PCE'S) 1 BUS= 1.5 PCE'S	NO. OF PASS. CARS	PEOPLE BY BUS	PEOPLE BY CAR	PERSON CAP.
1	1,800	0	0	1,800	0	2,700	2,700
2	1,800	50	75	1,775	2,500	2,660	5,160
3	1,800	100	150	1,750	5,000	2,630	7,630
4	1,800	150	225	1,725	7,500	2,590	10,090
5	1,800	200	300	1,700	10,000	2,550	12,550

Buses :	90 at 40 people/bus	= 3,600 (43 percent)
Cars :	3,400 at 1.4 people/car	= 4,760 (57 percent)
	Total	= 8,360 (100 percent)

Calculation 4—Effect of Buses on Arterials

1. *Description*—Sixty buses per hour operate along an arterial street with an average dwell time of 15 sec per stop. Find the reduction in available green time to the lane in which buses stop if (a) buses stop in the adjacent parking lane, and (b) buses stop in the through-traffic lane. Assume that the capacity of the through lane is 1,500 cars per hour of green, and that the green/cycle, g/C, time is 0.50, giving a capacity of 750 cars per hour.

What is the time loss per hour in each case? What percentage of total lane capacity is required for bus operation? How can this be translated into a PCE value?

2. *Solution*—For case (a), buses stop in parking lane, the time loss to a right-hand through lane when buses stop in the adjacent parking lane is due to acceleration and deceleration of the bus while entering and leaving the through lane. It has been noted in the section entitled "Effects of Buses on Vehicular Capacity" that this loss averages 3 to 4 sec per bus. Using 4 sec, it follows that:

$$\text{Time loss/hour} = 4 \times 60 = 240 \text{ sec/hour}$$

As the g/C ratio is 0.5, the total green time/hour available to the through lanes is $0.5 \times 3{,}600 = 1{,}800$ sec/hr. The percent loss in lane capacity may be expressed as:

$$\frac{240}{1{,}800} \times 100 = 13.3 \text{ Percent}$$

It results in *a capacity loss of 100 passenger cars per hour:*

$$750 \text{ pcph} \times 0.133 = 100 \text{ pcph}$$

In that one lane the passenger car equivalent (PCE) for this condition represents the ratio of the pcph loss in capacity divided by the number of buses/hr causing the loss, or:

$$\text{PCE} = 100/60 = 1.67$$

Note that the headway of each bus is, in effect, 4 sec as compared with 2.4 for cars. Thus, 900 buses/hour would be the equivalent of 1,500 cars. Each bus, therefore, has the equivalency of 1,500/900, or 1.67 cars.

For case (b), buses stop in through lanes, the time loss for buses stopping in a through lane is computed using Eq. 12-3:

$$T_L = (g/C) \times (N) \times (D + L)$$

where:

g/C = 0.50 (Given);
N = 60 buses/hour (Given);
D = 15 sec/bus (Dwell time, Given);
L = 6 sec/bus (Loss time, avg. conditions assumed); and
$T_L = (0.50)\ (60)\ (15 + 6) = 630$ sec/hour.

Then, the percent reduction in lane capacity is:

$$\frac{630}{1{,}800} \times 100 \text{ Percent} = 35 \text{ Percent}$$

and the capacity loss is:

$$0.35 \times 750 = 262 \text{ pcph}$$

This results in a PCE value of:

$$\text{PCE} = 262/60 = 4.37$$

Other lanes are not affected. Also note that buses stopping in a through lane have over 3 times the effect of buses stopping in a parking lane for this case.

Calculation 5—Passenger Service Times (Bus Stop)

1. *Description*—Field observations show that 15 passengers board each bus and 5 alight at a given stop during the peak hour. Assuming on-vehicle fare collection with an "exact fare" and a single door, find the passenger service and dwell times. If a rear door is available for alighting passengers, find the service time.

2. *Solution*—The passenger service dwell times can be estimated by applying Eqs. 12-8 (a,b,c), as follows, using passenger service rates of 3 sec per boarding passenger and 2 sec per alighting as drawn from the section on "Passenger Service Times" and Tables 12-9 and 12-10:

	Service Time	Clearance
1. Single door (entering and exiting)	$h = Aa + bB$	$+ t_c$
	$h = 15\ (3) + 5\ (2)$	$+ t_c = 55 + t_c$
2. Single door (entering only)	$h = bB$	$+ t_c$
	$h = 15\ (3)$	$+ t_c = 45 + t_c$

Thus, the passenger service times would be 55 sec for entering and exiting through a single door, and 45 sec if a rear door is available for exiting. The clearance times normally include the door opening and closing times, about 5 sec. Therefore, the total time spent at the stop for the two sets of conditions is 60 and 50 sec, respectively, when door opening and closing times are considered.

Calculation 6—Passenger Service Times (Bus Routes)

1. *Description*—The following values represent the number of passengers boarding and alighting each bus on a selected bus route:

Stop No. ⟶	1	2	3	4	5	6	7	8
Alighting Pass. (A)	0	2	2	5	8	15	25	10
Boarding Pass. (B)	20	10	10	15	10	1	1	0

Passengers board and alight through a single door. A $0.50 exact fare is used. Compute the dwell time at each stop. What is the total dwell time for the route? Consider the effects of lost time due to opening and closing doors.

2. *Solution*—From Tables 12-9 and 12-10, the average boarding time per passenger for the conditions given would be 2.6 to 3.0 sec (use $b = 2.8$ sec), and the typical alighting time is $a = 1.7$ sec.

For boarding and alighting through a single door, the dwell time is given by:

$$aA + bB$$

Thus, for each bus stop:

Stop		
1	0 (1.7) + 20 (2.8) =	56.0 sec
2	2 (1.7) + 10 (2.8) =	31.4 sec
3	2 (1.7) + 10 (2.8) =	31.4 sec
4	5 (1.7) + 15 (2.8) =	50.5 sec
5	8 (1.7) + 10 (2.8) =	41.6 sec
6	15 (1.7) + 1 (2.8) =	28.3 sec
7	25 (1.7) + 1 (2.8) =	45.3 sec
8	10 (1.7) + 0 (2.8) =	17.0 sec
Total Time		301.5 sec

The time lost in opening and closing doors would amount to another (8 × 5) or 40 sec. Thus, the total time lost at stops would be 341.5 sec, or almost 6 min.

Note that because of the heavy passenger interchange at stops 4 and 5, one could increase these time values about 20 percent, (i.e., 0.20 (50.5 + 41.6)). This would add 18.3 sec, resulting in a total dwell time of eactly 6 min.

Calculation 7—Planning Applications, Downtown Street, Level of Service

1. *Description*—Field observations show that a CBD street carries 4,500 passengers in 80 buses, during the peak hour, based on peak 15-min flow rates. At what level of service does this street operate?

2. *Solution*—The *approximate* level of service can be estimated from Table 12-11 or Table 12-12. The 80 buses per hour produce level-of-service D, verging on level-of-service E, in terms of bus flow. Referring to Table 12-12, level-of-service D from a passenger perspective has a passenger volume range of 4,000 to 5,000, based on 80 buses per hour. Thus, the bus routes operate at level-of-service D from both the traffic flow and passenger standpoint.

Note that the lower half of Table 12-11 and Table 12-12, pertaining to downtown streets was used in making this broad planning assessment.

Calculation 8—Bus Terminal (Transit Center)

1. *Description*—It is desired to estimate "base year" 1985, and "design year" 2000, berth requirements for an outlying transit center.

The bus lines serving the proposed transit center, as identified by the transit agency, are shown in Table 12-35. The 1985 data are based on actual schedules, while the 2000 data are based on a forecast of growth of 60 percent for local bus service and 100 percent for freeway bus service.

In 1985, 22 local buses and 16 express buses would use the Center in the peak direction, while some 10 local buses and 6 express buses in the off-peak direction. By 2000, some 35 local buses and 32 express buses would use the Center in the peak direction, while some 16 local buses and 12 express buses would use the Center in the off-peak direction.

Bus berths would be assigned according to principal "geographical" destinations.

Bus dwell times at the Transit Center would approximate 5 min per bus for buses passing through the Center and 8 min per bus for buses that begin and end trips there. These dwell times compare with about a 3-min passenger service time needed to fill an empty bus to seated capacity, assuming that exact fares are paid on the bus.

2. *Solution*—Estimated berth requirements for 1985 and 2000 are given in Tables 12-36 and 12-37. The berths were estimated as follows:

a. The bus routes were grouped by geographic destination in 3 categories.

b. The "capacity" of each type of service was obtained by the equation $f = 60/D$, where D was the specified dwell time, in minutes including clearance. Thus, a 5-min dwell time could accommodate 12 buses/berth/hour; an 8-min dwell time, 7.5.

c. The number of *inbound* berths for the AM peak hour were computed by dividing the number of buses by the berth capacity. Thus, for lines 42 and 68, in 1985, 12 buses would need 12/7.5 or 1.6 berths; This number was rounded up to 2.

d. The bus lines that *start* at the center would need only *inbound* berths. The other bus services would need an equal number of *outbound* berths to accommodate PM peak hour bus flows, and to ensure that each major geographic destination would have its specified own boundary area.

e. The total berth requirements represent the sum of the inbound and outbound berths. As a result, 10 loading positions would be needed for 1985 conditions; and 13 loading positions for 2000. Ideally 15 loading positions should be provided to account for growth and traffic fluctuations within the peak hour.

Note that 38 inbound buses with a berth capacity of 10 buses/berth/hour, would require only 4 inbound loading positions in 1985 if routes were not separated geographically. However, this is *not* advisable when one considers clarity to the riding public, so that 6 berths are to be anticiapted based on the grouping shown in Table 12-35.

Calculation 9—Berth Capacity for Loading

1. *Description*—A rail-bus interchange (intermodal terminal) is planned for two urban bus lines.

Passengers pay a "single-coin" fare, and enter via the front door. Each bus has a seating capacity of 50 people, and is equipped with single-width doors. It is assumed that loading would occur through the front door, and unloading through the rear. It is desired to determine the berths needed, assuming a minimum clearance time of 15 sec between buses. Bus frequency on line 1 is 20 buses/hour, and 30 buses/hour on line 2.

2. *Solution*—This problem can be analyzed by applying the

TABLE 12-35. ANTICIPATED PEAK-HOUR BUSES AT TRANSIT CENTER

ROUTE		PEAK DIRECTION 1985	PEAK DIRECTION (2000)	OFF-PEAK DIRECTION 1985	OFF-PEAK DIRECTION (2000)	TYPE OF SERVICE
LOCAL SERVICE						
42	Holman Crosstown	8	13			Terminating
68	Brays Bayou Crosstown	4	6			Terminating
76	Lockwood Crosstown	4	6	4	6	Through
77	MLK Limited	6	10	6	10	Through
Subtotal Local		22	35	10	16	
EXPRESSWAY SERVICE						
242	Clear Lake Park & ride	3	6			Through
245	Edgewood Park & ride	3	6			Through
250	Hobby Park & Ride	2	4			Through
255	Fuqua Park & Ride	4	8			Through
41	Garden Villas Limited	2	4			Through
147	Sagemont Express	2	4			Through
Off Peak Direction All Lines				6	12	
Subtotal Express		16	32	6	12	
TOTAL		38	67	16	28	

SOURCE: Adapted from Herbert Levinson and Texas Transportation Institute: *Conceptual Planning and Design, Lockwood Transit Center,* March 1983

TABLE 12-36. BUS BERTH REQUIREMENTS, YEAR-1985

BUS LINE	SERVICE TYPE	DWELL TIME/BUS (MINUTES) (ASSUMED)	BUSES/ BERTH/ HOUR $f = 60/D$	INBOUND BUSES AM PEAK HOUR (From Tab. 12-34)	INBOUND BERTHS	MAX. (OUTBOUND) BERTHS NEEDED FOR PM PEAK HOUR	TOTAL BERTHS
Local Service							
42-68 Holman Crosstown	Start	8 min	7.5	12	2	—	2
76 Crosstown	Through	5 min	12	4	1	1	2
77 MLK Limited	Through	5 min	12	6	1	1	2
Subtotal				22	4	2	6
Freeway							
Expressway Lines To City Center AM (From City Center PM)	Through	5 min	12	16	2	2	4
TOTAL				38	6	4	10

TABLE 12-37. BUS BERTH REQUIREMENTS, YEAR-2000

BUS LINE	SERVICE TYPE	DWELL TIME/BUS (MINUTES)	INBOUND BUSES AM PEAK HOUR (FROM TAB. 12-34)	INBOUND BERTHS	MAX. OUTBOUND BERTHS NEEDED FOR PM PEAK HOUR	TOTAL BERTHS
42-68 Holman Crosstown	Start	8 min	19	3	0	3
76 Crosstown	Through	5 min	6	1	1	2
77 MLK Limited	Through	5 min	10	1	1	2
Subtotal			35	5	2	7
Expressway Lines To City Center AM (From City Center PM)	Through	5 min	32	3	3	6
TOTAL			67	8	5	13

procedures for estimating berth requirements, assuming uninterrupted flow (Eq. 12-11).

In this example, each route is analyzed separately. Because both bus lines operate on short headways, and would continue after receiving and discharging passengers, no allowance is made for schedule recovery or layover; such provisions may be needed in practice and would have to be added to the dwell times.

The number of berths required for a given passenger volume can be computed from the following Eq. 12-13:

$$N_b = \frac{J\,(bB + t_c)}{(3{,}600)\,BR} = \frac{bB + t_c}{0.833\,h'}$$

where:

N_b = number of effective berths;
J = total number of passengers to be served per hour;
B = number of boarding passengers/bus;
b = dwell time per boarding passenger;
t_c = clearance time per bus in seconds;
h' = headway between buses, in seconds; and
R = 0.833.

Substituting the values of B = 50 passengers per bus, t_c = 15 sec and b = 3 sec (exact fare); and headways of 180 sec for line 1 and 120 sec for line 2, the number of berths become:

$$\text{Line 1: } N_1 = \frac{3\,(50) + 15}{180\,(0.833)} = \frac{165}{180\,(0.833)} = 1.10 \text{ (Use 1 berth)}$$

$$\text{Line 2: } N_2 = \frac{3\,(50) + 15}{120\,(0.833)} = \frac{165}{120\,(0.833)} = 1.67 \text{ (Use 2 berths)}$$

Note that using the alternative form of the equation,

$$N_b = \frac{J\,(bB + t_c)}{3{,}600\,BR}$$

$$J_1 = 50 \times 20 = 1{,}000 \text{ pass./hr}$$

$$J_2 = 50 \times 30 = 1{,}500 \text{ pass./hr}$$

$$N_1 = \frac{1{,}000\,[150 + 15]}{(3{,}600)\,50\,(0.833)} = \frac{10^3(165)}{10^3(180)(0.833)} = 1.10 \text{ (Use 1 berth)}$$

$$N_2 = \frac{1{,}500\,[150 + 15]}{(3{,}600)\,50\,(0.833)} = \frac{165}{120\,(0.833)} = 1.67 \text{ (Use 2 berths)}$$

During the peak 15 or 20 min, buses will probably load to their "design" or "crush" capacity. In this short period (a) dwell times will increase, and /or (b) clearance times between buses will decrease. The berths needed to accommodate loads of 75 to 80 passengers per bus are determined as follows:

$$N_1 = \frac{bB + C}{h_1\,(0.833)} = \frac{3\,(80) + 15}{180\,(0.833)} = 1.70 \text{ (Use 2 berths—case 1)}$$

$$N_2 = \frac{bB + C}{h_2\,(0.833)} = \frac{3\,(80) + 15}{120\,(0.833)} = 2.55 \text{ (Use 3 berths—case 2)}$$

Note also that the 75 passengers per bus in the peak 15 min as compared with 50 for the entire hour indicates a peak-hour factor of 0.67. Also note that the use of the R factor to reduce queuing, does not change the berth requirements that would otherwise be needed.

Calculation 10—Bus Berth Unloading

1. *Description*—A facility is being built in an outlying area to facilitate transfer between feeder buses and a rail rapid transit line. It is assumed that buses will enter the facility on 1-min headways and that each bus will discharge 50 passengers. This corresponds to a total passenger flow of (50 people/bus) × (60 buses/hour) = 3,000 people/hour. Clearance time required for

one vehicle to manuever out of the berth and for another to enter it is assumed as 20 sec.

It is desired to know the number of unloading berths that should be provided assuming the following bus configurations:

- Single-width door, one door used.
- Single-width door, two doors used.

2. *Solution*—The number of berths required for a given passenger volume can be computed from the variation of Eq. 12-13 which also applies to alighting:

$$N_b = \frac{J\,(aA + t_c)}{(3{,}600)\,AR} = \frac{aA + t_c}{h'\,(0.833)}$$

where:

N = number of effective berths;
A = number of alighting passengers per bus = 50;
t_c = clearance time per bus = 20 sec;
J = total passengers per hour to be served = 3,000;
a = dwell time per alighting passenger = (1.7 and 0.9 sec);
h' = headway between buses arriving at station; and
R = 0.833.

This equation is similar to Eq. 12-13, except that *unloading* rather than loading passenger flows and coefficients are used. Note that the uninterrupted flow equation is used since the unloading will not be affected by traffic signal delay.

Substituting yields:

$$N_b = \frac{3{,}000\,[a\,(50) + 20]}{3{,}600\,(50)\,(0.833)} = \frac{a\,(50) + 20}{60\,(0.833)}$$

The appropriate alighting service time factors are obtained from Tables 12-9 and 12-10 as follows (*note* that Table 12-33 suggests 117 to 210 sec):

- Single width door, 1 door used: a = 1.7 sec
- Single width door, 2 doors used: a = 0.9 sec

Case 1: $N_1 = \dfrac{1.7\,(50) + 20}{60\,(0.833)}$
= 2.1(Use 3 berths, although 2 would suffice)

Case 2: $N_2 = \dfrac{0.9\,(50) + 20}{60\,(0.833)} = 1.30$ (Use 2 berths)

In practice, allowance should be made for: (a) some buses carrying full or standing loads during part of the peak hour, (b) buses operating at closer headways during parts of the hour, and (c) imbalanced use of doors.

One approach is to assume that all buses would operate with standees for design purposes. Berth requirements, assuming 75 persons per bus would be 2.46/0.833, or 3 berths assuming availability of both doors for passenger discharge.

Given this condition which recognizes the likelihood of peak 15-min flow rates that are 25 percent greater, it is desirable to provide 3 unloading berths.

Calculation 11—Berth Capacity for Loading at Major Stops

1. *Description*—It is desired to estimate the capacity of a bus line where 10 people board each bus, passenger service time is 3 sec per passenger, and clearance time is 15 sec per bus. It is assumed that boarding conditions govern. The signal timing along the street has a g/C ratio of 0.45.

2. *Solution*—The problem may be analyzed in detail by use of Eq. 12-14a:

$$Q = (g/C)\,\frac{3{,}600\,BR}{t_c + Bb\,(g/C)}$$

where:

g/C = green time per cycle, 0.45;
t_c = clearance between buses, 15 sec;
B = boarding passengers per bus, 10;
b = passenger service time, 3 sec/pass.; and
R = 0.833.

Substituting gives:

$$Q = 0.45\left(\frac{(0.833)\;3600\;(10)}{15 + (10)\,(3)\,(0.45)}\right) = 0.45\left(\frac{(0.833)\;3600}{15 + 13.5}\right)$$
$$= 568(0.833) = 473$$

The number of buses per hour would be 473/10 or 47.

An approximate solution may be obtained from Table 12-18 using a g/C ratio of 0.5. Table 12-18 shows that for a g/C ratio of 0.5, 30-sec dwell time per stop (10 pass. × 3 sec/pass.), and 15-sec clearance that 50 buses per hour could be accommodated. This translates into 500 people. The difference between 473 and 500 results from the use of a 0.50 g/C ratio rather than 0.45.

Note that if there were no signal delays, 670 passengers per hour on 67 buses could be accommodated. In this case one could use Eq. 12-14a with g/C = 1.00 or Eq. 12-12 directly:

$$Q = \frac{R\;3{,}600\;b}{bB + t_c} = \frac{(3{,}600)\,(10)\,(0.833)}{3\,(10) + 15} = 666 \text{ pass. per hour}$$

Since 10 passengers board per bus, some 67 buses could be accommodated.

Calculation 12—Arterial Street Capacity

1. *Description*—A central business district "bus-only street" provides 4 loading positions at the busiest stop. There is a 15-sec clearance between buses and a maximum of 75 passengers per bus past the maximum load point, during the peak 15 min. An exact fare pay-as-you-enter system is used, with entry through a single door. Rear doors of buses are used for passenger exit. A g/C ratio of 0.52 is assumed. Field studies show that 25 percent of the passengers at the maximum load point board at the major stop and that the peak-hour load factor is 0.80.

It is desired to estimate the hourly passenger volumes and bus frequency at the maximum load point.

2. *Solution*—The number of people that can be carried past the maximum load point can be estimated from Eq. 12-21b:

$$P = \frac{3{,}600\ N_b\ (g/C)\ R}{Xb\ (g/C) + t_c/S}$$

where:

g/C = green time per cycle, 0.52;
t_c = clearance between buses, 15 sec;
b = service time per passenger, 3 sec;
S = pass./bus at maximum load point, 75;
P = pass./hour (flow rate) at max. load point;
N_b = number of *effective* berths, max. = 2.5 (Table 12-18, noting 4 loading positions provided);
X = proportion of passengers at maximum load point boarding at busiest stop, 0.25; and
R = reductive factor for queuing = 0.833.

Substituting gives:

$$P = \frac{3{,}600\ (2.5)\ (0.52)\ (0.833)}{[(0.25)\ (3)\ (0.52) + 15/75]} = \frac{4{,}680\ (0.833)}{[0.39 + 0.20]} = 6607$$

This represents the flow rate during the peak 15 min. Adjusting by the PHF of 0.80 gives 5,286 passengers at the maximum load point during the entire hour.

The 5,286 passenges at 75 passengers/bus would result in 70 buses/hour. If this service frequency were maintained for the entire hour, it would result in 60 passengers per bus. This is probably more realistic than reducing the service frequency to maintain 75 persons per bus during the entire 60-min period.

The number of people passing the maximum load point also can be estimated using Table 12-23, assuming a g/C ratio of 0.50. In using these exhibits, a value of 0.20 (i.e., 15/75) is used for the clearance time to passenger per bus ratio. They result in 2,610 passengers passing the maximum load point for each effective berth. This corresponds to 6,525 passengers per hour (flow rate) for 2.5 berths, or 5,220 when the peak-hour factor is applied. This approximation is sufficiently accurate for most planning purposes.

Calculation 13—CBD Busway

1. *Description*—A central business district busway serves 2,000 people past the maximum load point in the peak 15 min. The heaviest stop has a 15-min boarding volume of 1,000 people. It is desired to determine (a) the bus frequency, and (b) the number of berths required to accommodate the boarding passenger volume. It is assumed that "schedule design" bus volumes are 75 persons/bus at the maximum load point, clearance time between buses at each stop is 15 sec, and a pay-as-you-leave fare system is used in the downtown area.

2. *Solution*—Tables 12-9 and 12-10 give a range of 1.5 to 2.5 sec per passenger through a single door, pay-as-you-leave. A value of 2.0 sec per passenger will be used.

The number of buses per hour can be determined from Eq. 12-15, stated as:

$$f = P/S$$

where:

f = bus frequency at maximum load point;
P = demand at maximum load point, in passengers per peak 15 min; and
S = passenger capacity of bus (seated + standing).

Therefore: f = 2,000/75 = 26.7 buses per peak 15 min.

The number of berths can be computed from Eq. 12-19, because uninterrupted flow conditions can be assumed.

$$N_b = P\,\frac{Xb + t_c/S}{3{,}600\ R}$$

where:

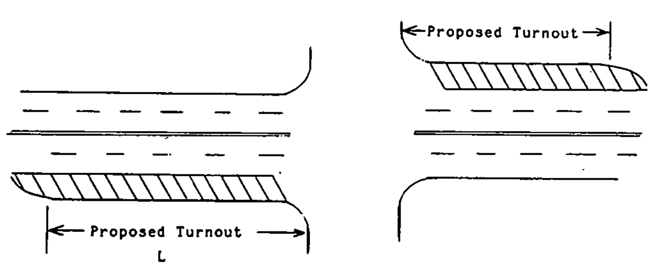
P = persons per hour (flow rate) = (1,500 × 4) = 6,000;
N_b = number of effective berths;
S = bus capacity (seated + standing) = 75;
t_c = clearance between buses = 15 sec;
b = boarding time per passenger = 2.0 sec;
X = proportion of maximum load point passengers which board at heaviest stop (X = 750/1,500 as given); and
R = 0.833.

Therefore, in this example:

$$N = 6{,}000\left(\frac{(0.5 \times 2) + 15/75}{3{,}600\ (0.833)}\right) = 6{,}000\,\frac{(1.2)}{3{,}600\ (0.833)}$$
$$= 2.0 \text{ Effective berths}$$

Thus, 2 effective berths should be provided. Allowing for berth "inefficiencies," 3 loading positions should be provided (Table 12-19). This corresponds to a cumulative capacity of 2.25 berths for "on-line" stations and 2.60 berths for "off-line" linear stations.

Calculation 14—Arterial Bus Turnout



1. *Description*—It is planned to build bus turnouts along an artery. Observations show that bus dwell times approximate 45 sec and clearance time 15 sec. The peak-hour factor is 0.67.

It is desired to find the desired number of buses per hour that can use the turnout, assuming that stopped buses will not back up onto traffic.

2. *Solution*—To provide for buses backing out onto traffic the turnout should be adequate 95 to 97.5 percent of the time. The corresponding R value is 0.5 (LOS B). This results in Eq. 12-24.

$$f' = \frac{1{,}800}{D + t_c} = \frac{1{,}800}{45 + 15} = 30 \text{ Buses/hour (flow rate)}$$

Applying the peak-hour factor of 0.67 results in 20 buses per hour.

Note that the *maximum* service volume at LOS B would occur if 5 loading positions are provided. Applying the berth efficiency factor 2.5 to the 20 buses would result in a service volume of 50 buses over the hour. In practice, one might accept a greater probability of queue formation by providing fewer bays. Alternatively, fare collection procedures could be improved to reduce the dwell times.

Calculation 15—Rail Rapid Transit

1. *Description*—A rail rapid transit line operates twenty 8-car trains per track per hour. Scheduled loads average 2.0 passengers per seat. How many people can the line carry? Cars are 75 ft long and can seat 75 people.

2. *Solution*—This number of people per hour per track can be estimated by applying Eqs. 12-5 or 12-6. For instance, applying Eq. 12-5

$$\text{Passengers per hour} = \frac{\text{Trains}}{\text{Hour}} \times \frac{\text{Cars}}{\text{Train}} \times \frac{\text{Seats}}{\text{Cars}} \times \frac{\text{Pass.}}{\text{Seat}}$$

$$= 20 \times 8 \times 75 \times 2.0 = 24{,}000 \text{ persons/hr}$$

Calculation 16—Light Rail Transit on City Street

1. *Description*—A light rail transit line operates within a city street median through signalized intersections. Service is provided by 2-car trains, with each car about 75 ft long. The g/C time is 0.50 and the passenger dwell times are 60 sec. How many people per hour can the trains carry?

2. *Solution*—Estimating the passenger capacity of the line requires three intermediate calculations. These are:

a. The train clearance times including: (1) minimum separation between trains, and (2) time for a train to clear the stop.

b. The maximum number of trains per hour.

c. The number of passengers that each train can carry.

The calculations are shown below.

a. Train clearance times: (1) minimum spacing between trains—estimated at 20 sec; (2) time for train to clear stop (station)—equals (length of train)/(average speed).

The train length is 75 × 2 or 150 ft. Assuming the train accelerates from rest to 15 mph (22 ft/sec), the average speed is 11 ft per sec. Therefore 150/11 or about 14 sec is needed for clearance. Total clearance, therefore, is 34 sec.

b. The maximum number of transit units per hour can be obtained from Eq. 12-10b, or Eq. 12-2a, adjusted for the g/C ratio of 0.50. For on-street operations, R is equal to 0.833.

$$c_v = \frac{g/C\,(3{,}600\,R)}{t_c + D\,(g/C)} = \frac{(0.50)\,(3{,}600)\,(0.833)}{34 + 60\,(0.50)} = \frac{1{,}500}{64}$$

$$= 23.4 \text{ units/hour, Say 23 units.}$$

c. Passengers per train values can be estimated in two ways: (1) Table 12-4 shows LRV's having a crush load of 400 to 460 passengers per pair of cars, and a maximum schedule load of 180 to 190 passengers per car; (2) Table 12-6 shows maximum schedule loads ranging from 3.3 to 3.9 persons per sq ft. Assuming a 75 × 8.8-ft car, this corresponds to 170 to 200 passengers per car. Selecting the midpoint, 3.6 sq ft per passenger, results in 185 persons per car.

Using the 185 persons per car gives a capacity of 185 × 2 or 370 persons per 2-car train.

d. Passenger capacity is computed as follows. The passengers per hour reflects the product of the passengers per train and the trains per hour. This gives 370 × 23 or 8,500 passengers per hour (rounded).

Note that the passenger capacity could be computed directly from Eq. 12-2c.

$$c_p = nSc_v \frac{(g/C)\,3{,}600\,nSR}{(g/C)\,D + t_c}$$

where $S = 185$ and $n = 2$.

This capacity can be realized, if there are at least two major stops, prepayment of fares, and at least two sets of double-width doors on each car available for boarding passengers.

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APPENDIX I

BUS CAPACITY EXPERIENCE

TABLE I.12-1. REPORTED THEORETICAL BUS LANE CAPACITIES

FACILITY OR SOURCE	BUSES PER HOUR	HEADWAY (SEC)	AVERAGE BUS STOP SPACING (FT)	AVERAGE BUS SPEED (MPH)	EQUIVALENT PASSENGERS PER HOUR[a]
Uninterrupted Flow					
G.M. Proving Grounds: Uninterrupted Flow (Initial Studies)	1,450[b]	2.5	No Stops	33	72,500
Highway Capacity Manual, 1985					
Freeway: Level-of-Service D	1,060	3.4	No Stops	40–47	53,000
Level-of-Service C	780	4.6	No Stops	48–50	39,000
Highway Capacity Manual, 1965					
Freeway: Level-of-Service D	940	3.8	No Stops	33	47,000
Level-of-Service C	690	5.2	No Stops	40–50	34,500
G.M. Proving Grounds: 6-Bus Platoons, 30-sec On-Line Stops	400	[c]	0.3 mile	15	20,000
City Streets					
Highway Capacity Manual, 1965 Arterial Streets—25-sec Loading Random Arrival (Approximate LOS C)	72	50	Not Cited	Not Cited	3,600
Toronto Transit Commission (Planning Criteria)	60	60	500–600 ft	10	3,000

[a] Equivalent passenger volume assumes 50 passengers per bus.
[b] Ref. *41*; subsequent studies have reported bus volumes of 900 to 1,000 vehicles per lane per hour; these are consistent with reported flows.
[c] 2.4 sec within the platoon with a platoon every 54 sec on the average.

SOURCE: Compiled from various bus-use studies.

TABLE I.12-2. OBSERVED PEAK-HOUR BUS VOLUMES ON STREETS AND FREEWAYS

FACILITY OR SOURCE	BUSES PER HOUR	HEADWAY (SEC)	AVERAGE BUS STOP SPACING (FT)	AVERAGE BUS SPEED (MPH)	PASS. PER HOUR	REMARKS
Freeway or Busway						
Lincoln Tunnel Uninterrupted Flow	735	4.9	No Stops	30	32,560	Connects to Midtown bus terminal
I-495 (New Jersey) Exclusive Bus Lane, Uninterrupted Flow	485	7.3	No Stops	30–40	21,600	
San Francisco Oakland Bay Bridge	350	10.3	No Stops	30–40	13,000	Pre-BART connects to Transbay terminal
Shirley Highway Busway, Wash., D.C.	200	18.0	No Stops	35(Freeway)	10,000	900-ft stop spacing in CBD
Bus-Only Mall						
State Street, Chicago	180	20.0	400	0–5	9,000	Based on peak 15-min rate
Portland, 5th at 6th Ave.	180	20.0	NA	5–10	9,000	
Arterial Street						
Michigan Ave., Chicago	228	15.0	NA	NA	11,400	Some multiple lane use, 5-min rate
Madison Ave., N.Y.C.	200 ±	18.0	1,000	NA	10,000	Two exclusive bus lanes
Hillside Ave., N.Y.C.	170	21.0	530	Not Cited	8,500[a]	Multiple lane use with lightly patronized stops
14th Street, Wash., D.C.	160	23.0	900	5–12	8,000	Approach to CBD
Market St., Philadelphia	150	24.0	300–600	5–10	6,100–9,900	Multiple lanes—Pre-Chestnut St. mall
K Street, Wash., D.C.	130	28.0	500	5–8	6,500	Pre-Metro
Main St., Rochester	80	45.0	1,000	5	4,000	Some platooning at stops
Downtown Streets with Stops (Various Cities)	80–120	30.0–45.0	500	5–10	4,500–6,000[a]	

[a] Estimated, assuming 50 passengers per bus; (1 ft = 0.305 m; 1 mph = 1.6 kph)

SOURCE: Compiled from various bus-use studies—1972–1978 conditions. Summarized in Ref. *34*.

TABLE I.12-3. OBSERVED BUS VOLUMES ON URBAN LIMITED ACCESS FACILITIES—PEAK DIRECTION OF FLOW, 1972–1976 CONDITIONS

		VEHICLES PER HOUR		PASSENGERS CARRIED[a]			PERCENT CARRIED
FACILITY	AREA	BUS	AUTO	BUS	AUTO	TOTAL	BY BUS
Lincoln Tunnel	New York	735	3,200	32,560	5,065	37,625	85.5
Bay Bridge, Post BART, San Francisco	San Francisco	200	8,700	8,900	16,000	24,900	35.7
Bay Bridge, Pre-BART, San Francisco	Oakland	327	8,115	13,000	10,400	23,400	55.5
Shirley Highway (I-95)	Wash., D.C.	200	3,600	10,000	5,000	15,000	67.0
Gowanus Expressway	New York	106	2,900	5,300	4,350	9,650	54.9 (1976)
Ben Franklin Bridge	Philadelphia	137	4,490	5,065	5,620	10,685	47.5
Long Island Expressway	New York	89	2,710	3,560	4,100	7,660	46.5
Memorial Bridge	Wash., D.C.	100	3,690	4,020	6,650	10,670	37.6
Lions Gate Bridge	Vancouver, BC	45	3,300	2,000	4,600	6,600	30.2
Schuylkill Expressway	Philadelphia	78	5,300	2,800	6,650	9,450	29.5
Southeast Expressway	Boston	65	4,200	2,450	6,000	8,450	29.0
I-71	Cleveland	35	3,200	1,850	4,500	6,350	29.0
Golden Gate Bridge	San Francisco	80	6,650	3,750	9,250	13,000	28.8
San Bernardino Freeway	Los Angeles	70	6,800	3,500	10,000	13,500	25.9
South Capitol St. Bridge	Wash., D.C.	32	3,335	1,920	5,000	6,920	27.7
George Washington Bridge	New York	108	9,440	4,245	13,215	17,460	24.3
14th St. Bridge	Wash., D.C.	79	6,565	3,295	10,425	13,720	24.0
North Lake Shore Drive	Chicago	80	9,500	4,000	14,200	18,200	22.0
John C. Lodge Freeway	Detroit	40	4,950	1,800	6,920	8,720	20.6
North Central Expressway	Dallas	32	4,000	1,200	5,600	6,800	17.5
Bayshore Freeway	San Francisco	35	6,800	2,270	10,880	13,150	17.3
South Lake Shore Drive	Chicago	24	5,700	1,400	8,000	9,400	14.9
I-5	Seattle	47	9,800	2,300	13,700	16,000	14.4
Hollywood Expressway	Los Angeles	36	7,650	1,755	10,500	12,255	14.4
North Expressway	Atlanta	24	4,550	1,070	6,380	7,450	14.4
East Memorial Shoreway	Cleveland	24	5,800	1,250	8,100	9,350	13.3
Memorial Drive	Houston	11	2,250	500	3,380	3,880	12.9
Stevenson Expressway	Chicago	16	4,600	840	6,900	7,740	10.9
Harbor Freeway	Los Angeles	23	7,200	1,050	10,000	11,050	9.5
I-45N	Houston	19	6,450	875	9,550	10,425	8.4
I-35W	Minneapolis, St. Paul	13	4,950	585	6,900	7,485	7.8
US 59	Houston	13	6,900	600	10,300	10,900	5.5
I-45S	Houston	11	6,000	505	9,000	9,505	5.3
I-10W	Houston	8	5,870	370	8,800	9,170	4.0
Jones Falls Expressway	Baltimore	3	2,780	125	3,900	4,025	3.1
Chrysler Freeway	Detroit	4	5,550	180	7,750	7,930	2.3

[a] Involves assumption in some cases as to car or bus occupancy.

SOURCE: Refs. *4* and *34*.

TABLE I.12-4. PEAK-HOUR BUS VOLUMES ON URBAN ARTERIALS, RANKED BY PERCENTAGE OF TOTAL PASSENGERS CARRIED BY BUS, IN DOMINANT DIRECTION OF FLOW UNDER 1972–1976 CONDITIONS

		VEHICLES PER HOUR			PASSENGERS CARRIED[a]			PERCENT CARRIED BY BUS
ARTERIAL LOCATION	CITY	BUS	AUTO	TOTAL	BUS	AUTO	TOTAL	
Nicollet Mall	Minneapolis	64	0	64	2,900	0	2,900	100.0
Market St. (East of Broad)	Philadelphia	143[b]	465	608	8,300	695	8,995	92.5
State St. at Madison	Chicago	151[b]	465	616	6,100	660	6,760	90.0
Hillside Ave.	New York	170[b]	630	800	8,500	950	9,450	90.0
Pennsylvania Ave. at Seventh St.	Washington, D.C.	120	600	720	6,000	900	6,900	87.0
Market St. at Van Ness	San Francisco	155[b]	1,200	1,355	9,900	1,550	11,450	86.5
Main St. at Fourth St.	Los Angeles	115	720	835	5,850	1,100	6,950	84.0
Main St. at Harwood St.	Dallas	100	635	735	4,400	900	5,300	83.0
Hill St. at Seventh St.	Los Angeles	109	800	909	5,250	1,200	6,450	81.5
Broad St. at Hunter St.	Atlanta	48	290	338	1,920	435	2,355	81.5
Seventh St. at Main St.	Los Angeles	91	705	796	4,500	1,050	5,550	81.0
Forbes Ave. at Wood St.	Pittsburgh	47	400	447	2,300	560	2,860	79.5
Fifth Ave. at Smithfield	Pittsburgh	47	420	467	2,300	590	2,890	79.5
Liberty St. at Sixth Ave.	Pittsburgh	66	650	716	3,250	910	4,160	78.2
K St. N.W. at 13th St.	Washington, D.C.	130	1,300	1,430	6,500	1,950	8,450	77.0
Eye St. at 13th St.	Washington, D.C.	104	1,100	1,204	5,200	1,600	6,800	76.5
Smithfield St. at Fifth Ave.	Pittsburgh	50	550	600	2,450	770	3,220	76.0
Thirteenth St. at F St.	Washington, D.C.	101	1,050	1,151	5,000	1,600	6,600	75.8
Broadway at Sixth St.	Los Angeles	78	850	928	4,000	1,390	5,390	74.5
Adams Street Bridge	Chicago	107	785	892	3,425	1,220	4,645	73.7
Granville St. at Georgia	Vancouver	70	900	970	3,150	1,200	4,350	72.5
Wisconsin Ave.	Milwaukee	78	935	1,013	3,100	1,200	4,300	72.0
Chestnut St. at 12th St.	Philadelphia	67	890	957	3,350	1,350	4,700	71.5
State St. at Roosevelt	Chicago	72	670	742	2,305	935	3,240	71.4
Washington St. at Wacker	Chicago	108	1,100	1,208	3,800	1,540	5,340	71.4
Wood St. at Forsyth Ave.	Pittsburgh	55	800	855	2,700	1,120	3,820	70.8
Seventh St. at Pennsylvania Ave.	Washington, D.C.	80	1,150	1,230	4,000	1,720	5,720	70.0
Main St. at Pratt	Hartford	75	625	700	1,875	815	2,690	70.0
Jackson Blvd. Bridge	Chicago	88	845	933	2,815	1,325	4,140	68.0
Sixth Ave. at Smithfield	Pittsburgh	33	560	593	1,620	780	2,400	67.6
Eglinton Ave. at Bathurst	Toronto	80	1,200	1,280	3,300	1,700	5,000	66.0
Elm St. at Harwood	Dallas	80	1,345	1,425	3,500	1,880	5,380	65.2
Sacramento St.	San Francisco	25	410	435	1,000	535	1,535	65.0
Constitution Ave. at 15th St.	Washington, D.C.	120	2,200	2,320	6,000	3,300	9,300	64.5
Spring St. at Seventh St.	Los Angeles	111	1,500	1,611	4,450	2,500	6,950	64.0
Sixteenth St. at Florida Ave.	Washington, D.C.	80	1,500	1,580	4,000	2,250	6,250	64.0

TABLE I.12-4. CONTINUED

ARTERIAL LOCATION	CITY	VEHICLES PER HOUR BUS	VEHICLES PER HOUR AUTO	VEHICLES PER HOUR TOTAL	PASSENGERS CARRIED[a] BUS	PASSENGERS CARRIED[a] AUTO	PASSENGERS CARRIED[a] TOTAL	PERCENT CARRIED BY BUS
Fourteenth St.at Constitution Ave.	Washington, D.C.	80	1,550	1,630	4,000	2,350	6,350	63.0
Connecticut Ave. at Cathedral Ave.	Washington, D.C.	90	1,800	1,890	4,500	2,700	7,200	62.5
Walnut at 15th St.	Philadelphia	48	960	1,008	2,400	1,450	3,850	62.5
Commerce St. at St. Paul St.	Dallas	72	1,415	1,487	3,300	2,120	5,420	61.0
Sheridan Rd. at Hollywood Ave.	Chicago	32	500	532	1,100	700	1,800	61.0
Michigan Ave. at Roosevelt Rd.	Chicago	77	770	847	1,815	1,210	3,025	60.0
Asylum St. at Main St.	Hartford	35	450	485	875	585	1,460	60.0
Michigan Ave. Bridge (Upper Level)	Chicago	116	1,590	1,706	3,580	2,390	5,970	60.0
Sutter St.	San Francisco	63	1,300	1,363	2,500	1,700	4,200	59.5
Madison Ave. at 42nd St.	New York	96	2,400	2,496	4,800	3,600	8,400	57.1
Second Ave. at 42nd St.	New York	110	2,800	2,910	5,500	4,200	9,700	56.8
First Ave. at 44th St.	New York	110	2,800	2,910	5,500	4,200	9,700	56.8
Sixth Ave. at Figueroa St.	Los Angeles	29	965	994	1,875	1,430	3,305	56.7
Georgia Ave. at Granville	Vancouver	45	1,200	1,245	2,000	1,600	3,600	55.5
Clay St.	San Francisco	26	650	676	1,050	850	1,900	55.3
Ninth St. at Market St.	Philadelphia	22	600	622	1,100	900	2,000	55.0
Second Ave. North	Birmingham, Ala.	44	1,400	1,444	2,300	1,950	4,250	54.0
Grand Ave. at Temple St.	Los Angeles	24	855	879	1,400	1,215	2,615	53.5
Geary St.	San Francisco	43	1,250	1,293	1,720	1,630	3,350	51.4
Howard St. at Fayette St.	Baltimore	30	470	500	790	755	1,545	51.0
Marietta at Spring St.	Atlanta	35	1,050	1,085	1,400	1,580	2,980	47.0
Peachtree St. at Ellis St.	Atlanta	55	1,700	1,755	2,200	2,550	4,750	46.5
Tyron St.	Charlotte, N.C.	40	1,150	1,190	1,200	1,700	2,900	41.4
Eighth St. at Los Angeles St.	Los Angeles	30	1,155	1,185	1,290	1,835	3,130	41.3
O'Farrell St.	San Francisco	27	1,200	1,227	1,080	1,550	2,630	41.2
Trade St.	Charlotte, N.C.	30	1,030	1,000	1,000	1,500	2,500	40.0
Pratt St. at Paca St.	Baltimore	64	2,390	2,454	2,215	3,825	6,040	36.7
Charles St. at Madison St.	Baltimore	33	1,915	1,948	1,480	3,060	4,540	32.6
Lombard St. at Greene St.	Baltimore	42	1,750	1,792	1,335	2,800	4,135	32.0
Eleventh St. Bridge	Washington, D.C.	54	4,120	4,174	2,870	7,735	10,605	27.1
Cathedral St. at Eager St.	Baltimore	36	1,545	1,581	880	2,470	3,350	26.3
St. Paul St. at Preston St.	Baltimore	45	2,815	2,860	1,375	4,505	5,880	23.4
Calvert St. at Lexington St.	Baltimore	39	2,645	2,684	1,185	4,230	5,415	21.9

[a] Data involve assumptions in some cases as to auto or bus occupancy.
[b] Buses operate in more than one lane.
SOURCE: Refs. *4* and *34*

TABLE I.12-5. OBSERVED BUS VOLUMES ON URBAN ARTERIALS—UPDATE, PEAK DIRECTION OF FLOW (1978–1984)

FACILITY	METROPOLITAN AREA	VEHICLE PER HR BUS	VEHICLE PER HR AUTO	TOTAL	PASSENGERS CARRIED BUS	PASSENGERS CARRIED AUTO	PASSENGERS CARRIED TOTAL	% CARRIED BY BUS
Main St.	Rochester	80	700	780	4,000	1,050	5,050	79.2
Madison Ave.	N.Y. City	200[a]	1,500	1,700	10,000	2,000	12,000	83.3
Temple St.	New Haven	35	575	610	1,380	750	2,130	64.8
W Chapel St.	New Haven	25	745	770	1,000	970	1,970	50.8
Church St.	New Haven	33	1,400	1,433	1,320	1,820	3,140	42.0
14th St.	Wash., D.C.	160[b]	1,480	1,640	6,400	1,930	8,330	76.8

[a] Estimated, based on 700 buses in 5 hr. This is in dual bus lane.
[b] Pre-Metro

SOURCE: Herbert S. Levinson

TABLE I.12-6. OBSERVED PASSENGERS AT MAJOR BUS TERMINALS

		PORT AUTHORITY BUS TERMINAL, NEW YORK, N.Y.	GEORGE WASHINGTON BRIDGE BUS TERMINAL, NEW YORK, N.Y.	GREYHOUND BUS TERMINAL, CLARK AND RANDOLPH STS. CHICAGO, ILL.[f]	TRANSBAY BUS TERMINAL, SAN FRANCISCO, CALIF.
Development Costs[b]		$58,000,000.00	$15,000,000.00	$8,000,000.00	$11,000,000.00
Type of Bus Service		Commuter and Intercity	Commuter and Intercity	Mainly Intercity	Intercity and Commuter
Date Completed		1950	1963	1952	1960
Number of Bus Levels		3	2	1	1
Number of Bus Loading Docks		184[c]	43	30	37
Contiguous Transportation Facilities		Subway, Local Bus, Auto Parking	Subway, Local Bus	Subway, Local Bus, Curb Parking	Streetcar and Bus, Auto Parking
Direct Ramp Connections		Lincoln Tunnel	George Washington Bridge	Garvey St. and Wacker Dr.	San Francisco Oakland Bay Bridge
Number of Passengers[a]	Daily	105,500	20,000	—	44,000, 35,000[d], 22,000[e]
	Peak Hr	32,600	4,200	10,000	13,000, 16,000[d], 10,000[e]
Number of Buses	Daily	3,350	850	—	1,150[d], 900[e]
	Peak Hr	730	108	—	400[d], 250[e]
Average Bus Occupancy	Daily	27.4	23.5	—	20.0
	Peak Hr	44.1	39.0	—	37.2
Avg. No. of Buses Per Dock	Daily	18.2	19.6	—	24.3–31.1
	Peak Hr	4.0	2.5	—	6.8–10.8
Avg. Bus Layover Time in Hours	Daily	1.32	1.22	—	0.40
	Peak Hr	0.25	0.4	—	0.16
Ancillary Land Uses		Retail convenience goods, restaurants	Retail convenience goods, restaurants	Retail convenience goods and offices over station	Retail convenience goods
Remarks		Saves buses 30 min over previous operations	Located over Cross Bronx Expressway	Designed to allow office building over station	Prior to 1960 Key System trains used terminal

[a] One-direction-only bus volumes.
[b] Data on maintenance costs and revenues unavailable.
[c] Before expansion.
[d] Before BART.
[e] After BART.
[f] This terminal is being replaced, 1985

SOURCE: Refs. *4* and *34*.

TABLE I.12-7. OBSERVED PEAK BUS BERTH VOLUMES AND FLOW RATES AT BUS TERMINALS

CITY AND TERMINAL	PEAK HOUR BUSES (ONE-WAY)	LOADING BERTHS	BUSES/BERTH
Eglinton, Toronto[b d]	250	13	19.2
Trans Bay, San Francisco (Pre-BART)	400	37	10.8
Jefferson Park, Chicago	140	14	10.0
69th St. and Ryan, Chicago	40	4	10.0
69th St., Philadelphia[a]	90	10	9.0
Southwest, Washington	80	10	8.0
Dixie, Cincinnati	48	6	8.0
Wilson Subway, Toronto[d]	136	18	7.6
Trans Bay, San Francisco (Post-BART)	250	37	6.8
95th St. and Ryan, Chicago	106	22	4.8
McKeesport, Pittsburgh	30	7	4.3
Midtown, New York[c]	730	184	4.0
George Washington Bridge, New York	108	43	2.5

[a] Includes buses and streetcars
[b] Before Yonge St. subway extension
[c] Includes 26 intercity bus bays; before terminal expansion
[d] Free transfer to subway

SOURCE: Refs. *4* and *35*.

APPENDIX II

RAIL CAPACITY EXPERIENCE

This appendix contains information on actual observed rail transit values, summarized in the first two tables. Table II.12-3 contains an enumeration of several theoretical relations for the minimum headway between trains, extracted from the literature. Using representative values of the parameters, the results for the several relations are computed in the same table. More detailed information on specific transit vehicle characteristics and capacities is contained in Ref. *39*.

TABLE II.12-1. OBSERVED PEAK-HOUR PASSENGER VOLUMES ON STREETCAR AND LRT LINES—EUROPE

CITY	LOCATION	YEAR	TRAINS PER HOUR	CARS PER HOUR	HEADWAY SECONDS	PASSENGERS IN PEAK DIRECTION	PASSENGERS PER TRAIN (ROUNDED)
Streetcars in Street	Vienna	1937	180	540	20	26,200[a]	150
	Stuttgart	1930	160	480	23	23,000[a]	140
	Hamburg	1957	120	300	30	19,200[a]	160
	Hong Kong	1978	96	38	38	8,000[a]	83
	Melbourne	1978	89	NA	40	4,400	49
	Dusseldorf	1975[b]	90	NA	40	NA	
	Hannover	1975[b]	80	NA	45	NA	
	Cologne	1975[b]	60	NA	60	NA	
	Belgrad	1978	51	79	46	4,200	60
	Koln	1978	32	NA	113	6,500	203
LRT-Tunnels	Koln	1978	48	48	75	10,000	208
Partial or Full	Hannover	1978	38	76	95	10,830[a]	
Signal Control	Stuttgart	1975[b]	38	NA	95	NA	

[a] Estimated by Ref. *9*
[b] Estimated herein

SOURCE: Refs. *8*, *9*, *29*.

TABLE II.12-2. RAPID TRANSIT CAR AND TRAIN CAPACITIES

		LENGTH (FT)	WIDTH (FT)	AREA (FT^2)	SEATED PASSENGERS	TOTAL PASSENGERS SCHEDULE	CRUSH	MAXIMUM CARS/TRAIN	SEATED PASSENGERS/ TRAIN
New York City Transit Authority	IRT	51.33	8.79	451.2	44	140	180	10–11	440–484
	IND	60.50	10.0	605	50	180	220	10	500
	R-44 R-46	75.00	10.0	750.0	72–76	225	225 290	8	576–608
Port Authority of N.Y. and N.J. (PATH)		51.25	4.23	473.0	42	140	200	7	294
Chicago Transit Authority		48.25	9.33	450.1	c.50	125	135	8	400
Philadelphia (SEPTA)									
Broad St.		67.50	10.00	675.0	67	NA	281 (est.)	6	450
Market St.		55.33	9.08	502.4	55	115	200	8 (est.)	440
Massachusetts Bay Transportation Authority									
Blue Line		48.75	8.58	418.3	48	125	191	4	192
Orange Line		55.31	9.28	513.3	54	175	240	4	216
Red Line		69.81	10.35	722.5	63	208	275	4	252
New Jersey (PATCO)		67.83	10.12	686.4	80	100	200	8	640
Toronto Transit Commission									
1962–1975		74.76	10.33	772.3	84	230	310	6	504
1953–1958		57.00	10.33	588.8	62	174	233	8	496
Bay Area Rapid Transit		75.00	10.5	787.5	72	144	216	8	576
Montreal Urban Community Transit Commission		56.42	8.25	465.5	39	157	208	29	351
Greater Cleveland Regional Transit Authority									
Airporter		70.25	10.41	731.3	80	120	140	4	320
Other		48.75	10.33	403.6	54	100	197	6	324
Washington Metropolitan Area Transit Authority		75.00	10.15	761.2	80	175	240	6	480

TABLE II.12-2. CONTINUED

		TOTAL PASSENGERS/TRAIN		SEATED PASSENGERS/FOOT OF LENGTH	TOTAL PASSENGERS/FOOT OF LENGTH		FT^2/SEATED PASSENGERS	FT^2/TOTAL PASSENGERS	
		DESIGN	CRUSH		SCHEDULE	CRUSH		SCHEDULE	CRUSH
New York City Transit Authority	IRT	1,400	1,800	0.86	2.72	3.51	10.2	3.22	2.50
	IND	1,800	2,200	0.83	2.97	3.64	12.1	3.36	2.75
	R-44 R-46	1,800	2,240	0.96–1.01	3.00	3.73	9.9–10.1	3.33	2.67
Port Authority of N.Y. and N.J. (PATH)		980	1,400	0.82	2.73	3.90	11.3	3.37	2.36
Chicago Transit Authority		1,000	1,480	1.03	2.59	3.83	9.0	3.60	2.43
Philadelphia (SEPTA)									
Broad St.		NA	1,686	0.99	NA	4.16	10.1	NA	2.40
Market St.		920	1,600	0.99	2.07	3.61	9.1	4.37	2.51
Massachusetts Bay Transportation Authority									
Blue Line		500	764	0.98	2.56	3.91	8.7	3.34	2.19
Orange Line		700	960	0.98	3.16	4.34	9.5	2.93	2.14
Red Line		832	1,100	0.90	2.98	3.94	11.4	3.47	2.62
New Jersey (PATCO)		800	1,600	1.01	1.47	2.95	8.6	6.68	3.43
Toronto Transit Commission									
1962–1975		1,380	1,860	1.12	3.08	4.14	9.2	3.36	2.49
1953–1958		1,392	1,864	1.09	3.05	4.09	9.5	3.38	2.52
Bay Area Rapid Transit		1,152	1,728	0.96	1.92	2.88	10.9	5.47	3.64
Montreal Urban Community Transit Commission		1,413	1,872	0.69	2.78	3.69	11.9	2.96	2.23
Greater Cleveland Regional Transit Authority									
Airporter		480	560	1.14	1.71	1.99	9.1	6.09	5.22
Other		600	1,182	1.11	2.05	4.04	9.3	5.04	2.55
Washington Metropolitan Area Transit Authority		1,050	1,440	1.07	2.33	3.20	9.52	4.35	3.17

SOURCE: Computed by Herbert S. Levinson from data obtained from Roster of North American Rapid Transit Cars 1945–1976, American Public Transit Association. Schedule and crush load data are based on information received from APTA.

TABLE II.12-3. THEORETICAL RAIL RAPID TRANSIT EQUATIONS

A. Equation

1. Lang and Soberman, 1980[a]

$$h = t_s + nL_1/V + V/2a + 5.05\,V/2b_n \quad (1)$$

2. Rice, 1977[b]

If maximum speed *is not* reached,

$$h = t_s + t_r + nL_1/V + V(1/b_n + 1/2b_e + \sqrt{2(D + nL_1)/a} \quad (2)$$

If maximum speed *is* reached,

$$h = t_s + t_r + 2nL_1/V + V(1/b_n + 1/2b_e + 1/2a) + D/V \quad (3)$$

3. Vuchic, 1981[c]

$$h = t_s + t_r + nL_1/V + V(k + 1)/2b_n + \sqrt{2\,nL_1/a} \quad (4)$$

$$h = t_s + t_r + nL_1/V + \sqrt{2nL_1\,b_1/a(a + b_1)} + V/b_2 \quad (5)$$

$b_1 = b_2$
b_1 , $b_2 = b_n$
b_1 , $b_2 = b_e$
(*Note*: excludes safety factor)

B. Symbols

h = minimum headway between trains, in sec;
t_r = reaction time, in sec, for driver response;
t_s = dwell time, in sec, in station;
k = safety factor;
L = length of train = nL_1, where: n = no. of cars and L_1 = length/car;
V = maximum approach speed, ft/sec;
a = acceleration rate from stop, ft/sec^2;
b_1 = braking rate of lead train, ft/sec/sec;
b_2 = braking rate of following car;
b_n = normal braking rate;
b_e = emergency braking rate; and
D = "run-out" distance, ft.

C. Typical Values

	English	*S.I.U.*
t_s	20–60 sec.	20–60 sec
t_r	3.0 sec	5.0 sec
k	1.5	1.5
$L = nL_1$	300–600 ft	91.5–183 ft
V	20–30 mph 29.4–44.1	
a	2.0 mph/sec 2.9 ft/sec/sec	0.9 m/sec^2
b_n	2.9 mph/sec 4.3 ft/sec/sec	3.0 m/sec^2
b_e	6.7 mph/sec 9.8 ft/sec/sec	
D	150 ft	45.7 m

D. Results of Computations for:
30 mph (13.4 m/sec)
600 ft (183-m train)

Equation

1. $h = t_s + 47.13$
2. $h = t_s + 47.30$ $D = 0$ ft
   49.74 $D = 150$ ft
3. $h = t_s + 50.29$ $D = 0$ ft
   53.70 $D = 150$ ft
4. $h = t_s + 49.71$
5. $h = t_s + 42.47$

For 30 mph and 600-ft long trains, the headway is: 50-sec plus station dwell time
For 60-sec station dwell times, this results, in a headway of 110 sec or 33 trains per hour.

[a] Lang, A. S., and Soberman, R. M., *Urban Rail Transit: Its Economics and Technology*. Massachusetts Institute of Technology Press, Cambridge, Mass (1964).
[b] Rice, P., "Practical Urban Railway Capacity—A World Review." *Proc. Seventh International Symposium on Transportation and Traffic Theory*, Sasaki T. and Yamaoka T., 1977, Kyoto, Japan, Institute of System Science Research.
[c] Vuchic, V. R., *Urban Public Transportation, Systems and Technology*. Prentice Hall Inc., Englewood Cliffs, N.J. (1981).

APPENDIX III

EXAMPLES OF BOARDING AND ALIGHTING TIME

TABLE III.12-1. TYPICAL CBD SERVICE TIMES PER PASSENGER

	SECONDS PER PASSENGER		
	AM	MIDDAY	PM
Philadelphia, 1977			
Chestnut St. Transitway	2.5 to 2.8	2.4 to 3.7	2.5 to 3.5
Walnut Street	2.5	3.6	2.9
Minneapolis, 1977			
Nicollet Mall	2.3 to 2.5	2.3 to 3.6	3.8 to 4.3
Other Streets (Second, Marquette)	1.4 to 1.7	1.9 to 3.8	1.3 to 4.4
New Haven 1979–1980			
15 Locations			2.9 to 3.1[a]
2 Locations			3.2 to 3.4[a]
12 Locations			1.0 to 2.0[b]
3 Locations			2.1 to 2.5[b]

[a] Boarding
[b] Alighting
SOURCE: H. S. Levinson, "INET Transit Travel Times Analysis." Final report prepared for UMTA, April 1982.

TABLE III.12-2. OBSERVED RAIL TRANSIT STATION DWELL TIMES, 1980

LINE	TIME	LOCATION	DWELL TIME (SEC)	REMARKS
Single Observations				
Lexington Ave. Expr., N.Y. City	AM Peak	42nd St.	77	
Lexington Ave. Local, N.Y. City	AM Peak	42nd St.	90	50 sec: Pass. dwell time; 40 sec: Wait for expr.
Evanston Express, Chicago	PM Peak	Howard St.	65	Major transfer
	PM Peak	Randolph Wells	47	
Red Line, Boston	PM Peak	Park St.	60	
Green Line (LRV), Boston	PM Peak	Park St.	95	Crowded car
Line Observations	TIME	MEAN	STAND. DEV	
Lexington Ave. Expr., N.Y. City	PM Peak	53	17	
Evanston Express, Chicago	PM Peak	42	14	
Green Line(LRV), Boston	PM Peak	58	24	
Milwaukee, Chicago	Post PM Peak	19	6	

Note: This is a sampling.
SOURCE: H. S. Levinson, "INET Transit Travel Times Analysis." Final report prepared for UMTA, April 1982.

TABLE III.12-3. BUS BOARDING AND ALIGHTING TIMES IN SELECTED URBAN AREAS

LOCATION	BUS TYPE	BOARDING AND ALIGHTING METHOD	FARE SCHEME	FARE COLLECTION	BOARDING AND ALIGHTING RELATIONSHIP[a]
Louisville, Ky.	One-man	Alighting only	Flat fare	Driver	$T = 1.8 + 1.1F$
	One-man	Boarding only	Flat fare	Driver	$T = -0.1 + 2.6N$
	One-man	Simultaneous	Flat fare	Driver	$T = 1.8 + 1.0F + 2 - 0.02FN$
London	Two-man	Consecutive	Graduated	Conductor	$T = 1.3 + 1.5\ (N + F)$
	One-man	Consecutive	Graduated	Driver	$T = 8 + 6.9N + 1.4F$
	One-man	Simultaneous	Flat fare		
			Single coin	Mechanical	$T = 7 + 2.0N$
			Two coin	Mechanical	$T = 5.7 + 3.3N$[b]
Toronto	One-man	Simultaneous	Zonal	Fare Box	$T = 1.7N$, $T = 1.25F$ $T = 1.4(N + F)$
Copenhagen	One-man	Simultaneous	Flat fare	Split entry[c]	$T = 2.2N$
Dublin	Two-man	Consecutive	Graduated	Conductor	$T = 1.4(N + F)$
	One-man	Consecutive	Graduated	Driver	$T = 6.5N + 3.0F$
France:					
Bordeaux	One-man	Simultaneous	Flat fare	Driver	$T = 15 + 3N$
Toulouse	One-man	Simultaneous	Flat fare	Driver	$T = 11 + 4.6N$
Paris	One-man	Simultaneous	Graduated	Driver	$T = 4 + 5N$
	Two-man	Simultaneous	Graduated	Conductor	$T = 2.3N$

[a] T = stop time, in sec; N = number of passengers boarding; F = number of passengers alighting.
[b] In peak time, $T = 5.7 + 5.0N$ in off-peak time.
[c] Driver and machine.

SOURCE: Refs. *19* and *20*.

TABLE III.12-4. MEANS AND VARIANCES OF OBSERVED PASSENGER SERVICE TIME DISTRIBUTIONS

LOCATION	DIRECTION OF FLOW	BUS TYPE	DOORS ON BUS	TIME IN SECONDS MEAN	TIME IN SECONDS VARIANCE	COEFFICIENT OF VARIATION (%)
Montreal, Canada	Boarding	Can. Car	2	2.097	0.727	40.67
Montreal, Canada	Boarding	GMC	2	2.034	0.834	44.89
New Brunswick, N.J.	Alighting	GMC	1	1.972	1.045	51.83
New Brunswick, N.J.	Boarding	GMC	1	3.471	3.499	53.90
San Diego, Calif.	Alighting	GMC	2	1.472	0.403	43.34
San Diego, Calif.	Boarding	GMC	2	2.180	0.868	42.75

SOURCE: Ref. *19*.

PEDESTRIANS

CONTENTS

I. INTRODUCTION

The purpose of this chapter is to describe the basic principles of pedestrian traffic flow, and to present a general framework and procedures for the analysis of pedestrian facilities. The scope is limited to sidewalks, crosswalks, and street corners, but the analysis techniques can be applied to other pedestrian facilities. The chapter includes examples illustrating several typical applications.

Pedestrian activity can be a major component in urban street capacity analysis, and pedestrian characteristics are an important factor in the design and operation of transportation systems. Concentrated pedestrian movement occurs at public events, in and near transit terminals, high-rise buildings, department stores, theaters, stadia, parking garages, and other major traffic generators. Pedestrian safety, trip patterns, and convenience are also a necessary consideration in all multimodal traffic and transportation studies. Table 13-1 presents some high pedestrian volumes observed in several major urban centers.

The concentration of pedestrian activity at street corners and crosswalks makes them critical traffic links for both sidewalk and street networks. An overloaded corner or crosswalk not

TABLE 13-1. OBSERVED PEDESTRIAN FLOW RATES IN URBAN AREAS*

LOCATION	TIME	WALKWAY WIDTH (FT)	AVG. FLOW RATES FOR FULL HOUR PED/MIN	AVG. FLOW RATES FOR FULL HOUR PED/MIN/FT	PEAK FLOW RATES FOR PERIODS LESS THAN 1 HOUR PED/MIN	PEAK FLOW RATES FOR PERIODS LESS THAN 1 HOUR PED/MIN/FT
BOSTON						
Washington St (1960)	12–1 PM	7.0	53	7.6	—	—
CHICAGO						
CTA (1976)	PM	—	—	5.2	—	—
State St/Wash (1960)	12–1 PM	25.0	112	4.5	—	—
State St/Wash (1972)	4–5 PM	25.0	93	3.7	—	—
State St/Wash (1939)	12–1 PM	25.0	206	8.2	—	—
State St/Mad (1929)	—	25.0	342	13.7	471 (15 min)	18.8
State St/Mad (1929)	—	20.0	287	14.4	368 (15 min)	18.4
Soldiers Fld (1940)	—	21.5	202	9.4	298 (1 min)	13.9
Dyche Stadium (1940)	—	10.0	114	11.4	167 (5 min)	16.7
LOS ANGELES						
Broadway (1940)	—	18.0	—	—	125 (12 min)	6.9
DES MOINES AND AMES, IOWA						
Veteran's Aud. (1975)	10 PM	8.2	—	—	—	20.0 (5 min) 22.2 (1 min)
College Creek Footbridge (1975)	12 Nn	6.0	—	—	—	22.3 (5 min) 31.8 (1 min)
CY Stephens Auditorium (1975)	4:40 PM	7.5	—	—	—	31.9 (5 min) 39.2 (1 min)
Iowa State Univ. Armory	1 PM	2.8	—	—	—	28.7 (1 min)
NEW YORK CITY						
Madison Av (1969)	12–1 PM	13.0	167	12.8	—	—
Fifth Av (1969)	12–1 PM	22.5	250	11.1	—	—
Lexington Av (1969)	12–1 PM	12.0	100	8.3	—	—
Eighth Av (1969)	PM	15.0	167	11.1	—	—
42nd Street (1969)	PM	20.0	105	5.3	—	—
Port Authority Bus Terminal (1965)	PM	—	—	25.0	—	—
WASHINGTON D.C.						
7th St SW (1968)	PM	10.0	42	4.2	—	—
F Street NW (1981)	PM	15.0	19	1.3	—	—
SEATTLE						
CBD (1976)	PM	—	—	—	—	9.6
SAN FRANCISCO						
CBD (1976)	PM	—	—	—	—	10.8
WINNEPEG						
CBD Street (1980)	3–4 PM	17.0	74	4.4	—	—

* Compiled by H. Levinson and R. Roess from:

1. *Chicago Loop Pedestrian Movement Study,* City of Chicago, Chicago, Ill., 1973.
2. Pushkarev, B., and Zupan, J., *Urban Space for Pedestrians,* Regional Plan Association, New York, N.Y., 1976.
3. *Traffic Circulation and Parking Plan-CBD Urban Renewal Area-Boston, Mass.,* Barton-Aschman Associates, 1968.
4. "Traffic Characteristics," *Traffic and Transportation Engineering Handbook,* Institute of Transportation Engineers, Prentice-Hall, Englewood Cliffs, N.J., 1976.
5. "Characteristics and Service Requirements of Pedestrians and Pedestrian Facilities," Informational Report, *ITE Journal,* Institute of Transportation Engineers, Washington, D.C., May 1976.
6. Carstens R., and Ring, S., "Pedestrian Capacity of Shelter Entrances," Technical Note, *Traffic Engineering,* Institute of Transportation Engineers, Washington, D.C., December 1970.

only affects pedestrian convenience, but can delay vehicle turning movements, thereby reducing the capacity of the intersection and connecting streets.

The principles of pedestrian flow analysis are similar to those used for vehicular flow. The fundamental relationships among speed, volume, and density are similar. As the volume and density of a pedestrian stream increases from free-flow to more crowded conditions, speed and ease of movement decreases. When the pedestrian density exceeds a critical level, volume and speed become erratic and rapidly decline.

Pedestrian flow on sidewalks is affected by reductions in effective walkway width caused by various items of street "furniture," such as parking meters, light standards, mail boxes, and trash cans, and by interruptions to flow caused by traffic signals. The traffic signal cycle also results in queues of waiting pedestrians at street corners, which decreases corner circulation capacity and concentrates crossing pedestrians into denser platoons.

The level-of-service (LOS) concept, first used to define relative degrees of convenience on highways, is also applicable to pedestrian facilities. With this concept, such convenience factors as the ability to select walking speeds, bypass slower pedestrians, and avoid conflicts with others are related to pedestrian density and volume. The concept can also be applied to degrees of crowding in queuing areas, such as sidewalk corners, transit platforms, and other waiting areas.

The following sections define pedestrian traffic terminology, develop the principles of pedestrian flow, present the concept of pedestrian level of service, and provide detailed analysis procedures for use.

PEDESTRIAN CAPACITY TERMINOLOGY

Pedestrian analysis uses some familiar traffic terms, as well as others not used elsewhere in the manual. The following listing defines the major terms used throughout this chapter:

1. *Pedestrian speed* is the average pedestrian walking speed, generally expressed in units of feet per second.
2. *Pedestrian flow rate* is the number of pedestrians passing a point per unit time, expressed as pedestrians per 15 minutes or pedestrians per minute; "point" refers to a perpendicular line of sight across the width of a walkway.
3. *Unit width flow* is the average flow of pedestrians per unit of effective walkway width, expressed as pedestrians per minute per foot.
4. *Platoon* refers to a number of pedestrians walking together in a group, usually involuntarily, because of signal control and other factors.
5. *Pedestrian density* is the average number of pedestrians per unit of area within a walkway or queuing area, expressed as pedestrians per square foot.
6. *Pedestrian space* is the average area provided for each pedestrian in a walkway or queuing area, expressed in terms of square feet per pedestrian; this is the inverse of density, but is a more practical unit for the analysis of pedestrian facilities.

PRINCIPLES OF PEDESTRIAN FLOW

The qualitative measures of pedestrian flow similar to those used for vehicular flow are the freedom to choose desired speeds and to bypass others. Other measures more specially related to pedestrian flow include the ability to cross a pedestrian traffic stream, to walk in the reverse direction of a major pedestrian flow, and to generally maneuver without conflicts and changes in walking speed or gait.

Additional environmental factors which contribute to the walking experience, and therefore to perceived level of service, are the comfort, convenience, safety, security, and economy of the walkway system.

1. *Comfort* factors include weather protection, climate control, arcades, transit shelters, and other pedestrian amenities.
2. *Convenience* factors include walking distances, pathway directness, grades, sidewalk ramps, directional signing, directory maps, and other features making pedestrian travel easy and uncomplicated.
3. *Safety* is provided by separation of pedestrians from vehicular traffic, horizontally in malls and other vehicle-free areas, and vertically using overpasses and underpasses. Traffic control devices can provide for time separation of pedestrian and vehicular traffic.
4. *Security* features include lighting, open lines of sight, and the degree and type of street activity.
5. *Economy* aspect relates to the user costs associated with travel delays and inconvenience, and to the rental value and retail development as influenced by pedestrian environment.

These supplemental factors can have an important effect on the pedestrian perception of the overall quality of the street environment. While auto users have reasonable control over most of these factors, the pedestrian has virtually no control over them. Although the bulk of this chapter emphasizes level-of-service analysis, which relates primarily to pedestrian flow measures, such as speed and space, these environmental factors should always be considered because they can greatly influence pedestrian activity.

Pedestrian Speed-Density Relationships

The fundamental relationship between speed, density, and volume for pedestrian flow is analogous to vehicular flow. As *volume* and *density* increase, pedestrian *speed* declines. As density increases, and pedestrian space decreases, the degree of mobility afforded the individual pedestrian declines, as does the average speed of the pedestrian stream.

Figure 13-1 shows the relationship between speed and density for a variety of pedestrian classes as determined by four researchers, including two European sources. The density term, when used to describe pedestrian streams and specified in persons per square foot, will have small values, generally under 0.50.

Flow-Density Relationships

The relationship between density, speed, and flow for pedestrians is of the same form as for vehicular traffic streams, that is:

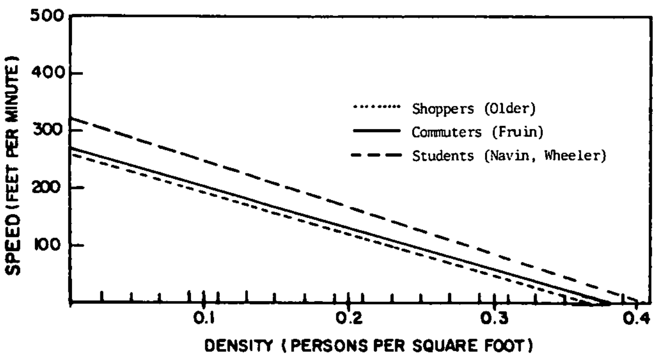


Figure 13-1. Relationships between pedestrian speed and density. (Source: Ref. *2)*

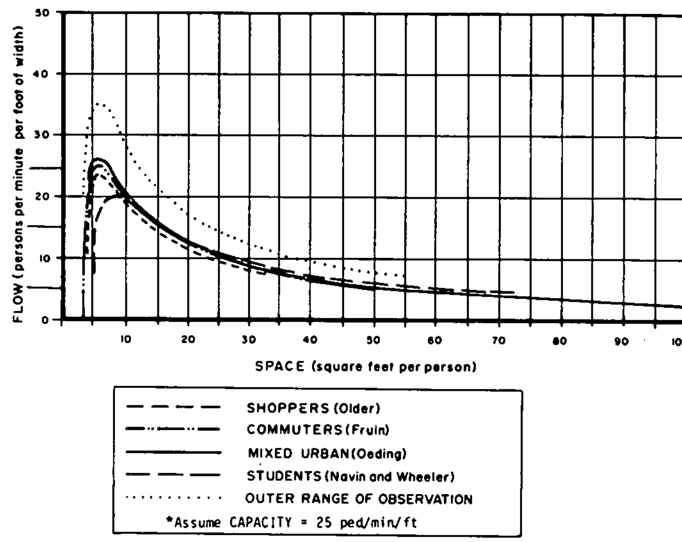


Figure 13-2. Relationships between pedestrian flow and space. (Source: Ref. *2)*

$$\text{Flow} = \text{Speed} \times \text{Density}$$
$$v = S \times D \qquad (13\text{-}1)$$

where *flow* is expressed as pedestrians per minute per foot, *speed* is expressed as feet per minute, and *density* is expressed as pedestrians per square foot.

The flow variable used in this expression is the "unit width flow" defined earlier. An alternative and more useful expression can be developed using the reciprocal of density, or *space,* as follows:

$$\text{Flow} = \text{Speed}/\text{Space}$$
$$v = S/M \qquad (13\text{-}2)$$

The basic relationship between flow and space, as recorded by several researchers, is illustrated in Figure 13-2.

The conditions at maximum flow are of interest because this represents the *capacity* of the walkway facility. From Figure 13-2, it is apparent that all observations of maximum unit flow fall within a very narrow range of density—that is, with the average space per pedestrian varying between 5 and 9 sq ft/ped. Even the outer range of these observations indicates that maximum flow occurs at this density, although the actual flow in this study is considerably higher than the others. As space is reduced to less than 5 sq ft/ped, the flow rate declines precipitously. All movement effectively stops at the minimum space allocation of 2 to 4 sq ft/ped.

These relationships show that pedestrian traffic can be evaluated qualitatively by using level-of-service concepts similar to vehicular traffic analysis. At flows near capacity, an average of 5 to 9 sq ft/ped is required for each moving pedestrian. However, at this level of flow, the limited area available restricts pedestrian speed and the pedestrian's freedom to maneuver within the pedestrian stream.

Speed-Flow Relationships

Figure 13-3 illustrates the relationship between pedestrian speed and flow. These curves, similar to vehicular flow curves, show that when there are few pedestrians on a walkway (low flow levels), space is available to choose higher walking speeds. As flow increases, speeds decline because of closer interactions

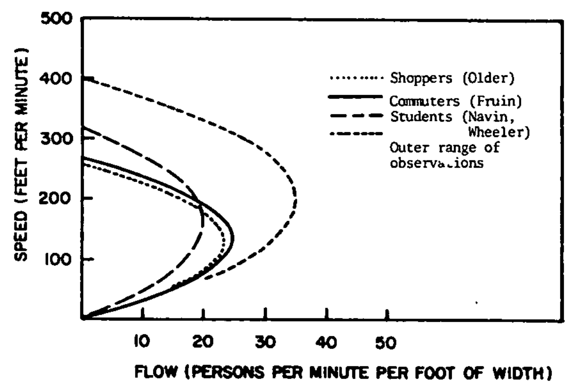


Figure 13-3. Relationships between pedestrian speed and flow. (Source: Ref. *2)*

with other pedestrians. When a critical level of crowding occurs, movement becomes more difficult, and both flow and speed decline.

Speed-Space Relationships

Figure 13-4 further confirms the relationships of walking speed and available space, and suggests some points of demarcation that can be used to develop level-of-service criteria. The outer range of observations shown on Figure 13-4 indicates that at an average space of about 15 sq ft/ped, even the slowest pedestrians cannot achieve their desired walking speed. Faster pedestrians wishing to walk at speeds up to 350 ft/min are not able to achieve such speeds until average space is 40 sq ft/ped or more. The space values of 15 and 40 sq ft/ped become critical points in defining level-of-service boundaries, as is illustrated in the "Methodology" section of this chapter.

EFFECTIVE WALKWAY WIDTH

The concept of a pedestrian "lane" has sometimes been used to analyze pedestrian flow, comparable to the analysis of a

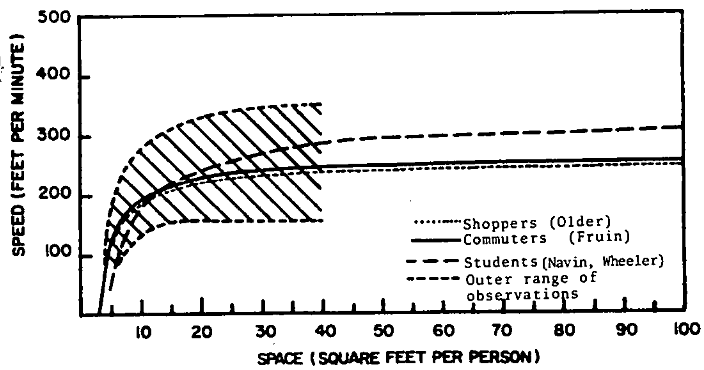


Figure 13-4. Relationships between pedestrian speed and space. (Source: Ref. *2)*

highway lane. The "lane" should not be used in pedestrian analysis, because photographic studies have shown that pedestrians do not walk in organized lanes. The "lane" concept is meaningful only in determining how many persons can walk abreast on a given walkway width, as in the case of determining the minimum sidewalk width to permit two pedestrians to conveniently pass by each other.

To avoid interference while passing each other, two pedestrians should *each* have at least 2.5 ft of walkway width, as observed by Oeding and Pushkarev (*2*). Pedestrians who know each other and are walking close together will each occupy a width of 2 ft, 2 in., a distance at which there is considerable likelihood of contact due to body sway. Lateral spacing less than this occurs only in the most crowded of situations.

The term "clear walkway width" is related to the portion of a walkway that can be effectively used for pedestrian movements. Moving pedestrians will shy away from the curb, and will not press closely against building walls. Therefore, unused space must be subtracted when determining pedestrian LOS. Further, a strip preempted by pedestrians standing near a building (as in window shopping) and/or near physical obstructions such as light poles, mail boxes, and parking meters, should also be excluded.

The degree to which point obstructions (poles, signs, hydrants) influence pedestrian movement and reduce effective walkway width is not extensively documented. While a single such obstruction would not reduce the effective width of an entire walkway, it would have such an effect in the immediate vicinity of the obstruction.

A list of typical obstructions and the estimated width of walkways which they preempt is provided in Table 13-2. Figure 13-5 shows the width of walkway preempted by curbs, buildings,

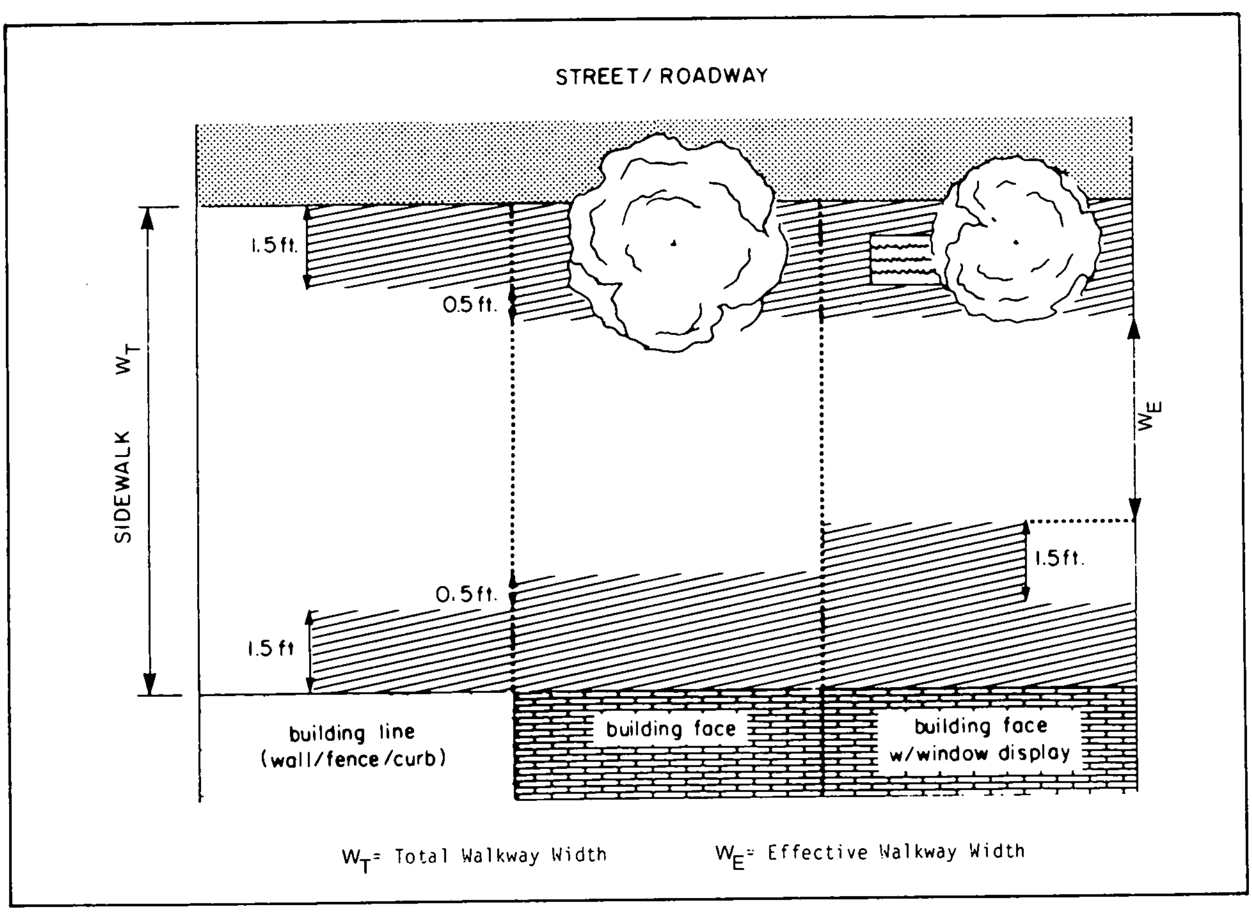


Figure 13-5. Preemption of walkway width. (Source: Adapted from Ref. *4)*

TABLE 13-2. FIXED OBSTACLE WIDTH ADJUSTMENT FACTORS FOR WALKWAYS*

OBSTACLE	APPROX. WIDTH PREEMPTED (FT)[a]
STREET FURNITURE	
Light Poles	2.5–3.5
Traffic Signal Poles and Boxes	3.0–4.0
Fire Alarm Boxes	2.5–3.5
Fire Hydrants	2.5–3.0
Traffic Signs	2.0–2.5
Parking Meters	2.0
Mail Boxes (1.7 ft by 1.7 ft)	3.2–3.7
Telephone Booths (2.7 ft by 2.7 ft)	4.0
Waste Baskets	3.0
Benches	5.0
PUBLIC UNDERGROUND ACCESS	
Subway Stairs	5.5–7.0
Subway Ventilation Gratings (raised)	6.0+
Transformer Vault Ventilation Gratings (raised)	5.0+
LANDSCAPING	
Trees	2.0–4.0
Planting Boxes	5.0
COMMERCIAL USES	
Newsstands	4.0–13.0
Vending Stands	variable
Advertising Displays	variable
Store Displays	variable
Sidewalk Cafes (two rows of tables)	variable, try 7.0
BUILDING PROTRUSIONS	
Columns	2.5–3.0
Stoops	2.0–6.0
Cellar Doors	5.0–7.0
Standpipe Connections	1.0
Awning Poles	2.5
Truck Docks (trucks protruding)	variable
Garage Entrance/Exit	variable
Driveways	variable

* To account for the avoidance distance normally occurring between pedestrians and obstacles, an additional 1.0 to 1.5 ft must be added to the preemption width for individual obstacles.
[a] Curb to edge of object, or building face to edge of object.
SOURCE: Ref. *2*

or fixed objects. Figure 13-5 may be used as a guideline when specific walkway configurations are not available.

PEDESTRIAN TYPE AND TRIP PURPOSES

The analysis of pedestrian flow is generally based on *mean,* or average, walking speeds of groups of pedestrians. Within any group, or among groups, there can be considerable differences in flow characteristics due to trip purposes, land use, type of group, age, and other factors. Figure 13-6 shows a typical distribution of free-flow walking speeds.

Pedestrians going to and from work, using the same facilities day after day, exhibit higher walking speeds than shoppers. This has been shown in Figure 13-1. Older or very young persons will tend to walk at a slower gait than other groups. Shoppers not only tend to walk slower than commuters, but may decrease the effective walkway width by stopping to window shop. Thus, in applying the techniques and numerical data in this chapter, the analyst should adjust for pedestrian behavior which deviates from the regular patterns represented in the basic speed, volume and density curves.

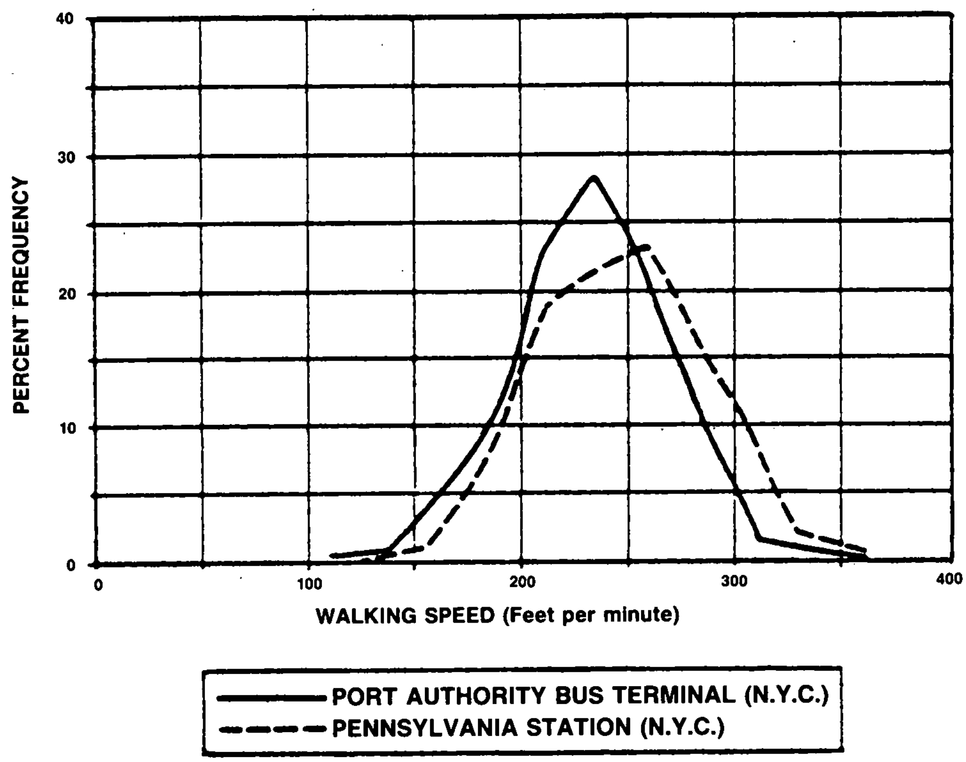


Figure 13-6. Typical free-flow walkway speed distribution. (Source: Ref. *3)*

II. METHODOLOGY

LEVELS OF SERVICE IN WALKWAYS

The criteria for various levels of service (LOS) for pedestrian flow are based on subjective measures that may be somewhat imprecise. However, it is possible to define ranges of *space per pedestrian, flow rates,* and *speeds* which can be used to develop quality of flow criteria.

Speed is an important level-of-service criterion because it can be easily observed and measured, and because it is a descriptor of the service pedestrians perceive. At speeds of 150 ft/min or less, most pedestrians resort to an unnatural "shuffling" gait. Figure 13-4 shows that this speed corresponds to a space per pedestrian in the range of 6 to 8 sq ft/ped. At 15 sq ft/ped or less, even the slowest walkers are forced to slow down (shown by the cross-hatching in Figure 13-4). The fastest walkers cannot reach their chosen speed of 350 ft/min until areas are over 40 sq ft/ped. Further, from Figure 13-2, it is evident that these three space values, 6, 15, and 40 sq ft/ped correspond approximately to the maximum flow at capacity, two-thirds of capacity, and one-third of capacity, respectively.

There are other significant indicators of service levels. For example, the ability of the pedestrian to cross a pedestrian stream is shown by Fruin (*3*) in Figure 13-7 to be impaired at areas below the 35- to 40-sq ft/ped range. Above that level, Fruin states that the probability of "stopping or breaking the normal walking gait" is reduced to zero. Below 15 sq ft/ped, virtually every crossing movement encounters a conflict. Similarly, the ability to pass slower pedestrians is unimpaired above 35 sq ft/ped, but becomes progressively more difficult as space allocations drop to 18 sq ft/ped, a point at which passing becomes virtually impossible.

Another level-of-service indicator is the ability to maintain flow in the minor direction in opposition to a major pedestrian flow. Here the quantitative evidence is somewhat less precise. For pedestrian streams of roughly equal flow in each direction, there is little reduction in the capacity of the walkway compared with one-way flow, because the directional streams tend to separate and occupy a proportional share of the walkway. However, if the bidirectional split is 90-10, and space is 10 sq ft/ped, capacity reductions of about 15 percent have been observed. This reduction is a consequence of the inability of the minority flow to utilize a proportional share of the walkway.

Photographic studies show that pedestrian movement on sidewalks is affected by the presence of other pedestrians, even at areas above 40 sq ft/ped. At 60 sq ft/ped, pedestrians have been observed walking in a "checkerboard" pattern, rather than

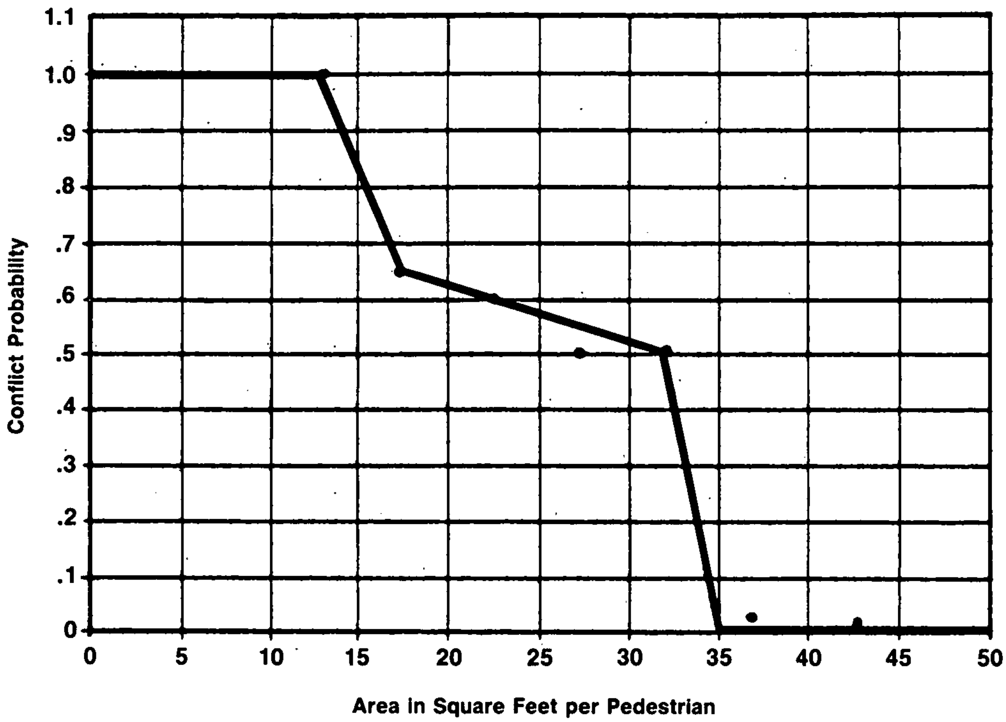


Figure 13-7. Cross-flow traffic—probability of conflict. (Source: Ref. *3)*

TABLE 13-3. PEDESTRIAN LEVEL OF SERVICE ON WALKWAYS*

LEVEL OF SERVICE	SPACE (SQ FT/PED)	EXPECTED FLOWS AND SPEEDS		
		AVE. SPEED, S (FT/MIN)	FLOW RATE, v (PED/MIN/FT)	VOL/CAP RATIO, v/c
A	$\geq$ 130	$\geq$ 260	$\leq$ 2	$\leq$ 0.08
B	$\geq$ 40	$\geq$ 250	$\leq$ 7	$\leq$ 0.28
C	$\geq$ 24	$\geq$ 240	$\leq$ 10	$\leq$ 0.40
D	$\geq$ 15	$\geq$ 225	$\leq$ 15	$\leq$ 0.60
E	$\geq$ 6	$\geq$ 150	$\leq$ 25	$\leq$ 1.00
F	< 6	< 150	----Variable----	

* Average conditions for 15 min.

directly behind or alongside each other. These same observations suggest that up to 100 sq ft/ped are required before completely free movement occurs without conflicts, and that at 130 sq ft/ped, individual pedestrians are no longer influenced by others (*5*). Bunching or "platooning" does not completely disappear until space is about 500 sq ft/ped or higher.

Walkway Level-of-Service Criteria

Table 13-3 shows the criteria for pedestrian level of service. The primary measure of effectiveness used in defining pedestrian level of service is *space,* the inverse of density. Mean speed and flow rate are shown as supplementary criteria. *Capacity* is taken to be 25 ped/min/ft, a representative value from Figures 13-2 and 13-3.

Graphic illustrations and descriptions of walkway levels of service are shown in Figure 13-8.

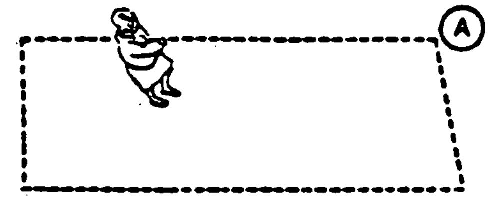
It should be noted that the pedestrian LOS, according to the criteria of Table 13-3, is quite good in most areas, as the high pedestrian flows required for the poorer levels generally occur only in and around major activity centers. In most areas, the design of walkways is based on the minimum widths required for voluntary pedestrian groups to pass each other and similar factors, rather than on the flow rate.

The LOS criteria apply to pedestrian flow and the space provided for that flow. Pedestrian facilities may also include extensive space intended to enhance the general environment that is not used or intended to handle basic pedestrian movements. When analyzing pedestrian flow rates per unit width of walkway, such space *should not be included.* Thus, pedestrian space intended to provide for window shopping, browsing, or

LEVEL OF SERVICE A

Pedestrian Space: ≥ 130 sq ft/ped Flow Rate: ≤ 2 ped/min/ft

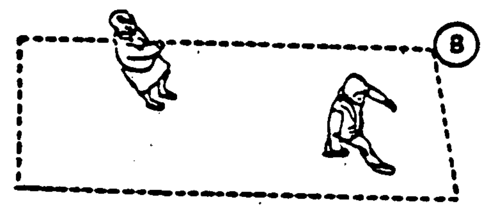
At walkway LOS A, pedestrians basically move in desired paths without altering their movements in response to other pedestrians. Walking speeds are freely selected, and conflicts between pedestrians are unlikely.



LEVEL OF SERVICE B

Pedestrian Space: ≥ 40 sq ft/ped Flow Rate: ≤ 7 ped/min/ft

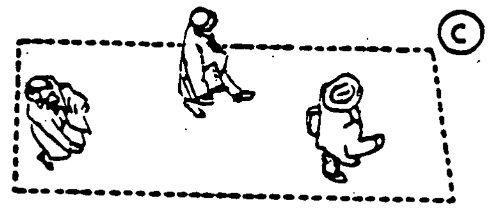
At LOS B, sufficient area is provided to allow pedestrians to freely select walking speeds, to bypass other pedestrians, and to avoid crossing conflicts with others. At this level, pedestrians begin to be aware of other pedestrians, and to respond to their presence in the selection of walking path.



LEVEL OF SERVICE C

Pedestrian Space: ≥ 24 sq ft/ped Flow Rate: ≤ 10 ped/min/ft

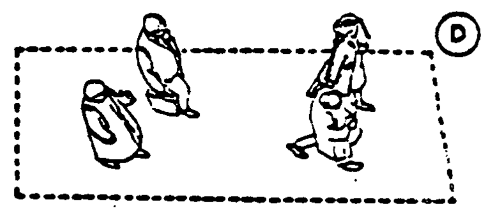
At LOS C, sufficient space is available to select normal walking speeds, and to bypass other pedestrians in primarily unidirectional streams. Where reverse-direction or crossing movements exist, minor conflicts will occur, and speeds and volume will be somewhat lower.



LEVEL OF SERVICE D

Pedestrian Space: ≥ 15 sq ft/ped Flow Rate: ≤ 15 ped/min/ft

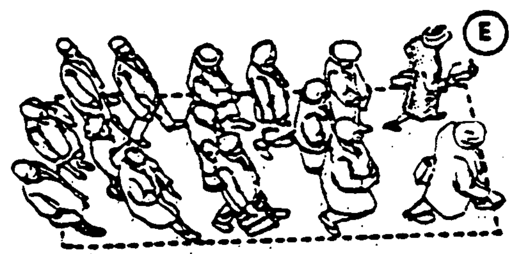
At LOS D, freedom to select individual walking speed and to bypass other pedestrians is restricted. Where crossing or reverse-flow movements exist, the probability of conflict is high, and its avoidance requires frequent changes in speed and position. The LOS provides reasonably fluid flow; however, considerable friction and interaction between pedestrians is likely to occur.



LEVEL OF SERVICE E

Pedestrian Space: ≥ 6 sq ft/ped Flow Rate: ≤ 25 ped/min/ft

At LOS E, virtually all pedestrians would have their normal walking speed restricted, requiring frequent adjustment of gait. At the lower range of this LOS, forward movement is possible only by "shuffling." Insufficient space is provided for passing of slower pedestrians. Cross- or reverse-flow movements are possible only with extreme difficulties. Design volumes approach the limit of walkway capacity, with resulting stoppages and interruptions to flow.



LEVEL OF SERVICE F

Pedestrian Space: ≤ 6 sq ft/ped Flow Rate: variable

At LOS F, all walking speeds are severely restricted, and forward progress is made only by "shuffling." There is frequent, unavoidable contact with other pedestrians. Cross- and reverse-flow movements are virtually impossible. Flow is sporadic and unstable. Space is more characteristic of queued pedestrians than of moving pedestrian streams.

Figure 13-8. Illustration of walkway levels of service.

simply sitting or standing in informal groups should not be considered to be part of the effective walkway width.

It should also be emphasized that the level-of-service criteria of Table 13-3 are based on the assumption that pedestrians distribute themselves uniformly throughout the effective walkway width. Pedestrian flow is subject to wide variability on a minute-by-minute basis, and the analyst must consider the effects of platooning as described in the next section.

Effect of Pedestrian Platoons

The average flow rates at different levels of service are of limited usefulness unless reasonable time intervals are specified. Figure 13-9 illustrates that "average flow rates" can be misleading. The data shown are for two locations in Lower Manhattan, but the pattern is generally characteristic of many concentrated CBD locations. The maximum 15-min flow rates average 1.4 and 1.9 ped/min/ft of effective walkway width during the periods measured. However, Figure 13-9 shows that flow during a 1-min interval can be more than double the rate in another, particularly at relatively low flows. Even during the peak 15-min period, incremental variations of 50 to 100 percent frequently occur from one minute to the next.

Depending on traffic patterns, it is clear that a facility designed for average flow can afford lower quality of flow for a proportion of the pedestrian traffic using it. However, it is extravagant to design for extreme peak 1-min flows which occur only 1 percent or 2 percent of the time. A relevant time period must therefore be determined through closer evaluation of the short-term fluctuations of pedestrian flow.

Short-term fluctuations are present in most unregulated pedestrian traffic flows because of random arrivals of pedestrians. On sidewalks, these random fluctuations are further exaggerated by the interruption of flow and queue formation caused by traffic signals. Transit facilities can create added surges in demand by releasing large groups of pedestrians in short time intervals, followed by pauses during which no flow occurs. Until they disperse, pedestrians in these types of groups move together as a platoon. Platoons can also form if passing is impeded because of insufficient space, and faster pedestrians slow down behind slower walkers.

It is important for the analyst to determine if platooning or other traffic patterns alter the underlying assumptions of average flow in LOS calculations, and to make appropriate adjustments where necessary.

In walkway sections having pronounced platooning effects, the duration and magnitude of these variations in demand should be established. This is done by timing and counting these short-term surges in demand. The magnitude and frequency of occurrence of the platoons would then be compared to the longer term 15-min average flow to provide a more accurate view of LOS conditions on the walkway segment.

The scatter diagram shown in Figure 13-10 indicates the platoon flow rate (i.e., the rate of flow within platoons of pedestrians) in comparison to the average flow rate for 58 data periods of 5- to 6-min duration. The dashed line approximates the upper limit of platoon flow observations.

The mathematical expression of this line relating maximum platoon flow rates to average flow rates is:

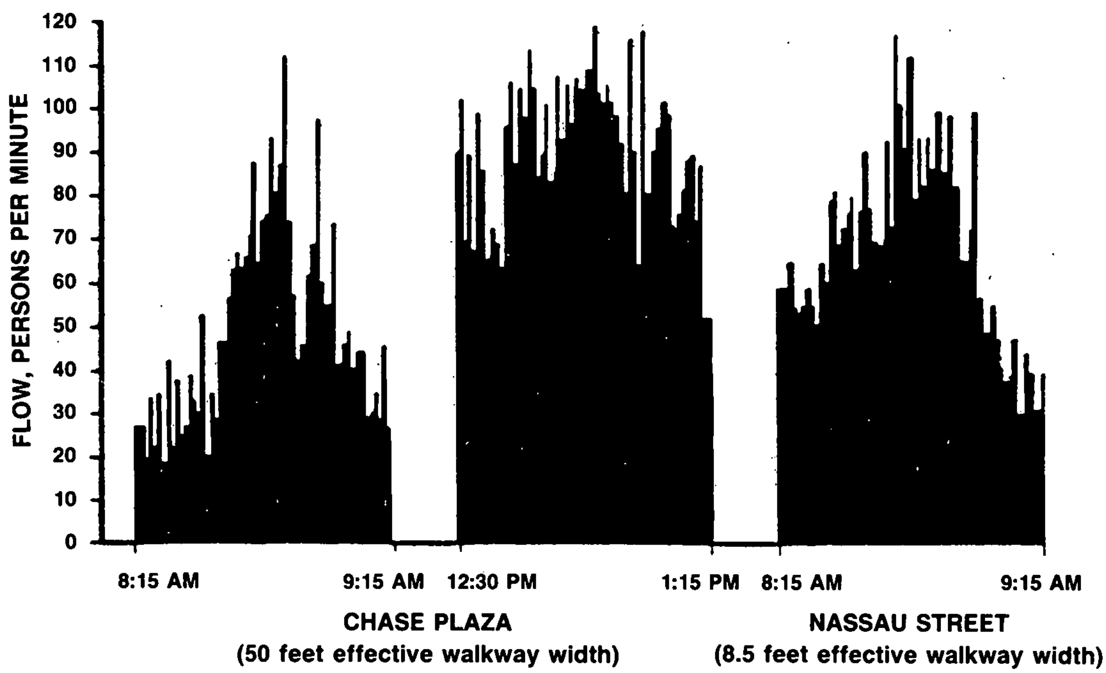


Figure 13-9. Minute-by-minute variations in pedestrian flow. (Source: Ref. *2*)

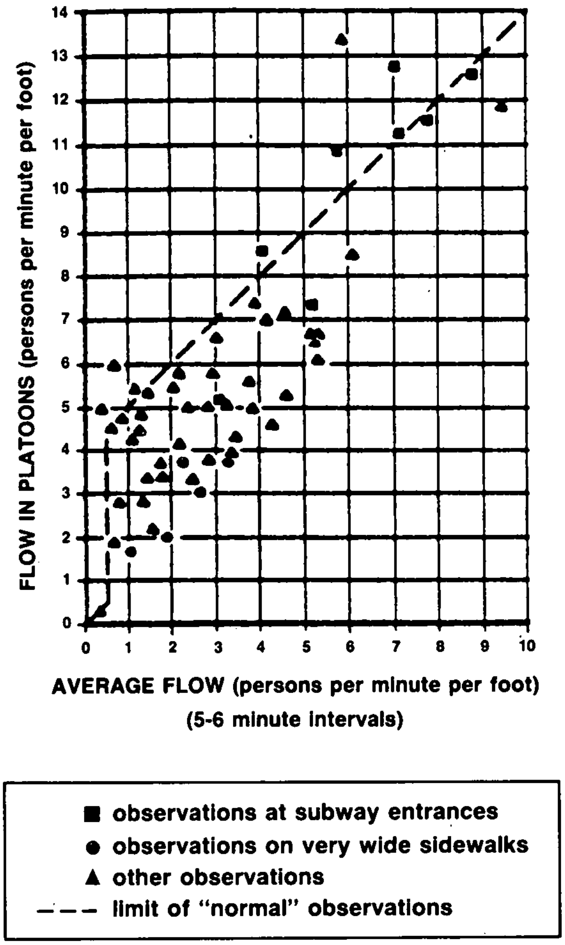


Figure 13-10. Relationship between platoon flow and average flow. (Source: Ref. *2*).

Platoon Flow = Average Flow + 4

$$v_p = v + 4 \qquad (13\text{-}3)$$

where both flows are expressed as pedestrians per minute per foot. This equation is valid for flows greater than 0.5 ped/min/ft. For lower flows, consult Figure 13-10 directly.

The form that this equation takes—a constant increment added to the average flow—shows that platooning has a relatively greater impact at low volumes than at high volumes. This pattern is logical, because gaps between platoons tend to fill up as flow increases. The equation can be used in general analyses where specific platooning data are not available.

Although the magnitude and frequency of platoons should be verified by field studies, the LOS occurring in platoons is generally *one level poorer than that determined by average flow criteria,* except for some cases of LOS E, which encompasses a broad range of pedestrian flow rates. The selection of an appropriate design objective to accommodate either average flows over a longer period, or the surges in demand occurring in platoons, depends on an evaluation of pedestrian convenience, available space, costs, and policy considerations.

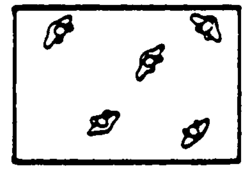
LEVELS OF SERVICE IN QUEUING AREAS

The concept of using the average space available to pedestrians as a walkway level-of-service measure can also be applied to queuing or waiting areas. In such areas, the pedestrian stands temporarily, while waiting to be served. The LOS of the waiting area is related to the average space available to each pedestrian and the degree of mobility allowed. In dense standing crowds, there is little room to move, but limited circulation is possible as average space per pedestrian is increased.

Level-of-service descriptions for standing spaces based on average pedestrian space, personal comfort, and degrees of internal mobility are shown on Figure 13-11. Standing areas in the LOS E category of 2–3 sq ft/ped are experienced only in the most crowded elevators or transit vehicles. LOS D, at 3–7 sq ft/ped, more typically exists where there is crowding, but where some internal maneuverability is still present. This commonly occurs at sidewalk corners where a large group of pedestrians is waiting to cross. Waiting areas where more space is required for circulation, such as theater lobbies and transit platforms, also require a higher LOS.

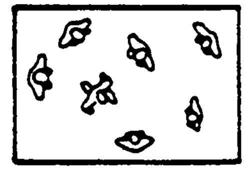
LEVEL OF SERVICE A

Average Pedestrian Area Occupancy: 13 sq ft/person or more
Average Inter-Person Spacing: 4 ft, or more
Description: Standing and free circulation through the queuing area is possible without disturbing others within the queue.



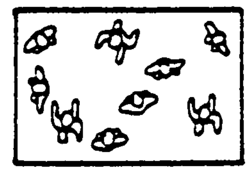
LEVEL OF SERVICE B

Average Pedestrian Area Occupancy: 10 to 13 sq ft/person
Average Inter-Person Spacing: 3.5 to 4.0 ft
Description: Standing and partially restricted circulation to avoid disturbing others within the queue is possible.



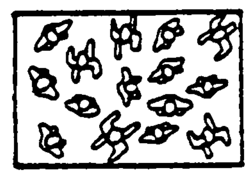
LEVEL OF SERVICE C

Average Pedestrian Area Occupancy: 7 to 10 sq ft/person
Average Inter-Person Spacing: 3.0 to 3.5 ft
Description: Standing and restricted circulation through the queuing area by disturbing others within the queue is possible; this density is within the range of personal comfort.



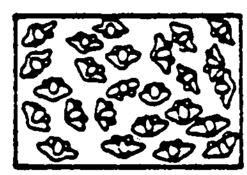
LEVEL OF SERVICE D

Average Pedestrian Area Occupancy: 3 to 7 sq ft/person
Average Inter-Person Spacing: 2 to 3 ft
Description: Standing without touching is possible; circulation is severely restricted within the queue and forward movement is only possible as a group; long term waiting at this density is discomforting.



LEVEL OF SERVICE E

Average Pedestrian Area Occupancy: 2 to 3 sq ft/person
Average Inter-Person Spacing: 2 ft or less
Description: Standing in physical contact with others is unavoidable; circulation within the queue is not possible; queuing at this density can only be sustained for a short period without serious discomfort.



LEVEL OF SERVICE F

Average Pedestrian Area Occupancy: 2 sq ft/person or less
Average Inter-Person Spacing: Close contact with persons
Description: Virtually all persons within the queue are standing in direct physical contact with those surrounding them; this density is extremely discomforting; no movement is possible within the queue; the potential for panic exists in large crowds at this density.

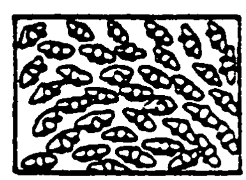


Figure 13-11. Levels of service for queuing areas. (Source: Ref. *3)*

APPLICATION OF CRITERIA

The application of these LOS criteria is relatively straightforward for walkways and waiting areas, as indicated in the previous sections. Two remaining pedestrian facilities of interest, however, present more complicated situations: street corners and crosswalks. Each of these is briefly discussed in the following sections.

Street Corners

The street corner is a more complex problem than the mid-block situation, involving intersecting sidewalk flows, pedestrians crossing the street, and others queued waiting for the signal to change. Because of the concentration of these activities, the corner is often the critical link in the pedestrian sidewalk network. An overloaded street corner can also affect vehicular operations by requiring added green crossing time or by delaying turning movements. There are two different types of pedestrian area requirements at corners:

1. *Circulation area*—Needed to accommodate (a) pedestrians crossing during the green signal phase, (b) those moving to join the red phase queue, and (c) those moving between the adjoining sidewalks, but not crossing the street.
2. *Hold area*—Needed to accommodate standing pedestrians waiting during the red signal phase.

Precise analysis of pedestrian activity at corners is difficult because of the many combinations of movements that are possible, as is illustrated in Figure 13-12. Each of the four directional movements into the corner may proceed straight ahead, or may turn left or right. This makes accurate collection of field data at busy intersections an almost impossible task. Methods for determining approximate LOS of street corners using more typically available crossing count data are given in the "Procedures for Application" section of this chapter.

The methodology is relatively straightforward and is adequate to establish problem locations which may require more detailed field study and possible remedial measures. Corrective measures could include sidewalk widenings, vehicle-turning restrictions, and/or changes in signal timing. Identifying problem areas is a primary objective of using LOS as an analytic tool.

Corners function as a "time-space" zone, with waiting pedestrians requiring less standing space, but occupying the corner for longer periods of time, and moving or circulating pedestrians requiring more space, but occupying the corner for only a few seconds. The total time-space available for these activities is simply the net area of the corner in square feet multiplied by the time of the analysis period. The analytical problem is allocation of this time-space in ways that provide a reasonable corner LOS for both waiting and moving pedestrians.

The method assumes that standing pedestrians waiting for the signal to change form a "competitive queue," in which each pedestrian occupies 5 sq ft/ped. This assumes midrange LOS D conditions within the queue, typical of many urban situations, and simplifies computational procedures. The average time moving pedestrians occupy the corner, typically in the range of 3 to 5 sec, is also assumed. This assumption of the travel time along the path of the longest dimension of a corner is actually conservative, as many pedestrians "short cut" corner edges, reducing their time-space requirements.

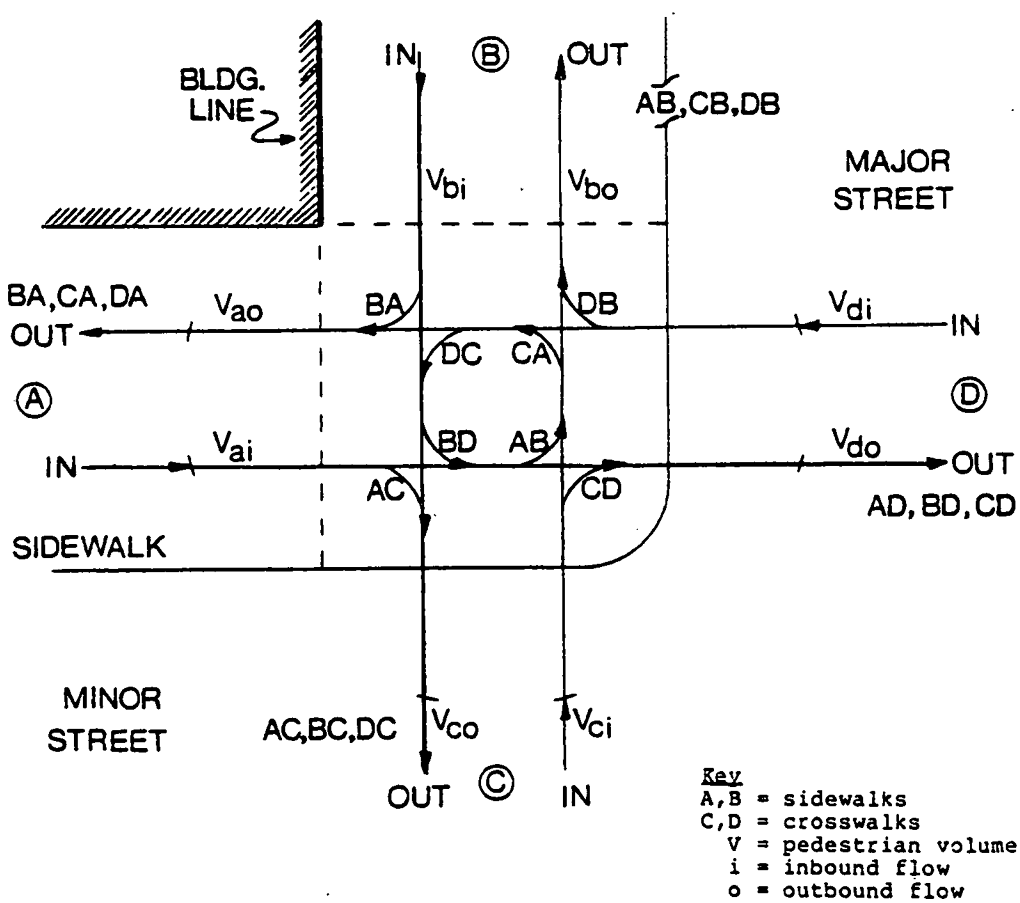


Figure 13-12. Pedestrian movements at a street corner.

Crosswalks

Pedestrian flow characteristics in crosswalks are similar to those on sidewalks, with the basic relationships of speed, density, space, and flow consistent with observed values for uninterrupted flow on walkways. However, traffic signals control movement on the crosswalk, collecting pedestrians into denser platoons, and altering the normal distribution of walking speeds. Average walking speed in crosswalks is frequently taken to be 4.5 ft/sec.

Level-of-service concepts developed primarily for movement of pedestrians on walkways can be applied to crosswalk analysis, but signal timing and the effects of turning vehicles during the pedestrian green phase can alter the underlying assumptions of the LOS analysis. Where crosswalk analyses show low pedestrian LOS, vehicle-turning restrictions must be seriously considered.

Like corners, the crosswalk can also be analyzed as a time-space zone. The available time-space is the product of the WALK phase time less a platoon start-up time, assumed to be 3 sec herein, and the area of the crosswalk in square feet. The product of pedestrian crossing flow and the average crossing time results in the demand for the space. Division of demand into the available time-space produces the space per moving pedestrian available during the green phase. This area can be compared with LOS criteria.

However, there is a brief maximum flow or surge condition during the WALK phase which must be examined. This occurs when the two lead platoons from opposite corners, formed during the waiting phase, are simultaneously in the crosswalk. Excessive pedestrian flows during this surge could cause pedestrians to drift out of the marked crosswalk area, potentially endangering them.

Neither the average nor the maximum estimate of crosswalk LOS accounts for the effects of turning vehicles during the pedestrian crossing phase. Rough estimates of pedestrian LOS degradation by turning vehicles can be made by assuming a vehicle swept path area and time in the crosswalk (time-space) decrement for each turning vehicle. An example of this is shown in the "Procedures for Application" section of this chapter. It should be noted, however, that the nature of pedestrian-vehicle interactions in the crosswalk may be greatly influenced by local right-of-way practices.

III. PROCEDURES FOR APPLICATION AND SAMPLE CALCULATIONS

In this chapter procedures for application and sample calculations are presented as a cohesive unit. Since procedures for analysis of walkways, street corners, and crosswalks are all relatively unique, illustrative calculations are shown with each procedural presentation.

ANALYSIS PROCEDURES FOR WALKWAYS

Computations for walkways are based on peak 15-min pedestrian counts. A midblock walkway should be counted for several different time periods during the day to establish variances in directional flows. For new locations or to analyze future conditions, forecasts of the flows must be made. Methods of forecasting pedestrian trip volumes and pedestrian trip generation rates for various types of land uses are contained in Ref. *8*.

Computational Steps

The methodology requires a specific sequence of computations which is presented below. Figure 13-13 is a worksheet which may be used in summarizing these computations.

1. Preliminary data needed to conduct an analysis include the following: For existing cases, field studies would be made to collect the information; for future cases, forecasts of demand and probable designs would be assumed:

- Peak 15-min pedestrian count, V_{P15}, in peds/15 min.
- Total walkway width, W_T, in ft
- Identification of obstacles in the walkway

2. The effective width of the walkway, W_E, must be determined by subtracting any unusable width from the total walkway width, W_T. Table 13-2 and Figure 13-5 can be used to estimate the unusable portion of walkway width.

3. The pedestrian unit flow rate, in ped/min/ft, is computed as:

$$v = V_{p15}/15W_E$$

4. The rate of flow within platoons may be estimated as:

$$v_p = v + 4$$

5. Levels of service for average or platoon conditions are found by comparing these flow rates to the criteria of Table 13-3.

Sample Calculation

1. *Description*—A given sidewalk segment on Third Street has a peak 15-min pedestrian flow of 1,250 ped/15 min. The 14-ft sidewalk has a curb on one side and stores with window shopping displays on the other. There are no other sidewalk obstructions. At what LOS does the sidewalk operate, on the average and within platoons?

WALKWAY ANALYSIS WORKSHEET

Location:____________________ *COUNTS*

City, State:____________________ Date:________

Time:________

Curb Line/Sidewalk Edge

PEAK 15-MIN FROM

______ to ______

W_{B1} (curb) = ft

W_{B2} (street furn.) = ft

W_T=____ W_E (effective width) = ft ← V_1 = ______

W_{B3} (window shop) = ft → V_2 = ______ (ped/15 min)

W_{B4} (bldg protrusions) = ft

W_{B5} (inside clearance = ft

Wall Line/Sidewalk Edge

Pedestrian Volume

V_1 = ______ ped/15 min

V_2 = ______ ped/15 min

$V_p = V_1 + V_2$ = ______ ped/15 min

Walkway Width

W_T = ______ ft

$W_B = W_{B1} + W_{B2} + W_{B3} + W_{B4} + W_{B5}$ = ______ ft

$W_E = W_T - W_B$ = ______ ft

Average Walkway LOS

$v = V_p/15W_E$ = ______ ped/min/ft

Average LOS = ______ (Table 13-3)

Platoon Walkway LOS

$v_p = v + 4$ = ______ ped/min/ft

Platoon LOS = ______ (Table 13-3)

Figure 13-13. Worksheet for walkway analysis.

2. *Solution*—The total sidewalk width of 14 ft must be reduced to account for unused "buffer" areas at the curb and building line. From Figure 13-5, the curb buffer is 1.5 ft, and the building buffer (with window shopping assumed) is 3.0 ft. Thus, the effective walkway width is 14.0 − 1.5 − 3.0 = 9.5 ft, and it is this figure that is used to determine the average and platoon flow rates.

The average unit width flow rate is computed as:

$$v = V_{P15}/15W_E$$
$$v = 1{,}250/(15 \times 9.5) = 8.8 \text{ ped/min/ft}$$

The rate of flow within platoons may then be estimated as:

$$v_p = v + 4$$
$$v = 8.8 + 4 = 12.8 \text{ ped/min/ft}$$

Table 13-3 is entered with these flow values to estimate the level of service. The LOS for average conditions is *C*, while the LOS within platoons is estimated to be *D*.

These computations can be summarized on the walkway analysis worksheet, as illustrated in Figure 13-14.

ANALYSIS PROCEDURES FOR STREET CORNERS AND CROSSWALKS

As noted previously, the analysis of street corners requires consideration of the amount of *circulation area* available for pedestrians moving through the corner, and the amount of *holding area* required for standing pedestrians waiting to cross the street. Figure 13-15 illustrates the geometrics of a typical street

WALKWAY ANALYSIS WORKSHEET

Location: *Third Street*

City, State: *Hempstead NY*

COUNTS

Date: *6/20/84*
Time: *PM*

PEAK 15-MIN FROM *4:15* to *4:30*

Curb Line/Sidewalk Edge

W_T = *14'*

W_{B1} (curb) =	1.5	ft
W_{B2} (street furn.) =	0	ft
W_E (effective width) =	9.5	ft
W_{B3} (window shop) =	3.0	ft
W_{B4} (bldg protrusions) =	0	ft
W_{B5} (inside clearance =	0	ft

Wall Line/Sidewalk Edge

V_1 = *625*
V_2 = *625*
(ped/15 min)

Pedestrian Volume

V_1 = 625 ped/15 min

V_2 = 625 ped/15 min

$V_p = V_1 + V_2$ = 1250 ped/15 min

Walkway Width

W_T = 14.0 ft

$W_B = W_{B1} + W_{B2} + W_{B3} + W_{B4} + W_{B5}$ = 4.5 ft

$W_E = W_T - W_B$ = 9.5 ft

Average Walkway LOS

$v = V_p/15W_E$ = 8.8 ped/min/ft

Average LOS = C (Table 13-3)

Platoon Walkway LOS

$v_p = v + 4$ = 12.8 ped/min/ft

Platoon LOS = D (Table 13-3)

Figure 13-14. Illustration of solution to walkway problem.

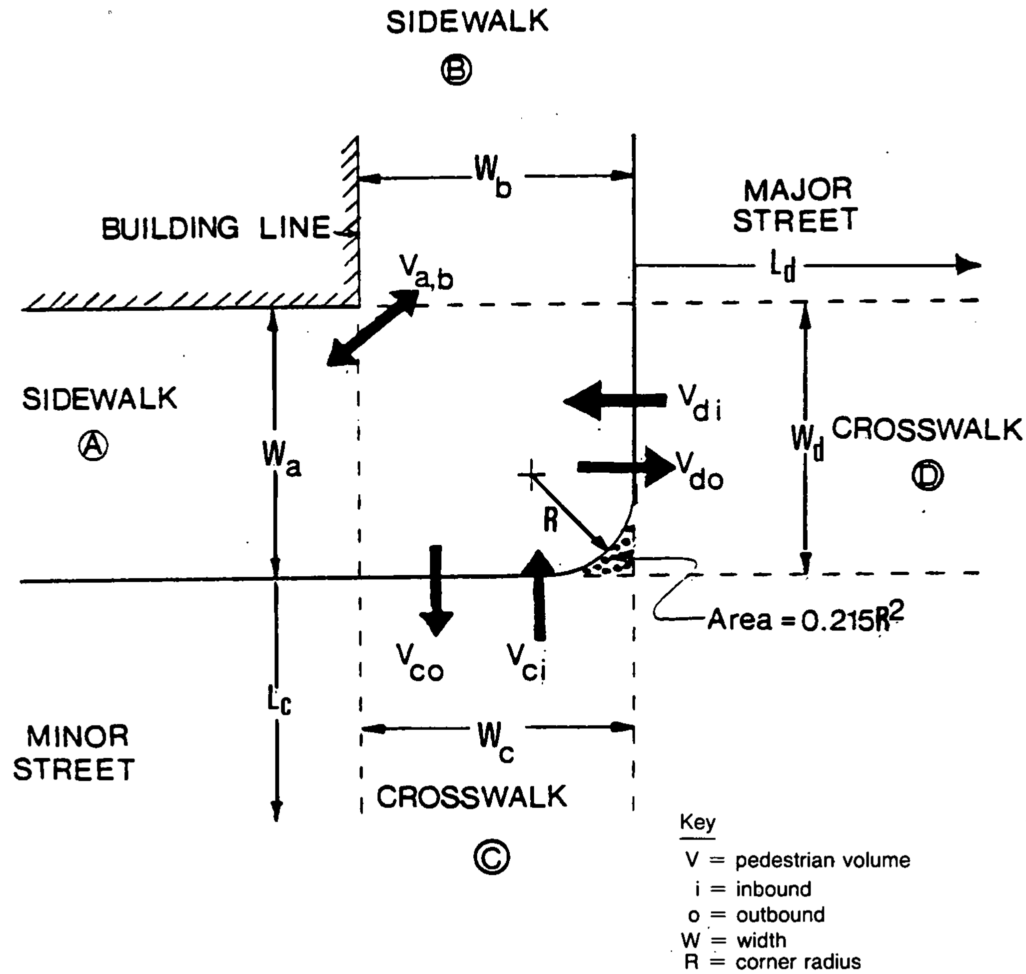


Figure 13-15. Intersection corner geometrics and pedestrian movements.

corner, and also the directional flow variables which will be used in subsequent LOS analyses.

Figures 13-16 and 13-17 show the two signal phase conditions which are analyzed both in corner and crosswalk computations. *Condition 1* is the minor street crossing phase during the major street green, with pedestrians held in a queue on the major street side during the minor street red phase. *Condition 2* is the major street crossing phase, with pedestrians crossing during the minor street green, and held in a queue on the minor street side by the major street red phase.

When making street corner computations, it is advisable to refer to Figures 13-15, 13-16, and 13-17 for graphic illustrations of the various parameters used.

The point of maximum pedestrian queuing and minimum available circulation space on the corner occurs just before the signal phase change. At this time, there is an average flow of outbound pedestrians leaving the corner, a more concentrated platoon of inbound pedestrians approaching from the opposite side of the street, and an average flow joining the pedestrian queue waiting to cross at the signal change. At this same time, there are also pedestrians moving between the intersecting sidewalks, not crossing the street.

The analysis of street corners and crosswalks is based on a comparison of available time and space to pedestrian demand. The product of time and space, i.e., time-space, is the critical parameter for consideration, because physical design limits available space and signalization controls available time.

In order to simplify the presentation and application of the time-space analysis approach, the development of relationships (equations) is presented in parallel with the solution of a sample calculation. Worksheets are illustrated in Figures 13-18 and 13-19 for crosswalk and street corner calculations respectively.

The sample calculation illustrated in the analysis of street corners and crosswalks is as follows:

1. *Description*—The sidewalks at a major and minor street intersection are each 16 ft wide, with a corner radius of 20 ft. The roadway width for the major street is 46 ft; and for the minor street, 28 ft. The signal cycle length, C, is 80 sec with a two-phase split of 48 sec of green plus amber, G_{mj}, for the major street (60 percent) and 32 sec of green plus amber, G_{mi}, for the minor street (40 percent). The 15-min peak period pedestrian crossing and sidewalk counts are shown below. Refer to Figure 13-15 for a graphic definition of flows.

Flow	Peak 15-Min Pedestrian Count	Average Flow Rate (ped/min)	Average Flow Per Cycle (ped/cycle)
v_{ci}	540	36	48
v_{co}	300	20	27
v_{di}	450	30	40
v_{do}	240	16	21
$v_{a,b}$	225	15	20
Totals	1,755	117	156

Note that flow rates in pedestrians per minute are rounded to the nearest integer. Pedestrians per cycle are computed by multiplying pedestrians per minute by the signal cycle length (in seconds) divided by 60 sec. For this calculation, the multiplier is 80 sec/60sec = 1.33. Pedestrians per cycle are also rounded to the nearest integer.

2. *Find—*

- The average LOS for pedestrian circulation at the street corner during a typical peak-period signal cycle.
- The average LOS for pedestrians crossing the minor and major streets.
- The decrement in average crosswalk pedestrian LOS due to five turning vehicles per cycle on the major street crossing.

Procedures for analysis of street corners and crosswalks are presented in a step-by-step fashion, along with the solution of the sample calculation.

Street Corner Analysis (Computational Steps and Sample Calculation)

Step 1—Determine Total Available Time-Space

The total time-space available in the intersection corner for circulation and queuing, for an analysis period of *t* minutes, is the product of the net corner area, *A,* and the time *t.* For street corner and crosswalk analysis, *t* is taken to be one signal cycle and is, therefore, equal to the cycle length, *C.* The net corner

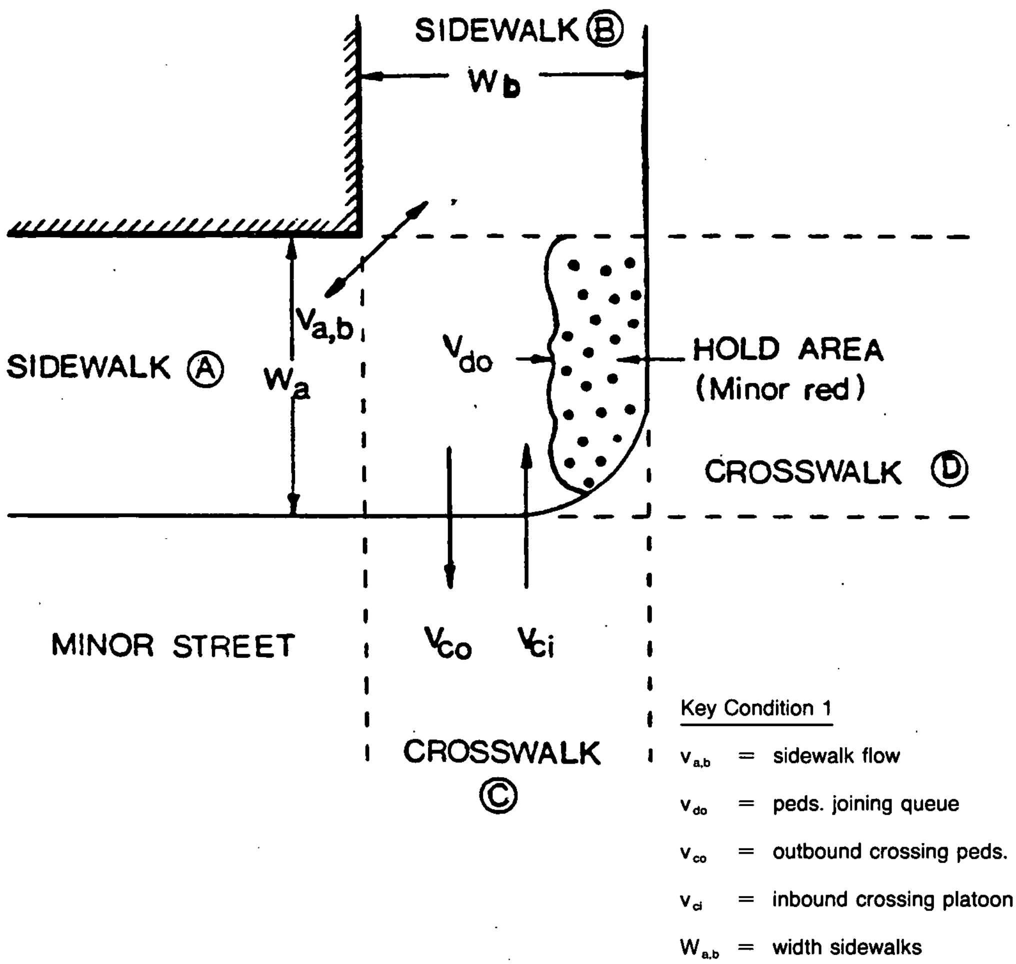


Figure 13-16. Intersection corner Condition 1—minor street crossing.

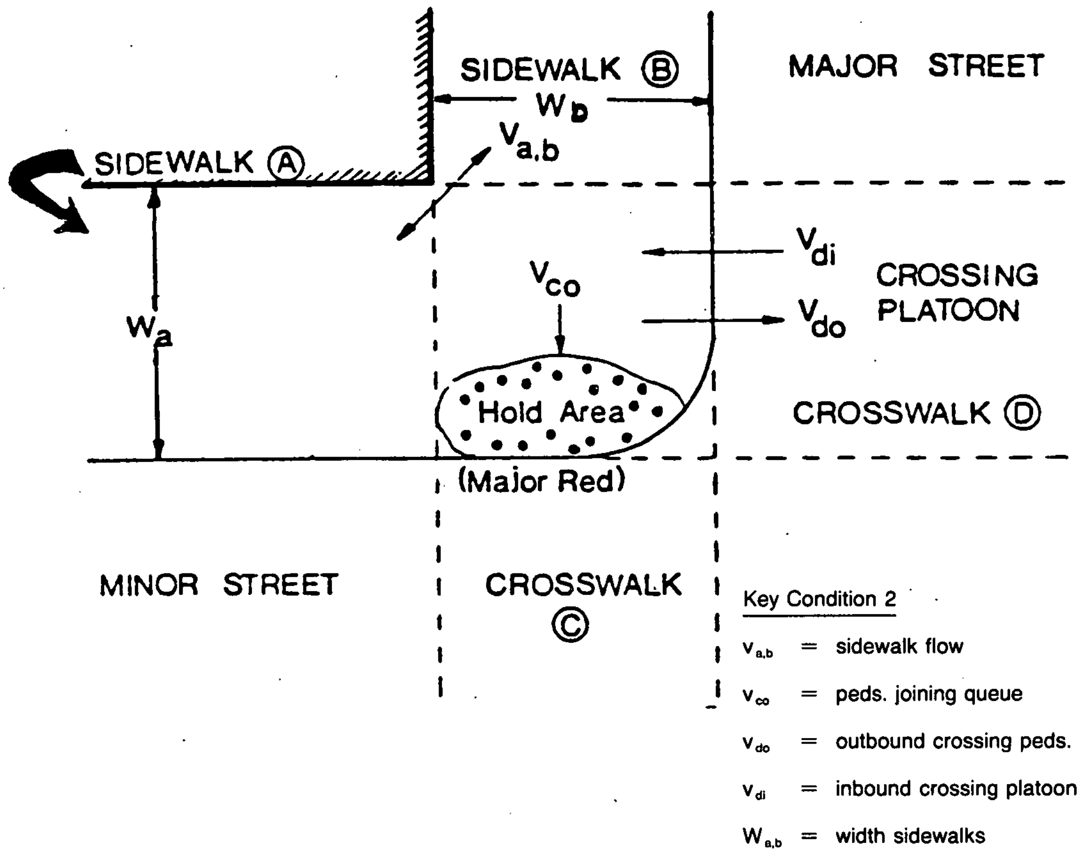


Figure 13-17. Intersection corner Condition 2—major street crossing.

area is found by multiplying the intersecting sidewalk widths, W_a and W_b, and deducting the area lost due to the corner radius and any obstructions. Then, assuming there are no obstructions in the corner area:

$$A = W_a W_b - 0.215 R^2 \quad (13\text{-}4)$$

$$TS = A \times C/60 \quad (13\text{-}5)$$

where:

A = area of the street corner, in sq ft;
W_a = width of the sidewalk a, in ft;
W_b = width of sidewalk b, in ft;
R = radius of corner curb, in ft.
C = cycle length, in sec; and
TS = total time-space available, in sq ft-min.

For the sample calculation described earlier, the following values may be computed:

$$A = (16 \times 16) - 0.215\,(20^2) = 170 \text{ sq ft}$$
$$TS = 170 \times 80/60 = 227 \text{ sq ft-min}$$

Step 2—Compute Holding Area Waiting Times

If uniform arrivals are assumed at the crossing queues, the average pedestrian holding times, Q_{tco} and Q_{tdo}, of persons waiting to use crosswalks C and D, respectively, is 1/2 the product of the outbound flows during a signal cycle (v_{co} and v_{do}, in ped/cycle), the proportion of cycle that these flows are held up, and their holding time based on the red signal phase:

For Condition 1, the minor street crossing, which occurs during the major street WALK or green phase:

$$Q_{tdo} = [v_{do} \times (R_{mi}/C) \times (R_{mi}/2)]/60 \quad (13\text{-}6)$$

For Condition 2, the major street crossing, which occurs during the minor street WALK or green phase:

$$Q_{tco} = [v_{co} \times (R_{mj}/C) \times (R_{mj}/2)]/60 \quad (13\text{-}7)$$

where:

Q_{tdo} = total time spent by pedestrians waiting to cross the major street during one signal cycle, in ped-min;
Q_{tco} = total time spent by pedestrians waiting to cross the minor street during one signal cycle, in ped-min;
v_{do} = the number of pedestrians per cycle crossing the minor street, in ped/cycle;
v_{co} = the number of pedestrians per cycle crossing the major street;
R_{mi} = the minor street red phase, or the DON'T WALK phase where pedestrian signals exist, in sec;

R_{mj} = the major street red phase, or the DON'T WALK phase where pedestrian signals exist, in sec; and
C = cycle length, in sec.

The term R/C is used to estimate the number of pedestrians per cycle that must wait for the green indication. The number is esimated as $v \times R/C$. Assuming that arrivals are uniformly distributed, each pedestrian that waits, does so for an average duration of R/2 sec. The division by 60 converts time from seconds to minutes.

For the sample calculations, the following values are computed:

$$Q_{tco} = [27 \times 0.40 \times 32/2]/60 = 2.9 \text{ ped min}$$
$$Q_{tdo} = [21 \times 0.60 \times 48/2]/60 = 5.0 \text{ ped-min}$$

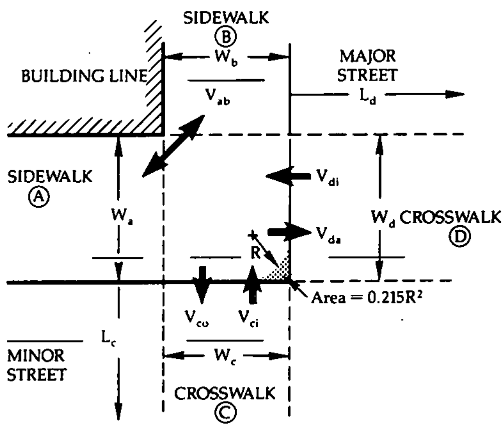
CROSSWALK ANALYSIS WORKSHEET

Location: ____________

City, State: ____________

SIGNAL TIMING (sec)

C = ______
G_{mj} = ______ R_{mj} = ______
G_{mi} = ______ R_{mi} = ______



PEDESTRIAN VOLUMES

Flow	Ped/Min	Ped/Cyc
v_{ci}		
v_{co}		
v_{di}		
v_{do}		
$v_{a,b}$		
v_{tot}		

CROSSWALK AREAS

$A_c = L_cW_c$ = ______ sq ft
$A_d = L_dW_d$ = ______ sq ft

CROSSWALK TIME-SPACE

$TS_c = A_c (G_{mj} - 3)/60$ = ______ sq ft-min
$TS_d = A_d (G_{mi} - 3)/60$ = ______ sq ft-min

CROSSING TIMES

$t_{wc} = L_c/4.5$ = ______ sec
$t_{wd} = L_d/4.5$ = ______ sec

CROSSWALK OCCUPANCY TIME (use ped/cycle)

$T_{wc} = (v_{ci} + v_{co})(t_{wc}/60)$ = ______ ped-min
$T_{wd} = (v_{di} + v_{do})(t_{wd}/60)$ = ______ ped-min

AVERAGE PEDESTRIAN SPACE AND LOS

$M_c = TS_c/T_{wc}$ = ______ sq ft/ped; LOS = ______ (Table 13-3)
$M_d = TS_d/T_{wd}$ = ______ sq ft/ped; LOS = ______ (Table 13-3)

MAXIMUM SURGE (use ped/min)

$V_{mc} = (v_{ci} + v_{co})(R_{mj} + 3 + t_{wc})/60$ = ______ ped
$V_{md} = (v_{di} + v_{do})(R_{mi} + 3 + t_{wd})/60$ = ______ ped

SURGE PEDESTRIAN SPACE AND SURGE LOS

$M_c\,(Max) = A_c/V_{mc}$ = ______ sq ft/ped; LOS = ______ (Table 13-3)
$M_d\,(Max) = A_d/V_{md}$ = ______ sq ft/ped; LOS = ______ (Table 13-3)

Figure 13-18. Worksheet for crosswalk analysis.

Step 3—Determine Holding Area Time-Space Requirements

The holding area needs of waiting pedestrians is the product of the total waiting times determined in Step 2 (Q_{tdo} and Q_{tco}), and the average area used by a waiting pedestrian, which is taken to be 5 sq ft/ped for a competitive queue. Then:

$$TS_h = 5\ (Q_{tdo} + Q_{tco}) \qquad (13\text{-}7)$$

where TS_h equals the total time-space holding area requirements for the intersection, in sq ft-min.

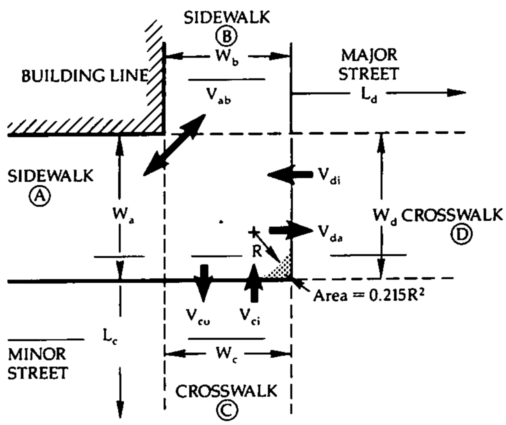
For the sample calculation, the following value is determined:

$$TS_h = 5\ (5.0 + 2.9) = 39.5, \text{ SAY } 40 \text{ sq ft-min}$$

STREET CORNER ANALYSIS WORKSHEET

Location: ____________________

City, State: ____________________



SIGNAL TIMING (sec)

C = ______

G_{mj} = ______ R_{mj} = ______

G_{mi} = ______ R_{mi} = ______

PEDESTRIAN VOLUMES

Flow	Ped/Min	Ped/Cyc
v_{ci}		
v_{co}		
v_{di}		
v_{do}		
$v_{a,b}$		
v_{tot}		

NET CORNER AREA $A = W_aW_b - 0.215R^2 =$ ______ sq ft

AVAILABLE TIME-SPACE $TS = A \times C/60 =$ ______ sq ft-min

HOLD AREA WAITING TIMES
(use ped/cycle)

$Q_{tco} = [(v_{co})\ (R_{mj}/C)\ (R_{mj}/2)]/60 =$ ______ ped-min

$Q_{tdo} = [(v_{do})\ (R_{mi}/C)\ (R_{mi}/2)]/60 =$ ______ ped-min

HOLD AREA TIME-SPACE

$TS_h = 5\ (Q_{tco} + Q_{tdo}) =$ ______ sq ft-min

CIRCULATION TIME-SPACE

$TS_c = TS - TS_h =$ ______ sq ft-min

TOTAL CIRCULATION VOLUME

$v_c = v_{ci} + v_{co} + v_{do} + v_{di} + v_{a,b} =$ ______ ped

TOTAL CIRCULATION TIME

$t_c = v_c \times 4/60 =$ ______ ped-min

PEDESTRIAN SPACE AND LOS

$M = TS_c/t_c =$ ______ sq ft/ped; LOS = ______
(Table 13-3)

Figure 13-19. Worksheet for street corner analysis.

Step 4—Determine the Net Corner Time-Space Available for Circulation

The total time-space available for circulation is the total intersection time-space minus that used for holding waiting pedestrians; or:

$$TS_c = TS - TS_h \qquad (13\text{-}8)$$

where TS_c equals the total time-space available for circulating pedestrians, in sq ft-min.

For the sample calculation:

$$TS_c = 227 - 40 = 187 \text{ sq ft-min}$$

Step 5—Determine the Total Number of Circulating Pedestrians Per Cycle

The number of pedestrians which must use the available circulation time-space during each cycle is the sum of all pedestrian flows, each flow is expressed in units of ped/cycle:

$$v_c = v_{ci} + v_{co} + v_{di} + v_{do} + v_{a,b} \qquad (13\text{-}9)$$

where v_c equals total number of circulating pedestrians, in ped/cycle.

For the sample calculation:

$$v_c = 48 + 27 + 40 + 21 + 20 = 156 \text{ ped}$$

Step 6—Determine the Total Circulation Time Utilized by Circulating Pedestrians

The time that pedestrians consume while walking through the corner area is taken as the product of the total circulation volume and an assumed average circulation time of 4 sec, or:

$$t_c = v_c \times 4/60 \qquad (13\text{-}10)$$

where t_c equals the total circulation time, in ped-min.

For the sample calculation:

$$t_c = 156 \times 4/60 = 10.4 \text{ ped-min}$$

Step 7—Determine the Circulation Area Per Pedestrian

The circulation area per pedestrian is referred to as the "pedestrian area module," and given the symbol, M. It is computed as the net time-space available for circulation, TS_c, divided by the total circulation time, t_c:

$$M = TS_c/t_c \qquad (13\text{-}11)$$

For the sample calculation:

$$M = 187/10.4 = 18.0 \text{ sq ft/ped}$$

Step 8—Determine the Corner Level of Service

The corner LOS is found by comparing the pedestrian area module, M, to the criteria found in Table 13-3. Values below LOS C indicate a potential problem that should be the subject of further field study and possible remedial actions, which could include changes in the signal timing, prohibition of vehicle-turning movements, sidewalk widening, and removal of sidewalk obstructions.

From Table 13-3, for a pedestrian area module of 18.0 sq ft/ped, the LOS for the sample calculation is found to be D. The need for further field study and possible remedial action is indicated.

Figure 13-20 illustrates the solution of the sample calculation on the street corner worksheet.

Crosswalk Analysis (Computational Steps and Sample Calculation)

Analysis procedures for crosswalks use the same basic principles of accounting for time-space. The procedure is explained in the following steps.

Step 1—Determine the Total Available Time-Space

The total time-space available in the crosswalk during one signal cycle is the product of the crosswalk area and the WALK interval for the crosswalk. Where pedestrian signals are not present, the green time minus 3 sec is substituted for WALK time. Note that in computing crosswalk area, the effect of the corner radius is not considered. Then:

$$A_w = W \times L \qquad (13\text{–}12)$$

$$TS_w = A_w \times G_w/60 \qquad (13\text{–}13)$$

where:

A_w = area of the crosswalk, in sq ft;
W = width of the crosswalk, in ft;
L = length of the crosswalk, in ft;
TS_w = Total time-space available in the crosswalk during one signal cycle, in sq ft-min; and
G_w = WALK interval, in sec.

Then, for Crosswalk C in the illustrative calculation:

$$A = 16 \times 28 = 448 \text{ sq ft}$$

$$TS_w = 448 \times (48 - 3)/60 = 336 \text{ sq ft-min}$$

and for Crosswalk D:

$$A = 16 \times 46 = 736 \text{ sq ft}$$

$$TS_w = 736 \times (32 - 3)/60 = 356 \text{ sq ft-min}$$

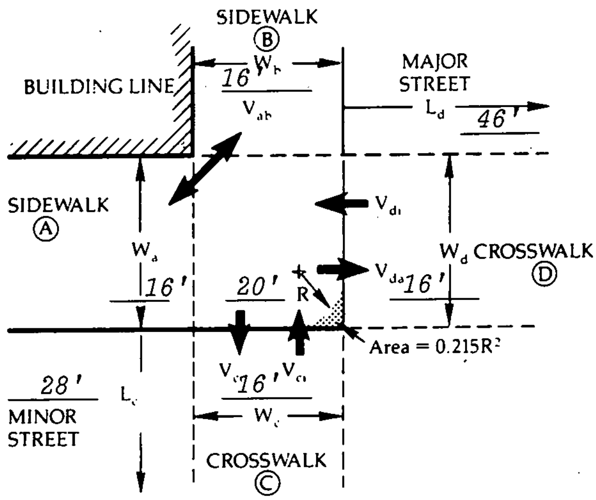
Step 2—Determine the Average Crossing Times

The average time a pedestrian occupies each crosswalk is

STREET CORNER ANALYSIS WORKSHEET

Location: *Ralph Ave. & Crossway Blvd.*

City, State: *Townsville, KY*



SIGNAL TIMING (sec)

C = 80
G_{mj} = 48 R_{mj} = 32
G_{mi} = 32 R_{mi} = 48

PEDESTRIAN VOLUMES

Flow	Ped/Min	Ped/Cyc
v_{ci}	36	48
v_{co}	20	27
v_{di}	30	40
v_{do}	16	21
$v_{a,b}$	15	20
v_{tot}	117	156

NET CORNER AREA $A = W_a W_b - 0.215R^2 =$ 170 sq ft

AVAILABLE TIME-SPACE $TS = A \times C/60 =$ 227 sq ft-min

HOLD AREA WAITING TIMES
(use ped/cycle)

$Q_{tco} = [(v_{co})(R_{mj}/C)(R_{mj}/2)]/60 =$ 2.9 ped-min

$Q_{tdo} = [(v_{do})(R_{mi}/C)(R_{mi}/2)]/60 =$ 5.0 ped-min

HOLD AREA TIME-SPACE

$TS_h = 5(Q_{tco} + Q_{tdo}) =$ 39.5~40 sq ft-min

CIRCULATION TIME-SPACE

$TS_c = TS - TS_h =$ 187 sq ft-min

TOTAL CIRCULATION VOLUME

$v_c = v_{ci} + v_{co} + v_{do} + v_{di} + v_{a,b} =$ 156 ped

TOTAL CIRCULATION TIME

$t_c = v_c \times 4/60 =$ 10.4 ped-min

PEDESTRIAN SPACE AND LOS

$M = TS_c/t_c =$ 18.0 sq ft/ped; LOS = D
(Table 13-3)

Figure 13-20. Worksheet for street corner analysis of sample calculation.

obtained by dividing the length of the crosswalk (street width) by the assumed walking speed. Average walking speed in crosswalks is taken to be 4.5 ft/sec. Then:

$$t_w = L/4.5 \tag{13-14}$$

where:

t_w = average time spent by pedestrian in the crosswalk, in sec; and

L = length of the crosswalk, in ft.

Then, for the sample calculation Crosswalk C:

$$t_w = 28/4.5 = 6.2 \text{ sec}$$

and for Crosswalk D:

$$t_w = 46/4.5 = 10.2 \text{ sec}$$

Step 3—Determine the Total Crosswalk Occupancy Time

The total crosswalk occupancy time is the product of the average crossing time and the number of pedestrians using the crosswalk during one signal cycle. Then:

$$T_w = (v_i + v_o)\ t_w/60 \tag{13-15}$$

where:

T_w = total crosswalk occupancy time, in ped-min;

v_i = incoming pedestrian volume for the subject crosswalk, in ped/cycle; and

v_o = outgoing pedestrian volume for the subject crosswalk, in ped/cycle.

For the sample calculation, Crosswalk C:

$$T_w = (48 + 27)\ 6.2/60 = 7.8 \text{ ped-min}$$

and for Crosswalk D:

$$T_w = (40 + 21)\ 10.2/60 = 10.4 \text{ ped-min}$$

Step 4—Determine the Average Circulation Space per Pedestrian and the Average Level of Service

The average circulation space provided for each pedestrian is determined by dividing the time-space available for crossing by the total occupancy time. This yields the average area module provided for each pedestrian, which is related to level of service by the criteria of Table 13-3.

$$M = TS_w/T_w \tag{13-16}$$

For Crosswalk C:

$$M = 336/7.8 = 43 \text{ sq ft/ped (LOS B, Table 13-3)}$$

and for Crosswalk D:

$$M = 356/10.4 = 34 \text{ sq ft/ped (LOS C, Table 13-3)}$$

Step 5—Determine the Level of Service for the Maximum Surge Condition

Step 4 yields an analysis of conditions that are average for the WALK interval. The point at which the maximum number of pedestrians are in the crosswalk should also be examined. This occurs when the lead pedestrians in opposing crossing platoons reach the opposite corner. The area module for the surge condition is the area of the crosswalk divided by the maximum number of pedestrians in the crosswalk. Crosswalk flows in pedestrians per minute (NOT the ped/cycle units which have been used for other analysis steps) are multiplied by the DON'T WALK interval + the crossing time, t_w. The DON'T WALK interval is used to estimate the number of pedestrians queued when the WALK interval is given, and the crossing time is added to estimate the number of new arriving pedestrians during the period that the queued pedestrians cross the street. Where pedestrian signals are not present, the red interval + 3 sec is substituted for the DON'T WALK interval. Then:

$$V_m = (v_i + v_o)\ (R_w + t_w)/60 \tag{13-17}$$

$$M = A/V_m \tag{13-18}$$

where:

V_m = maximum number of pedestrians occupying crosswalk;

v_i = incoming crosswalk volume, in ped/min;

v_o = outgoing crosswalk volume, in ped/min; and

R_w = DON'T WALK interval, in sec.

For the sample calculation, Crosswalk C:

$$V_m = (36 + 20) \times (32 + 3 + 6.2)/60 = 38.5 \text{ ped}$$

$$M = 448/38.5 = 11.6 \text{ sq ft/ped (LOS E, Table 13-3)}$$

and for Crosswalk D:

$$V_m = (30 + 16) \times (48 + 3 + 10.2)/60 = 46.9, \text{ SAY 47 ped}$$

$$M = 736/47 = 15.7 \text{ sq ft/ped (LOS D, Table 13-3)}$$

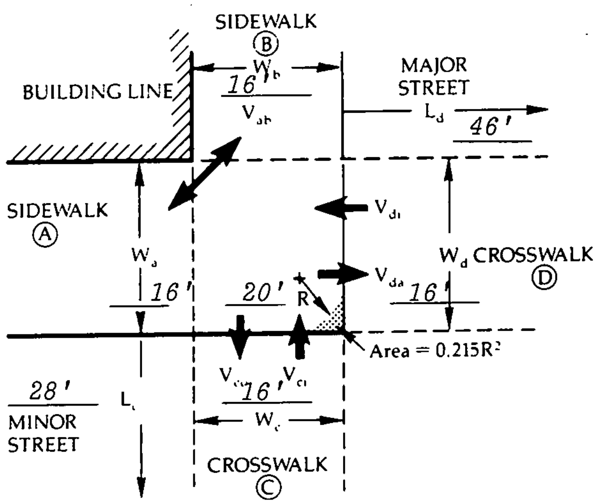
Note that the surge LOS is worse than the average LOS, particularly for Crosswalk C, where the value fell from B for average conditions to E for surge conditions. This emphasizes the need to consider both conditions.

Figure 13-21 shows the worksheet for the sample calculation discussed herein.

CROSSWALK ANALYSIS WORKSHEET

Location: *Ralph Ave. & Crossway Blvd.*

City, State: *Townsville, KY*



SIGNAL TIMING (sec)

C = *80*

G_{mj} = *48* R_{mj} = *32*

G_{mi} = *32* R_{mi} = *48*

PEDESTRIAN VOLUMES

Flow	Ped/Min	Ped/Cyc
v_{ci}	36	48
v_{co}	20	27
v_{di}	30	40
v_{do}	16	21
$v_{a,b}$	15	20
v_{tot}	117	156

CROSSWALK AREAS

$A_c = L_c W_c =$ *448* sq ft

$A_d = L_d W_d =$ *736* sq ft

CROSSWALK TIME-SPACE

$TS_c = A_c (G_{mj} - 3)/60 =$ *336* sq ft-min

$TS_d = A_d (G_{mi} - 3)/60 =$ *356* sq ft-min

CROSSING TIMES

$t_{wc} = L_c/4.5 =$ *6.2* sec

$t_{wd} = L_d/4.5 =$ *10.2* sec

CROSSWALK OCCUPANCY TIME (use ped/cycle)

$T_{wc} = (v_{ci} + v_{co})(t_{wc}/60) =$ *7.8* ped-min

$T_{wd} = (v_{di} + v_{do})(t_{wd}/60) =$ *10.4* ped-min

AVERAGE PEDESTRIAN SPACE AND LOS

$M_c = TS_c/T_{wc} =$ *43* sq ft/ped; LOS = *B* (Table 13-3)

$M_d = TS_d/T_{wd} =$ *34* sq ft/ped; LOS = *C* (Table 13-3)

MAXIMUM SURGE (use ped/min)

$V_{mc} = (v_{ci} + v_{co})(R_{mj} + 3 + t_{wc})/60 =$ *38.5* ped

$V_{md} = (v_{di} + v_{do})(R_{mi} + 3 + t_{wd})/60 =$ *46.9* ped

SURGE PEDESTRIAN SPACE AND SURGE LOS

$M_c\,(\text{Max}) = A_c/V_{mc} =$ *11.6* sq ft/ped; LOS = *E* (Table 13-3)

$M_d\,(\text{Max}) = A_d/V_{md} =$ *15.7* sq ft/ped; LOS = *D* (Table 13-3)

Figure 13-21. Worksheet for crosswalk analysis of sample calculation.

Estimating the Decrement to Crosswalk LOS Due to Right-Turning Vehicles

The time-space method allows for an approximate estimate to be made of the effect of turning vehicles on the average LOS for pedestrians crossing during a given green phase. This is done by assuming an average area occupancy of a vehicle in the crosswalk, based on the product of vehicle swept-path and crosswalk widths, and an estimate of the time that the vehicle preempts this space. The swept-path for most vehicles may be estimated at an average of 8 ft, and it is assumed that a vehicle occupies the crosswalk for a period of 5 sec.

For the sample calculation, each turning vehicle will preempt:

[8 ft × 16 ft (crosswalk width) × 5 sec]/60 = 10.7 sq ft-min/veh

If 5 vehicles turn during an average green phase, the total decrement to available time-space would be: 10.7 × 5 = 54 sq ft/min.

For the major street crossing (Crosswalk D), the total available time-space was computed to be 356 sq ft-min. Deducting 54 sq ft-min, only 302 sq ft-min remain for pedestrian use. The pedestrian space module is now recomputed using this figure in Eq. 13-16:

M = 302/10.4 = 29 sq ft/ped (LOS C, Table 13-3)

In this case, the decrement has not caused a reduction in the LOS, although the area per pedestrian is clearly reduced. This is an indication that the crosswalk can handle both the pedestrian demands and the turning vehicle demands without experiencing a capacity or delay problem. Where the decrement causes a significant decline in LOS, particularly where LOS F would result, further field studies and remedial action should be pursued.

IV. REFERENCES

The basic pedestrian characteristics used in this chapter were presented in Transportation Research Board Circular 212 (*1*). Pioneering references of great interest were authored by Pushkarev and Zupan (*2*) and Fruin (*3*). References *4* through *8* offer additional information for interested users of this manual.

1. "Interim Materials on Highway Capacity." *Transportation Research Board Circular 212,* Transportation Research Board, Washington, D.C. (1980).
2. PUSHKAREV, B., and ZUPAN, J., *Urban Space for Pedestrians.* MIT Press, Cambridge, Mass. (1975).
3. FRUIN, J., *Pedestrian Planning and Design.* Metropolitan Association of Urban Designers and Environmental Planners, New York, N.Y. (1971).
4. *Feasibility Analysis and Design Concepts and Criteria for Communitywide Separated Pedestrian Networks.* Phase III, Draft Pedestrian Planning Procedures Manual, Vols. I–III, RTKL Associates, Baltimore, Md. (1977).
5. HALL, D., *The Hidden Dimension.* Doubleday and Co., New York, N.Y. (1966).
6. VIRKLER, M., and GUELL, D., "Pedestrian Crossing Time Requirements at Intersections." *Transportation Research Record 959,* Transportation Research Board, Washington, D.C. (1984) pp. 47-51.
7. FRUIN, J, and BENZ, G., "Pedestrian Time-Space Concept for Analyzing Corners and Crosswalks." *Transportation Research Record 959,* Transportation Research Board, Washington, D.C. (1984) pp. 18-24.
8. KAGAN, L., ET AL., "*A Pedestrian Planning Procedures Manual.*" Vols. I–II, *FHWA Report Nos. RD-78-45, RD-78-46, RD-79-47,* Federal Highway Administration, Washington, D.C. (Nov. 1978).

APPENDIX I

WORKSHEETS FOR USE IN ANALYSIS OF WALKWAYS, CROSSWALKS, AND STREET CORNERS

WORKSHEETS

WALKWAY ANALYSIS WORKSHEET

Location: ____________________

City, State: ____________________

COUNTS

Date: __________

Time: __________

PEAK 15-MIN FROM

______ to ______

Curb Line/Sidewalk Edge

W_T = ____

W_{B1} (curb) = ____ ft

W_{B2} (street furn.) = ____ ft

W_E (effective width) = ____ ft ← V_1 = ______

W_{B3} (window shop) = ____ ft → V_2 = ______ (ped/15 min)

W_{B4} (bldg protrusions) = ____ ft

W_{B5} (inside clearance = ____ ft

Wall Line/Sidewalk Edge

Pedestrian Volume

$V_1 =$ ______ ped/15 min

$V_2 =$ ______ ped/15 min

$V_p = V_1 + V_2 =$ ______ ped/15 min

Walkway Width

$W_T =$ ______ ft

$W_B = W_{B1} + W_{B2} + W_{B3} + W_{B4} + W_{B5} =$ ______ ft

$W_E = W_T - W_B =$ ______ ft

Average Walkway LOS

$v = V_p/15W_E =$ ______ ped/min/ft

Average LOS = ______ (Table 13-3)

Platoon Walkway LOS

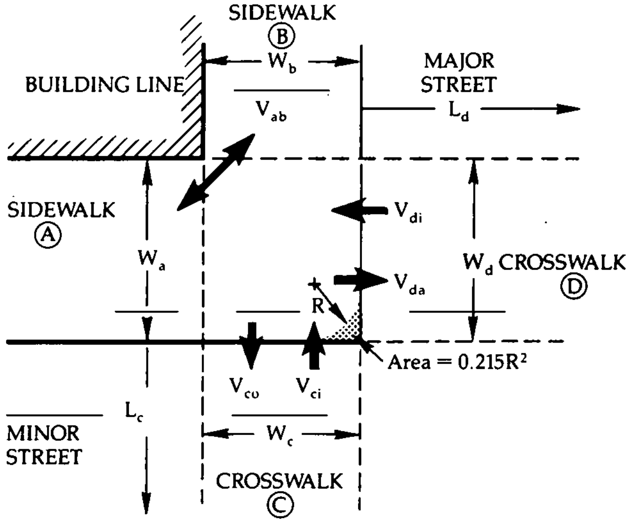
$v_p = v + 4 =$ ______ ped/min/ft

Platoon LOS = ______ (Table 13-3)

CROSSWALK ANALYSIS WORKSHEET

Location: ______________________________

City, State: ______________________________



SIGNAL TIMING (sec)

C = ________

G_{mj} = ________ R_{mj} = ________

G_{mi} = ________ R_{mi} = ________

PEDESTRIAN VOLUMES

Flow	Ped/Min	Ped/Cyc
v_{ci}		
v_{co}		
v_{di}		
v_{do}		
$v_{a,b}$		
v_{tot}		

CROSSWALK AREAS

$A_c = L_cW_c =$ ________ sq ft

$A_d = L_dW_d =$ ________ sq ft

CROSSWALK TIME-SPACE

$TS_c = A_c(G_{mj} - 3)/60 =$ ________ sq ft-min

$TS_d = A_d(G_{mi} - 3)/60 =$ ________ sq ft-min

CROSSING TIMES

$t_{wc} = L_c/4.5 =$ ________ sec

$t_{wd} = L_d/4.5 =$ ________ sec

CROSSWALK OCCUPANCY TIME
(use ped/cycle)

$T_{wc} = (v_{ci} + v_{co})(t_{wc}/60) =$ ________ ped-min

$T_{wd} = (v_{di} + v_{do})(t_{wd}/60) =$ ________ ped-min

AVERAGE PEDESTRIAN SPACE AND LOS

$M_c = TS_c/T_{wc} =$ ________ sq ft/ped; LOS = ________ (Table 13-3)

$M_d = TS_d/T_{wd} =$ ________ sq ft/ped; LOS = ________ (Table 13-3)

MAXIMUM SURGE
(use ped/min)

$V_{mc} = (v_{ci} + v_{co})(R_{mj} + 3 + t_{wc})/60 =$ ________ ped

$V_{md} = (v_{di} + v_{do})(R_{mi} + 3 + t_{wd})/60 =$ ________ ped

SURGE PEDESTRIAN SPACE AND SURGE LOS

$M_c(\text{Max}) = A_c/V_{mc} =$ ________ sq ft/ped; LOS = ________ (Table 13-3)

$M_d(\text{Max}) = A_d/V_{md} =$ ________ sq ft/ped; LOS = ________ (Table 13-3)

STREET CORNER ANALYSIS WORKSHEET

Location: ____________________

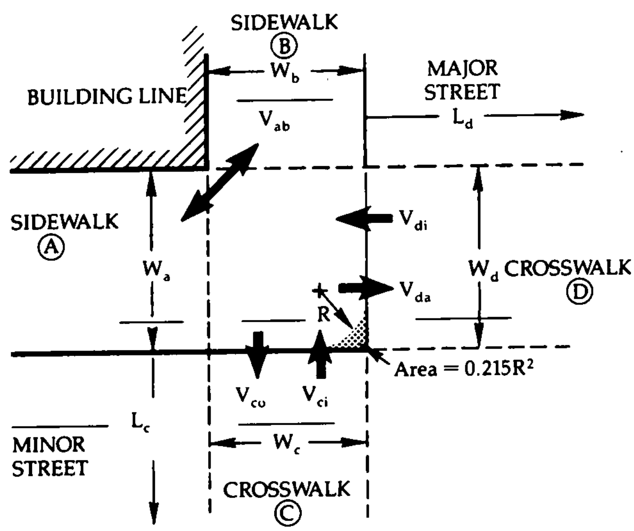
City, State: ____________________

SIGNAL TIMING (sec)

C = ______

G_{mj} = ______ R_{mj} = ______

G_{mi} = ______ R_{mi} = ______



PEDESTRIAN VOLUMES

Flow	Ped/Min	Ped/Cyc
v_{ci}		
v_{co}		
v_{di}		
v_{do}		
$v_{a,b}$		
v_{tot}		

NET CORNER AREA

$A = W_a W_b - 0.215R^2 =$ ______ sq ft

AVAILABLE TIME-SPACE

$TS = A \times C/60 =$ ______ sq ft-min

HOLD AREA WAITING TIMES
(use ped/cycle)

$Q_{tco} = [(v_{co})(R_{mj}/C)(R_{mj}/2)]/60 =$ ______ ped-min

$Q_{tdo} = [(v_{do})(R_{mi}/C)(R_{mi}/2)]/60 =$ ______ ped-min

HOLD AREA TIME-SPACE

$TS_h = 5\,(Q_{tco} + Q_{tdo}) =$ ______ sq ft-min

CIRCULATION TIME-SPACE

$TS_c = TS - TS_h =$ ______ sq ft-min

TOTAL CIRCULATION VOLUME

$v_c = v_{ci} + v_{co} + v_{do} + v_{di} + v_{a,b} =$ ______ ped

TOTAL CIRCULATION TIME

$t_c = v_c \times 4/60 =$ ______ ped-min

PEDESTRIAN SPACE AND LOS

$M = TS_c/t_c =$ ______ sq ft/ped; LOS = ______
(Table 13-3)

BICYCLES

CONTENTS

I. INTRODUCTION

A *bicycle* is defined as a vehicle having two tandem wheels, propelled solely by human power, upon which any person or persons may ride.

Bicycles make up a small percentage of the traffic stream at most locations in North America. Nevertheless, there are many locations where the impact of bicycles on the vehicular traffic stream is noticeable. Many cities have initiated extensive programs to provide facilities for bicycles in the form of designated bicycle lanes on streets and highways and bikeways with physically separated rights-of-way. The use of bicycles as a regular means of personal transportation has increased, particularly in warm climates. The bicycle is a popular mode in and around many university campuses, and is an attractive alternative in congested city areas where vehicular traffic is difficult.

While the state of knowledge concerning specific impacts of bicycles on the capacity and level of service of highway facilities is not advanced, this chapter presents some insights and procedures for approximately analyzing the effects of bicycles in the traffic stream. It also presents approximate information on the capacity of various types of bicycle facilities. Specifically, this chapter addresses the following aspects of bicycle capacity:

1. The impacts of bicycle presence on intersection capacity.
2. The impacts of bicycle presence on roadway segments between intersections.
3. The capacity of designated bicycle facilities.

The sections that follow detail these types of analyses, and illustrate their use with sample calculations.

II. METHODOLOGY AND PROCEDURES FOR APPLICATION

IMPACTS ON INTERSECTION CAPACITY

Bicycles affect the capacity and operating conditions at intersections in two principal ways:

1. Where bicycles share a lane with other vehicles, they utilize a portion of the lane's capacity. This effect is accounted for by assigning an appropriate "passenger-car equivalent" (pce) for each bicycle.
2. Where vehicles execute turning movements through a conflicting bicycle stream, they encounter opposition in addition to that normally presented by opposing vehicle streams and pedestrians. The intersection analysis techniques of Chapters 9 and 10 should be modified to account for this conflict.

Passenger-Car Equivalents for Bicycles

Table 14-1 presents the recommended values of passenger-car equivalents for bicycles. The equivalent varies with lane width and depends on whether the bicycle movement in question is "opposed" or "unopposed."

A bicycle moving straight through an intersection, encountering no significant interference from vehicles or pedestrians, is considered to be unopposed. A left-turning bicycle must cross an opposing vehicular flow on two-way streets, and would be considered to be opposed. Right-turning bicycles may or may not encounter significant pedestrian interference, and could be classified as either opposed or unopposed. Where the conflicting crosswalk flow exceeds 100 peds/hour, it is recommended that right-turning bicycles be considered opposed.

As indicated in Table 14-1, the impact of bicycles sharing vehicular lanes increases as lane width decreases. When lane widths are 14 ft or greater, bicycles tend to use a portion of the lane as a bike lane, and have little impact on vehicular flow. It should also be noted that these factors are conservative, as they assume that most bicyclists move through the intersection on the green signal.

Table 14-1 is used as follows. The number of bicycles (segregated by type of movement) is multiplied by the appropriate passenger-car equivalent values. The result is added to the vehicular volume, yielding a total equivalent vehicular volume which is used in subsequent computations. Consider a signalized intersection with a vehicular volume of 500 vph which shares a 10-ft lane with a bicycle volume of 100 bicycles/hour, one-half of which are opposed.

Then:

$$\begin{aligned}\text{Equivalent volume} &= 500 + 100(0.5)(1.2) \\ &\quad + 100(0.5)(1.0) \\ &= 500 + 60 + 50 = 610 \text{ vph}\end{aligned}$$

where 1.2 and 1.0 are the passenger-car equivalent values for opposed and unopposed bicycle movements selected from Table 14-1. Further computations would proceed using a volume of 610 vph in the procedures of Chapter 9, "Signalized Intersections."

TABLE 14-1. PASSENGER-CAR EQUIVALENT FOR BICYCLES

BICYCLE MOVEMENT	LANE WIDTH (ft) < 11	11–14	> 14
Opposed	1.2	0.5	0.0
Unopposed	1.0	0.2	0.0

Effect of Bicycles on Right-Turning Vehicles

At intersections where a curb bicycle lane is provided, right-turning vehicles encounter not only a conflicting pedestrian flow, but a conflicting bicycle flow as well. Figure 14-1 illustrates these conflicts.

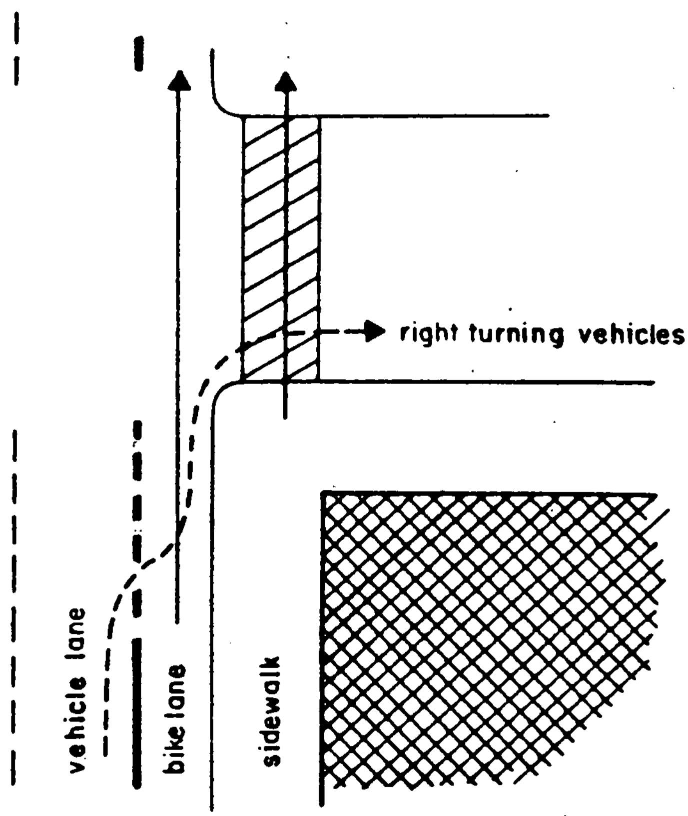


Figure 14-1. Illustration of right-turn conflicts with bicycles and pedestrians.

Where such conflicts exist, right-turning vehicles experience considerably more friction than in situations where no bike lane exists. Table 9-12, of Chapter 9, "Signalized Intersections," gives adjustment factors used in correcting for the impact of pedestrian interference on right-turn saturation flow. Where a bicycle lane exists, it is recommended that this table be entered with total number of pedestrians plus bicycles which interfere with the subject right-turn movement. Thus, if a right-turn movement must cross a pedestrian flow of 100 peds/hour and a bicycle flow of 150 bicycles/hour, Table 9-11 would be entered as if the conflicting pedestrian flow were 100 + 150 = 250 peds/hour.

Where bicycles share a vehicular lane, it is not necessary to include this adjustment because the approach volume is already inflated to account for bicycle presence. Where shared-lane width is 14 ft or greater, however, it was assumed that bicycles separate into the right portion of the lane, using it essentially as a bike lane. In such cases, their impact on right-turning vehicles should be considered as indicated in this section.

Left-Turning Bicycles from Bike Lanes

Bicycles turning left out of a bike lane must mix with other vehicles as they approach the intersection and execute the left-turn maneuver. An appropriate passenger-car equivalent value is selected from Table 14-1 and added to the vehicular volume in the leftmost lane. The passenger-car equivalent value for bicycles is also added to the volume in each lane the bicycles must cross in transferring from the right-hand bike lane to the leftmost traffic lane.

EFFECTS OF BICYCLES ON ROADWAY SEGMENTS BETWEEN INTERSECTIONS

There is little existing data or information on the impacts of bicycles on capacity or operating conditions between intersections. Bicycles are not expected to have any impact on flow where curb-lane widths exceed 14 ft. Where bicycle volumes are less than 50/hour, impacts are also believed to be negligible, except where lanes are narrow ($\leq$ 11 ft).

One study (*1*) has indicated that vehicular intersection approach speeds are reduced by approximately 2.5 mph when bicycles are present in an adjacent bike lane.

BICYCLE FACILITIES

Bicycle facilities separated from vehicular traffic can be provided in two basic forms:

1. *Bike lane*—A portion of a roadway which has been designated by striping, signing, and pavement markings for the preferential or exclusive use of bicyclists.
2. *Bike path*—A bikeway physically separated from motorized vehicular traffic by an open space or barrier, either within the highway right-of-way or within an independent right-of-way.

There is not a great deal of information available concerning the capacity of such facilities. Planning and design criteria for bicycle facilities are available from a number of sources (*2–5*), including the *Transportation and Traffic Engineering Handbook* (*6*). A summary of available data was compiled from Ref. *2*, and is presented in Table 14-2.

Reference *3* cites the capacity of a bicycle facility as 0.22 bicycles per second per foot of bikeway. This is equivalent to 2,376 bicycles/hour for a 3-ft bikeway.

TABLE 14-2. REPORTED ONE-WAY AND TWO-WAY HIGH VOLUMES OF BICYCLE FACILITIES

TYPE OF FACILITY	NO. OF LANES[a]	RANGE OF REPORTED CAPACITIES (BICYCLES/HOUR)
One Way Bike Lane or Path	1	1,700–2,530
Two Way Bike Path	1	850–1,000
	2	500–2,000

[a] Lane widths 3–4 ft/lane.
SOURCE: Adapted from Refs. *2* and *6*

It should be noted that the wide variation of reported high volumes reflects a similarly wide range in environmental conditions, skill and familiarity of cyclists, and specific geometric features of the facilities reported. Bikeway capacity is also rarely observed in practice, as demand levels are generally well below the capacity of the facility. Indeed, the planning and design documents referenced previously all emphasize the need to have bicycle facilities that provide sufficient capacity to allow good-to-excellent operating conditions if they are to be successful in encouraging bike use.

III. SAMPLE CALCULATIONS

CALCULATION 1—PASSENGER-CAR EQUIVALENTS

1. *Description*—An intersection approach with one 12-ft lane has a vehicular demand of 500 vph. It is shared by 50 bicycles/hour, 10 of which turn left and 15 of which turn right across a flow of 110 peds/hour. Convert the approach volume to an equivalent which accounts for the effects of bicycles.

2. *Solution*—Both left-turning and right-turning bicycles are considered to be "opposed." From Table 14-1, the following passenger-car equivalent values are found:

1 Through bicycle = 0.2 pce
1 Left-turning bicycle = 0.5 pce
1 Right-turning bicycle = 0.5 pce

The total equivalent demand volume on the intersection approach may then be expressed as:

$$\text{Equivalent volume} = 500 + 25(0.2) + 10(0.5) + 15(0.5)$$
$$= 500 + 5 + 5 + 7.5$$
$$= 517.5, \text{ SAY } 518 \text{ vph}$$

Note that this is *not* the final conclusion of the analysis of the intersection in question. If the intersection were signalized, the analysis would proceed using the procedures of Chapter 9, but with a demand volume for 518 vph on the subject approach rather than 500 vph, which is the actual vehicular demand volume. If the intersection were unsignalized, the procedures of Chapter 10 would be applied to complete the analysis.

CALCULATION 2—LEFT-TURN IMPACTS ON A MULTILANE APPROACH

1. *Description*—An intersection approach has three traffic lanes and a right-hand curb bicycle lane. The three lanes have the following approach volumes: left lane, 250 vph; center lane, 350 vph; right lane, 220 vph. There are 50 bicycles/hour executing left turns. How should the vehicular volumes be adjusted to reflect the impact of these bicycles. Traffic lanes are 11 ft wide.
2. *Solution*—From Table 14-1, each bicycle has an equivalent of 1.2 (opposed, 11-ft lanes). Thus, the 50 left-turning bicycles/hour are equivalent to $50 \times 1.2 = 60$ vph. These passenger-car equivalents should now be added to the volume in all three approach lanes. Thus, any additional analysis would proceed using the following adjusted approach volumes:

Left lane: 250 + 60 = 310 vph
Center lane: 350 + 60 = 410 vph
Right lane: 220 + 60 = 280 vph

Note that the equivalents are added to each lane that is crossed by bicycles transferring from the bike lane to the leftmost traffic lane.

CALCULATION 3—IMPACTS OF A BIKE LANE ON RIGHT-TURNING VEHICLES

1. *Description*—A single-lane approach at a signalized intersection is adjacent to a curb bike lane carrying 400 bicycles/hour. What right-turn adjustment factor would be selected if right-turning vehicles also interfere with a pedestrian flow of 200 pedestrians/hour? Right turns make up 20 percent of the total volume in the single lane.
2. *Solution*—Right-turn adjustment factors for right turns at signalized intersections are selected from Table 9-11 (Ch. 9). Single-lane approaches are represented by Case 7 in that table. A factor would normally be selected for 200 pedestrians/hour and 20 percent right turns, yielding an adjustment factor of 0.86, which is applied to the saturation flow rate for the approach.

Where a bicycle lane is present, however, the factor is selected as if the pedestrian volume were the total of pedestrians and bicycles. Thus, a factor is selected for 200 + 400 = 600 pedestrians and 20 percent right turns. This factor would be 0.82. Thus, the presence of the bicycle reduces the capacity of the single-lane approach by 0.86 − 0.82 = 0.04, or 4 percent.

IV. REFERENCES

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APPENDIX A

GLOSSARY

Adjustment factor—A multiplicative factor that adjusts a capacity or service flow rate from one representing an ideal or base condition to one representing a prevailing condition.

Alighting time—Time for a passenger to leave a transit vehicle, expressed as the time per passenger or total time for all passengers.

Approach—A set of lanes accommodating all left-turn, through, and right-turn movements arriving at an intersection from a given direction.

Arterial—Signalized streets that serve primarily through-traffic and provide access to abutting properties as a secondary function, having signal spacings of 2 miles or less and turning movements at intersections that usually do not exceed 20 percent of total traffic.

Arterial class—A categorization of arterials involving functional and design categories and free-flow speed.

Arterial section—The aggregation of a sequence of consecutive arterial segments of comparable length and characteristics.

Arterial segment—A one-way length of arterial from one signal to the next, including the downstream signalized intersection, but not the upstream signalized intersection.

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.

Average approach delay—Average stopped-time delay at a signalized intersection plus average time lost due to deceleration to and acceleration from stopping; generally estimated as 1.3 times the average stopped-time delay.

Average stopped-time delay—The total time vehicles are stopped in an intersection approach or lane group during a specified time interval divided by the volume departing from the approach or lane group during the same time period, in seconds per vehicle.

Average running speed—The average speed of a traffic stream computed as the length of a highway segment divided by the average running time of vehicles traversing the segment, in miles per hour.

Average running time—The average time vehicles are in motion while traversing a highway segment of given length; excludes stopped-time delay; in seconds per vehicle or minutes per vehicle.

Average travel speed—The average speed of a traffic stream computed as the length of a highway segment divided by the average travel time of vehicles traversing the segment, in miles per hour.

Average travel time—The average time spent by vehicles traversing a highway segment of given length, including all stopped-time delay, in seconds per vehicle or minutes per vehicle.

Balanced operation—An operating condition in a weaving area in which both weaving and nonweaving vehicles achieve the same level of service.

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 approach to cross through or merge with the major traffic stream, unadjusted for geometric and other site-specific conditions, in seconds.

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

Berth—A position for a bus to pick up and discharge passengers, including curb bus stops and other types of boarding/discharge facilities.

Bicycle—A vehicle having two tandem wheels, propelled solely by human power, upon which any person or persons may ride.

Bike lane—A portion of a roadway which has been designated by striping, signing, and pavement markings for the preferential or exclusive use of bicycles.

Bike path—A bikeway physically separated from motorized traffic by an open space or barrier, either within the highway right-of-way or within an independent right-of-way.

Bikeway—Any road, path, or way, which in some manner is specifically designated as being open to bicycle travel, regardless of whether such facilities are designated for the exclusive use of bicyclists, or are to be shared with other vehicles.

Boarding time—The time for a passenger to board a transit vehicle, expressed as time per passenger or total time for all passengers; a function of fare collection procedures.

Bus—A heavy vehicle involved in the transport of passengers on a for-hire, charter, or franchised transit basis.

Bus lane—A lane restricted to usage by buses by special regulations and markings.

Busway—A right-of-way restricted to usage by buses by physical separation from other traffic lanes.

Capacity—The maximum rate of flow at which persons or vehicles can be reasonably expected to traverse a point or uniform segment of a lane or roadway during a specified time period under prevailing roadway, traffic, and control conditions; usually expressed as vehicles per hour or persons per hour.

Change interval—The "yellow" plus "all red" intervals which occur between phases of a traffic signal to provide for clearance of the intersection before conflicting movements are released.

Circulation area—The portion of a sidewalk street corner used by moving pedestrians passing through the area, in square feet.

Clearance lost time—The portion of the time between signal phases during which an intersection is not used by any traffic movement, in seconds.

Clearance time—The minimum possible time interval between one bus departing a bus berth and another entering it.

Collector street—Surface streets providing land access and traffic circulation service within residential, commercial, and industrial areas.

Composite grade—A series of adjacent grades along a highway having a cumulative effect on operations which is more severe than if each grade were considered separately.

Conflicting traffic volume—The volume of traffic which conflicts with a specific movement at an unsignalized intersection.

Constrained operation—An operating condition in a weaving area where weaving vehicles are unable to occupy as large a portion of available lanes as required to achieve balanced operation because of geometric constraints.

Control conditions—Prevailing conditions concerning traffic controls and regulations in effect for a given segment of street or highway, including the type, phasing, and timing of traffic signals, STOP or YIELD signs, lane use and turn controls, and similar measures.

Crawl speed—The maximum sustained speed that can be maintained by a specified type of vehicles on a constant upgrade of a given percent, in miles per hour.

Critical density—The density at which capacity occurs for a given facility, usually expressed as vehicles per mile per lane.

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 approach to cross through or merge with the major traffic stream under prevailing traffic and roadway conditions, in seconds.

Critical speed—The speed at which capacity occurs for a given facility, usually expressed as miles per hour.

Critical *v/c* ratio—The proportion of available intersection capacity used by vehicles in critical lane groups.

Crosswalk—The marked crossing area for pedestrians crossing the street at an intersection or designated midblock location.

Crown line—A lane marking which directly connects the nose of the entry gore area to the nose of the exit gore area in a weaving section.

Crush capacity—The maximum number of passengers that can physically be accommodated on a transit vehicle.

Cycle—Any complete sequence of signal indications.

Cycle length—The total time for a signal to complete one cycle.

Delay—Additional travel time experienced by a driver, passenger, or pedestrian beyond what would reasonably be desired for a given trip.

Demand volume—The traffic volume expected to desire service past a point or segment of the highway system at some future time, or the traffic currently arriving or desiring service past such a point, usually expressed as vehicles per hour.

Density—The number of vehicles occupying a given length of lane or roadway averaged over time; usually expressed as vehicles per mile or vehicles per mile per lane.

Design analysis—A usage of capacity analysis procedures to determine the size (number of lanes) required on a given segment of a facility in order to provide a specified level of service.

Design category—A type of arterial defined by geometric features and roadside environment.

Design hour factor—Proportion of 24-hour volume occurring during the design hour for a given location or area.

Direction 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, in vehicles per hour.

Direct ramp—A ramp roadway in which vehicles turn only in the direction of their intended directional change, i.e., a ramp providing a left-turn connection that does not require vehicles to turn right or vice-versa.

Diverge—A movement in which a single lane of traffic separates into two separate lanes without the aid of traffic control devices.

Downstream—The direction to which traffic is flowing.

Downtown street—Surface facilities primarily providing access to abutting lands in a downtown area.

Dwell time—The time that a transit vehicle is stopped in a berth for the purposes of boarding or discharging passengers.

Effective green time—The time allocated for a given traffic movement (green plus yellow) at a signalized intersection, less the start-up and clearance lost times for the movement.

Effective red time—The time during which a given traffic movement or set of movements is directed to stop; cycle length minus effective green time.

Effective walkway width—The width of a walkway which is usable by pedestrians; the total walkway width minus the width of unusable "buffer" zones at the curb and building line and other unusable portions due to obstacles and obstructions in the walkway, in feet.

Flow ratio—The ratio of actual flow rate to the saturation flow rate for a given lane group at a signalized intersection.

Free-flow speed—(1) The theoretical speed of traffic when density is zero, i.e., there are no vehicles present; (2) the average speed of vehicles over an arterial segment not close to signalized intersections under conditions of low volume.

Freeway—A multilane divided highway having a minimum of two lanes for exclusive use of traffic in each direction and full control of access and egress.

Freeway surveillance—A system in which freeway operations are monitored and controlled in real time.

Fully actuated control—Signal control of an intersection in which the occurrence and length of every phase is controlled by actuations of vehicle detectors placed on each approach to the intersection.

Functional category—A type of arterial defined by the type of traffic service provided.

Gore area—The area located immediately between the left edge of a ramp pavement and the right edge of the roadway pavement at a merge or diverge area.

Green ratio—The ratio of the effective green time for a given movement at a signalized intersection to the cycle length.

Green time—The actual length of the "green" indication for a given movement at a signalized intersection.

Headway—The time between two successive vehicles in a traffic lane as they pass a point on the roadway, measured from front bumper to front bumper, in seconds.

Heavy vehicle—Any vehicle with more than four wheels touching the pavement during normal operation.

High-occupancy vehicle lane—A lane of a freeway reserved for the use of vehicles with more than a preset number of occupants; such vehicles often include buses, taxis, and carpools.

Ideal conditions—Characteristics for a given type of facility which are assumed to be the best possible from the point of view of capacity; i.e., characteristics which if further improved would not result in increased capacity.

Impedance—The effect of congestion in higher priority movements at a STOP- or YIELD-controlled approach on lower priority movements, which reduces the capacity of lower priority movements.

Interrupted flow—A category of traffic facilities having traffic signals, STOP or YIELD signs, or other fixed causes of periodic delay or interruption to the traffic stream; examples include intersections and arterials.

Interval—A period of time in a signal cycle during which all signal indications remain constant.

Jam density—The density at which congestion becomes so severe that all movement of persons or vehicles stops; usually expressed as vehicles per mile (per lane) or pedestrians per square foot.

Lane 1—The highway lane adjacent to the shoulder.

Lane balance—A condition at a diverge point where the number of lanes leaving the diverge is equal to the number of lanes approaching it plus one.

Lane group—A set of lanes on an intersection approach which has been established for separate capacity and level of service analysis.

Level of service—A qualitative measure describing operational conditions within a traffic stream; generally described in terms of such factors as speed and travel time, freedom to maneuver, traffic interruptions, comfort and convenience, and safety.

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

Load factor—The number of passengers occupying a transit vehicle divided by the number of seats on the vehicle.

Loop ramp—A ramp serving a left-turn movement by requiring vehicles to execute that movement by turning right; typically, a 90-degree left turn is accomplished by making a 270-degree right turn.

Lost time—Time during which the intersection is not effectively used by any movement; clearance lost time plus start-up lost time.

Major weaving section—A weaving area having at least three entry and exit legs with two or more lanes.

Maximum load point—The section of a transit line which has the highest passenger demand during a specified time interval.

Maximum service flow rate—The highest 15-minute rate of flow that can be accommodated on a highway facility under ideal conditions, while maintaining the operating characteristics for a stated level of service, expressed as passenger cars per hour per lane.

Measures of effectiveness—Parameters describing the quality of service provided by a traffic facility to drivers, passengers, or pedestrians; examples include speed, density, delay, and similar measures.

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.

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

Movement capacity—The capacity of a specific movement at a STOP- or YIELD-controlled intersection approach, assuming that the movement has exclusive use of a separate lane, in passenger cars per hour.

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.

Nonweaving flows—Traffic movements in a weaving area not actually engaged in weaving movements.

No passing zone—A segment of a two-lane, two-way highway along which passing is prohibited in one or both directions.

One-sided weaving section—A weaving area in which vehicles entering the highway approach from the same side of the roadway as exiting vehicles depart it.

Operational analysis—A use of capacity analysis to determine the prevailing level of service on an existing or projected facility, with known or projected traffic, roadway, and control conditions.

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

Passenger service time—The time required for a passenger to board or alight from a transit vehicle, in seconds per passenger.

Passing sight distance—The visibility distance required to allow drivers to execute safe passing maneuvers in the opposing traffic lane of a two-lane, two-way highway.

Peak-hour factor—The hourly volume during the maximum volume hour of the day divided by the peak 15-minute rate of flow within the peak hour; a measure of traffic demand fluctuation within the peak hour.

Pedestrian—An individual traveling on foot.

Pedestrian area module—The space provided per pedestrian in a pedestrian facility, expressed as square feet per pedestrian; space.

Pedestrian flow rate—The number of persons passing a point per unit time, usually expressed as pedestrians per 15 minutes, or pedestrians per minute.

Pedestrian speed—The average walking speed of pedestrians, in feet per second.

Permitted turns—Left or right turns at a signalized intersection which are made against an opposing or conflicting vehicular or pedestrian flow.

Person-capacity—The maximum number of persons that can be carried past a given point on a highway or transit right-of-way during a given time period under specified operating conditions without unreasonable delay, hazard, or restriction, in persons per hour.

Person level of service—The quality of service offered the passenger within a transit vehicle, as determined by the available space per passenger.

Phase—The part of the signal cycle allocated to any combination of traffic movements receiving the right-of-way simultaneously during one or more intervals.

Planning analysis—A use of capacity analysis procedures to estimate the number of lanes required by a facility in order to provide for a specified level of service based on approximate and general planning data in the early stages of project development.

Platoon—A group of vehicles or pedestrians traveling together as a group, either voluntarily or involuntarily due to signal control, geometrics, or other factors.

Platoon flow rate—The rate of flow of vehicles or pedestrians within a platoon.

Potential capacity—The capacity of a specific movement at a STOP- or YIELD-controlled intersection approach, assuming that it is unimpeded by other movements and has exclusive use of a separate lane, in passenger cars per hour.

Pretimed control—Traffic signal control in which the cycle length, phase plan, and phase times are preset, and are repeated continuously according to the preset plan.

Productive capacity—A measure of transit efficiency or performance; the product of passenger capacity and speed along a section of a transit line.

Protected turns—Left or right turns at a signalized intersection made with no opposing or conflicting vehicular or pedestrian flow.

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 limited access facility 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 the mainline of a freeway.

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.

Ramp-street junction—The roadway area over which an on- or off-ramp joins with a surface street or arterial.

Ramp-weave section—A weaving area formed by a one-lane on-ramp followed by a one-lane off-ramp where the two are joined by a continuous auxiliary lane.

Rate of flow—The equivalent hourly rate at which vehicles or persons pass a point on a lane, roadway, or other trafficway for a period of time less than one hour; computed as the number of persons or vehicles passing the point divided by the time interval in which they passed (in hours); expressed as vehicles or persons per hour.

Recreational vehicle—A heavy vehicle, generally operated by a private motorist, engaged in the transportation of recreational equipment or facilities; examples include campers, boat trailers, motorcycle trailers, and the like.

Reserve capacity—The capacity of a lane at an unsignalized intersection minus the demand for that lane, where all terms are stated in passenger cars per hour.

Roadway conditions—Geometric characteristics of a street or highway, including the type of facility, number and width of lanes (by direction), shoulder widths and lateral clearances, design speed, and horizontal and vertical alignments.

Rolling terrain—Any combination of horizontal and vertical alignments causing heavy vehicles to reduce their speed substantially below that of passenger cars, but not causing heavy vehicles to operate at crawl speeds for any significant amount of time.

Saturation flow rate—The equivalent hourly rate at which vehicles can traverse an intersection approach under prevailing conditions, assuming that the green signal was available at all times, and no lost times are experienced, in vehicles per hour of green or vehicles per hour of green per lane.

Saturation headway—The average headway between passenger cars in a stable moving queue as they pass through a signalized intersection, in seconds.

Seat capacity—The number of seats on a transit vehicle.

Service flow rate—The maximum hourly rate at which persons or vehicles can be reasonably expected to traverse a point of uniform section of a lane or roadway during a given time period (usually 15 minutes) under prevailing roadway, traffic, and control conditions while maintaining a designated level of service, expressed as vehicles per hour or vehicles per hour per lane.

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.

Space—The average area provided for pedestrians in a moving pedestrian stream or pedestrian queue, in square feet per pedestrian.

Space mean speed—The average speed of the traffic stream computed as the length of the highway segment divided by the average travel time of vehicles to traverse the segment; average travel speed; in miles per hour.

Spacing—The distance between two successive vehicles in a traffic lane, measured from front bumper to front bumper, in feet.

Speed—A rate of motion expressed as distance per unit time.

Standees—The number of passengers standing in a transit vehicle.

Start-up lost time—Additional time consumed by the first few vehicles in a queue at a signalized intersection above and beyond the saturation headway due to the need to react to the initiation of the green phase and to accelerate to ambient speed, in seconds.

Street corner—The area encompassed within the intersection of two sidewalks.

Three-lane highway—A highway having a three-lane cross section; the third lane (center) may be used in a variety of ways including as a passing lane, a two-way left-turn lane, or a climbing lane.

Time mean speed—The arithmetic average of individual vehicle speeds passing a point on a roadway or lane, in miles per hour.

Traffic conditions—The distribution of vehicle types in the traffic stream, directional distribution of traffic, lane use distribution of traffic, and type of driver population on a given facility.

Truck—A heavy vehicle engaged primarily in the transport of goods and materials, or in the delivery of services other than public transportation.

Turnout—A short section of a lane added to a two-lane, two-way highway for the purpose of allowing slow-moving vehicles to leave the main roadway and stop to allow faster vehicles to pass.

Two-lane highway—A roadway having a two-lane cross section with one lane for each direction of flow, on which passing maneuvers must be made in the opposing lane.

Two-sided weaving section—A weaving area in which vehicles entering the highway approach on the right and vehicles departing the highway depart on the left, or vice-versa; weaving vehicles must essentially cross the mainline highway flow.

Two-way left-turn lane—The center lane on a three-lane or multilane highway which is used continuously for vehicles turning left in either direction of flow at midblock locations.

Unconstrained operation—An operating condition in a weaving area where geometric constraints do not limit the ability of weaving vehicles to achieve balanced operation.

Uninterrupted flow—A category of facilities having no fixed causes of delay or interruption external to the traffic stream; examples of such facilities include freeways and unsignalized sections of multilane and two-lane rural highways.

Unit width flow rate—The pedestrian rate of flow expressed as pedestrians per minute per foot of walkway or crosswalk width.

Unsignalized intersection—Any intersection not controlled by traffic signals.

***v/c* ratio**—The ratio of demand flow rate to capacity for a traffic facility.

Volume—The number of persons or vehicles passing a point on a lane, roadway, or other trafficway during some time interval, often taken to be one hour, expressed in vehicles.

Walkway—A facility provided for pedestrian movement, segregated from vehicular traffic by a curb, or provided on a separate right-of-way.

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 lane-changing maneuvers; formed between merge and diverge points, as well as between on-ramps and off-ramps on limit access facilities.

Weaving configuration—The organization and continuity of lanes in a weaving area; determines lane-changing characteristics in the weaving area.

Weaving diagram—A schematic drawing of flows in a weaving area, used as an aid to analysis.

Weaving flows—Traffic movements in a weaving area actually engaged in weaving movements.

Weaving length—The length of a weaving area measured from a point at the entrance gore where the right edge of the shoulder highway lane and the left edge of the ramp are separated by 2 feet to a point at the exit gore where the lane edges are separated by 12 feet, expressed in feet.

Work zone—An area of a highway in which maintenance and construction operations are taking place which impinge on the number of lanes available to moving traffic or affect the operational characteristics of traffic flowing through the area.

SYMBOLS

A............total area of a pedestrian facility, or portion thereof, sq ft; also the average number of alighting passengers per bus during a peak 15-minute period, passengers/bus

a............alighting service time per passenger discharging from a transit vehicle, sec

A_c............circulation area of a pedestrian facility, sq ft

A_h............holding area of a pedestrian facility, sq ft

A_n............net area available on a bus for standees, sq ft

A_w............crosswalk area, sq ft

$AADT$........average annual daily traffic, veh/day

ART SPD....average travel speed on an arterial segment, mph

B............average number of boarding passengers per bus during a peak 15-minute period, passengers/bus

b............boarding service time per passenger entering a transit vehicle, sec

C............cycle length, sec

c............capacity, vph

c'_b............design capacity of a bus stop, buses/hr

c_I............approximate capacity of a multilane intersection approach, vph

c_i............capacity of lane group i at a signalized intersection, vph

c_j............capacity per lane for a freeway or multilane highway under ideal conditions, for design speed j, pcphpl

c_{LT}..........left-turn capacity at a signalized intersection, vph

c_{mi}...........movement capacity for movement i at an unsignalized intersection, pcph

c_{pi}...........potential capacity for movement i at an unsignalized intersection, pcph

c_R............reserve capacity at an unsignalized intersection, pcph

c_{SH}..........shared-lane capacity at an unsignalized intersection, pcph

c_T............capacity of a climbing lane under prevailing conditions, vph

c_v............maximum number of buses per hour per channel or bus berth

c_{vi}............maximum number of buses per hour per channel or bus berth at level of service i

D............density, pc/mi/ln, veh/mi/ln, or pedestrians/sq ft; also the directional distribution factor used in converting AADT to DDHV; also approach delay on an arterial intersection approach, sec; also bus dwell time at a bus stop, sec

d............average stopped-time delay per vehicle, unadjusted for arrival type, sec/veh

d_1............first-term delay, accounting for uniform delay, sec/veh

d_2............second-term delay, accounting for incremental delay over and above uniform delay, sec/veh

d_A............average stopped-time delay for Approach A at a signalized intersection, sec/veh

d_i ....average stopped-time delay for lane group i at a signalized intersection, sec/veh
d_I ....average stopped-time delay for a signalized intersection, sec/veh
D_d............distance to a downstream adjacent ramp, ft
D_u............distance to an upstream adjacent ramp, ft
D_j............jam density, pc/mi/ln or veh/mi/ln
$DDHV$........directional design hour traffic, vph

E............passenger car equivalent for a standard mix of vehicles on a specific grade on a two-lane, two-way rural highway
E_B............passenger car equivalent for buses
E_{HV} ....passenger car equivalent for the prevailing mix of heavy vehicles on a two-lane, two-way rural highway
E_L............through passenger car equivalent for left turns at a signalized intersection
E_o............passenger car equivalent for a standard mix of vehicles on a level section of two-lane, two-way rural highway
E_R ....passenger car equivalent for recreational vehicles
E_T............passenger car equivalent for trucks

f ....bus frequency at the maximum load point, buses/hr
f' ....maximum bus frequency at a berth, buses/hr
f_a............area type adjustment factor
f_{bb}............bus blockage adjustment factor
f'_c............bus frequency at a berth, buses/cycle
f'_d............design bus frequency, buses/berth/hr
f_d ....directional distribution factor
f_E ....multilane highway type and environment adjustment factor
f_{HV} ....heavy vehicle adjustment factor
f_g............grade adjustment factor for passenger cars on specific grades on a two-lane, two-way rural highway, and for all vehicles at a signalized intersection
f_{LT} ....left-turn adjustment factor
f_m............adjustment factor for permitted left turns in a shared or exclusive lane at a signalized intersection—applied only to flow in the lane from which left turns are made
f_p............driver population adjustment factor; parking condition adjustment factor for signalized intersections
f_{RT} ....right-turn adjustment factor
f_s............opposing flow saturation factor, used in estimating the left-turn adjustment factor for permitted left turns at a signalized intersection
f_w ....lane width and lateral clearance adjustment factor

g_f ....initial portion of a green phase during which through vehicles may move in a shared left-turn, through-lane until the arrival of the first left-turning vehicle, sec
G_i............green time for phase i at a signalized intersection, sec
g_i............effective green time for phase i at a signalized intersection, sec
G_p............minimum pedestrian green phase at a signalized intersection, sec
g_q ....portion of a green phase during which left turns are blocked by the clearance of an opposing queue of vehicles, sec
g_u ....portion of a green phase during which left turns are not blocked by the clearance of an opposing queue of vehicles, sec
G/C ....ratio of green time to cycle length
g/C..........ratio of effective green time to cycle length

h ....saturation headway, sec/veh; also average headway of a transit facility at its maximum load point, sec
h' ....minimum bus headway at a bus stop or berth, sec
H ....alighting passenger capacity per berth per hour
H_v............hourly transit volume, passengers/hr
HV...........heavy vehicle

I_p ....impedance factor for the effect of specific grades on the operation of passenger cars on a two-lane, two-way highway grade

J ....number of passengers boarding a bus line at the heaviest stop, passengers/hr

K............ratio of design hour traffic to AADT; also number of passengers alighting a bus line at the heaviest stop, passengers/hr

L............length of a weaving area, ft; also lost time per cycle at a signalized intersection, sec; also additional lost time due to buses stopping, starting, and queuing near bus stops, sec
L_h............length of a weaving area, in hundreds of ft
L_i............net square ft per standee for level of service i
L_t............length of a queue upstream of a work zone, ft
l_1.............start-up lost time, sec
l_2.............clearance lost time, sec
LT ....left turn

M ....pedestrian space, sq ft/ped
MSF.........maximum per lane service flow rate for a given level of service, pcphpl

N............number of lanes on a facility or in a lane group at an intersection, generally in one direction; also number of the vehicle in a signalized intersection queue at which start-up lost times no longer exist; also number of buses per hour stopping at a given bus stop, veh
N_B ....number of local buses stopping at an intersection to pick up or discharge passengers, buses/hr
N_b...........number of effective berths at a bus stop
N'_b...........number of berths provided at a multiberth bus station

N_m ....number of parking maneuvers per hour within 250 ft of an intersection

O_1 ....bus occupancy (during peak 15 minutes) along a freeway, passengers/hr
O_2 ....car occupancy (during peak 15 minutes) along a freeway, passengers/hr

P ....line-haul capacity of a bus facility past the maximum load point, persons/hr
P_b ....unit line-haul capacity of a bus facility, assuming a single berth at the heaviest stop, persons/hr
P_B ....proportion of buses in the traffic stream
P_{HV} ....proportion of heavy vehicles on a two-lane, two-way highway grade
P_L ....proportion of left turns in a lane from which left turns are made at a signalized intersection
P_{LT} ....proportion of left turns at a signalized intersection
P_{LTo} ....proportion of left turns in the opposing flow at a signalized intersection
P_P ....proportion of passenger cars in the traffic stream
P_R ....proportion of recreational vehicles in the traffic stream
P_{RT} ....proportion of right turns at a signalized intersection
P_T ....proportion of trucks in the traffic stream
$P_{T/HV}$ ....proportion of trucks among heavy vehicles on a two-lane, two-way rural highway grade
PEDS ....number of pedestrians per hour conflicting with a given right-turn movement at a signalized intersection
PF ....progression factor
PHF............peak-hour factor
P_i ....impedance factor for movement *i* at an unsignalized intersection
PVG............proportion of vehicles arriving during the green phase at a signalized intersection
PTG............*G/C* ratio

Q ....maximum number of passengers per berth per hour
Q_i ....number of pedestrians in a holding area, for flow *i* during one signal cycle, peds
Q_t ....number of vehicles in a queue upstream of a work zone, veh
Q_u ....total time spent in a holding area by pedestrians in flow *i* during one signal cycle, pedestrian-minutes

R ....weaving ratio; also reductive factor used to compensate for variations in bus dwell times in transit analysis
r ....corner radius, ft
r_i ....length of red for phase *i*, sec
R_p ....platoon ratio
RT ....right turn

s ....saturation flow rate under prevailing conditions, vphg or vphgpl
s_n ....seats per transit vehicle
s_o ....saturation flow rate under ideal conditions, pcphgpl
s_{op} ....saturation flow rate in an opposing lane group at a signalized intersection
S ....average travel speed, mph; also average pedestrian speed, fps
S_c ....critical speed, the speed at which capacity occurs, mph
S_w ....average travel speed of weaving vehicles in a weaving area, mph
S_{nw} ....average travel speed of nonweaving vehicles in a weaving area, mph
SF............service flow rate for a given level of service, vph
SFL ....service flow rate per lane, vphpl

T ....total time spent by a bus in a berth or stop, the sum of dwell time plus clearance time, sec
t_c ....clearance time between successive buses at a bus berth, sec
T_{ci} ....critical gap for movement *i* at an unsignalized intersection, sec
TH............through vehicle
t_i ....start-up lost time for the *i*th vehicle in a queue at a signalized intersection; also travel time for the *i*th vehicle traversing a highway section, sec
T_L ....time loss resulting from a bus blocking other vehicles at an intersection, sec
TS ....time-space in a pedestrian area, ped-min
TS_c ....circulation time-space in a pedestrian area, ped-min
TS_h ....holding time-space in a pedestrian area, ped-min
t_w ....average time a pedestrian spends in a crosswalk, sec/ped

U ....lane utilization factor; also bus berth utilization factor

V ....hourly volume, vph or vphpl
v ....rate of flow, vph, vphpl, or pedestrians/min/ft of walkway
v_a ....total flow rate on a signalized intersection approach, vph
v_g ....lane group flow rate at a signalized intersection, unadjusted for lane utilization, vph
V_1 ....volume in lane 1 (shoulder lane) of a freeway immediately upstream of a ramp junction
V_c ....volume in a diverging lane immediately upstream of a major diverge point, vph
v_c ....flow rate at which capacity occurs on a two-lane, two-way rural highway grade
V_{ci} ....conflicting volume for movement *i* at an unsignalized intersection, vph
V_d ....total diverge volume, vph
v_d ....total diverge rate of flow, vph
V_f ....total freeway volume at a ramp junction, vph
v_{LE} ....equivalent left-turn flow rate at a signalized intersection, in through passenger cars/hr
v_{LT} ....left-turn flow rate, vph

V_m .... total merge volume, vph; also maximum number of pedestrians occupying a crosswalk during one signal cycle, peds
v_m .... total merge rate of flow, vph
v_M .... mainline flow in a signalized intersection approach, vph
v_o .... opposing flow rate at a signalized intersection, vph
v_p .... movement flow rate during peak 15-min. at a signalized intersection, vph; also pedestrian flow rate in platoons, peds/min/ft
V_r .... total ramp volume, vph
v_r .... total ramp rate of flow, vph
v_{RT} .... right-turn flow rate, vph
V_u .... total volume on an adjacent upstream ramp, vph
v_{w1} .... weaving flow rate with the larger numeric value among the two weaving flows, vph
v_{w2} .... weaving flow rate with the smaller numeric value among the two weaving flows, vph
v_w .... total weaving flow rate in a weaving area, vph
V_{15} .... volume per 15-minute interval
$V_{1\text{-}12}$ .... volume for various movements at an unsignalized intersection, vph
$v_{1\text{-}12}$ .... volume or rate of flow for various movements at an unsignalized intersection, pcph
VR .... volume ratio

W_B .... buffer width on a walkway, not usable by moving pedestrians, ft
W_E .... effective walkway width, ft
W_T .... total walkway width, ft

X .... proportion of maximum load point passengers that board at the heaviest stop in a transit line
X_c .... critical flow rate-to-capacity ratio for a signalized intersection
X_i .... flow rate-to-capacity ratio for lane group i at a signalized intersection

Y .... proportion of maximum load point passengers that alight at the heaviest stop in a transit line; also flow ratio

INDEX

D

E

F

N

O

P

Q

R

S