

3-55(3)

Copy No. _____

CAPACITY AND QUALITY OF SERVICE OF TWO-LANE HIGHWAYS

Final Report

by

**Douglas W. Harwood, Adolf D. May, Ingrid B. Anderson,
Lannon Leiman, and A. Ricardo Archilla**

Prepared for

**National Cooperative Highway Research Program
Transportation Research Board
National Research Council**

**Midwest Research Institute
University of California-Berkeley
MRI Project No. 104215
November 1999**

Acknowledgment of Sponsorship and Disclaimer

Acknowledgment

This work was sponsored by the American Association of State Highway and Transportation Officials, in cooperation with the Federal Highway Administration, and was conducted in the National Cooperative Highway Research program which is administered by the Transportation Research Board of the National Research Council.

Disclaimer

This copy is an uncorrected draft as submitted by the research agency. A decision concerning acceptance by the Transportation Research Board and publication in the regular NCHRP series will not be made until a complete technical review has been made and discussed with the researchers. The opinions and conclusions expressed and implied in the report are those of the research agency. They are not necessarily those of the Transportation Research Board, the National Research Council, the Federal Highway Administration, the American Association of State Highway and Transportation Officials, or of the individual states participating in the National Cooperative Highway Research Program.

CONTENTS

FIGURES	v
TABLES	ix
ACKNOWLEDGMENTS	xi
SUMMARY	xv
 CHAPTER 1. INTRODUCTION	 1
RESEARCH PROBLEM STATEMENT	1
RESEARCH OBJECTIVES AND SCOPE	1
RESEARCH APPROACH	2
ORGANIZATION OF THIS REPORT	3
 CHAPTER 2. CAPABILITIES AND DEFICIENCIES OF EXISTING METHODS ...	 5
HCM METHODOLOGIES FOR TWO-LANE HIGHWAYS	5
ASSESSMENTS OF HCM CHAPTER 8	35
HCM USER SURVEY	40
 CHAPTER 3. TRAFFIC OPERATIONAL FIELD DATA	 61
TYPES OF FIELD DATA COLLECTED	61
FIELD DATA COLLECTION METHODS	62
DATA COLLECTION LOCATIONS AND AMOUNT OF FIELD DATA COLLECTED	65
 CHAPTER 4. IMPROVEMENT OF THE TWOPAS SIMULATION MODEL	 69
SELECTION OF THE TWOPAS MODEL	69
HISTORY AND BACKGROUND OF THE TWOPAS MODEL AND THE UCBRURAL INTERFACE	69
IMPROVEMENT OF THE TWOPAS MODEL AND THE UCBRURAL INTERFACE	71
CALIBRATION AND VALIDATION OF THE TWOPAS MODEL	88
 CHAPTER 5. OPERATIONAL ANALYSIS PROCEDURES FOR TWO-LANE HIGHWAYS	 93
CONCEPTUAL FRAMEWORK	93
FUNDAMENTAL RELATIONSHIPS	106
CAPACITY	115
FREE-FLOW SPEED	119
DEMAND FLOW RATES	124
OPERATIONAL ANALYSIS PROCEDURE FOR TWO-WAY SEGMENTS	137
OPERATIONAL ANALYSIS PROCEDURE FOR DIRECTIONAL SEGMENTS	139

OPERATIONAL ANALYSIS PROCEDURES FOR DIRECTIONAL SEGMENTS CONTAINING PASSING LANES IN LEVEL AND ROLLING TERRAIN	143
OPERATIONAL ANALYSIS PROCEDURE FOR DIRECTIONAL SEGMENTS CONTAINING CLIMBING LANES ON UPGRADES	152
OTHER TRAFFIC PERFORMANCE MEASURES	153
OTHER ELEMENTS OF THE REVISED HCM CHAPTERS	154
CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS	159
REFERENCES	163

Appendices

Appendix A—Published and Unpublished Assessments of HCM Chapter 8
Appendix B—HCM User Survey
Appendix C—HCM User Survey Questionnaire
Appendix D—Excerpt from Draft of HCM2000 Chapter 12 (Highway Concepts)
Appendix E—Draft of HCM 2000 Chapter 20 (Two-Lane Highways)

FIGURES

Figure 1. Speed-Flow Relationships for Two-Lane Highways Used in the 1965 HCM	8
Figure 2. Relationships of Percent Time Delay and Average Travel Speed to Flow Rate Used in the 1985 HCM	9
Figure 3. Example of the Effect of a Passing Lane on Two-Lane Highway Operations	15
Figure 4. Sensitivity of Service Volume to Percent No-Passing Zones in the General Terrain Segment Procedure	18
Figure 5. Sensitivity of Service Volume to Directional Distribution of Traffic in the General Terrain Segment Procedure	18
Figure 6. Sensitivity of Service Volume to Lane Width in the General Terrain Segment Procedure	19
Figure 7. Sensitivity of Service Volume to Usable Shoulder Width in the General Terrain Segment Procedure	19
Figure 8. Sensitivity of Service Volume to Truck Percentage for Level Terrain in the General Terrain Segment Procedure	21
Figure 9. Sensitivity of Service Volume to Truck Percentage for Rolling Terrain in the General Terrain Segment Procedure	21
Figure 10. Sensitivity of Service Volume to Truck Percentage for Mountainous Terrain in the General Terrain Segment Procedure	22
Figure 11. Sensitivity of Service Volume to RV Percentage for Level Terrain in the General Terrain Segment Procedure	22
Figure 12. Sensitivity of Service Volume to RV Percentage for Rolling Terrain in the General Terrain Segment Procedure	23
Figure 13. Sensitivity of Service Volume to RV Percentage for Mountainous Terrain in the General Terrain Segment Procedure	23
Figure 14. Relationship Between Percent Time Delay and Flow Rate for “Worst Cases” in the General Terrain Segment Procedure	26
Figure 15. Combinations of Truck Percentage, Percent Grade, and Length of Grade to Which the Specific Grade Procedure Applies	28
Figure 16. Combinations of RV Percentage, Percent Grade, and Length of Grade to Which the Specific Grade Procedure Applies	29
Figure 17. Speed-Flow Relationships Provided by the Specific Grade Procedure for Trucks on a 1.6-km (1-mi), 4 Percent Upgrade	30
Figure 18. Speed-Flow Relationships Provided by the Specific Grade Procedure for Trucks on a 2.4-km (1.5-mi), 4 Percent Upgrade	30
Figure 19. Speed-Flow Relationships Provided by the Specific Grade Procedure for Trucks on a 3.2-km (2-mi), 4 Percent Upgrade	31
Figure 20. Speed-Flow Relationships Provided by the Specific Grade Procedure for Trucks on a 4.8-km (3-mi), 4 Percent Upgrade	31
Figure 21. Speed-Flow Relationships Provided by the Specific Grade Procedure for Trucks on a 6.4-km (4-mi), 4 Percent Upgrade	32

Figure 22. Speed-Flow Relationships Provided by the Specific Grade Procedure for RVs on a 4.8-km (3-mi), 4 Percent Upgrade	32
Figure 23. Speed-Flow Relationships Provided by the Specific Grade Procedure for RVs on a 6.4-km (4-mi), 4 Percent Upgrade	33
Figure 24. Speed-Flow Relationships Provided by the Specific Grade Procedure for Trucks on a 4.8-km (3-mi), 6 Percent Upgrade	33
Figure 25. Speed-Flow Relationships Provided by the Specific Grade Procedure for RVs on a 4.8-km (3-mi), 6 Percent Upgrade	34
Figure 26. Typical Equipment Setup for Two-Lane Highway Data Collections	64
Figure 27. Flow Diagram of Interactions Between TWOPAS, TWOSUM, and UCBRURAL	72
Figure 28. Comparison of Old and New Vehicle Populations for 100% Passenger Car Flows as a Function of Percent Grade	82
Figure 29. Graphical Illustration of LOS Thresholds for Class I Highways	102
Figure 30. Flow Diagram of Two-Lane Highway Operational Analysis Methodology	104
Figure 31. Scatter Plot of Speed-Flow Data for Field Sites Evaluated During the Research	108
Figure 32. Speed-Flow Relationships Used in the Two-Way and Directional Segment Procedures	110
Figure 33. Scatter Plots of PTSF-Flow Data For Site CA02	113
Figure 34. PTSF-Flow Relationship Used in the Two-Way Segment Procedure	114
Figure 35. PTSF-Flow Relationships Used in the Directional Segment Procedure	114
Figure 37. Example of the Operational Effect of a Passing Lane on Percent Time Spent Following	145
Figure 38. Effect of a Passing Lane on Percent time Spent Following as Represented in the Operational Analysis Methodology	148
Figure 39. Effect of Passing Lane on Average Travel Speed as Represented in the Operational Analysis Methodology	150

TABLES

Table 1. Level of Service Criteria for Two-Lane Highways Used in the 1965 HCM	7
Table 2. Level of Service Criteria for Operational Analysis of General Terrain Segments on Two-Lane Highways in the 1985 HCM	9
Table 3. Adjustment Factors for Directional Distribution of Traffic, Lane Width, and Shoulder Width in General Terrain Segment Procedure	11
Table 4. Passenger Car Equivalents of Heavy Vehicle Used in the General Terrain Segment Procedure	14
Table 5. Comparison of Level of Service Criteria for General Terrain Segments and Specific Grades on Two-Lane Highways in the 1985 HCM	15
Table 6. Variation of Service Volumes as a Function of Terrain and Percent Trucks	24
Table 7. Variation of Service Volumes as a Function of Terrain and Percent Recreational Vehicles	25
Table 8. Priorities for Future Two-Lane Highway Research Recommended by the HCQS Two-Lane Roads Subcommittee	36
Table 9. Response Rate for HCM User Survey	41
Table 10. Assessment by Respondents of the Adequacy of Specific Features of the HCM Chapter 8 Analysis Procedures	45
Table 11. Assessment by Respondents of Features that Should be Added to the HCM for Analysis of Two-Lane Highways	52
Table 12. Potential HCM Enhancements Ranked by Percentage of Respondents Who Favor Them	53
Table 13. Potential HCM Enhancement Ranked by Priority Assigned by Respondents Who Favor Them	53
Table 14. Assessment by Respondents of Alternative Level of Service Measures for Two-lane Highways (Based on First Choice Selections)	54
Table 15. Respondent Assessment of Alternative Level of Service Measures for Two-lane Highways (Based on Weighing of First, Second, and Third Choices)	55
Table 16. Assessment by Respondents of the Need for Design Speed as an Explicit Factor in Determine Level of Service of Two-Lane Highways	55
Table 17. Assessment by Respondents of the Appropriate Level of Service for Two-lane Highways With Lower Design Speeds But Relatively Low Traffic Volumes	56
Table 18. Priorities Recommended for Specific Issues in the Canadian Survey	58
Table 19. Summary of Field Data Collection Activities	66
Table 20. List of Candidate TWOPAS Model Improvements Made as Part of the Research	73
Table 21. Recommended Changes in TWOPAS Input for Vehicle Performance Characteristics	81

Table 22. Comparison of TWOPAS and Field Values for Mean Speed and Percent Following for Site CA02	90
Table 23. Comparison of TWOPAS and Field Values for Mean Spot Speed for the Trans Canada Highway Site	91
Table 24. Comparison of TWOPAS and Field Values for Spot Percent Following for the Trans Canada Highway Site	92
Table 25. Comparison of Percent Time Spent Following as Determined with the TWOPAS Model to Various Headway Criteria	98
Table 26. Level-of-Service Criteria for Two-lane Highways in Class I	101
Table 27. Level-of-Service Criteria for Two-lane Highways in Class II	101
Table 28. Values of Coefficients used in Estimating Percent Time Spent Following for Directional Segments	115
Table 29. Traffic Data for Highest Flow Periods at Site CA02	116
Table 30. Reported Maximum Volumes on Selected Two-Lane Rural Highways	118
Table 31. Reduction in Free-Flow or Desired Speed Due to Lane Width and Shoulder Width (f_{LS})	121
Table 32. Observed Speed Differences at Lane and Shoulder Width Transitions	122
Table 33. Reduction in Mean Speed Due to Narrow Shoulders	123
Table 34. Reduction in Mean Speed Due to Narrow Lanes	123
Table 35. Adjustments for Access Point Density	123
Table 36. Computation of Grade Adjustment Factor (f_G) for Level and Rolling Terrain	127
Table 37. Computation of Grade Adjustment Factor (f_G) for Average Travel Speed on Specific Grades	128
Table 38. Computation of Grade Adjustment Factor (f_G) for Percent Time Spent Following on Specific Grades	129
Table 39. Passenger-Car Equivalents for Trucks (E_T) in Level and Rolling Terrain ...	132
Table 40. Passenger-Car Equivalents for RVs (E_R) in Level and Rolling Terrain	132
Table 41. Computation of Passenger Car Equivalents for Trucks (E_T) for Average Travel Speed on Specific Grades	133
Table 42. Computation of Passenger-Car Equivalents for Trucks (E_T) for Percent Time Spent Following on Specific Grades	134
Table 43. Computation of Passenger-Car Equivalents for RVs (E_R) for Average Travel Speed on Specific Grades	135
Table 44. Computation of Passenger-Car Equivalents for RVs (E_R) for Percent Time Spent Following on Specific Grades	136
Table 46. Optimal Lengths of Passing Lanes	147
Table 47. Factors (f_{pl}) for Estimation of Average Travel Speed and Percent Time Spent Following Within a Passing Lane	147
Table 48. Factors (f_{pl}) for Estimation of Average Travel Speed and Percent Time Spent Following within a Climbing Lane	152
Table 49. Maximum Two-Way Service Volume (pc/h) Versus Level of Service for a Class I Two-Lane Rural Highway	156
Table 50. Maximum AADT (veh/day) Versus Level of Service for a Class I Two-Lane Rural Highway	157

ACKNOWLEDGMENTS

The work reported herein was performed under NCHRP Project 3-55(3) by Midwest Research Institute (MRI) and the University of California-Berkeley (UCB). The work was performed in MRI's Applied Engineering Division, directed by Dr. Thomas J. Grant.

Mr. Douglas W. Harwood, Principal Traffic Engineer at MRI, was the principal investigator of the research. Other MRI staff members who contributed to the research include Ms. Ingrid B. Anderson, Ms. Karin M. Bauer, and Mr. Brian K. Rosson. The subcontract work at UCB was directed by Prof. Adolf D. May. Key UCB contributors to the research were Ms. Lannon Leiman and Mr. A. Ricardo Archilla. Mr. Andrew D. St. John and Dr. John M. Morrall served as consultants to the project team.

The research team received valuable assistance from the staffs of the California Department of Transportation, Florida Department of Transportation, Missouri Department of Transportation, and Oregon Department of Transportation in arranging field studies and providing existing field data. We are also grateful to 102 HCM users who made the effort to respond to a questionnaire survey concerning the strengths and weaknesses of the existing HCM Chapter 8. We hope they will find that their ideas and assessments have been considered in the revised operational analysis procedure presented in this report. We appreciate the efforts of Catalina Engineering, the contractor for NCHRP Project 3-55(6), for converting the draft HCM chapters to the HCM2000 format and for organizing the material for consistency with other chapters. Finally, we appreciate the help, guidance, support, and constructive criticism provided by the TRB Committee on Highway Capacity and Quality of Service. Their continuing involvement has had a key impact on the final product of this research.

ABSTRACT

Two-lane highways constitute the vast majority of the road system of the United States and of most other countries. Operational analysis procedures that can be used to assess the capacity and level of service of two-lane highways are presented in the *Highway Capacity Manual* (HCM). This research identified the strengths and weaknesses of the existing operational analysis procedures in of the 1985 edition of HCM Chapter 8 and developed improved operational analysis procedures for incorporation in the new HCM edition to be published in the Year 2000 (HCM2000). The research included a thorough assessment of the current HCM chapter, collection of new traffic operational field data on two-lane highways, improvement of an existing computer simulation model of two-lane highway traffic operations known as TWOPAS, and the development of updated operational analysis procedures for the HCM2000. Key features of the improved operational analysis procedures are revised level-of-service measures and definitions, revised factors for the effects of grades and heavy vehicles, separate computational procedures for two-way and directional segments, provision of operational analysis procedures for passing lanes in level and rolling terrain, climbing lanes on steep upgrades, and steep downgrades on which some trucks must use crawl speeds.

SUMMARY

The objective of this research was to develop improved operational analysis procedures for two-lane highways. Two-lane highways constitute the vast majority of the highway system of the United States and most other countries. Operational analysis procedures that can be used to assess the capacity and level of service of two-lane highways are presented in the *Highway Capacity Manual* (HCM). Existing two-lane highway operational analysis procedures are presented in the 1985 edition of HCM Chapter 8. These procedures have remain unchanged in the 1994 and 1997 updates of the HCM.

A thorough literature review and a survey of users of HCM Chapter 8 was conducted to identify the strengths and weaknesses of the existing HCM Chapter 8. These included: confusion among HCM users about the meaning of the term percent time delay, used as the service measure in HCM chapter 8; inconsistency between the level-of-service assessments made with the general terrain segment and specific grade procedures; lack of an effect of roadway design speed on level of service; and lack of a procedure to evaluate the operational effects of passing and climbing lanes.

Traffic operational field data were collected at a total of 20 sites in four states and one Canadian province. These data included traffic volume, speed, and platooning data on high-volume two-lane highways, comparison of speeds upstream and downstream of shoulder width transitions on two-lane highways, collection of truck crawl speeds on a steep upgrade, and evaluation of traffic operations on a steep downgrade.

The field data were used in improving an existing computer simulation model of two-lane highway traffic operations known as TWOPAS. Improvements were made both to the TWOPAS model itself and to a companion user interface known as UCBRURAL. The UCBRURAL interface provides a user-friendly environment to specific input data for the TWOPAS model and to display and analyze output. A total of 25 distinct improvements were made to the TWOPAS model and the UCBRURAL interface. Key improvements to TWOPAS capabilities included the capability for the user to specify roadway lane and shoulder widths in input data, capability to specific reduced speed zones, capability to vary the standard deviation of driver desired speeds by vehicle type, updating of vehicle performance capabilities of both passenger cars and heavy vehicles for the current vehicle fleet, improvement of logic for simulating vehicle speed reductions on horizontal curves, and development of logic to automatically compute sight distance and locations of passing/no-passing zones where these are not specified by the user. A key improvement to the UCBRURAL interface is the ability to make multiple runs in batch mode and to transfer the results of those runs to a spreadsheet for comparison and analysis.

The TWOPAS model was validated by comparison to field data from two sites, one in California and one in western Canada. The validation sites included a relatively short site with high traffic volumes and several reduced-speed horizontal curves and an extended site with a variety of terrain and several passing and climbing lanes. It was found that the

TWOPAS model generally provides speed estimates within 5 to 8 percent of the field-measured values and spot platooning estimates within 4 to 8 percent of field-measured values.

Improved operational analysis procedures for two-lane highways were developed with use of both the TWOPAS model and the field data collected in the study. The improved procedures use a combined service measure for two-lane highways in which each level of service is defined by an upper limit of percent time spent following and a lower limit of average travel speed. Percent time spent following is a revised name for the measure known in the previous procedure as percent time delay. The new term, percent time spent following, is intended to reduce user confusion about the meaning of this traffic performance measure.

Key features of the revised operational analysis procedures are: (1) separate classes of two-lane roads, with differing level-of-service criteria, based on the HCM user's assessment of whether motorists expect to travel at relatively high speeds on a given facility; (2) incorporation of free-flow speed to represent differences in the quality of alignment and cross section for two-lane highways; (3) provision of separate, but comparable, procedures for analysis of two-way and directional two-lane highway segments; (4) revised speed-flow and percent time spent following-flow relationships; (5) revised factors for the effects of grades and heavy vehicles; (6) revised adjustments for the effects of the percentage of no-passing zones and the directional distribution of traffic; (7) provision of procedures for operational analysis of passing lanes in level and rolling terrain; (8) provision of procedures for operational analysis of climbing lanes on upgrades; and (9) provision of procedures for operational analysis of steep downgrades on which some trucks must use crawl speeds.

The revised operational analysis procedures are intended for incorporation in a new HCM edition to be published in the Year 2000, known as the HCM2000.

CHAPTER 1.

INTRODUCTION

This report documents the development of a revised chapter on two-lane highways for the new edition of the *Highway Capacity Manual* (HCM) to be published in the year 2000. The chapter features a revised operational analysis procedure for two-lane highways with added capabilities to address two-lane highways with lower design speeds, passing lanes, climbing lanes, and downgrade crawl regions. This first chapter presents the research problem statement, the research objectives and scope, the research approach, and the organization of this report.

RESEARCH PROBLEM STATEMENT

The Transportation Research Board (TRB) Committee on Highway Capacity and Quality of Service (HCQS) intends to publish a new version of the *Highway Capacity Manual* by the year 2000 (HCM2000). A program of research needed for the HCM2000 was developed and documented in TRB *Transportation Research Circular 371*. One of the last remaining research needs is to improve the procedures for analyzing the capacity and quality of service of two-lane highways.

The procedures in Chapter 8 of TRB Special Report 209, *Highway Capacity Manual* (HCM), for analyzing two-lane highways have several limitations. Low-cost improvements such as passing lanes and two-way left-turn lanes can be very effective in improving the operation of two-lane highways and can reduce the need to widen highways to four lanes. Chapter 8 of the HCM, however, lacks a procedure to estimate the effectiveness of these improvements. It also lacks methods for evaluating the operational effects of roadside development, wide roadway cross sections, and lower design standards. Finally, different measures of effectiveness (MOEs) are used in different procedures within Chapter 8 making comparisons of analytical results difficult.

In addition to the HCM, simulation models are used to analyze the capacity and quality of service of two-lane highways. The simulation models also have limitations, both in analytical capabilities and in the user interface. One particular limitation is the inability to import data from computer-aided design files.

RESEARCH OBJECTIVES AND SCOPE

The objective of the research is to develop improved methods and procedures for capacity and quality of service analysis of two-lane highways, including a revised chapter to the HCM2000. The revised chapter will be applicable to a wider range of user

applications will be more compatible with other HCM procedures than the existing chapter.

The scope of the project was based on an assessment of the strengths and weaknesses of the existing chapter and user needs for improved capacity and level of service procedures. Key decisions made in the research include the choice of an appropriate service measure or measures for two-lane highways; the determination of which additional geometric features (e.g., passing lanes, turnouts, shoulder use sections, lower design speed curves, intersections, driveways, etc.) will be evaluated with the revised procedures; the determination of the extent to which urban and suburban conditions will be addressed by the revised procedures; the determination of how the revised procedures can be used to make comparisons in quality of service between a two-lane highway and a multilane highway, and between a two-lane highway and an urban or suburban arterial; the determination of the need for planning, design, and operational analysis procedures; and the role of manual work sheets versus automated analytical procedures versus simulation models in the HCM2000.

RESEARCH APPROACH

The research approach to address the need for a revised HCM Chapter on two-lane highways included the following elements:

- Identify the capabilities and deficiencies of the existing HCM Chapter 8 analysis procedures for two-lane highways and the capabilities and deficiencies of other existing or proposed methods for traffic operational analysis of two-lane highways.
- Establish a recommended conceptual framework for improved capacity and level of service procedures.
- Collect field data on the traffic operational performance of two-lane highways representing a range of conditions.
- Identify a suitable computer simulation model of two-lane highway operations and improve its capabilities to serve as a tool for making improvements to the existing HCM Chapter.
- Using the field data and the improved simulation model, develop a revised operational analysis procedure for two-lane highways.
- Incorporate the revised operational analysis procedure in a draft chapter for publication as part of the HCM2000.

ORGANIZATION OF THIS REPORT

The remainder of this report is organized as described below. Chapter 2 presents the assessment of existing methods for operational analysis of two-lane highways. Chapter 3 documents the traffic operational field data that were collected for use in the research. Chapter 4 describes the selection of the TWOPAS simulation model and its improvement in the research. Chapter 5 presents the operational analysis procedures for two-lane highways that were developed using the field data and the TWOPAS model. Chapter 6 presents the conclusions and recommendations of the study.

Appendix A presents published and unpublished assessments of HCM Chapter 8. Appendix B presents the results of the HCM user survey. The survey questionnaire is presented in Appendix C. Appendix D presents an excerpt from the draft of Chapter 12 of the HCM 2000 which presents background and defines concepts used in the two-lane highway operational analysis procedure. Appendix E presents the draft of Chapter 20 of the HCM 2000 which presents the operational analysis procedure developed in this research.

CHAPTER 2.

CAPABILITIES AND DEFICIENCIES OF EXISTING METHODS

This section presents an evaluation of the capabilities and deficiencies of existing methods for capacity and quality of service analysis of two-lane highways. This evaluation focuses on the procedures of the existing HCM Chapter 8. Alternative methodologies that have been proposed or put into actual practice are also considered.

HCM METHODOLOGIES FOR TWO-LANE HIGHWAYS

The most widely accepted two-lane highway capacity and quality-of-service analysis procedures are those presented in the *Highway Capacity Manual* (HCM) published by the Transportation Research Board. The HCM was first published in 1950 and has been updated in subsequent editions in 1965, 1985, and 1994.(1,2,3,4) The updating of the HCM and its various capacity and quality-of-service analysis procedures is overseen by the TRB Committee on Highway Capacity and Quality of Service (HCQS). The following discussion reviews the historical development of the HCM procedures.

2.1.1 1950 HCM

The earliest procedures for analysis of two-lane highway capacity, developed by O.K. Normann, determined the “practical” capacity of a two-lane road (the forerunner of service volume) by comparing the demand for passing with observed actual passing rates at various flow rates.(1)

2.1.2 1965 HCM

The 1965 HCM(2) first presented the level-of-service concept for highway capacity analysis that is still in use today. Under this concept, a quantitative measure is selected to represent the quality of service provided to motorists on a given facility type, relationships between that measure of service and traffic flow rate are developed, and certain combinations of the service measure and flow rate are defined to represent capacity and the boundaries between levels of service A through E.

For two-lane highways, the 1965 HCM provided an operational analysis procedure that was based on two service measures: the operating speed of traffic over a roadway section and the volume-to-capacity ratio. The 1965 HCM estimated the capacity of a two-

lane highway as 2,000 passenger cars/hr (pc/h) for both directions of travel combined, regardless of the directional split of traffic. Table 1 presents the level of service criteria for two-lane highways that were presented in the 1965 HCM. Figure 1 illustrates the speed-flow relationships on which the 1965 procedure was based.

2.1.3 1985 HCM

Research leading to the updated two-lane highway capacity and level of service analysis procedure for the 1985 HCM(3) was undertaken by Texas Transportation Institute (TTI) in NCHRP Project 3-28A.(5) This research reexamined the level of service measure used for two-lane highway analyses and concluded that mean speed did not adequately reflect the relative balance between passing demand and passing supply that is fundamental to traffic operations on extended sections of two-lane highways. It was also found that, on a given highway facility, the mean speed of traffic was much less sensitive to traffic flow rate than previously supposed. The research developed a new level of service measure that was subsequently named “percent time delay.” Percent time delay is the percentage of their cumulative travel time that drivers on a particular roadway section spend following in platoons behind other vehicles. Thus, percent time delay tends to increase when passing demand along a section of road exceeds passing supply, and tends to decrease when passing supply exceeds passing demand. Percent time delay is measured over an extended section of highway, so it is a space-averaged measure like average travel speed, but further analyses in the research established that percent time delay could be approximated in the field by a spot measure—the percentage of vehicles traveling at headways of 5 sec or less at one or more representative points within the section.

Another key revision in the 1985 HCM was that the capacity of a two-lane highway was determined to be a function of the directional split of traffic, ranging from a capacity of 2,800 pc/h in both directions of travel combined for a 50/50 directional split to 2,000 pc/h for a 100/0 split (i.e., all traffic in one of the two directions).

The two-lane highway capacity and level of service analysis procedures are presented in Chapter 8 of the 1985 HCM. Figure 2, based on HCM Figure 8-1, illustrates the typical relationships of average travel speed and percent time delay to the flow rate for both directions of travel combined. This figure illustrates that percent time delay is more sensitive to flow rate than is speed.

Table 2 presents the level of service criteria for general terrain segments on two-lane highways, as shown in Table 8-1 of the 1985 HCM.

Table 1. Level of Service Criteria for Two-Lane Highways Used in the 1965 HCM(2)

Level of service	Traffic flow conditions		Passing sight distance > 1,500 ft (%)	Limiting service volume/capacity (v/c) ratio					
	Description	Operating		Basic limiting value ^a for average highway speed of 70 mi/h ^b	Working value for restricted average highway speed of: ^{b,c}				
					60 mi/h	50 mi/h	45 mi/h	40 mi/h	35 mi/h
A	Free flow	≥ 60	100	≤ 0.20	–	–	–	–	–
			80	0.18	–	–	–	–	–
			60	0.15	–	–	–	–	–
			40	0.12	–	–	–	–	–
			20	0.08	–	–	–	–	–
			0	0.04	–	–	–	–	–
B	Stable flow (upper speed range)	≥ 50	100	≤ 0.45	≤ 0.40	–	–	–	–
			80	0.42	0.35	–	–	–	–
			60	0.38	0.30	–	–	–	–
			40	0.34	0.24	–	–	–	–
			20	0.30	0.18	–	–	–	–
			0	0.24	0.12	–	–	–	–
C	Stable flow	≥ 40	100	≤ 0.70	≤ 0.66	≤ 0.56	≤ 0.51	–	–
			80	0.68	0.61	0.53	0.46	–	–
			60	0.65	0.56	0.47	0.41	–	–
			40	0.62	0.51	0.38	0.32	–	–
			20	0.59	0.45	0.28	0.22	–	–
			0	0.54	0.38	0.18	0.12	–	–
D	Approaching unstable flow	≥ 35	100	≤ 0.85	≤ 0.83	≤ 0.75	≤ 0.67	≤ 0.58	–
			80	0.84	0.81	0.72	0.62	0.55	–
			60	0.83	0.79	0.69	0.57	0.51	–
			40	0.82	0.76	0.66	0.52	0.45	–
			20	0.81	0.71	0.61	0.44	0.35	–
			0	0.80	0.66	0.51	0.30	0.19	–
E	Unstable flow	≥ 30 ^d	Not applicable ^e	-----	≤ 1.00	-----	-----	-----	-----
F	Forced flow	< 30 ^d	Not applicable ^e	-----	≤ 1.00	-----	-----	-----	-----

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

^b Average highway speed is the weighted average of the design speeds of roadway elements that comprise the analysis section.

^c Where no entry appears, the operating speed required for this level is unattainable even at low volumes.

^d Approximately.

^e No passing is possible at this flow rate.

^f Demand volume/capacity ratio may well exceed 1.00, indicating overloading.

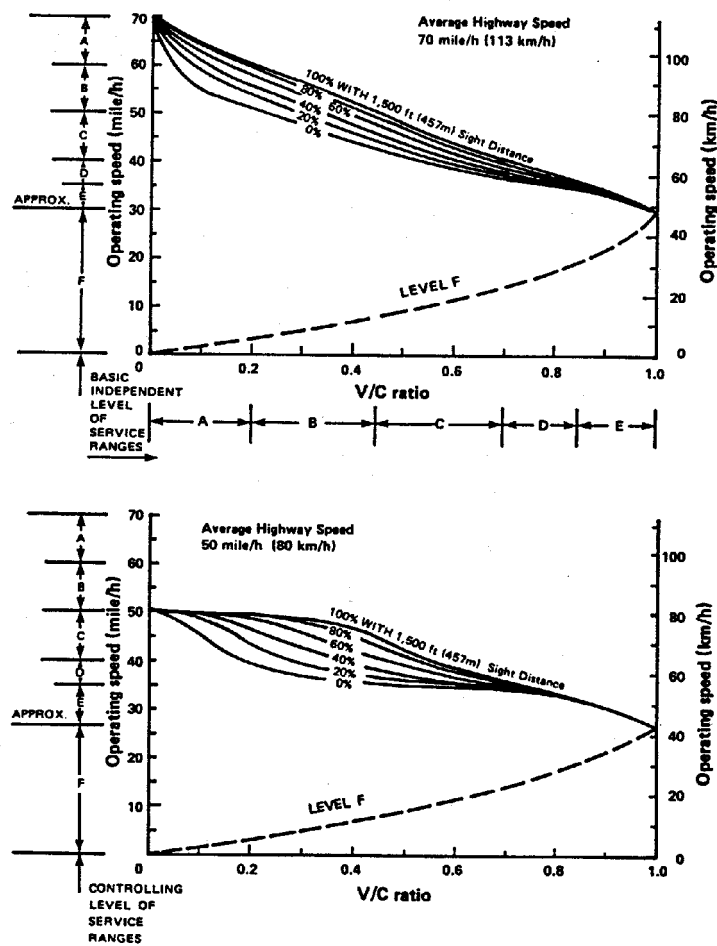


Figure 1. Speed-Flow Relationships for Two-Lane Highways Used in the 1965 HCM(2)

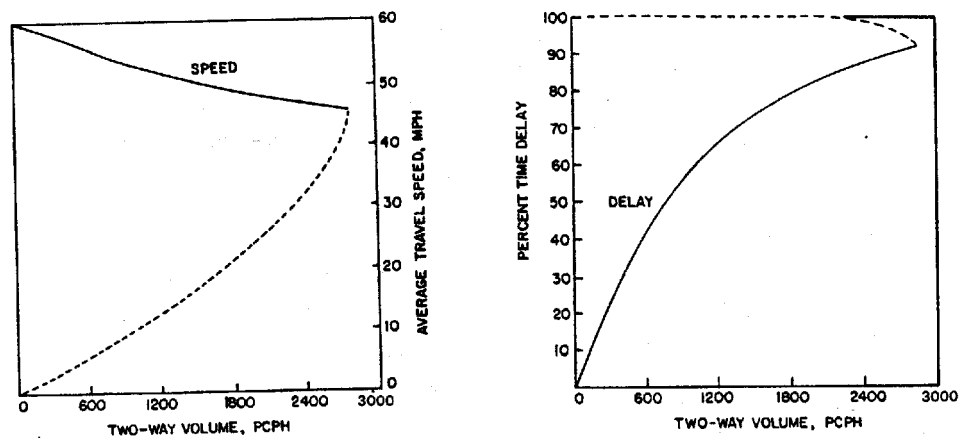


Figure 2. Relationships of Percent Time Delay and Average Travel Speed to Flow Rate Used in the 1985 HCM(3,4)

Table 2. Level of Service Criteria for Operational Analysis of General Terrain Segments on Two-Lane Highways in the 1985 HCM(3,4)

TABLE 8-1. LEVEL-OF-SERVICE FOR GENERAL TWO-LANE HIGHWAY SEGMENTS

LOS	PERCENT TIME DELAY	v/c RATIO ^a																				
		LEVEL TERRAIN							ROLLING TERRAIN							MOUNTAINOUS TERRAIN						
		AVG ^b SPEED	PERCENT NO PASSING ZONES						AVG ^b SPEED	PERCENT NO PASSING ZONES						AVG ^b SPEED	PERCENT NO PASSING ZONES					
			0	20	40	60	80	100		0	20	40	60	80	100		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 These speeds are provided for information only and apply to roads with design speeds of 60 mph or higher.

HCM Equation 8-1, presented below, is used to compute the service flow rate for a two-lane highway:

where:
$$SF_i = 2800 \times (v/c)_i \times f_d \times f_w \times f_{HV} \quad (1)$$

where: SF_i = service volume (two-way) for level of service i for prevailing roadway and traffic conditions (pc/h)

$(v/c)_i$ = volume-to-capacity ratio for level of service i (from HCM Table 8-1)

f_d = adjustment factor for directional distribution of traffic (from HCM Table 8-4)

f_w = adjustment factor for narrow lanes and restricted lateral clearance (from HCM Table 8-5)

f_{HV} = adjustment factor for the presence of heavy vehicles in the traffic stream (from HCM Equation 8-2 and HCM Table 8-6)

Although it does not appear to be explicitly, Equation (1) is functionally a prediction equation to estimate percent time delay from the values of the other factors, since the service flow rate for any given level of service corresponds to a particular value of percent time delay.

The values of $(v/c)_i$ presented in HCM Table 8-1 represent the percent time delay vs. flow relationship shown in Figure 2 (HCM Figure 8-1), for each level of service, as affected by terrain and percent no-passing zones. The capacity of a two-lane road under otherwise ideal conditions is assumed to vary from 2,000 to 2,800 pc/h as a function of the directional distribution of traffic, based on an adjustment factor (f_d). Other adjustment factors are provided for lane width and usable shoulder width (f_w) and heavy vehicles (f_{HV}). The values of the adjustment factors f_d , f_w , f_{HV} are assumed to be independent of one another. The values of the f_w and f_{HV} vary with level of service (i.e., with flow rate), while the values of f_d do not. The level of service is determined by comparing the actual hourly flow rate for the highest-flow 15-min period of the peak hour with the service flow rates computed with Equation (1).

Equation (1) incorporates adjustment factors to account for the effects of the directional distribution of traffic, narrow lanes and restricted shoulder width, and the presence of heavy vehicles in the traffic stream. Table 3 summarizes the adjustment factors for directional distribution of traffic (f_d) from HCM Table 8-4, and for narrow lanes and usable shoulder width (f_w) from HCM Table 8-5. In the development of Chapter 8 of the 1985 HCM, the passenger car equivalents (PCEs) used in the heavy vehicle adjustment factors for trucks and buses were revised and PCEs for recreational vehicles (RVs) were introduced for the first time. The PCEs for heavy vehicles in HCM Chapter 8 were based

Table 3. Adjustment Factors for Directional Distribution of Traffic, Lane Width, and Shoulder Width in General Terrain Segment Procedure(3,4)

<i>Directional Split</i>	<i>Total Capacity (pcph)</i>	<i>Ratio of Capacity to Ideal Capacity</i>
50/50	2,800	1.00
60/40	2,650	0.94
70/30	2,500	0.89
80/20	2,300	0.83
90/10	2,100	0.75
100/0	2,000	0.71

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 ^b		11-FT LANES ^b		10-FT LANES ^b		9-FT LANES ^b	
	LOS A-D	LOS E	LOS A-D	LOS E	LOS A-D	LOS E	LOS A-D	LOS 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 For analysis of specific grades, use LOS E factors for all speeds less than 45 mph.

on the work of Werner and Morrall(6,7) and Cunagin and Messer(8). The PCEs for general terrain segments are summarized in Table 4.

The previous discussion has stated that the operational analysis procedure for general terrain segments is based on percent time delay, which is a measure of traffic platooning. Figure 3 uses a related measure, the percent of traffic traveling in platoons at any given spot on the road, to illustrate how useful platooning-based measures can be in describing the effectiveness of two-lane highway improvements, such as passing lanes, that are intended to increase the availability of passing opportunities(9). The figure shows that a passing lane can result in a substantial reduction of platooning (and, thus, a substantial increase in level of service) over the length of the passing lane. Furthermore, the reduced platooning levels may persist for 3 to 8 mi (5 to 13 km) downstream, so that the effective length of roadway over which the passing lane provides traffic operational benefits is much greater than its actual length. HCM Chapter 8 contains a section identifying design and operational treatments that can be used on two-lane highways, but does not include a procedure to quantify their operational effects.

A separate operational analysis procedure is provided to determine the level of service for specific grades, which generally consist of upgrades of 3% or more that are at least 0.5 miles in length. The specific grade procedure operates similarly in that level of service is determined by comparing the actual flow rate to specific service flow rates, but the service measure used for the specific grade procedure is average upgrade speed. Table 5 shows the differences between the level of service criteria for general terrain segments and those used for specific grades. The specific grade procedure uses a service flow rate relationship that is similar to that used in the general terrain segment procedure:

$$SF_i = 2800 \times (v/c)_i \times f_d \times f_w \times f_g \times f_{HV} \quad (2)$$

where: SF_i = total service flow rate (two-way) for level of service i for prevailing roadway and traffic conditions (pc/h)

$(v/c)_i$ = volume-to-capacity ratio for level of service i (from HCM Table 8-7)

f_d = adjustment factor for directional distribution of traffic (from HCM Table 8-8)

f_w = adjustment factor for narrow lanes and restricted shoulder width (from HCM Table 8-5)

f_g = adjustment factor for the operational effects of grades on passenger cars (from HCM Equations 8-4 and 8-5)

f_{HV} = adjustment factor for the presence of heavy vehicles in the traffic stream (from HCM Equation 8-6 and HCM Table 8-9)

Although it does not appear to be explicitly, Equation (2) is functionally a method to estimate average upgrade speed from the values of the other parameters, since the service flow rate for any given level of service corresponds to a particular value of average upgrade speed.

The differences between Equations (1) and (2) are as follows:

- The $(v/c)_i$ term in Equation (2) represents the speed-flow relationship on various grades, as affected by percent grade and percent no-passing zones, rather than the percent time delay-flow relationship.
- The adjustment factors f_d and f_{HV} have different values.
- A new adjustment factor f_g represents the slowing of passenger cars on the grade (in effect, a passenger car equivalent for passenger cars).

HCM Chapter 8 also contains a planning analysis procedure, which is essentially a simplified version of the operational analysis procedure for general terrain segments in which a number of the parameters in the operational analysis procedure are set to default values. The planning analysis procedure explicitly incorporates assumptions concerning the design hour factor (K) and the peak hour factor (PHF) so that results are expressed in terms of ADTs that can be served by particular facility conditions, rather than peak 15-min volumes expressed as hourly flow rates, which are typically used in HCM operational analysis procedures.

The analysis procedures in HCM Chapter 8 are accompanied by manual work sheets that can be used to implement the procedures. The work sheets are accompanied by six computational examples for the operational analysis procedure and three computational examples for the planning procedure.

2.1.4 1994 and 1997 HCM

TRB has issued a new editions of the HCM in 1994 and 1997 with a number of updated chapters. However, because of the lack of funded research on two-lane highway capacity over the intervening years, the two-lane chapter in these editions remains unchanged from 1985.

HCM 2000

TRB plans to publish a new edition of the HCM in the year 2000. This edition will contain revised operational analysis procedures for two-lane highways. The development of these revised procedures is documented in this report.

Table 4. Passenger Car Equivalents of Heavy Vehicle Used in the General Terrain Segment Procedure(3,4)

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

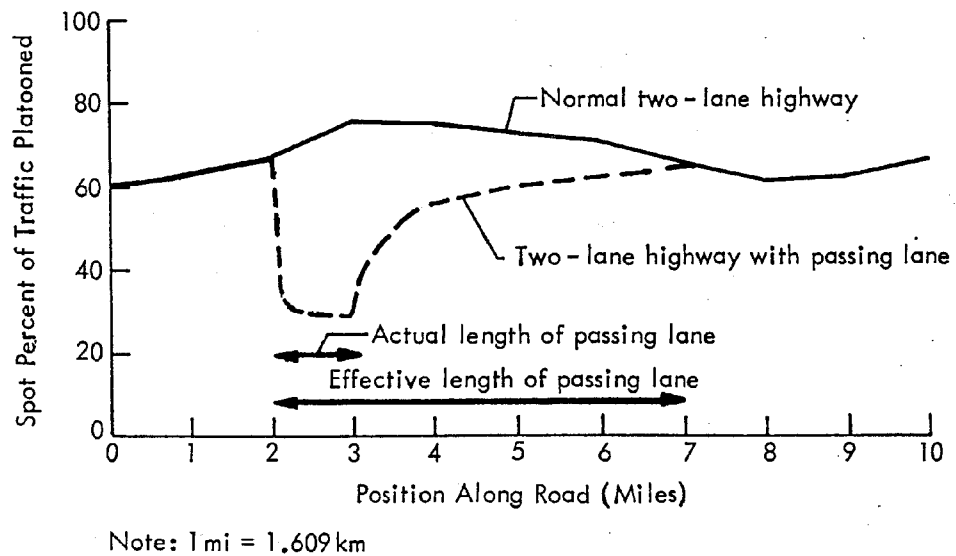


Figure 3. Example of the Effect of a Passing Lane on Two-Lane Highway Operations(8)

Table 5. Comparison of Level of Service Criteria for General Terrain Segments and Specific Grades on Two-Lane Highways in the 1985 HCM(3,4)

Level of Service	Percent Time Delay on General Segments	Average Upgrade Speed (mi/hr) on Specific Grades
A	≤ 30	≥ 55
B	≤ 45	≥ 50
C	≤ 60	≥ 45
D	≤ 75	≥ 40
E	> 75	$\geq 25-40$
F	100	$< 25-40$

SENSITIVITY ANALYSIS OF THE CURRENT HCM CHAPTER 8 METHODOLOGY

A sensitivity analysis has been conducted by the research team to illustrate the effects of the various adjustment factors on service flow rates and capacity in the current version of HCM Chapter 8. This sensitivity analysis has addressed both the general terrain segment procedure and the specific grade procedure. The results are presented below. The computations of service flow rates for particular roadway and traffic conditions was performed with the HCS.

2.2.1 General Terrain Segment Procedure

The figures presented in the following discussion illustrate the sensitivity of the percent time delay vs. flow rate relationship to the various adjustment factors used in Equation (1) in the general terrain segment procedure in HCM Chapter 8. Each figure shows the effect of the adjustment factors in the procedure in comparison to ideal conditions, which are defined in the chapter as:

- Design speed greater than or equal to 97 km/h (60 mi/h)
- Lane widths greater than or equal to 3.6 m (12 ft)
- Clear shoulders wider than or equal to 1.8 m (6 ft)
- No “no-passing zones” on the highway
- All passenger cars in the traffic stream (i.e., no heavy vehicles)
- A 50/50 directional split of traffic
- No impediments to through traffic due to traffic control or turning vehicles
- Level terrain

Explicit factors are included in the procedures to account for the effects of each of these factors under non-ideal conditions except design speed and impediments to through traffic. The following discussion addresses each of these adjustment factors.

Effect of No-Passing Zones

Figure 4 illustrates the sensitivity of service volume (i.e., the sensitivity of the percent time delay vs. flow rate relationship as assumed in HCM Chapter 8) to the percentage of no-passing zones on the roadway being analyzed. Curves are shown for 0, 20, 40 60, 80, and 100 percent no-passing zones. The adjustment factors for percent no-passing zones

are embedded within v/c ratios in HCM Table 8-1 (see Table 2). Figure 4 shows that, logically, percent no-passing zones have no effect on service volumes at very low or very high flow rates, but can have a substantial effect on service volumes at intermediate flow rates. The service volumes in Figure 4 and subsequent figures are the flow rates at the boundaries between levels of service illustrated at the right-hand edge of each graph.

Effect of Directional Distribution of Traffic

Figure 5 illustrates the effect on service volumes on the directional distribution of traffic. The effect of directional split on service volume increases as the flow rate increases. As the directional distribution of traffic varies from a balanced 50/50 split to an unbalanced 100/0 split, the capacity (service volume for LOS E) decreases from 2,800 pc/h to approximately 2,000 pc/h for both directions of travel combined.

Effect of Lane Width

Figure 6 illustrates the sensitivity of service volume to lane width. The effect of reduced lane width on service volume increases as the flow rate increases. At capacity, with all other factors held at ideal conditions, a roadway with 2.7-m (9-ft) lanes has a service volume that is 76 percent of the service flow rate for a roadway with 3.6-m (12-ft) lanes.

Effect of Usable Shoulder Width

Figure 7 illustrates the sensitivity of service volume to usable shoulder width. The effect of reduced shoulder width on service volume increases as the flow rate increases up to a flow rate of 1,200 pc/h and then the curves shown in Figure 7 begin to converge slightly. With all other factors held at ideal conditions, a roadway with no shoulders has a service volume that is, at worst, 70 percent of the service volume for a roadway with 1.8-m (6-ft) shoulders.

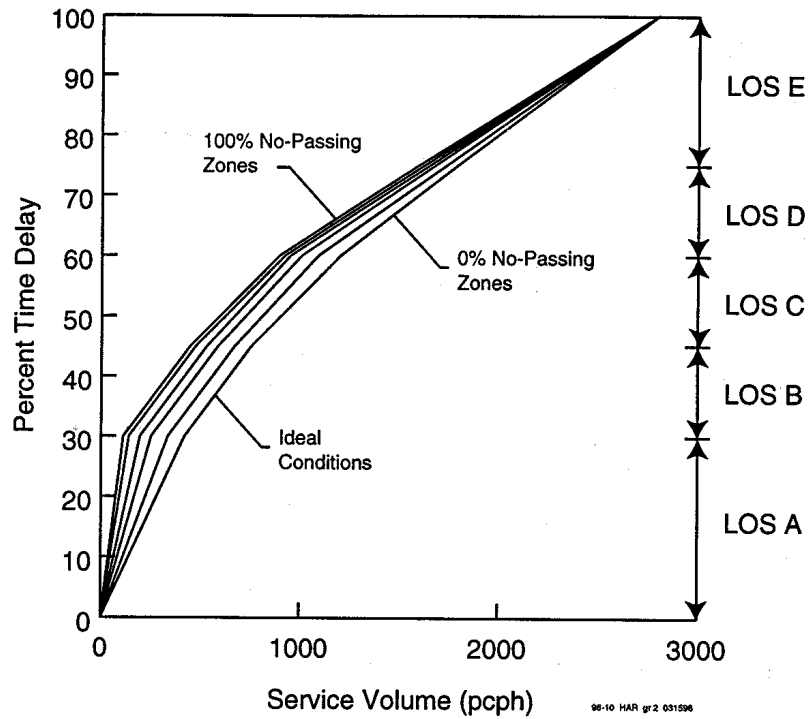


Figure 4. Sensitivity of Service Volume to Percent No-Passing Zones in the General Terrain Segment Procedure

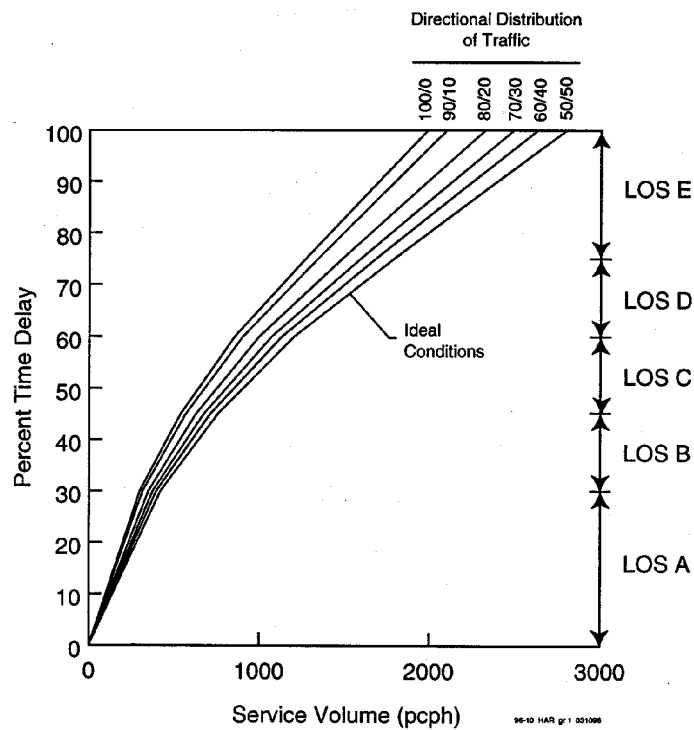


Figure 5. Sensitivity of Service Volume to Directional Distribution of Traffic in the General Terrain Segment Procedure

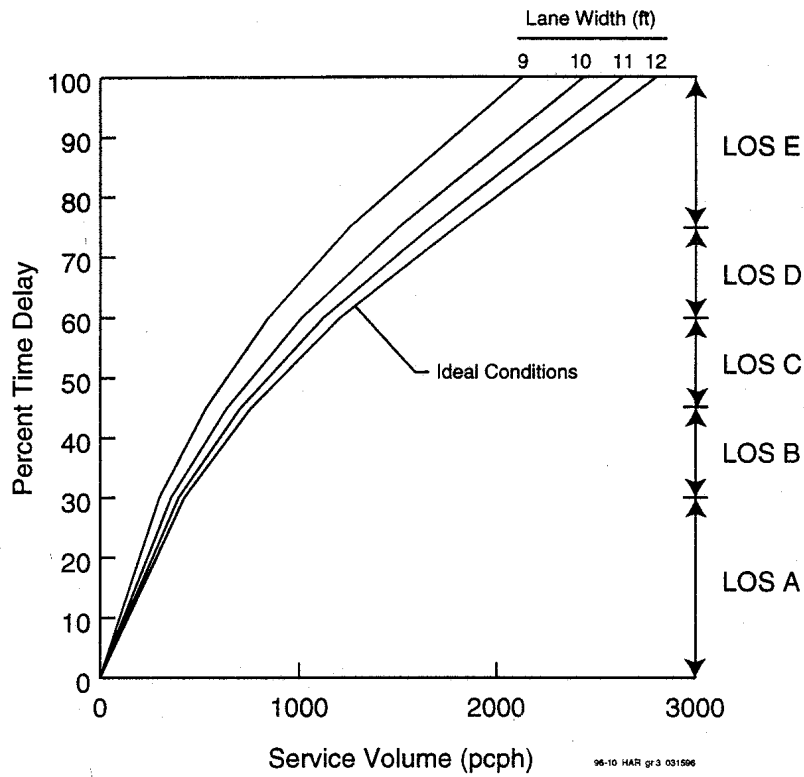


Figure 6. Sensitivity of Service Volume to Lane Width in the General Terrain Segment Procedure

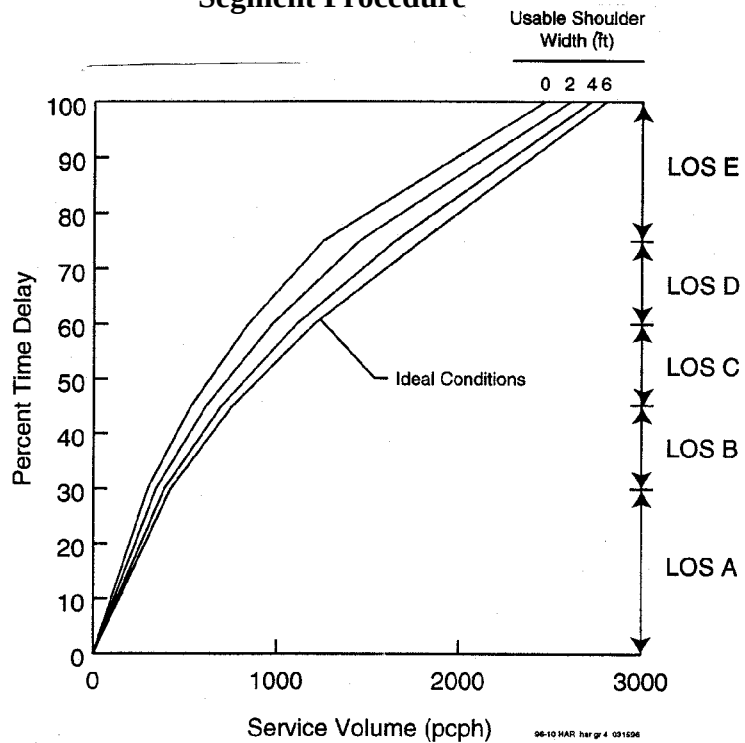


Figure 7. Sensitivity of Service Volume to Usable Shoulder Width in the General Terrain Segment Procedure

Effect of Truck Percentage

Figures 8 through 10 illustrate the effect of truck percentage on service volume for level, rolling, and mountainous terrain, respectively. Each figure compares the percent time delay vs. flow rate relationship for various truck percentages (0, 5, 10, and 20 percent) to the relationship for ideal conditions (i.e., in level terrain). The figures show that the effect of trucks on service volume increases as the flow rate increases over the entire range of flow rates. The magnitude of the truck effects increase progressively for rolling and mountainous terrain, in comparison to level terrain. When all other factors are held at ideal conditions, the presence of 20 percent trucks in the traffic stream decreases capacity to 83% of its ideal value for level terrain, 54 percent of ideal for rolling terrain, and 28 percent of ideal for mountainous terrain, as shown in Table 6.

Effect of RV Percentage

Figures 11 through 13 illustrate the effect of truck percentage on service volume for level, rolling, and mountainous terrain, respectively. Each figure compares the percent time delay vs. flow rate relationship for various RV percentages (0, 5, 10, and 20 percent) to the relationship for ideal conditions (i.e., in level terrain). The figures show that the effect of RVs on service volume generally increases as the flow rate increases, except for level terrain at higher flow rates where the RV effects do not appear to change much with flow. The magnitude of the RV effects increase progressively for rolling and mountainous terrain, in comparison to level terrain. When all other factors are held at ideal conditions, the presence of 20 percent RVs in the traffic stream decreases capacity to 89 percent of its ideal value for level terrain, 66 percent of ideal for rolling terrain, and 49 percent of ideal for mountainous terrain, as shown in Table 7.

Combination of Effects

Figure 14 illustrates the effect on the percent time delay vs. flow rate relationship when all of the adjustment factors in HCM Chapter 8 are set at their “worst case” or “near worst case” levels. The “worst case” curve in the figure represents a roadway with the following conditions:

- 3.6-m (9-ft) lane width
- No usable shoulder width
- 100 percent no-passing zones
- 20 percent trucks in the traffic stream
- 100/0 directional split
- Mountainous terrain

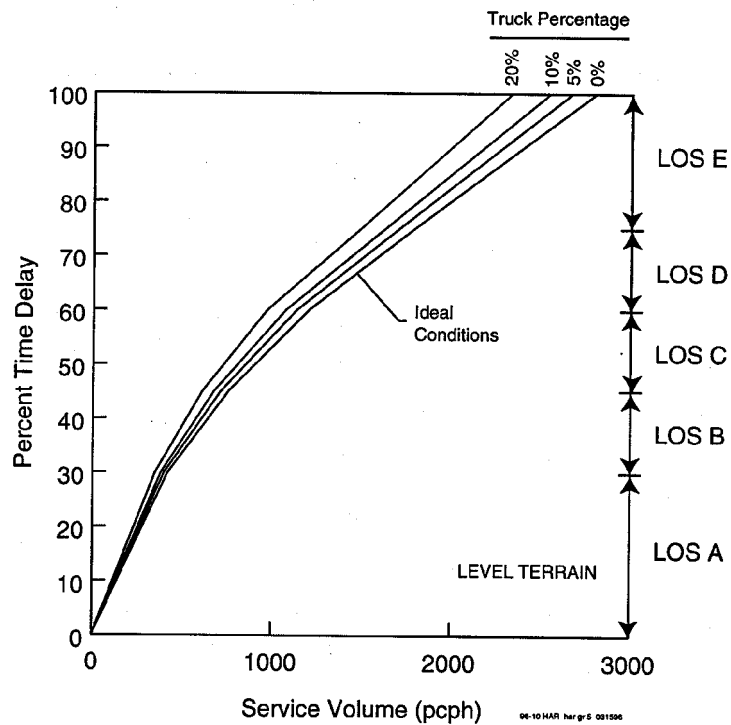


Figure 8. Sensitivity of Service Volume to Truck Percentage for Level Terrain in the General Terrain Segment Procedure

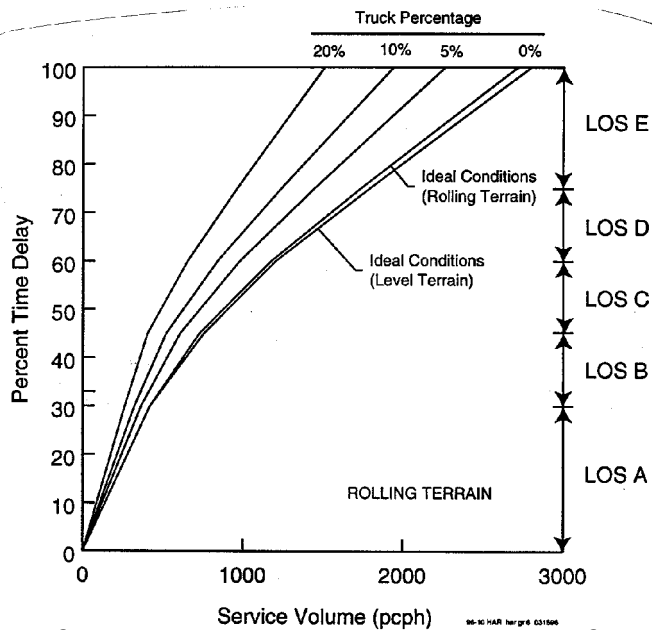


Figure 9. Sensitivity of Service Volume to Truck Percentage for Rolling Terrain in the General Terrain Segment Procedure

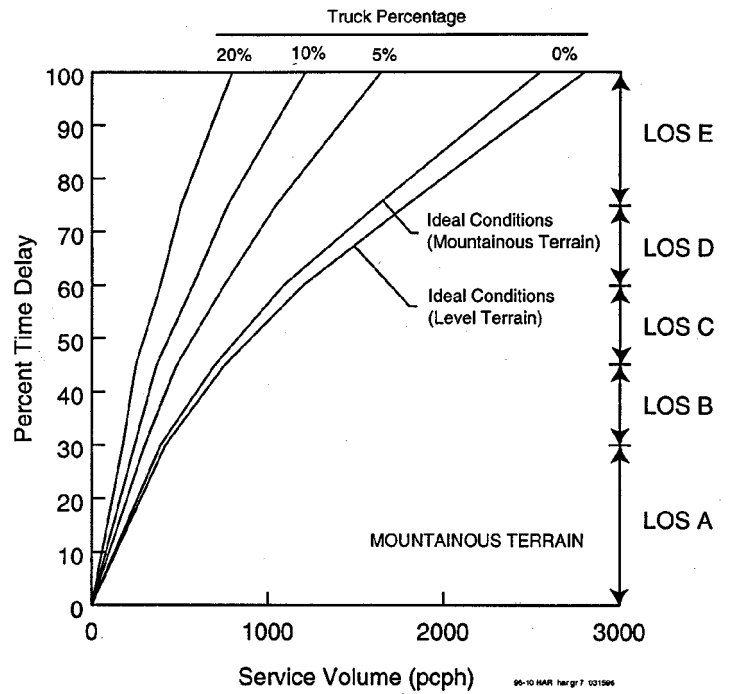


Figure 10. Sensitivity of Service Volume to Truck Percentage for Mountainous Terrain in the General Terrain Segment Procedure

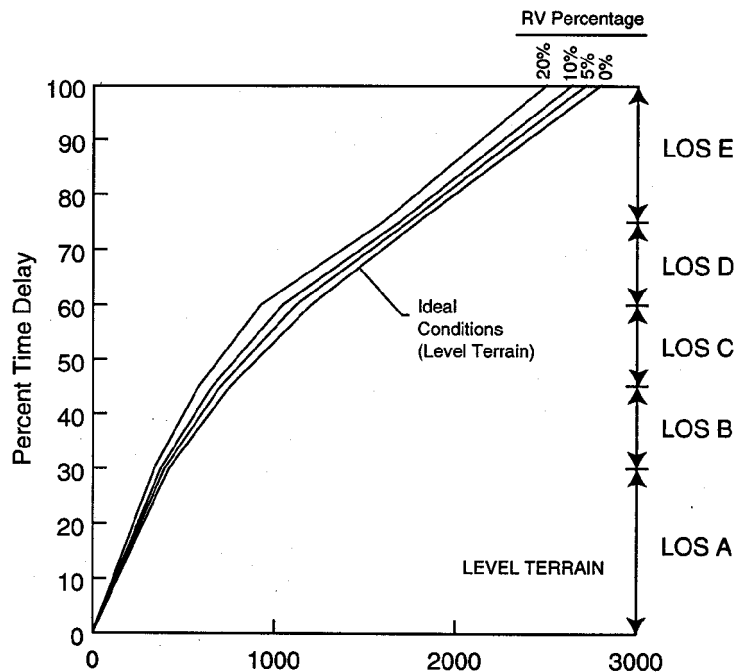


Figure 11. Sensitivity of Service Volume to RV Percentage for Level Terrain in the General Terrain Segment Procedure

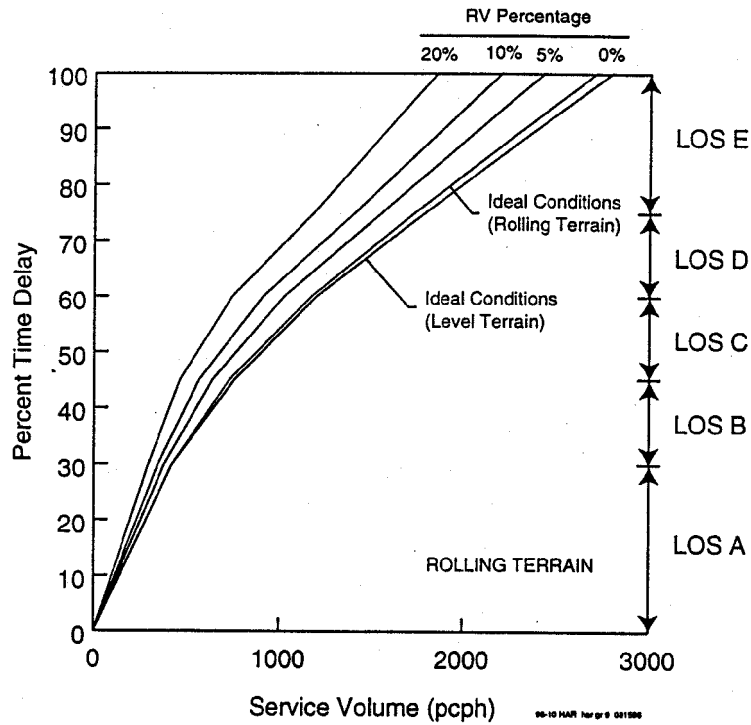


Figure 12. Sensitivity of Service Volume to RV Percentage for Rolling Terrain in the General Terrain Segment Procedure

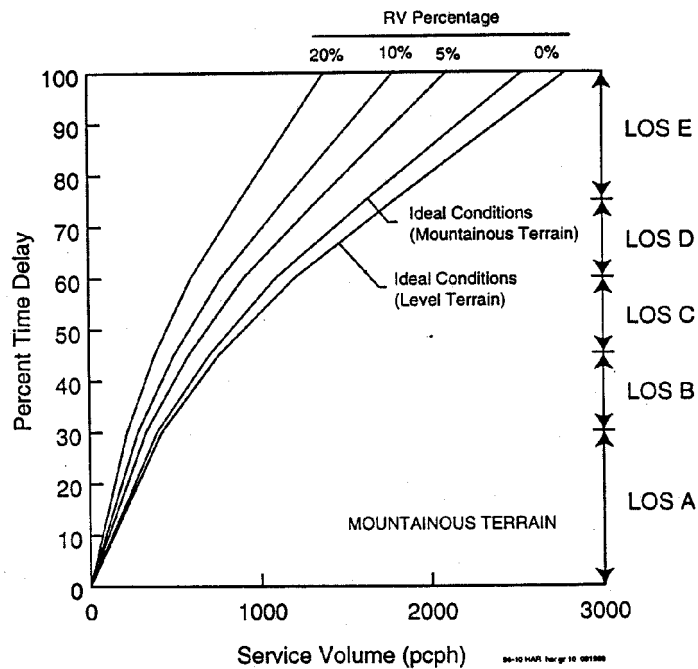


Figure 13. Sensitivity of Service Volume to RV Percentage for Mountainous Terrain in the General Terrain Segment Procedure

Table 6. Variation of Service Volumes as a Function of Terrain and Percent Trucks

Terrain	Percent trucks	Service volume for level of service:					Capacity expressed as a percentage of ideal capacity
		A	B	C	D	E	
Level ^a	0	420	756	1204	1792	2800	100%
	5	400	713	1136	1707	2667	95%
	10	382	675	1075	1629	2545	91%
	20	350	610	971	1493	2333	83%
Rolling ^b	0	420	728	1176	1736	2716	97%
	5	365	607	980	1447	2263	81%
	10	323	520	840	1240	1940	69%
	20	263	404	653	964	1509	54%
Mountainous ^c	0	392	700	1092	1624	2548	91%
	5	302	483	753	1048	1644	59%
	10	245	368	575	773	1213	43%
	20	178	250	390	507	796	28%

^a Based on data shown in Figure 8.

^b Based on data shown in Figure 9.

^c Based on data shown in Figure 10.

Table 7. Variation of Service Volumes as a Function of Terrain and Percent Recreational Vehicles

Terrain	Percent RVs	Service volume for level of service:					Capacity expressed as a percentage of ideal capacity
		A	B	C	D	E	
Level ^a	0	420	756	1204	1792	2800	100%
	5	396	703	1120	1740	2718	97%
	10	375	657	1047	1691	2642	94%
	20	339	582	926	1600	2500	89%
Rolling ^b	0	420	728	1176	1736	2716	97%
	5	378	636	1027	1557	2436	87%
	10	344	564	912	1411	2208	79%
	20	292	461	744	1189	1860	66%
Mountainous ^c	0	392	700	1092	1624	2548	88%
	5	327	579	902	1342	2106	75%
	10	280	493	769	1144	1794	64%
	20	218	380	593	883	1385	49%

^a Based on data shown in Figure 11.

^b Based on data shown in Figure 12.

^c Based on data shown in Figure 13.

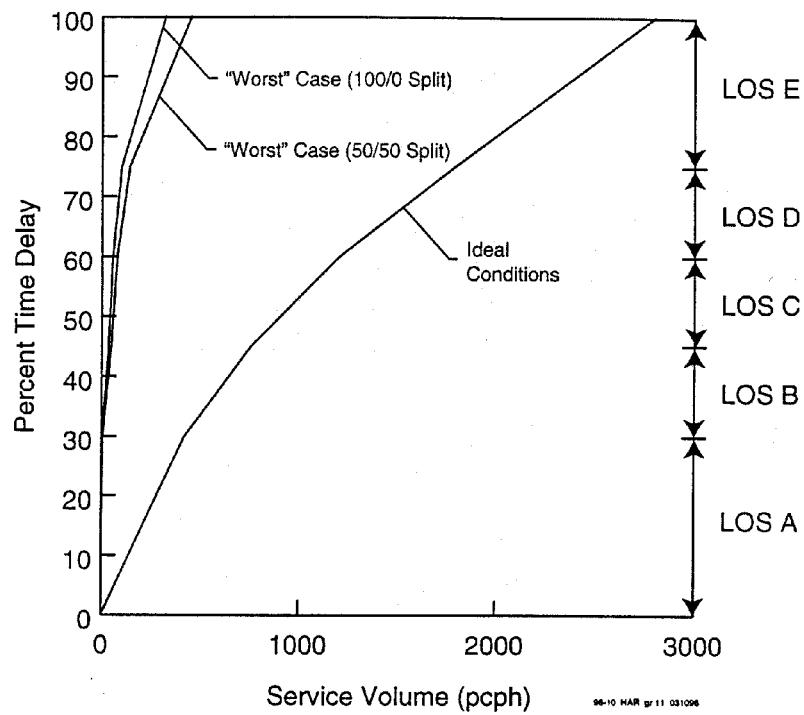


Figure 14. Relationship Between Percent Time Delay and Flow Rate for “Worst Cases” in the General Terrain Segment Procedure

A “near worst case” curve represents the same conditions except a 50/50 directional split. For comparative purposes, the figure also shows the percent time delay vs. service volume relationship for ideal conditions. The figure shows that the capacity for the “worst case” conditions is 11 percent of the capacity for ideal conditions and the capacity for the near “worst case” conditions is 16 percent of the capacity for ideal conditions.

2.2.2 Specific Grade Procedure

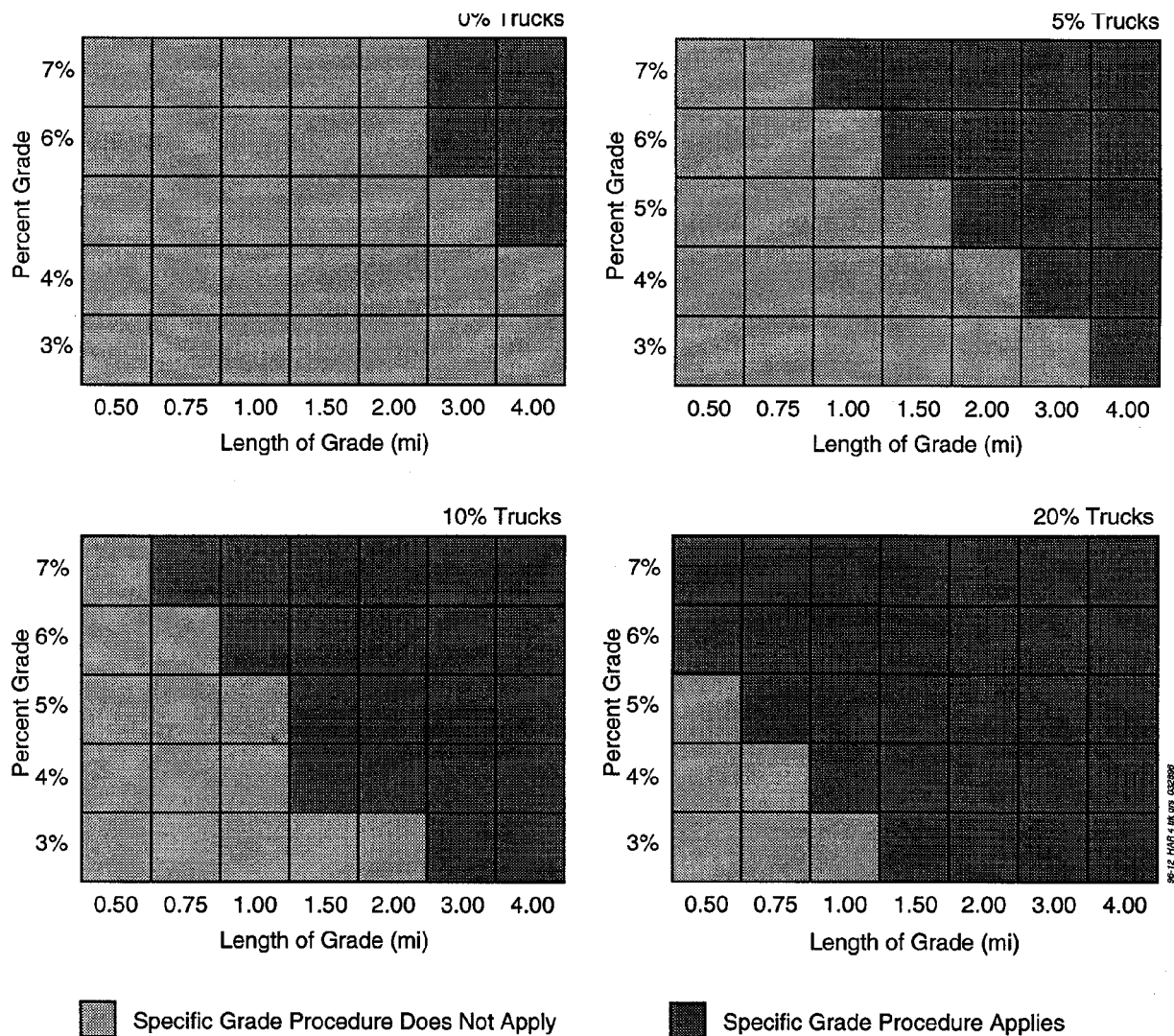
A sensitivity analysis was also performed for the specific grade procedure is HCM Chapter 8. The specific grade procedure uses passenger car equivalency factors for heavy vehicles on extended grades that vary with the percent grade and length of grade. Where percent grade varies along the length of a particular upgrade, the equivalent grade method is used to combine grades. The specific grade procedure includes passenger car equivalency factors for passenger cars, because even traffic streams without any heavy vehicles will slow down on an extended grade.

The specific grade procedure applies only to upgrades for which the capacity is less than 2,000 pc/h. Where the capacity equals or exceeds 2,000 pc/h, the specific grade methodology does not apply and the analysis in question should be performed with the general terrain segment procedure. Figure 15 illustrates for various truck percentages (0, 5, 10, and 20 percent), combinations of percent grade and length of grade to which the specific grade procedure does and does not apply; the figure assumes that all factors other than percent grade, length of grade, and vehicle mix represent ideal conditions. Comparable data for various RV percentages is presented in Figure 16.

Figures 17 through 21 illustrate the speed-flow relationships for trucks on 4 percent upgrades for each truck percentage and length of grade to which Figure 15 indicates that the specific grade procedure applies. The lengths of grade shown in the figures range from 1.6 to 6.4 km (1 to 4 mi). In other words, if the specific grade procedure does not apply to a given truck percentage and length of grade, then no speed-flow relationship is shown for that combination of truck percentage and length of grade. For comparative purposes, each figure shows the speed-flow relationship for ideal conditions (i.e., in level terrain).

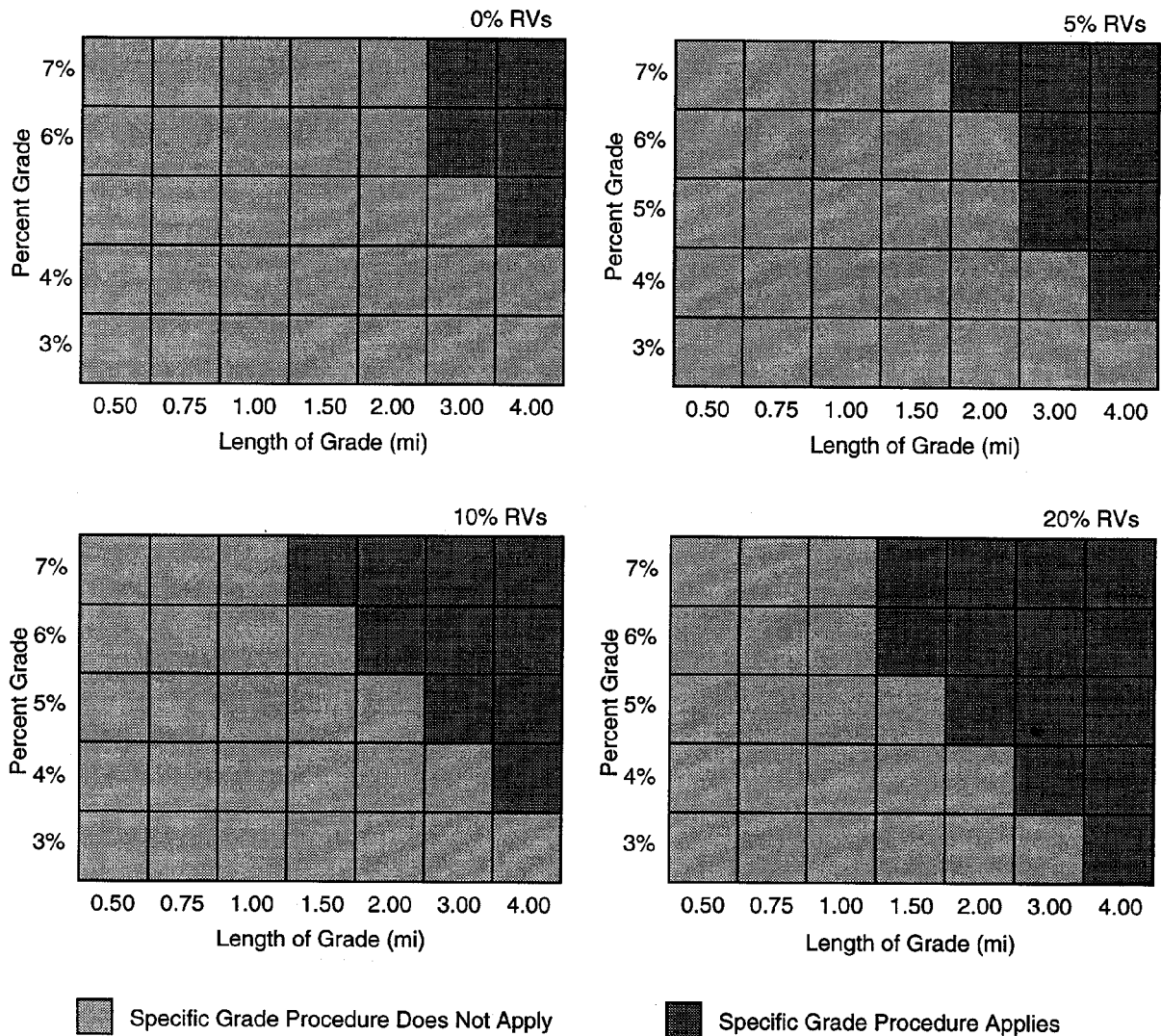
Comparable data for RVs on 4% upgrades are shown in Figures 22 and 23. The lengths of upgrades to which the specific grade procedure applies includes only the range from 4.8 to 6.4 km (3 to 4 mi) for flows with RVs, but no trucks, in the traffic stream.

Finally, Figures 24 and 25 show comparable data for trucks and RVs, respectively, on two steeper grades: a 6 percent upgrade with a length of 4.8 km (3 mi). This figure shows that average upgrade speeds can be pulled as low as 45 km/h (28 mi/h) by trucks and as low as 53 km/h (33 mi/h) by RVs.



Note: All factors other than percent grade, length of grade, and percent trucks represent ideal conditions.

Figure 15. Combinations of Truck Percentage, Percent Grade, and Length of Grade to Which the Specific Grade Procedure Applies



Note: All factors other than percent grade, length of grade, and percent RVs represent ideal conditions.

Figure 16. Combinations of RV Percentage, Percent Grade, and Length of Grade to Which the Specific Grade Procedure Applies

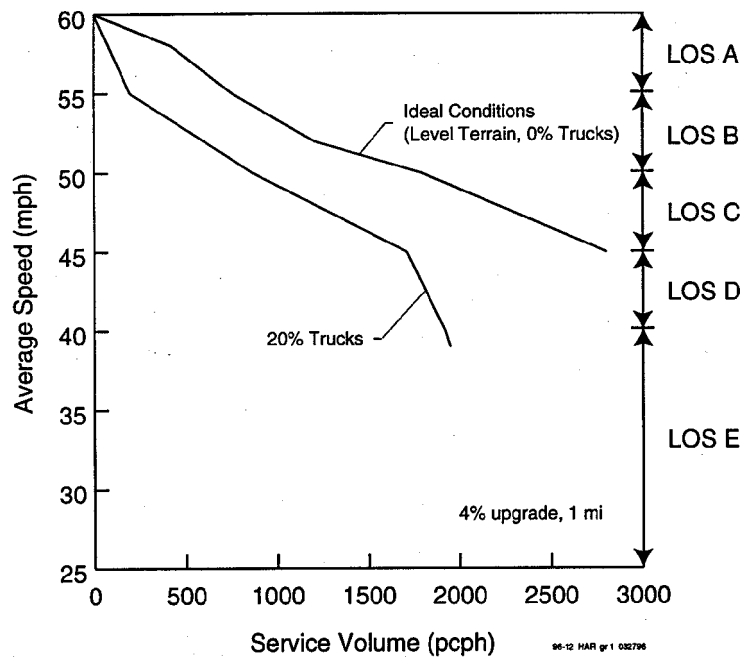


Figure 17. Speed-Flow Relationships Provided by the Specific Grade Procedure for Trucks on a 1.6-km (1-mi), 4 Percent Upgrade

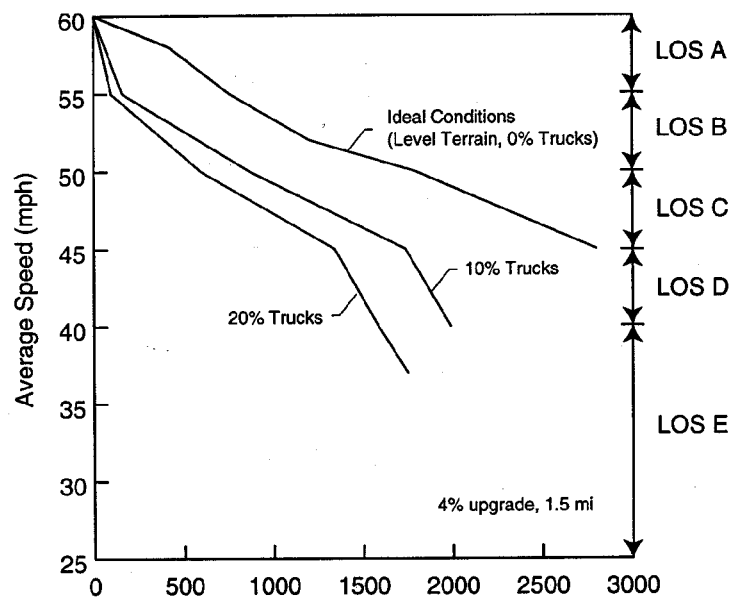


Figure 18. Speed-Flow Relationships Provided by the Specific Grade Procedure for Trucks on a 2.4-km (1.5-mi), 4 Percent Upgrade

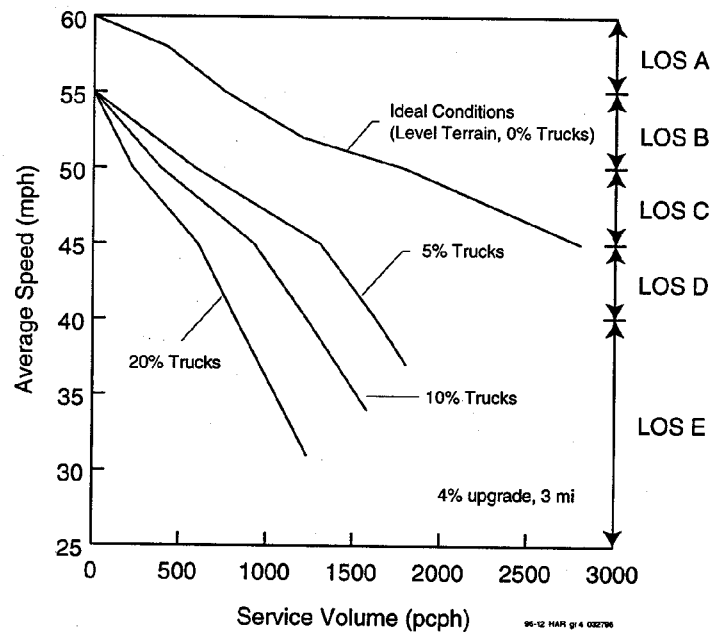


Figure 19. Speed-Flow Relationships Provided by the Specific Grade Procedure for Trucks on a 3.2-km (2-mi), 4 Percent Upgrade

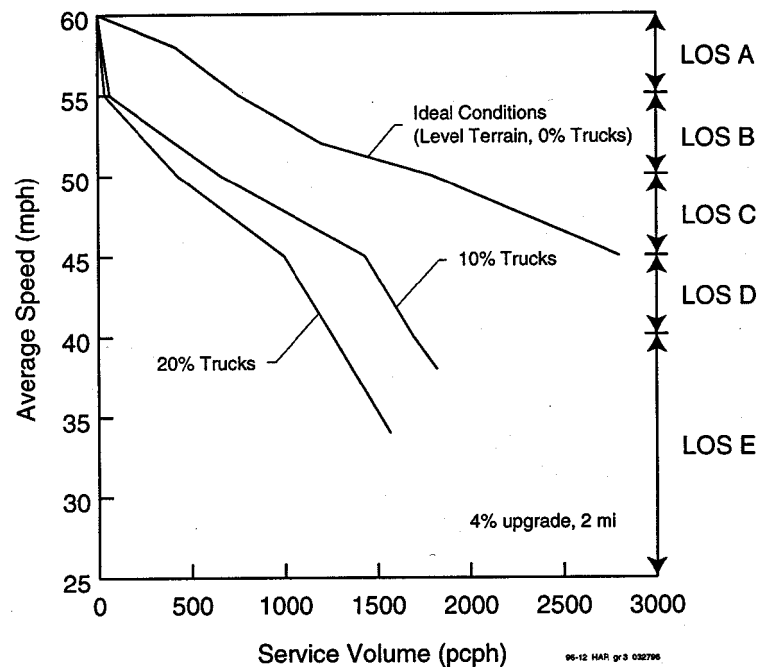


Figure 20. Speed-Flow Relationships Provided by the Specific Grade Procedure for Trucks on a 4.8-km (3-mi), 4 Percent Upgrade

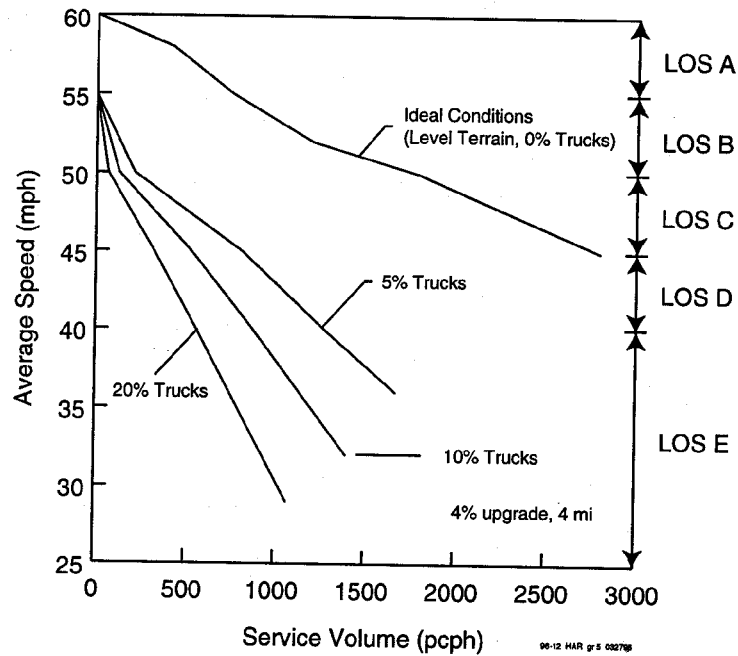


Figure 21. Speed-Flow Relationships Provided by the Specific Grade Procedure for Trucks on a 6.4-km (4-mi), 4 Percent Upgrade

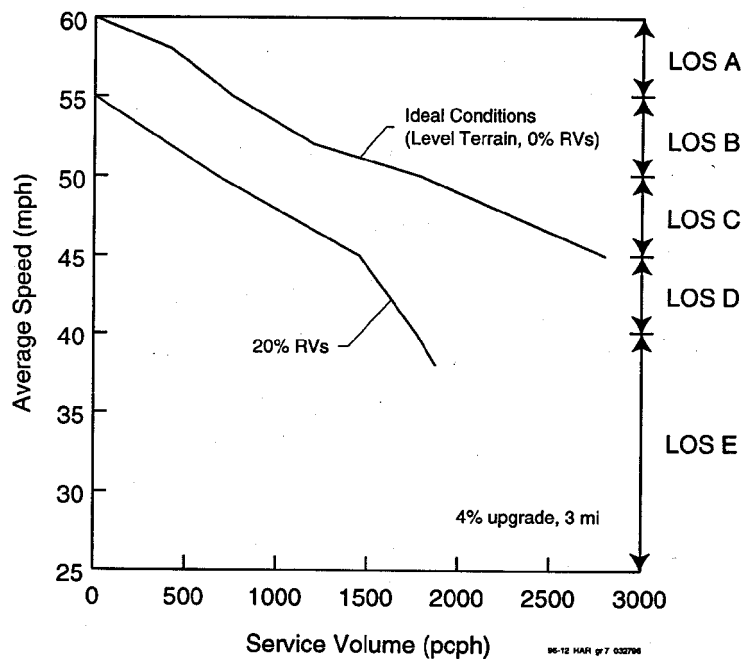


Figure 22. Speed-Flow Relationships Provided by the Specific Grade Procedure for RVs on a 4.8-km (3-mi), 4 Percent Upgrade

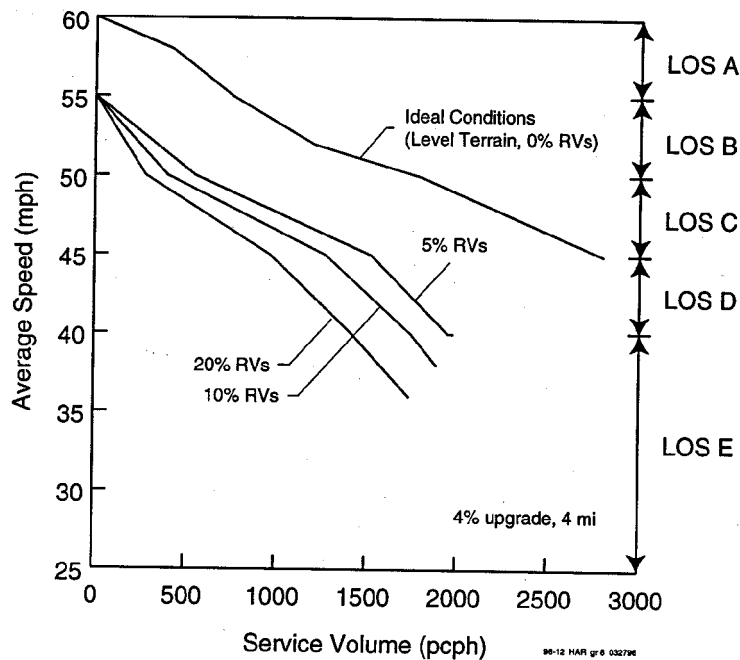


Figure 23. Speed-Flow Relationships Provided by the Specific Grade Procedure for RVs on a 6.4-km (4-mi), 4 Percent Upgrade

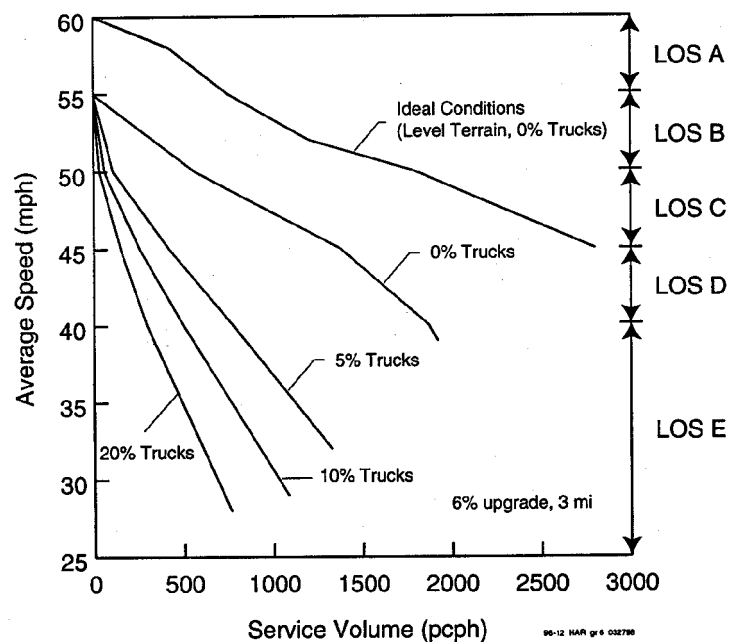


Figure 24. Speed-Flow Relationships Provided by the Specific Grade Procedure for Trucks on a 4.8-km (3-mi), 6 Percent Upgrade

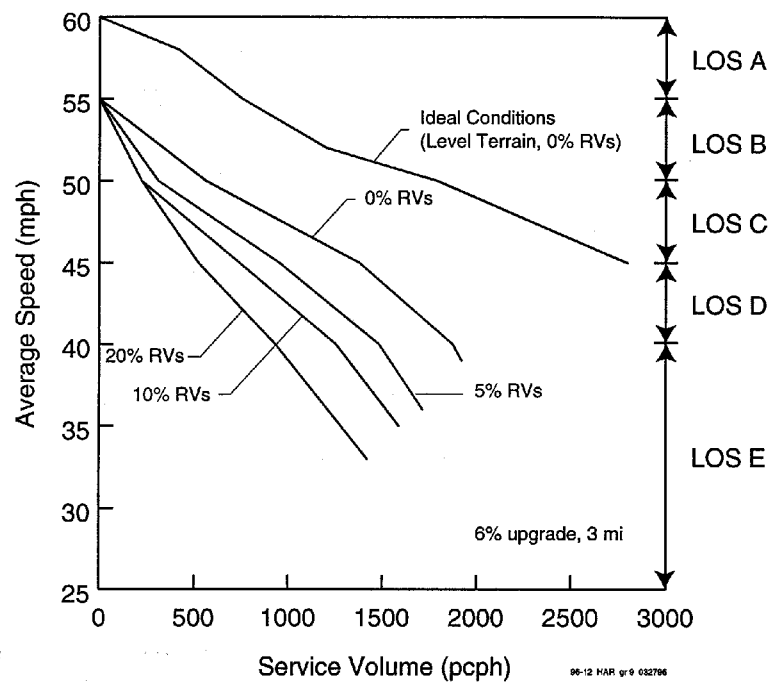


Figure 25. Speed-Flow Relationships Provided by the Specific Grade Procedure for RVs on a 4.8-km (3-mi), 6 Percent Upgrade

ASSESSMENTS OF HCM CHAPTER 8

This section summarizes a number of published and unpublished assessments of the strengths and weaknesses of the current HCM Chapter 8, including sources that recommend specific modifications or improvements to the chapter. Assessments by both researchers and practitioners are included. Each source is discussed in more detail in Appendix A.

2.3.1 HCQS Two-Lane Roads Subcommittee Assessment

The most complete assessment of the strengths and weaknesses of HCM Chapter 8 was performed in the early 1990's by the HCQS Two-Lane Roads Subcommittee. The subcommittee went through a major effort to assess the existing HCM Chapter 8 by gathering published and unpublished research, user comments and complaints, and the viewpoints of subcommittee members and other researchers. This resulted in the preparation of a report entitled "Research Needs for Capacity and Level of Service Analysis,"(10) which reviews the strengths and weaknesses of the chapter and identifies the research needed leading to a revised chapter. Table 8 summarizes the priorities for future research recommended in that report. The high-priority issues in Table 8 are obvious candidates for attention in this research.

2.3.2 Other Assessments

Many other sources were reviewed in the process of assessing HCM Chapter 8. The key comments of researchers and practitioners are summarized below.

Level of Service Definitions and Boundaries

Both McLean(11) and Hoban(12) noted that the service flow rates determined by the general terrain segment procedure in HCM Chapter 8 are lower than the service flow rates from the comparable procedure in the 1965 HCM. Thus, for any given flow rate, the level of service estimated by the chapter is lower than in previous procedures.

Guell and Virkler(13) were concerned that, even under the best roadway and traffic conditions, any two-way flow rate greater than 43 percent of the capacity falls into LOS D or E. This suggests that the boundaries LOS D and E are too broad to provide definitive information about the flows within these levels.

Table 8. Priorities for Future Two-Lane Highway Research Recommended by the HCQS Two-Lane Roads Subcommittee(10)

Issue No.	Description of Issue	Priority Rating		
		Average	Minimum	Maximum
HIGH-PRIORITY ISSUES				
24	Agreement between general terrain segment and specific grade procedures	8.6	3	10
27	Design and operational treatments (e.g., passing lanes)	8.1	5	10
3	LOS criterion -- percent time delay	7.8	3	10
11	Overestimation of terrain effects	7.8	3	10
5	LOS boundaries and descriptions	7.6	3	10
2	Length of segments for operational analysis	7.4	5	10
18	Specific grades -- LOS criterion	6.9	1	10
15	Separate LOS for each direction of travel	6.7	4	10
12	Lack of a design speed effect	6.6	2	10
25	LOS determination for specific downgrades	6.6	3	10
MEDIUM-PRIORITY ISSUES				
9	Lane width and lateral clearance adjustments	6.4	1	10
4	Percent time delay vs flow rate relationship	6.2	2	9
20	PCEs for specific grades	6.1	2	9
16	Effect of access points and roadside development	6.1	4	10
7	Effect of directional split	5.9	2	10
10	Heavy vehicle PCEs -- general terrain segments	5.8	2	9
22	Site-to-site variations in truck population	5.6	4	9
17	Effect of non-motorized traffic	5.4	2	9
14	Estimation of actual travel speeds	5.3	1	10
23	Grade adjustment factor -- PCEs of passenger cars	5.1	2	7
LOW-PRIORITY ISSUES				
22	System planning procedure	4.4	1	10
19	Specific grades: speed-flow curve at capacity	4.3	2	8
6	Basic service flow rate relationship (Eq. 8-1)	4.0	1	10
8	Use of peak hour factor	3.6	1	10
21	Method of combining truck, bus, and RV effects	3.3	1	6
13	Variations in driver desired speeds	2.4	1	5
1	Definition of ideal conditions	1.6	1	3

Priority Ratings (1 to 10 scale):

- 1 = Current chapter is adequate
10 = Revisions to chapter are essential

MRI-AED\4215-08.wpd

Service Measures

Hoban(12) noted that the use of traffic platooning as the service measure for two-lane roads was a major improvement in the 1985 HCM. However, Hoban stated that he preferred the Australian term “time spent following,” rather than “percent time delay,” as more clearly communicating the nature of the measure.

Krumins(14) expressed a concern based on extensive field data collected in Canada that the percent time delay values predicted with the HCM Chapter 8 procedure were generally higher than those observed in the field.

Both Guell and Virkler(13) and Johnson(15) expressed dissatisfaction with the 5.0-s headway criterion recommended in HCM Chapter 8 for use in estimating percent time delay from spot platooning data in the field. Guell and Virkler suggested that a headway criterion of 4.0 s would provide more useful LOS categories. Johnson stated that the 5.0-s value has been replaced with 2.5 to 3.5 s in work for California counties.

Morrall and Werner(16) suggested the possible use of the overtaking or passing ratio as a service measure for two-lane highways. This ratio is defined as the actual number of passing maneuvers to the desired number of overtaking or passing maneuvers (or the total number of passing maneuvers possible on a two-lane highways with continuous passing lanes and with vertical and horizontal geometry similar to the two-lane highway in question).

DeArazoza and McLeod(17) expressed their concern that the existing HCM Chapter 8 procedures were not appropriate for application to developed, tourist-oriented sections of US Route 1 in the Florida Keys. DeArazoza and McLeod recommended speed as a more appropriate service measure for a roadway of this type. They proposed also proposed a specific LOS boundary structure in which the LOS was tied to the posted speed limit of the roadway section being analyzed.

Botha, et al.(18,19) examined alternative service measures for two-lane highways with lower design speeds. No specific recommendation was made, but Botha appeared most intrigued by the combination of density and percent time delay to define the levels of service.

Adjustment Factors

Hoban(12) commented that the volume/capacity ratios in HCM Table 8-1 appear to have been retained only for nostalgic reasons, since they are based on the ideal capacity of 2,800 pc/h rather than the actual computed capacity of the roadway. Furthermore, Hoban noted that the first thing that one does with one of the volume/capacity ratios in HCM Chapter 8 is to multiply it by 2,800 and convert it back to a service flow rate. In other

words, Hoban's argument is that it would be simpler and more logical to tabulate the service flow rates themselves, rather than expressing them as volume/capacity ratios.

Cassidy and May(20,21) cautioned that the use of percent no-passing zones to represent the availability of available sight distance on the roadway did not consider the actual lengths of individual passing and no-passing zones.

Archilla(22) questions the directional split factors used in HCM Chapter 8. The directional split factor appears to be overestimated, and may even be in the opposite direction to that shown in the HCM. In particular, Archilla found that, for space mean speed, traffic delay rate, and percent time delay, the TWOPAS model indicates that uneven directional splits produced slightly better operational performance than 50/50 splits. By contrast, the HCM methodology considers the 50/50 split to be ideal conditions and to have the best operational performance.

Both Krumins(14) and Archilla(22) concluded from research that the passenger car equivalents (PCEs) of heavy vehicles, and therefore the adjustment factors for heavy vehicles, in the current HCM Chapter 8 procedures are too high. Krumins reached this conclusion from field data and Archilla from computer simulation studies with the TWOPAS model. Archilla also concluded that the operational effects of heavy vehicles should vary with flow rate.

McLean(11) noted that there had been little change in the values of the lane width and shoulder width factors from those first presented in the 1950 HCM. However, Hoban(12) noted that the lane width and shoulder width factors had been adjusted by Messer(5) to assume a convex instead of a concave transition with reducing road width. In other words, in the HCM Chapter 8 procedures, a small width reduction produces little reduction in service flow, but the effect becomes greater as the width is reduced further. While there is no firm supporting evidence from capacity or traffic operations research for either set of factors, Hoban noted that the convex form is consistent with the results of safety studies.

Research by Crowley, et al.(23) had the objective of collecting field data to revise the lane width and shoulder width factors in HCM Table 8-5. However, the study encountered difficulties in finding suitable sites to fit their experimental design and the research was abandoned without proposing any replacement for HCM Table 8-5.

Archilla(22) expressed an overall concern that HCM Chapter 8 treats the various adjustment factors as having independent effects when, in fact, they do not.

Specific Grade Procedure

McLean(11) found that, for certain situations, the 1985 HCM predicts unrealistically high capacities and service volumes on specific grades. In rolling or mountainous terrain, McLean found that higher service volumes would be indicated for most highways if they

were analyzed as a sequence of separate grades instead of a general segment. This anomaly arises for the use of different service measures in segments and specific grades.

Guell and Virkler(13) noted the same problem. Given otherwise identical traffic and roadway conditions, a two-lane highway will often be categorized as having a better level of service on a specific grade than on a level or rolling terrain segment, which is opposite to what one would expect.

Botha, et al.(18,19) also noted the lack of agreement between the general terrain segment and specific grade procedures.

Cassidy and May(20,21) noted that the specific grade procedure assumes that the approach roadway is level and doesn't deal with initial or entry speeds less than 55 mph.

Concerning the grade factor (f_g) in the specific grade procedure, Archilla(22) noted that passenger cars are slowed on grades under low-volume conditions, but not under high-volume conditions. Under high-volume conditions, the slowing of passenger cars due to traffic volumes is greater than the slowing due to the grade, so there is no apparent effect of grade. Archilla suggests that factors to adjust speed be applied using a limiting or minimum values, rather than applying all factors in succession.

Archilla questions the PCEs for passenger cars and heavy vehicles used in the specific grade procedure in HCM Table 8-9. Archilla found that for a 6 percent upgrade and a 64-km/h (40-mi/h average upgrade speed, the PCEs in Table 8-9 have a shape that is concave upwards when plotted as a function of length of grade. Archilla argues that, because steep grades reduce heavy trucks to crawl speeds, which they then maintain for the remainder of the grade, that the relationship between PCE and length of grade should be convex upward and should asymptotically approach a constant value. This issue is discussed further in Appendix A.

Passing Lanes and Other Operational Improvements

Both Hoban(12) and Cassidy and May(20,21) noted that HCM Chapter 8 is limited by the lack of operational analysis procedures for design and operational treatments such as passing lanes, climbing lanes, shoulder use sections, etc.

Effect of Lower Design Speeds

Botha, et al.(18,19) found that the current HCM Chapter 8 procedures do not address two-lane roads with lower design speeds. In particular, the procedures do not address the effects of horizontal curves on traffic operation on two-lane roads.

Downgrades

Messer(5) recommended that truck performance as related to steepness and length of downgrade should be documented. This issue was explored further by Archilla and Morrall,(24) who demonstrated that, because of slow-moving vehicles (such as recreational vehicles and heavy trucks), many downgrades actually experience lower level of service than suggested by the current chapter and lower levels of service than the adjacent upgrade or adjacent general terrain segments. The treatment of grades by the chapter appears inconsistent because the chapter addresses upgrades explicitly, but does not provide comparable attention to downgrades. Archilla and Morrall recommend an approach to determining level of service on downgrades that combines the World Bank HDM III model and the TRARR model. Further development of this method, including the collection of field data for calibration purposes, is recommended.

LOS by Direction of Travel

Cassidy and May(20,21) noted that the current procedures assess level of service for both directions of travel combined. They stated that it would be desirable to assess level of service separately for each direction of travel.

Other Operational Effects

Messer(5) stated that the effect of analysis section length on perceived level of service should be identified. Messer also noted that the effects of speed variance and desired speed variance on two-lane highway operations should be determined and that further study was needed of passing practices as related to volume and vehicle performance capabilities.

HCM USER SURVEY

A survey of users of HCM Chapter 8 was conducted in the early stages of the research to determine user experience with the current chapter and user assessment of needs for improvement of the chapter. This survey addressed the views of practitioners who use HCM Chapter 8, as opposed to researchers. This section summarizes the survey results, which are presented in more detail in Appendix B. Appendix C presents the questionnaire that was used to conduct the survey.

2.4.1 Survey Recipients

The mailing list for the survey included:

- 50 state highway agencies
- 55 local highway agencies
- 93 consultants

Thus, a total of 198 survey questionnaires were mailed.

The questionnaires for state highway agencies were generally sent to the state traffic engineers. The names and addresses of the state traffic engineers were determined from the membership roster of the AASHTO Highway Subcommittee on Traffic Engineering.

Most of the local highway agency engineers and consultants on the mailing list for the questionnaires were obtained from FHWA's HCM user data base. This data base was assembled by FHWA from the responses to the survey card that was distributed with each copy of the 1994 HCM. FHWA provided the research team with the names and addresses of 240 individuals who indicated in response to the survey that they devoted at least 40 percent of their time to rural highway analysis. This sample constitutes less than 10 percent of the total HCM user data base, but includes those most interested in rural highways. The mailing list for the survey conducted as part of this research included each local highway agency or consulting engineer with a U.S. address in the sample of 240 names from the FHWA data base. In addition, the names of a number of practitioners in local highway agencies and consulting firms known to the research team to be users of HCM Chapter 8 were added to the mailing list.

2.4.2 Response Rate

Table 9 summarizes the responses to the HCM user survey received to date. A total of 102 responses were received, out of 198 questionnaires that were mailed, for an overall response rate of 52 percent. The response rate was 80 percent for state highway agencies, 47 percent for local highway agencies, and 39 percent for consultants. We are especially gratified that 40 of the 50 state highway agencies responded to the survey.

Table 9. Response Rate for HCM User Survey

Agency type	Number of questionnaires mailed	Number of responses received	Response rate (%)
State agencies	50	40	80.0
Local agencies	55	26	47.3
Consultants	<u>93</u>	<u>36</u>	<u>38.7</u>
Total	198	102	51.5

2.4.3 Use of HCM Chapter 8

The survey shows that, overall, 76 percent of the respondents use the existing version of HCM Chapter 8 for capacity and quality of service analysis of two-lane highways. The reported usage rate for HCM Chapter 8 is 90 percent in state highway agencies, 69 percent in local highway agencies, and 67 percent in consulting firms.

The specific uses of HCM Chapter 8 reported by users, in order of descending frequency, are:

- Level of service analysis (72 percent)
- Capacity analysis (69 percent)
- Operational analysis of general terrain segments (61 percent)
- Operational analysis of specific upgrades (48 percent)
- Highway systems planning (36 percent)

In addition, one local highway agency stated that they use the chapter to evaluate the impact of specific land development proposals. Several agencies noted that they do capacity and level of service analyses to justify the need for specific improvement projects.

2.4.4 Analysis Segment Lengths

The respondents to the survey were asked to identify the range of segment lengths they consider in planning applications and in operational analysis of general terrain segments. For planning applications, the data show that the analysis segment lengths considered can range from 0.2 to 80 km (0.1 to 50 mi), with an average range from 2.4 to 15 km (1.5 to 9.5 mi). For operational analysis of general terrain segments, the analysis segments can also range from 0.2 to 80 km (0.1 to 50 mi), with a more typical average range from 1.6 to 13 km (1 to 8 mi). The data show that state highway agencies generally consider longer analysis segments than local highway agencies.

2.4.5 Analysis Software

The respondents were asked to identify whether they conduct operational analyses of two-lane highways using the Highway Capacity Software (HCS) maintained by the McTrans Center or, if not, whether they use some alternative software. The responses show that approximately 83 percent of state highway agencies, 46 percent of local highway agencies, and 50 percent of consultants use the HCS software for two-lane highway

analyses. The following alternative software packages were mentioned by respondents that do not use the HCS:

- Use own in-house software that implements the Chapter 8 procedures (4 respondents)
- Use own in-house software to implement planning procedures that are not based on Chapter 8 (1 respondent)
- Use planning software obtained from state highway agency (2 respondents)
- Use computerized traffic simulation (1 respondent) (see responses to next question)

The remaining 31 respondents apparently have no software for operational analyses of two-lane highways.

2.4.6 Usage of Computerized Traffic Simulation Models

Respondents were asked whether they made use of computerized simulation models in operational analyses of two-lane highways. The responses show that approximately 13 percent of state highway agencies, 12 percent of local highway agencies, and 8 percent consultants indicated that they have used simulation models of traffic operations on two-lane highways. The models used are as follows:

- Have used the TRARR model to evaluate passing lane alternatives along a steep grade. Have also funded development of a user interface for both the TRARR and TWOPAS models (1 response)
- Have used the TRARR model (2 respondents)
- Have had two-lane highway simulation performed by a consultant [the model used was not stated] (1 response)
- Have used TRAF-NETSIM to evaluate signalization and STOP sign alternatives on two-lane arterials (6 responses)

2.4.7 Use of Chapter 8 in Economic Analyses

Only 7 respondents indicated that they have used results from Chapter 8 in economic analyses. Approximately 8 percent of state highway agencies and 11 percent of consultants have used results from Chapter 8 in economic analyses. No local highway agencies have used results from Chapter 8 in economic analyses. The primary performance measure from

Chapter 8 used in economic analyses was average speed, which was used directly by some agencies and was used indirectly by others to estimate delay. One consulting firm indicated that they compare socioeconomic factors and land use to HCM results.

Respondents were asked about improvements to Chapter 8 that would make its results more useful in economic analyses. One state highway agency responded that it would be useful if the HCM procedures could give an estimate of delay per vehicle that could be used in economic analyses. Another state highway agency responded that they would like more details on the measures of effectiveness and the impact of each on user operating costs. Still another state highway agency responded that they would like better information on the relationship of V/C ratio and intersection delay.

2.4.8 Adequacy of Specific Features of HCM Chapter 8

Respondents were asked to assess whether several specific aspects of the existing HCM Chapter 8 procedures meet their needs. The responses to this question are summarized in Table 10. The key features of the existing chapter, in descending order of their adequacy as rated by HCM users, are as follows:

- Heavy vehicle adjustment factor (95 percent)
- Peak hour factor (92 percent)
- Directional split factor (92 percent)
- Lane width and lateral clearance factors (86 percent)
- Level of service boundaries (82 percent)
- Level of service measure (percent time delay) (78 percent)
- Specific grade procedure (78 percent)
- Planning analysis procedure (64 percent)

2.4.9 User-Recommended Modifications to Procedures of HCM Chapter 8

Respondents to the survey were asked to identify ways in which they would like to see the existing procedures of HCM Chapter 8 improved. This question specifically addressed the existing features of the chapter that were rated in Chapter 8, but many of the comments received also addressed new features that should be added to the HCM. The following responses were obtained:

Table 10. Assessment by Respondents of the Adequacy of Specific Features of the HCM Chapter 8 Analysis Procedures

Agency type	Number (percentage) of respondents who stated that specific features of HCM Chapter 8 adequately meet their needs														Number of respondents that use HCM Chapter 8		
	LOS measure (percent time delay)		LOS Boundaries		Heavy vehicle adjustment factors		Lane width/ lateral clearance factors		Directional split factors		Peak hour factor		Specific grade procedure			Planning analysis procedure	
State agencies	27	(75.0)	30	(83.3)	34	(94.4)	31	(86.1)	34	(94.4)	32	(88.9)	27	(75.0)	24	(66.7)	36
Local agencies	13	(72.2)	14	(77.8)	17	(94.4)	15	(83.3)	16	(88.9)	18	(100.0)	15	(83.3)	13	(72.2)	18
Consultants	<u>21</u>	(87.5)	<u>20</u>	(83.3)	<u>23</u>	(95.8)	<u>21</u>	(87.5)	<u>22</u>	(91.7)	<u>22</u>	(91.7)	<u>19</u>	(79.2)	<u>13</u>	(54.2)	<u>24</u>
Total	61	(78.2)	64	(82.1)	74	(94.9)	67	(85.9)	72	(92.3)	72	(92.3)	61	(78.2)	50	(64.1)	78

General

- Make it simple.
- The procedures should estimate capacity and level of service by direction of travel, especially in mountainous areas.
- There should be more distinction between the end of the “stable flow” levels of service and the “unstable flow” levels of service. It is difficult to visualize a 84 km/h (52-mi/h) traffic stream on a two-lane highway as unstable.
- A more detailed discussion regarding the conditions under which the use of the Chapter 8 procedures is warranted should be included in the HCM. Particularly important is allowable segment length and allowable distances between signals. The discussion should address the appropriateness of the Chapter 8 procedures to segment length and signal spacing.
- The range covered by levels of service D and E is very wide. The LOS D/E boundary sometimes occurs at only 52 percent of capacity. The procedure can show a poor level of service at relatively low volumes.
- With the recent emphasis on congestion management systems, it has become apparent that the customers need performance measures that are easily understood. Speed or travel rate are two types that seem acceptable to the public.
- Better information is needed on design features to use in upgrading existing two-lane facilities to improve percent time delay; e.g., for a given value of percent time delay, provide suggestions for improvements that will improve percent time delay.
- Level of service needs to be tied more tightly to percent time delay. We have the capability to easily collect headway/gap data and we should be doing this. The procedure needs the flexibility to work on roads with speeds less than 50 mph.
- Level of service is a qualitative measure of operating conditions. It is supposed to be defined consistent with motorists perception of their mobility and comfort. When HCM methods are used to analyze conditions on certain roads, the analysis results indicate LOS F, but citizen advisory committees perceive these roads as perhaps LOS D, or even LOS C, in mountainous terrain.
- A more general method of two-lane analysis needs to be developed which takes into account: (1) driveway frequency (number per unit length); (2) the presence of a continuous two-way left-turn lane; (3) the presence of other left-turn lanes; and (4) the travel speed of the roadway (not the design speed; and (5) the other items listed in Question 8.

- In our experience, the HCM analysis consistently underestimates the actual capacity of rural roads. It is not clear why.
- Need more information/data in establishing factors to be used.
- Inconsistencies with software. (NOTE: The respondent did not state what the specific inconsistencies were and what specific software these inconsistencies were found in.)
- Percent time delay is vague. “Drivers are delayed up to 45 percent of the time.” How long are they delayed? 1 minute? 10 minutes?
- Need more options for improvements and how they will work with future flows.
- Need better modeling of delay (state-of-the-art methodology).
- Ability to maintain design or “expected” (not desired) speed should be the criterion. The first step probably needs to separate higher design facilities (i.e., state highways) from lower design facilities (i.e., county collectors). Length of trip and terrain also heavily influence drivers' expectations.

Lane Width and Lateral Clearance Factors

- Roadway widths narrower than 5.5 m (18 ft) should be addressed by Table 8-5. We occasionally need to evaluate low-volume roads that are only 4.3 m (14 ft) wide.
- Table 8-5 should address lanes wider than 3.6 m (12 ft).
- Many county roadway widths are less than 5.5 m (18 ft); i.e., in range of 4.5 to 4.9 m (15 to 16 ft).
- Decrease the amount of impact that shoulder widths have on capacity. Decreasing capacity by 30 percent because a roadway has a shoulder width of less than 0.6 m (2 ft) seems extreme.

Heavy Vehicle Effects

- Users should be able to input into the method any power-to-weight ratio for design vehicles that we know we have in the field. Users should be able to input the speed of the truck if already measured.

- On two-lane facilities, the truck factor does not represent the actual delay and impact to capacity when no passing is allowed or oncoming volumes are high.

Peak Hour Factor

- Peak hour factors don't account for heavy but short duration peaks.

Specific Grade Procedure

- The levels of service that are provided by the specific grade methodology seem not to follow the levels of service that are expected to result from traffic operations as defined by the HCM for both rolling and mountainous terrain.
- Users need the ability to vary entering speeds of specific grades. Charts and software should go to speeds greater than 88 km/h (55 mi/h).

Passing and Climbing Lanes

- A better way to evaluate the effect of passing lanes is needed.
- The procedures of Chapter 8 should address climbing lanes.
- There may be a need to develop an “aggravation” factor. An example of it could be, say, commuter traffic on a roadway with many crossroads and entrances. There, the “aggravation” factor will be high, as opposed to high-volume commuter traffic on a roadway with few crossroads and entrances where the “aggravation” factor may be low. Also, a long stretch of roadway with narrow shoulders will have a high “aggravation” factor, and two-lane highways with wide shoulders and available passing lanes may have a low “aggravation” factor.

Effect of Roadside Development and Turning Maneuvers

- The procedures of Chapter 8 should address two-way left-turn lanes.
- There is a need to develop a consistent level of service measure to address situations where a rural two-lane road passes through “village” areas where posted speeds are less than those considered in the current methodology. In many cases, these areas cannot be considered urban or suburban and, thus, there is not an appropriate method to assess level of service.

- The procedure should address levels of service for roads with design speeds down to 40 km/h (25 mi/h). In projects for county agencies, we have had to extrapolate/invent our own methods to determine LOS for low design speed roads with steep grades and high truck volumes.
- Procedures should include a higher range of speeds for rural highways (48 to 105 km/h or 30 to 65 mi/h) and the effects of access conditions.
- There is a need to determine the impact of a center two-way left-turn lane on existing highway.

Role of Design Speed and Speed Limits

- HCM should consider facilities with lower design speeds (56 to 72 km/h or 35 to 45 mi/h). Need alternate accurate methods in determining percent time delay.
- Our agency has many miles of foothill or mountain roadways with free-flow speeds of 40 to 72 km/h (25 to 45 mi/h). The HCM does not work in these situations.
- Need adjustments for lower speed limits (i.e., 72-km/h or 45-mi/h segments).
- Make analysis responsive to roadways with posted speed limits less than 88 km/h (55 mi/h).

Downgrade Analysis

- There is no procedure for significant downhill sections of a roadway.

Planning Procedure

- We don't use the planning procedure because it is too vague.

Other Comments

- Add a factor for driver type (recreation/tourism versus commuter traffic or through traffic versus local traffic).
- Chapter should include effects of pedestrians, bicycle lanes, and scenic routes.
- Add factors for pedestrian/bicycle traffic.

2.4.10 User-Initiated Modifications or Enhancements to HCM

Users were asked whether they had made modifications or enhancements to the HCM Chapter 8 procedures for their own use. Eight respondents indicated that they had made formal or informal modifications to the Chapter 8 procedures. The modifications are as follows:

- For rural two-way left-turn lanes, use 100 percent no-passing and exclude the left-turn volumes from the determination of level of service. For truck climbing lanes, use HCM Chapter 7 with a typical directional split of 60 percent of DHV and assume that the maximum service flow rates are decreased by 15 percent to represent reduced efficiency.
- The upper limit of LOS D has been changed from 75 percent to 85 percent.
- For truck climbing lanes, put all trucks in the climbing lane and then analyze the two remaining lanes (i.e., one in each direction of travel) using the procedures for level terrain with no trucks.
- Use a 3.5-s headway instead of a 5.0-s headway based on procedures developed by the local district office of the state highway agency.
- Use a 2.5- or 3.5-s headway, rather than 5.0-s headway.
- Use agency-developed capacity table for long range planning analysis to replace HCM Table 8-10.
- Use simplified procedure developed by a consulting firm.

2.4.11 Priorities for Improvements to Chapter 8

Table 11 summarizes the user responses concerning the assessment of priorities for improvement of HCM Chapter 8. The table shows the number and percentage of respondents who favor adding specific features to Chapter 8. Table 12 presents these features presented in order of descending priority, based on the number of respondents who favor adding each specified feature to the chapter. Users were also asked to assign a priority rating for those features they favored on a scale from 1 (not very important) to 10 (very important). Table 13 presents the same features as Table 12, presented in descending order of the average priority ratings assigned by those respondents who favor adding the feature to Chapter 8.

As identified in the “Other” column in Table 11, six respondents suggested specific improvements to Chapter 8 other than those listed explicitly on the questionnaire. These suggestions, and the accompanying comments provided by the respondent, have been

included with the respondent comments on needs for improvement of Chapter 8 that are presented above.

Service Measures

Respondents were asked to assess seven alternative service measures for two-lane highways by selecting their first, second, and third choices. The respondents were asked to choose the most appropriate measures of quality of service, as perceived by the motorist. An opportunity was also provided for respondents to suggest other candidate service measures as a first, second, or third choice.

Table 14 provides the HCM user ratings of alternative service measures based on the respondent's first choice selections. The table shows that the combination of average travel speed and percent time delay and the delay rate were chosen as first choice by the highest number of users. These were followed closely by average travel speed and percent time delay. Only five respondents picked density as a first choice and only two respondents picked the overtaking or passing ratio.

Table 15 is analogous to Table 14, but considers the first, second, a third choice ratings by respondents, rather than just the first choice ratings. The ratings in Table 15 are based on a weighting system with three points assigned for a first-choice rating, two points assigned for a second-choice rating, and one point assigned for a third-choice rating. The ranking of alternative service measures by this weighting system was very similar to Table 14. The highest rated service measure in Table 15 is the combination of percent time delay and average travel speed followed by (in descending order) delay rate, average travel speed, and percent time delay.

2.4.12 Consideration of Design Speed

Respondents were asked to indicate whether the design speed of a two-lane highway should be an explicit factor in the procedure for determining its level of service. The results, presented in Table 16, indicate that 55 percent of respondents would like design speed to be an explicit factor in determining level of service and 45 percent of respondents do not want design speed as an explicit factor. Generally, a higher percentage of consultants than highway agency engineers favored explicit consideration of design speed.

Table 11. Assessment by Respondents of Features that Should be Added to the HCM for Analysis of Two-Lane Highways

Agency type	Number (percentage) of respondents who favor addition of specific features to HCM Chapter 8											Number of agencies responding
	Passing and climbing lanes	Lower design speed curves	Unsignalized intersections	Signalized intersections	Driveways and roadside development	Two-way left-turn lanes	Slow-vehicle turnouts	Wide cross section/shoulder driving	Steep down-grades	Upgrading from two to four lanes	Other	
State agencies	36 (90.0)	27 (67.5)	16 (40.0)	15 (37.5)	28 (70.0)	30 (75.0)	12 (30.0)	9 (22.5)	14 (35.0)	21 (52.5)	3 (7.5)	40
Local agencies	14 (53.8)	10 (38.5)	10 (38.5)	8 (30.8)	17 (65.4)	15 (57.7)	6 (23.1)	3 (11.5)	3 (11.5)	10 (38.5)	3 (11.5)	26
Consultants	<u>19</u> (52.8)	<u>21</u> (58.3)	<u>20</u> (55.6)	<u>12</u> (33.3)	<u>20</u> (55.6)	<u>18</u> (50.0)	<u>11</u> (30.6)	<u>3</u> (8.3)	<u>9</u> (25.0)	<u>17</u> (47.2)	<u>0</u> (0.0)	<u>36</u>
Total	69 (67.6)	58 (56.9)	46 (45.1)	35 (34.3)	65 (63.7)	63 (61.8)	29 (28.4)	15 (14.7)	26 (25.5)	48 (47.1)	6 (5.9)	102

Table 12. Potential HCM Enhancements Ranked by Percentage of Respondents Who Favor Them

Potential enhancement	Respondents favoring	
	Number	(Percentage)
Passing and climbing lanes	69	67.6
Driveways and roadside development	65	63.7
Two-way left-turn lanes	63	61.8
Horizontal curves with lower design speeds	58	56.9
Upgrading from two to four lanes	48	47.1
Unsignalized intersections	46	45.1
Signalized intersections	35	34.3
Slow-vehicle turnouts	29	28.4
Steep downgrades	26	25.5
Wide cross section to encourage shoulder driving	15	14.7
Other	6	5.9

Table 13. Potential HCM Enhancement Ranked by Priority Assigned by Respondents Who Favor Them

Potential enhancement	Priority rank ^a
Two-way left-turn lanes	6.8
Passing and climbing lanes	6.7
Driveways and roadside development	6.7
Unsignalized intersections	6.5
Signalized intersections	6.4
Horizontal curves with lower design speeds	6.3
Upgrading from two to four lanes	6.2
Steep downgrades	5.2
Slow-vehicle turnouts	5.0
Wide cross section to encourage shoulder driving	4.0

^a On a scale from 1 (not very important) to 10 (very important).

Table 14. Assessment by Respondents of Alternative Level of Service Measures for Two-lane Highways (Based on First Choice Selections)

Level of service measure	Number (percentage) of respondents selecting the specified measure as first choice							
	State agencies		Local agencies		Consultants		Combined	
Percent time delay	9.0	(23.1)	3.5 ^a	(13.5)	5.0	(17.2)	17.5	(18.6)
Average travel speed	9.0	(23.1)	6.0	(23.1)	5.5	(19.0)	20.5	(21.8)
Combination of percent time delay and average travel speed	10.0	(25.6)	7.5	(28.8)	8.0	(27.6)	25.5	(27.1)
Density	1.0	(2.6)	2.0	(7.7)	1.5	(5.2)	4.5	(4.8)
Passing ratio	1.0	(2.6)	0.0	(0.0)	1.0	(3.4)	2.0	(2.1)
Delay rate	9.0	(23.1)	7.0	(26.9)	8.0	(27.6)	24.0	(25.5)
Other	<u>0.0</u>	(0.0)	<u>0.0</u>	(0.0)	<u>0.0</u>	(0.0)	<u>0.0</u>	(0.0)
Total	39		26		29		94	

^a Fractional values results from ties in first choice ratings.

Table 15. Respondent Assessment of Alternative Level of Service Measures for Two-lane Highways (Based on Weighing of First, Second, and Third Choices)

Level of service measure	Weighted rating for specified level of service measures ^a			
	State agencies	Local agencies	Consultants	Combined
Percent time delay	56.0	23.5 ^b	29.3	108.8
Average travel speed	49.0	32.5	29.5	111.0
Combination of percent time delay and average travel speed	46.0	38.0	52.0	136.0
Density	16.5	20.0	18.8	55.3
Passing ratio	15.5	7.5	4.0	27.0
Delay rate	48.0	28.5	36.3	112.8
Other	0.0	1.0	1.0	2.0

^a Weighted ratings are based on three points for each first choice selection, two points for each second choice selection, and one point for each third choice selection.

^b Fractional values result from ties in respondent's first, second, or third choice selections.

Table 16. Assessment by Respondents of the Need for Design Speed as an Explicit Factor in Determine Level of Service of Two-Lane Highways

Agency type	Number (percentage) of respondents who recommend design speed as an explicit factor				
	Yes		No		Total
State agencies	14	(45.2)	17	(54.8)	31
Local agencies	13	(54.2)	11	(45.8)	24
Consultants	<u>18</u>	(66.7)	<u>9</u>	(33.3)	<u>27</u>
Total	45	(54.9)	37	(45.1)	82

To explore HCM user's viewpoints on this issue further, respondents were asked if, for example, a winding, rural two-lane highway contains an extended series of horizontal curves with lower design speeds (e.g., 48 to 56 km/h or 30 to 35 mph) and long no-passing zones, but has relatively low traffic volumes, should have its quality of service classified as:

- A high level of service (e.g., LOS A or B) because there is little operational congestion.
- A low level of service (e.g., LOS D or E) because drivers are forced to travel at much lower speeds than they desire and are unable to pass other vehicles due to long no-passing zones.

The results, presented in Table 17, indicate that 56 percent of respondents would prefer a high level of service for this situation and 44 percent of respondents would prefer a low level of service. This creates a difficult decision about how to deal with design speed in a future HCM chapter because users are so evenly split on this issue.

Table 17. Assessment by Respondents of the Appropriate Level of Service for Two-Lane Highways With Lower Design Speeds But Relatively Low Traffic Volumes

Agency type	Number (percentage) of respondents that consider that a winding, rural two-lane highway with horizontal curves of lower design speed but relatively low traffic volumes should be assigned:			
	High LOS		Low LOS	
State agencies	21	(58.3)	15	(41.7)
Local agencies	17	(65.4)	9	(34.6)
Consultants	<u>11</u>	(42.3)	<u>15</u>	(57.7)
Total	49	(55.7)	39	(44.3)

A number of the responses appeared at first glance to be internally inconsistent because users stated that design speed should be an explicit factor in determining level of service but that the low-volume winding road described above should be assigned a high level of service. Based on follow-up contact with selected respondents, these users apparently do not want lower speed highways assigned a low level of service, but want level of service judged by a comparison between average travel speed and design or “expected” speed.

2.4.13 Canadian Survey

A survey of Canadian practitioners was conducted by Werner and Morrall(25) in 1994 to determine the procedures used in Canada for operational and planning analyses of two-lane highways and to obtain a user assessment of the HCM Chapter 8 procedures. The survey questionnaire used for the HCM user survey in the United States (see Appendix C) was, to some extent, based on the experience gained in this Canadian survey.

The Canadian survey was mailed to 73 individual practitioners within government, universities, and consulting firms. A total of 15 responses were received, which represented the views of 23 of the 73 individuals on the mailing list (i.e., in some cases, multiple recipients of the questionnaire within the same agency decided to combine their responses). Thus, the overall response rate to the questionnaire was 32 percent.

All of the respondents indicated that they use HCM Chapter 8 for two-lane highway operational analyses. All but two respondents indicated that they use the HCS software. Seven respondents indicated that they had used computerized simulation models of two-lane highway traffic; six of these respondents had used TRARR and one had used TWOPAS.

With respect to the appropriate service measure for two-lane highways, most respondents to the Canadian survey stated that percent time delay was a good measure, but some thought that other such as net passing opportunities should be considered. One respondent noted that the 5-s criterion for delay is too long and suggested 3.5 or 4 s. Most respondents stated that average travel speed and V/C ratio were not appropriate service measures. Other service measures that were mentioned included net passing opportunities, overtaking supply and demand, overtaking ratio, frequency of delay, and average queue length. One respondent stated that the service measure should be based on actual driver behavior and expectations.

Table 18 summarizes the issues identified by the Canadian survey classified into four priority rankings: high priority issues, medium priority issues, low priority issues, and other issues.

Table 18. Priorities Recommended for Specific Issues in the Canadian Survey(25)

HIGH PRIORITY ISSUES

1. Need for a procedure to determine the impact of passing and climbing lanes on level of service.
2. Need to link procedures for determining LOS on general terrain segments and specific upgrades.
3. Criteria to determine upgrading from two to four lanes. What is the right volume to twin?
4. Guidelines for the determination of the design hourly volume.
5. Impact of access points and roadside development on capacity and LOS.
6. Need for a set of metric procedures (but not necessarily upgrade Chapter G).
7. Criteria for length of highway segments for LOS analysis.
8. Need for a simplified procedure for determining LOS.
9. Need for a detailed procedure including the use of simulation to determine LOS for a range of conditions.
10. Need for a procedure on how to classify terrain.

MEDIUM PRIORITY ISSUES

1. Need for a LOS analysis procedure that can be integrated with an economic analysis of highway projects.
2. In conjunction with #1, a need for a procedure to prioritize passing lane and climbing lane projects.
3. Evaluation of other measures of effectiveness for two-lane performance including net passing opportunities, net passing supply, overtaking ratio, in addition to % time delay.
4. Review of current LOS boundaries with special attention to the large LOS C/D/E boundaries.
5. Determination of the effect of the distribution of passing and no passing zones on LOS.
6. Incorporation of a design speed effect into LOS procedures.
7. Need to develop techniques for rural highway intersection analysis.
8. Procedures to determine LOS for each direction of travel especially on specific upgrades and downgrades and for level and rolling terrain subject to heavy directional flows.
9. Review of HV, PCE's for general terrain segments.
10. Procedure to estimate actual travel speeds.

Table 18 (Continued)

LOW PRIORITY ISSUES

1. Procedure to determine LOS on specific downgrades.
2. Impact of wide paved shoulder on LOS.
3. Procedures for turnouts.
4. Lane width and lateral clearance adjustments.
5. Peak hour factor adjustments.
6. Review of ideal conditions definition.

OTHER ISSUES

1. No consensus on whether to delete Chapter G or upgrade it. Respondents would like to have access to LOS metric procedures.
2. Respondents would like to see the formation of a LOS users Group.
3. Respondents would like feedback on this survey.

CHAPTER 3.

TRAFFIC OPERATIONAL FIELD DATA

A key activity in the research was the collection of traffic operational field data for two-lane highways. These data were used in improving, calibrating, and validating the TWOPAS simulation model and in developing revised operational analysis procedures for the two-lane highway chapter of the HCM 2000.

The following discussion identifies the types of field data that were collected, the data collection methods, the data collection locations and amounts of field data collected.

TYPES OF FIELD DATA COLLECTED

Traffic operational field data were collected for the study to represent traffic operational conditions at several types of sites that were of potential interest in making simulation model improvements and developing improved HCM procedures. The types of sites that were selected for field evaluation included:

- Sites with high traffic volumes
- Sites in both level and rolling terrain
- Sites with high truck volumes on extended steep upgrades
- Sites on steep downgrades with auxiliary lanes
- Sites with varying levels of access point density with and without center two-way left-turn lanes (TWLTL)
- Sites upstream and downstream of shoulder width transitions
- Sites upstream and downstream of lane width transitions

Field data for the study were collected at a total of 20 field sites in four states (California, Florida, Missouri, and Oregon) and one Canadian province (British Columbia). These sites adequately covered all of the desired site types identified above, with the following exceptions.

First, sites with high traffic volumes were sought to characterize the relationships of speed and percent following to flow rate at higher flow rates and, if possible, to observe traffic operational conditions at or approaching capacity. The former goal was met, but two-lane highway sites with traffic volumes at or approaching capacity were difficult to

find. The research team was successful in finding and collecting traffic operational field data at four sites with peak two-way flow rates over 2,000 veh/h, including one site with peak flow rates over 3,000 veh/h. Additional high-volume sites were sought, but the volumes of reported high-volume sites were generally found to be under 2,000 veh/h for both directions of travel combined. It is apparent that most two-lane highways are widened to four lanes before traffic volumes approach capacity. The use of the traffic operational data from high-volume sites in establishing estimated capacity volumes and fundamental relationships of speed and percent following to capacity is discussed in Chapter 5 of this report.

Second, the research team was successful in finding four pairs of sites upstream and downstream of shoulder width transitions that were used to investigate the effect of shoulder width on free-flow speed. However, no comparable sites upstream and downstream of lane width transitions were found. As a result, the effect of shoulder width on free-flow speed incorporated in the TWOPAS model and the revised operational analysis procedure is based on field data, while the effect of lane width is based on assumed values from the 1985 HCM procedures. The analysis of the shoulder width data is discussed in Chapter 5 of this report.

The traffic characteristics, vehicle characteristics, and traffic operational performance measures that were determined directly or indirectly in the field studies include:

- Flow rates
- Vehicle mix
- Speeds and travel times
- Headways
- Percent time delay
- Truck weight/power ratio

Both macroscopic (section-based) and microscopic (spot) speed data were collected. The field data collection methods are described below.

FIELD DATA COLLECTION METHODS

The techniques that were used in the field data collection include:

- Automated traffic recorders to count volumes and collect data on the speeds, headways, and lengths of individual vehicles.

- Laser speed guns to determine spot speeds and speed profiles of vehicles.
- Video recorders to film traffic events for later review and data reduction in the office.
- Manual observers to count volumes or events with manual counting boards and record traffic events on clipboards

The automated traffic recorders used in field data collection were Unicorn traffic classifiers manufactured by Diamond Traffic Products. Vehicle axle passages to determine speeds, headways, and vehicle lengths were detected with piezoelectric cables taped to the pavement surface. The laser speed guns use in the field were Kustom ProLaser units. The video recorders were Panasonic recorders using 13-mm (0.5-in) VHS cassettes.

Data collection for extended two-lane highway sites was performed using equipment deployed as shown in Figure 26. To investigate a site 0.8 km (0.5 mi) or more in length, traffic sensors were placed on the roadway and connected to automated traffic recorders in both directions of travel at each end of the section. The automated traffic recorders provided data on each vehicle passing over the sensors including:

- Arrival time
- Headway
- Spot speed
- Vehicle length
- Vehicle type (based on vehicle length)

These data were retained in the memory of the automated traffic recorder and subsequently transferred to a laptop computer for analysis. At selected locations, an additional pair of sensors and automated traffic recorders was deployed within the roadway section of interest.

Video cameras were placed on the roadside at each end of the roadway section near the automated traffic recorders to allow determination of vehicle travel times through the section and computation of section-averaged speed. The video cameras filmed the traffic stream in a particular direction of travel of interest. The cameras included a character generation that superimposed elapsed time on the recorded videotape. A hand-held stop watch was filmed with the video cameras at each end of the section, so that the internal elapsed times of the two cameras could be placed on a common time scale.

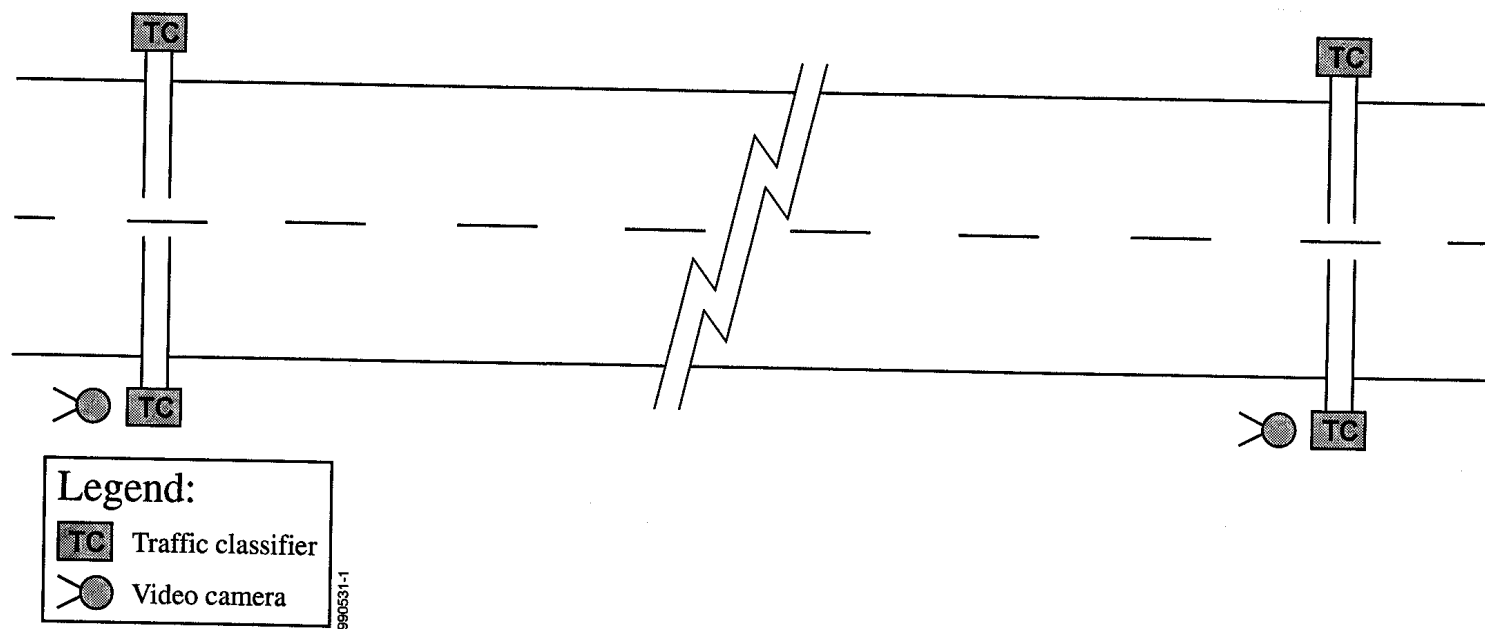


Figure 26. Typical Equipment Setup for Two-Lane Highway Data Collections

Video recordings to obtain vehicle travel times were made at nearly all sites. However, because of the laborious nature of the data reduction required, while travel times were determined only for selected periods of interest.

At four specific locations, two in California and two in Missouri, the extended roadway section like that shown in Figure 26 included a shoulder width transition. Automated traffic recorders were deployed upstream and downstream of the shoulder width transition to obtain data on the effect of shoulder width on free-flow speeds of vehicles. At two locations, only one direction of travel was studied. However, at the other two locations, both directions of travel were studied so that the effect on speed of both types of shoulder width transitions (narrower to wider and wider to narrower) could be evaluated.

At one site in California, data on truck crawl speeds on a steep grade were collected using a laser speed gun. These speed data were used to estimate the weight/power ratios of the trucks.

DATA COLLECTION LOCATIONS AND AMOUNT OF FIELD DATA COLLECTED

Table 19 summarizes the locations at which field data were collected and the amount of data collected. The table includes a total of 20 data collection sites including:

- 7 sites in California
- 4 sites in Florida
- 6 sites in Missouri
- 2 sites in Oregon
- 1 site in British Columbia

Data were collected for a total of 540.3 hours using automated traffic recorders. Video data were collected for a total of 60.4 hours, or 11 percent of the total study period. All of the field data were collected during the period from September 1996 to September 1997.

The table shows the highest 15-min two-way and directional flow rates measured at the field sites. The highest two-way flow rate measured was 3,350 veh/h and the highest directional flow rate measured was 1,920 veh/h.

Table 19. Summary of Field Data Collection Activities

State/ Province	Site Number	County	Route	Location	Length (mi)	Date(s)	Number of Hours of Data Collected		Highest 15-min flow rate (veh/h)		Comments
							Automated	Video	Directional	Two-Way	
British Columbia	BC01	--	TCH	downgrade from Great Divide	2.0	9/3/96	2.0	2.0	--	--	Downgrade site with added passing lane
Oregon	OR01*	Polk	SR 18	near Grand Ronde (MP 17.92 to 24.30)	6.4	9/19/96 - 9/23/96	97.0	**	1,470	2,300	High-volume site
Oregon	OR02*	Polk	SR 18	Sheridan to McMinnville (MP 32.91 to 45.76)	12.9	9/19/96 - 9/23/96	97.0	**	1,020	1,440	High-volume site
California	CA01A	San Joaquin	SR 12	between SR 160 and I-5 (MP 5.54 to 9.14)	3.6	11/12/96	4.0	1.1	460	--	Shoulder width comparison site Westbound: 8-ft shoulders; 12-ft lanes
California	CA01B	San Joaquin	SR 12	between SR 160 and I-5 (MP 0.84 to 3.94)	3.1	11/12/96	4.0	1.3	460	--	Shoulder width comparison site Westbound: 4-ft shoulders; 12-ft lanes
California	CA02	Contra Costa	SR 4	between I-80 and Cummings Skyway (MP 1.75 to 3.00)	1.3	11/13/96-11/14/96	12.0	6.0	1,730	3,110	High-volume site
California	CA03A	Contra Costa	SR 4	near Discovery Bay (MP 46.71 to 47.31)	0.6	6/2/97-6/6/97	94.8	12.0	1,920	3,350	
California	CA03A	Contra Costa	SR 4	near Discovery Bay (MP 47.41 to 47.81)	0.4	11/18/96	7.0	2.0	310	--	Shoulder width comparison site Eastbound: 8-ft shoulders; 12.5-ft lanes
California	CA04	Monterey	SR 1	Castroville to Watsonville (MP 93.65 to 95.10)	1.5	11/20/96 - 11/21/96	24.0	4.0	1,110	2,160	Shoulder width comparison site Eastbound; 2-ft shoulders; 11.5-ft lanes
Florida	FL01	Monroe	US 1	mainland north of Key Largo (MP 108.0 to 112.5)	4.5	3/18/97 - 3/19/97	27.8	8.0	750	1,570	High-volume site
Florida	FL02	Monroe	US 1	Big Coppit Key (MP 10.0 to 10.6)	0.6	3/20/97	7.0	4.0	1,150	1,900	Conventional two-lane highway with turning traffic
Florida	FL03	Monroe	US 1	Saddlebunch Key (MP 11.5 to 14.7)	3.2	3/24/97 - 3/25/97	28.0	6.0	1,004	1,580	High-volume site
Florida	FL04	Monroe	US 1	Plantation Key (MP 87.8 to 89.2)	1.4	3/26/97	7.0	4.0	1,090	2,080	Three-lane section with two-way left-turn lane
Missouri	MO01A	Newton	SR 43	0.85 mi south of Kapok Rd to 1.95 mi south of Kapok Rd	1.1	5/20/97	22.0	--	440	760	Shoulder width comparison site 11.5-ft lanes, 0.6-ft shoulders
Missouri	MO01B	Newton	SR 43	1.95 mi south of Kapok Road to 0.4 mi north of Seneca city limit	0.5	5/20/97	22.0	--	400	610	Shoulder width comparison site 11.5-ft lanes, 0.6-ft shoulders
Missouri	MO02	McDonald	US 71	0.2 mi south of SR 76 to 0.15 mi north of Hwy W	4.0	5/21/97	21.8	4.0	520	860	Terrain comparison section: rolling terrain (see also MO03)
Missouri	MO03	McDonald	US 71	0.3 mi north of Brush Creek to 0.3 mi south of Hwy H	2.4	5/22/97	6.0	4.0	490	860	Rolling terrain Terrain comparison section: Level terrain(see also MO02)

Table 19 (Continued)

State/ Province	Site Number	County	Route	Location	Length (mi)	Date(s)	Number of Hours of Data Collected		Highest 15-min flow rate (veh/h)		Comments
							Automated	Video	Directional	Two-Way	
California	CA05	Siskiyou	US 97	4.6 mi north of Mt Hebron summit to 0.4 mi north of Mt. Hebron summit (southbound)	4.2	6/9/97-	--	--	--	--	Measurements of 260 truck speeds on grade by laser gun (12 hr of data)
Missouri	MO04A	Ray	SR 210	0.8 mi east of Hwys O/Z to 1.7 mi east of Hwys O/Z west of Richmond	0.9	6/10/97 7/24/97 - 7/25/97	18.0	--	460	830	Shoulder width comparison site 11.7-ft lanes, 10.5 to 11.5-ft paved shoulders
Missouri	MO04B	Ray	SR 210	3.3 mi west of Hwy O/Z to 1.8 mi west of Hwy O/Z west of Richmond	1.5	09/16/97 7/24/97 - 7/25/97	7.0 18.0	--	390	740	Shoulder width comparison site 11-ft lanes, 4.5-ft gravel shoulders
						9/16/97	7.0				
						TOTAL	540.3	60.4			

* Data collected by the Oregon DOT in cooperation with the research team.

** No video travel time data are available but a limited number of floating car runs were made.

TCH = Trans-Canada Highway

Site CA02 shown in the table was of great interest because it had higher flow rates than any other two-lane highway site evaluated during the research. This site is located in the San Francisco Bay area of California. It is fed continuously by traffic from a four-lane freeway at one end of the site and, at the other end of the site, it connects with another freeway. Thus, this two-lane highway has higher traffic demand than most two-lane highways and serves as a useful location to observe high flow rates on a two-lane highway. Sites CA02 was first studied in the field during November 1996. The research team returned to collect additional data at this site in June 1997 to verify the high flow rates during a different season of the year and to increase the available sample of data under high-volume conditions. A table of data for the highest-volume periods at site CA02 is presented in Chapter 5 of this report.

CHAPTER 4.

IMPROVEMENT OF THE TWOPAS SIMULATION MODEL

This chapter of the report describes the improvement of the TWOPAS computer simulation model for use in revision of two-lane highway operational analysis procedures. The discussion first addresses the selection of the TWOPAS model for this application, the history and background of the TWOPAS model, and the improvements made to the TWOPAS model as part of the current research.

SELECTION OF THE TWOPAS MODEL

The research approach presented in Chapter 1 of this report included identification of a suitable computer simulation model of two-lane highway operations and improvement of its capabilities to serve as a tool for making improvements to the existing two-lane highway operational analysis procedures. Two candidate models that were suitable for this application were identified and compared during the research. These models were the TWOPAS model (26, 27, 28), developed by Midwest Research Institute and others for the Federal Highway Administration, and the TRARR model (29, 30), developed by the Australian Road Research Board (now ARRB Transport Research, Ltd.).

An extensive comparison of the capabilities and features of the two models was undertaken in the early stages of the research. It was found that their capabilities and features are comparable in many respects. There were no features of either model that would have limited its use as intended in the research. The decision to select TWOPAS for use in the research was based primarily on the two factors. First, TWOPAS was developed with U.S. data and, therefore, was thought to be better representative of U.S. conditions. Second, the developers of TWOPAS were available to participate in the research team, while the developers of TRARR were overseas.

HISTORY AND BACKGROUND OF THE TWOPAS MODEL AND THE UCBRURAL INTERFACE

The TWOPAS model is a microscopic computer simulation model of traffic on two-lane highways. The predecessor of the TWOPAS model was originally developed by Midwest Research Institute (MRI) in NCHRP Project 3-19, "Grade Effects on Traffic Flow Stability and Capacity," which resulted in the publication of *NCHRP Report 185*(31) in 1978. The model was originally known as TWOWAF (for TWO WAY Flow). TWOWAF was improved by MRI in 1981 in an FHWA study entitled, "Implications of Light-Weight, Low-Powered Vehicles in the Traffic Stream." (32) Then, in 1983, Texas Transportation

Institute (TTI) and KLD and Associates(5) made further updates to TWOWAF, which resulted in the version of the model that was used in the development of Chapter 8 for the 1985 HCM. TWOWAF had the capability to simulate traffic operations on normal two-lane highways, including both passing and no-passing zones, as well as the effects of horizontal curves, grades, vertical curves and sight distance. Subsequent to the publication of the 1985 HCM, MRI developed the TWOPAS model by adding to TWOWAF the capability to simulate passing lanes, climbing lanes, and short four-lane sections on two-lane highways.(26, 27, 28) A modified version of TWOWAF known as ROADSIM was also developed and included in FHWA's TRAF model facility at about this time.

As a microscopic model, TWOPAS simulates the operation of each individual vehicle on the roadway. The operation of each vehicle as it advances along the road is influenced by the characteristics of the vehicle and its driver, by the geometrics of the roadway, and by the surrounding traffic situation. The following features are found in TWOPAS:

- Three general vehicle types—passenger cars, recreational vehicles, and trucks.
- Roadway geometrics specified by the user in input data, including horizontal curves, grades, vertical curves, sight distance, passing lanes, climbing lanes, and short four-lane sections.
- Traffic controls specified by the user, particularly passing and no-passing zones marked on the roadway.
- Entering traffic streams at each end of the simulated roadway generated in response to user-specified flow rate, traffic mix, and percent of traffic platooned.
- Variations in driver performance and preferences based on field data.
- Driver speed choices in unimpeded traffic based on user-specified distribution of driver desired speeds.
- Driver speed choices in impeded traffic based on a car-following model that simulates driver preferences for following distances (headways), based on relative leader/follower speeds, driver desired speeds, and desire to pass the leader.
- Driver decisions concerning initiating passing maneuvers in the opposing lane, continuing/aborting passing maneuvers, and returning to normal lane, based on field data.
- Driver decision concerning behavior in passing/climbing/four-lane sections, including lane choice at beginning of added lane, lane changing/passing within added lanes and at lane drops, based on field data.

- Processing of traffic and updating of vehicle speeds, accelerations, and positions at intervals of 1 second of simulated time.

TWOPAS is written in the FORTRAN language. TWOPAS was originally developed for use on a mainframe computer and the original mainframe version of TWOPAS is not very user friendly. An improved user interface is needed to increase the user friendliness of TWOPAS and to allow it to be used effectively in the PC environment. As described below, a user interface for TWOPAS was available for use in the research and that interface has been substantially improved.

In recent work for the California Department of Transportation (Caltrans), the Institute of Transportation Studies (ITS) at the University of California-Berkeley (UCB) has developed a user interface, known as UCBRURAL, for use with the TRARR and TWOPAS models. The interface provides a convenient tool for users (1) to enter input data on traffic volumes, traffic characteristics, and geometric features of two-lane roads; (2) to run either the TRARR or TWOPAS model; and (3) to display the output in a convenient graphical format. The UCBRURAL interface has not involved any substantive changes to either model; however, it provides a more convenient means for users to run either model.

The user interface was originally developed for use with the TRARR model and, thus, it was based in many respects on TRARR's capabilities. In later work for Caltrans, the capability of the interface was expended to include the TWOPAS model as well. However, the existing version of the interface did not take full advantage of the capabilities of TWOPAS.

A useful adjunct to the TWOPAS model is another FORTRAN program known as TWOSUM. TWOSUM is a postprocessing program which summarizes the very detailed TWOPAS output report in a more succinct version. The UCBRURAL interface actually uses the TWOSUM output report, rather than the more detailed TWOPAS output report, as the source of the output data it presents to users.

Figure 27 illustrates how the TWOPAS and TWOSUM programs and the UCBRURAL interface work together to provide a user-friendly environment for simulation of traffic operations on two-lane highways.

IMPROVEMENT OF THE TWOPAS MODEL AND THE UCBRURAL INTERFACE

An initial review of the TWOPAS model and the UCBRURAL interface, and later reviews as work progressed, identified a total of 78 candidate improvements to make TWOPAS and UCBRURAL more accurate and/or more functional. However, the implementation of all 78 candidate improvements would have required far more time and funds than were available. Priorities were established and 26 of the 78 candidate improvements were selected for implementation. Table 20 identifies these 26

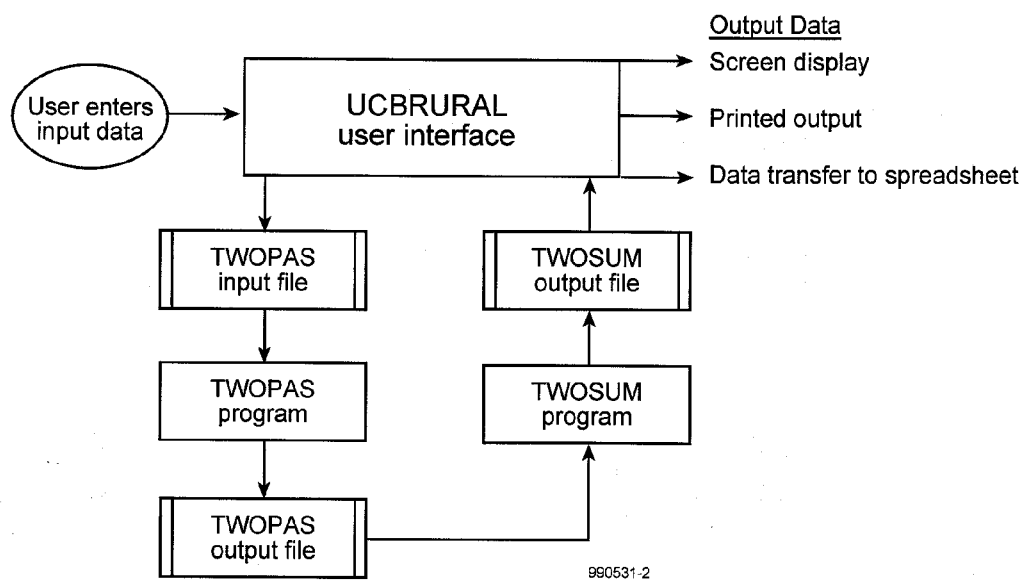


Figure 27. Flow Diagram of Interactions Between TWOPAS, TWOSUM, and UCBRURAL

Table 20. List of Candidate TWOPAS Model Improvements

Table 20 (Continued)

Made as Part of the Research

Candidate model improvements	
B	New model capabilities—additions to vehicle processing logic
1	Add capability to simulate the speed-suppressing effects of narrow lanes and shoulders
4	Add capability to simulate turning maneuvers by vehicles entering or leaving the roadway at specific intersections or driveways, including the effects of turning speed and the presence or absence of left- and right-turn lanes
9	Add capability for driver responses to reduced speed zones
D	Existing model features—improvements to input data
2	Include range checks for reasonableness of input data. Detect and reject input data which might result in an execution error (reduced scope from original plan)
3	Make further increases in dimensions (i.e., number of features or regions), if necessary, for input variables such as horizontal curves, passing/no-passing zones, sight distance regions, and intervals (needed for H4)
E	Existing model features—improvements to vehicle processing logic
3	Allow standard deviation of desired speed (not just the mean desired speed) to vary by vehicle type
4	Check possible problem with assignment of desired speeds and percent following for RVs
10	Investigate method of defining and collecting the predicted number of passes in passing lane (predictions may be too high)
F	Existing model features—improvements to output data
2	Review and improve content and layout of output file
G	Updates to incorporate changes in driver and vehicle characteristics
1	Update acceleration and speed maintenance capabilities of vehicle population
3	Update reduction in speeds selected by drivers on horizontal curves
H	UCBRURAL interface—improvements to input data
1	Improve existing UCBRURAL interface to deal with input data by zones rather than by fixed-length intervals (temporary fix for use within the research rather than permanent fix for the ultimate user)
4	Use the existing user-specified interval feature to add a graphical display of the longitudinal display of passing rates to the UCBRURAL output (requires D3)
5	Implement automatic calculation of sight distance from geometric data and user-specified offset to roadside obstructions
6	Implement automatic calculation of passing/no-passing zones from sight distance data for normal two-lane roadway sections and for the opposing direction to a passing lane
7	Move any values that users may wish to vary, including those used for calibration, from the source code of the TWOPAS program to an input file which can be updated through the interface. Provide password access to portions of these data, if appropriate.
8	Allow user to vary random number seeds from the interface
10	Modify the interface to allow user-specified data collection stations and intervals in Direction 2 to be specified in Direction 1 coordinates

Table 20 (Continued)

Candidate model improvements	
I	UCBRURAL interface—improvements to output data
1	Modify the interface to add additional information on printed graphs (e.g., analysis section limits, buffer section limits, duration of simulation, etc.)
J	UCBRURAL interface—improvements to file management capabilities
3	Provide capability to print files (input and output) from within the interface
K	UCBRURAL interface—improvements to other features
1	Add capability to make multiple TWOPAS runs in batch operation
5	Update on-line context sensitive help provided by the interface as appropriate for the new enhancements
N	Other TWOPAS/UCBRURAL Improvements
1	Allow interface to take advantage of TWOPAS capability to model vertical curves at changes of grade
2	Generate driven desired speeds for correct distributions when more than one vehicle category is present
4	Allow interface to use two user-defined data intervals in TWOPAS
5	Remove TRARR from the interface

improvements, organized into 11 categories (designated Categories B through N) based on the nature of the improvement. Each of these improvements to TWOPAS and/or UCBRURAL was undertaken as part of the research. Twenty-five of the 26 improvements were fully completed and the remaining two improvements were partially completed. Each of these improvements is described briefly below. More complete documentation of the improvements is provided in a technical memorandum prepared by the University of California-Berkeley (33).

In addition to the improvements to the functional improvements to the TWOPAS model described below, the source code for the larger and more important TWOPAS subroutines to more modern FORTRAN standards. Upgrading of source code was performed for 15 of the 99 TWOPAS subroutines which were selected base on the clarity of the existing code, the extensiveness of the subroutine, and its importance within TWOPAS. Flow diagrams and improved documentation were prepared for the improved subroutines.

4.3.1 Improvement B1—Effects of Narrow Lanes and Shoulders

The original TWOPAS model allow users to adjust for the effect on traffic operations of narrow lanes or shoulders by permitting users to modify the input values of desired speed for each direction of travel. Lower values of desired speed would be appropriate as lane or shoulder width decreased. However, no guidance was provided to the user concerning the magnitude of the effect that changes in lane and shoulder width might have on desired speed.

Field data collected as part of the research were used to quantify the effect of shoulder width on free-flow or desired speeds. This effect was based on analysis of speed data collected upstream and downstream of shoulder with transitions on two-lane highways. No suitable sites with lane width transitions were found. Therefore, the effect of lane width on free-flow speed was estimated from the operational analysis procedure of the 1985 HCM. Chapter 5 of this report presents the lane and shoulder width effects that were developed from this analysis.

The lane and shoulder width effects presented in Chapter 5 were incorporated in the UCBRURAL interface. The user of the interface can specify the lane and shoulder width for each direction of travel for every location along a roadway segment. Where the user specifies a lane with less than 3.6 m (12 ft) or a shoulder width less than 1.8 m (6 ft), the desired speed value provided by the user will be adjusted accordingly.

4.3.2 Improvement B4—Turning Maneuvers at Intersections and Driveways

The original TWOPAS model was a pipeline flow model in which all traffic entered at one end of the simulated roadway and traveled through the entire simulated roadway length to the other end. No capability was provided for vehicles to turn onto or off of the roadway at any intervening point.

It would be highly desirable for TWOPAS to have the capability to simulate turning maneuvers at intersections or driveways along the simulated roadway. Turning maneuvers onto and off of the roadway have the potential to delay other vehicles and, thus, to affect key traffic performance measures such as speed and platooning.

An approach to simulating traffic operations at intersections and driveways was developed as part of the research. This approach incorporates intersection (or driveway) locations, turning volumes, and geometrics specified by the TWOPAS user. TWOPAS logic has been developed to simulated turning maneuvers at intersections and driveways and that logic has been tested to the maximum extent possible on a stand-alone basis without being incorporated into the model. Incorporating the turning maneuver logic into TWOPAS and test in both the new logic and its compatibility with existing TWOPAS logic was found to require greater effort than was available within the contract. It is hoped that this effort will be completed in future FHWA work.

4.3.3 Improvement B9—Driver Responses to Reduced Speed Zones

In the original TWOPAS model, the user-specified desired speeds for each direction of travel applied throughout the entire length of the simulated roadway for that direction of travel except within horizontal curves, downgrade truck crawl regions, or their approaches. Thus, the original TWOPAS model provided no capability to simulated the effect of a reduced speed limit or any other factor that would reduce motorists speeds for only a portion of the simulated roadway.

Logic was developed and incorporated into TWOPAS to enable user to specify the locations of reduced speed zones in either direction of travel on the simulated roadway. The UCBRURAL interface was adapted to enable users to enter the locations of such zones in the interface and pass them on to TWOPAS. This capability for user-specified reduced speed zones was also used within the UCBRURAL interface to implement the effects of narrow lanes and shoulders discussed in Improvement B1.

4.3.4 Improvement D2—Range Checks on Input Data

Numerous range checks on the reasonableness of TWOPAS input data were implemented within the UCBRURAL interface.

4.3.5 Improvement D3—Increase Array Dimensions

The dimensions of many of the arrays in the TWOPAS model were increased. This change allows users to specify roadway geometrics in finer detail and to model a roadway up to 48 km (30 mi) in length.

4.3.6 Improvement E3—Vary Standard Deviation of Desired Speed by Vehicle Type

The original TWOPAS model allowed the user to enter an overall mean desired speed and a separate bias from the overall mean speed for each of three categories of vehicles: passenger cars, RVs, and trucks. However, only one standard deviation of desired speeds was permitted. Further, the three biases to the mean desired speed and the one standard deviation were used for both directions of travel. This represented two problems. First, in most cases, the standard deviations of desired speed should be different for different vehicle categories. Second, there are situations in which the mean desired speed is different for the same vehicle category in the two opposing directions of travel. For example, this may occur when most trucks travel heavily loaded in one direction of travel and empty in the other. Therefore, a change was made to TWOPAS to allow the model to accept different values of mean desired speed and standard deviation of desired speed for each vehicle category and for each direction of travel. Only minor changes to the UCBRURAL interface were required.

4.3.7 Improvement E4—RV Speed/Percent Following Problem

A potential problem was identified in test runs with the TWOPAS model which found that the directional average travel speed for RVs was lower than the desired speed, even under ideal conditions, when the desired speed was 96 km/h (60 mi/h) or more. It was also found that an increase in desired speed caused an increase in percent following, as well. These findings seemed unrealistic.

This potential problem was investigated and it was found that the RV performance data specified in TWOPAS corresponded to vehicles that could not maintain a speed of 96 km/h (60 mi/h) in level terrain. These RVs were representative of the lower performance end of the RV population in the 1970s and that these input parameters had not been updated since. Thus, it was established that the observed problem resulted from the input data specified rather than from any structural problem within the model itself. This problem was corrected by updating the vehicle population used as input to TWOPAS (see Improvement G1).

4.3.8 Improvement E10—Investigate High Frequency of Passes in Passing Lane Sections

Several previous users of TWOPAS reported that the model appeared to overestimate the number of passes that occur in roadway sections with added passing lanes. This problem was investigated and it was concluded that there was no evident problem with the model. However, the investigation was limited by the scarcity of field data on passing rates in passing lane sections available for comparative purposes.

The only relatively recent reference found with passing rates in a form that could be compared was the work of May (34). A very crude comparison found that the passing rates observed in the field were two to three times higher than those simulated by TWOPAS. However, this comparison was only approximate because desired speed of the drivers whose passing rates were measured in the field was unknown, because the vehicle mix in the field and simulated data differed, and because the simulated passing rates were based on longer passing lanes than were used in the field studies.

The passing rates in passing lanes simulated by TWOPAS were compared with the model of theoretical passing rates developed by Wardrop (35):

$$P = \frac{1}{\sqrt{\pi}} \frac{\sigma Q^2}{V_s} \quad (10)$$

where:

P	=	passing demand (passes/mi/h)
Q	=	directional traffic flow (veh/h)
V_s	=	mean driven desired speed (mi/h)
σ	=	standard deviation of driven desired speed (mi/h)

Reasonably good agreement between the TWOPAS and Wardrop results was found and, in fact, the passing rates simulated by TWOPAS appeared to be in better agreement with Wardrop than those simulated by TRARR. Thus, it was concluded that the potential problem reported in TWOPAS was not, in fact, present and that the passing rates simulated by TWOPAS in passing lane sections are reasonable.

4.3.9 Improvement F2—Improvement to TWOSUM Output

The UCBRURAL interface was enhanced to give the TWOPAS user better access to the TWOSUM output data. The summary data that can now be printed from the TWOPAS output data consists of three tables. The first table includes the specifications for the simulation run including simulation time, warm-up time, the random number seeds, the name of the user input files, and the lengths of the simulated roadway and warm-up sections. The second table includes input data including flow rates, vehicle mix, desired speed, standard deviation of desired speed, and specified percent platooning of entering

vehicles. The third table consists of output for the entire simulated roadway and for subsections within the simulated roadway. The traffic performance measures in this table include simulated flow rate, average percent following, average speed, trip time, total trip delay, trip delay due to traffic, trip delay due of geometrics, number of passes, total travel time, and total travel.

4.3.10 Improvement G1—Update Vehicle Performance Characteristics

A study was undertaken to update the vehicle performance characteristics used as input to TWOPAS. The vehicle performance characteristics for passenger cars and trucks had last been updated in the mid 1980s and, as discussed above in connection with Improvement E4, the RV performance characteristics had not been updated since the 1970s.

The truck performance data used as input to TWOPAS were revised using field data for truck crawl speeds on grade that were collected at Site CA05 in northern California. When a truck is slowed by a grade to a crawl speed (i.e., a speed that it maintains without decelerating but from which it cannot accelerate), that speed is primarily a function of two factors: the steepness of the grade and the weight-to-horsepower ratio of the truck. Other factors that influence the relationship between crawl speed and wieght-to-horsepower ratio, such as aerodynamic drag coefficient, can be set to default values. Thus, when truck speed data are collected near the top of a long grade whose steepness (percent grade) is known, the weight-to-horsepower ratios of the trucks can be estimated. The analysis results showed that the weight-to-horsepower ratios of the highest-performance trucks have changed very little since the mid-1980s, but the weight-to-horsepower ratios of the lowest performance trucks have decreased (i.e., their performance has improved). This change has occurred primarily because truck engines with greater horsepower have become available. There are strong economic incentives based on engine maintenance and operating costs for truckers to use these more powerful engines when hauling heavy loads.

Passenger car performance characteristics were revised through analysis of published data assembled by Oak Ridge National Laboratory (36, 37) as part of their ongoing monitoring of fuel economy trends for the U.S. Department of Energy. These data show vehicle performance characteristics as well as fuel economy. The performance characteristics of the current passenger car population were determined by weighting together (in proportion to their presence in the current vehicle fleet) a set of current vehicles, a set of early 1990s vehicles, and a set of 1980s vehicles. As part of this process, we have included not only the performance characteristics of passenger cars, but also of light trucks and vans, which now constitute about one-third of passenger vehicles.

Recreational vehicle performance characteristics were also updated with newer data, incorporating the information on improved passenger car and light truck performance with assumptions about additional weight carried and additional aerodynamic drag.

Table 21 compares the vehicle performance data used previously as input to TWOPAS with the revised values recommended for use in develop the chapter. Figure 28 illustrates how passenger car speeds vary with percent grade for the old and new vehicle populations. This plot is based on a TWOPAS run for a traffic stream composed of 100% passenger cars (in the mixes shown in Table 21) with drivers whose desired speed is 113 km/h (70 mph). As shown in the plot, the change in vehicle population has essentially no effect on downgrades and in level terrain, but there are increasingly larger effects on vehicle speed as the percent grade increases on upgrades, with the new vehicle population being less affected by grade than the old. At desired speeds of 60 mph and below, there is almost no effect of grade on average travel speed until the grade exceeds 6 percent. These results reflect the effects of both speed decreases on upgrades by lower performance passenger cars and delays to higher performance passenger cars that catch up with them.

4.3.11 Improvement G3—Investigate Speeds on Horizontal Curves

An investigation was undertaken of the logic used by TWOPAS to reduce driver desired speeds on horizontal curves. In the original TWOPAS model, speeds on horizontal curves were generally unaffected until a relatively sharp radius was reached at which the curve effect was triggered. A more realistic effect of horizontal curvature on speed was introduced by changing the manner in which the maximum speed on a horizontal curve is computed.

4.3.12 Improvement H1—Allow Data Input by Zones or Regions

The purpose of this improvement was to reduce the effort required by the user to enter road data by allowing the user to utilize the zone input capability of TWOPAS. TWOPAS does not require the user to input geometric and traffic control data at fixed intervals, but only at points of change (i.e., beginnings and ends of “zones”). The goal of this model improvement was to devise a simple procedure that could be incorporated into the general logic of the existing road data screen in TWOPAS to take advantage of this TWOPAS capability. Such a method has been developed and implemented.

Table 21. Recommended Changes in TWOPAS Input for Vehicle Performance Characteristics

Vehicle category	TWOPAS vehicle type	Percent of truck population	Weight to net horsepower ratio (lb/hp)		Weight to projected frontal area ratio (lb/ft ²)		
			Old	New	Old	New	
Truck	1	12.0	266	228	620	682	lowest performance truck
	2	25.6	196	176	420	462	
	3	34.0	128	140	284	340	highest performance truck
	4	28.4	72	76	158	174	
Vehicle category	TWOPAS vehicle type	Percent of RV population	Maximum acceleration (ft/sec ²)		Maximum speed (ft/sec)		
			Old	New	Old	New	
Recreational vehicle	5	10.0	8.22	9.0	102.67	110.0	lowest performance RV
	6	40.0	8.64	11.0	105.92	115.0	
	7	40.0	8.75	12.5	110.81	120.0	highest performance RV
	8	40.0	8.76	14.0	114.07	125.0	
Vehicle category	TWOPAS vehicle type	Percent of PC population	Maximum acceleration (ft/sec ²)		Maximum speed (ft/sec)		
			Old	New	Old	New	
Passenger car	9		9.277	11.17	109.14	112.8	lowest performance PC
	10		9.766	11.99	114.89	117.8	
	11		10.089	12.77	118.69	121.1	
	12		10.429	13.22	122.69	127.0	
	13		11.201	14.10	131.78	142.7	highest performance PC

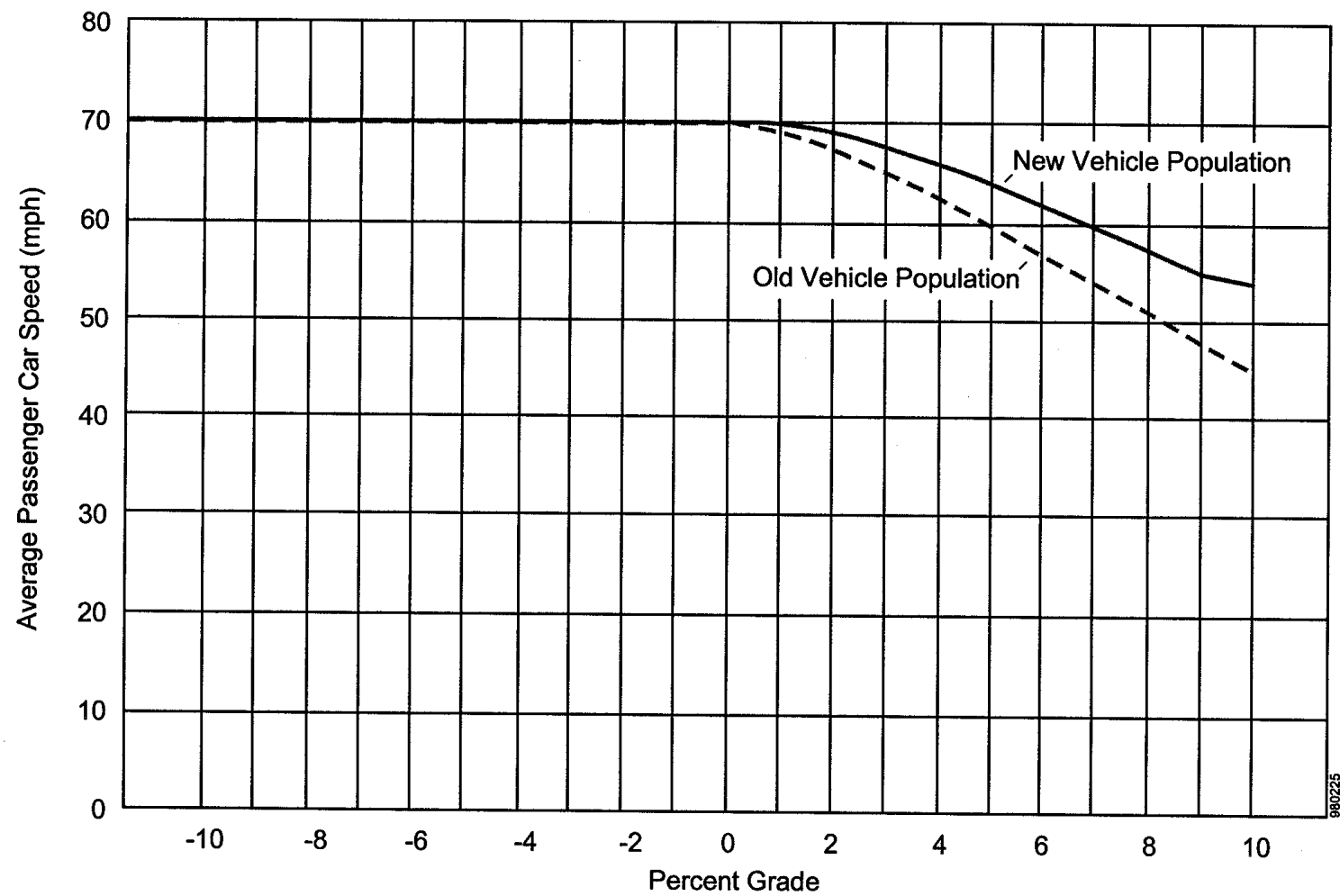


Figure 28. Comparison of Old and New Vehicle Populations for 100% Passenger Car Flows as a Function of Percent Grade

The general procedure for beginning a new study in the UCBRURAL interface has been changed so that the user specifies the total length of the road and the length of constant-length road sections that will be used to describe that road. The constant-length road sections can be as short as 0.01 mi or 52.8 ft. The interface then generates all the road sections with default values. Beginning at the top of the road file, the user enters all the data for one variable for the entire roadway. Each time a value is entered, it is automatically repeated for the remaining road sections to the end of the roadway. In this way, the user enters values for that variable only where they change. When the road data is being edited after the data has been entered by this method, a similar procedure takes place. In this case, each time the user enters a value for the given variable, it is automatically stored in all the following road sections if they have the same value as the one that was replaced by the user.

In order to allow for the possibility of editing road data for each road section after the initial data is entered, the user can set a flag which will specify whether road data is to be entered by this new zonal technique or for each individual section. This option can be toggled back and forth to accommodate changing needs during data entry.

4.3.13 Improvement H4—Add Profile Graph of Number of Passes

TWOPAS has the capability to assemble data on the number of passes that occur within specific user-specified intervals within the simulated roadway. The UCBRURAL interface had been set up originally to capture and graph data on the number of passes from TRARR, but this capability had never been implemented for TWOPAS. Appropriate changes were made to enable UCBRURAL to capture data on the number of passes from TWOPAS. The capability to graph these data as output was already available in UCBRURAL.

4.3.14 Improvement H5—Automatic Sight Distance Computation

The purpose of this improvement, was to relieve users of the TWOPAS model of the necessity to determine available sight distances either from plans and profiles or in the field. An algorithm to determine sight distance from the geometric data available in TWOPAS input was developed. This algorithm was implemented in the UCBRURAL interface so that once sight distances were automatically determined, they could be edited by the user. The horizontal and vertical sight distances are computed separately for several points on each road section, and for each point the minimum of the two is selected as the available sight distance for that point.

The sight distance algorithm uses as input the horizontal and vertical alignment of the road, the drivers eye height (h_1), the height of the object (h_2), and the distances from the centerline to the obstructions to both sides of the road. Only circular curves are modeled,

and nighttime conditions and the restrictions posed by overpasses are not modeled. The sight distance is computed from a position on the centerline of the road to another position on the centerline, i.e., the vehicle evaluating a pass and the opposing vehicle are both assumed to be on the roadway centerline.

4.3.15 Improvement H6—Automatic Passing Zone Calculation

This improvement has the purpose of simplifying the data input process in the UCBRURAL interface by allowing the locations of passing and no-passing zone markings (barrier stripes) to be computed automatically. By creating the passing/no-passing zones automatically, the user needs to change the barrier line codes only for those locations at which the computed barrier line locations disagree with the field marking. The algorithm uses the sight distance and passing lane location information to obtain passing/no-passing zones automatically. No-passing zones are established on sections with inadequate passing sight distance. The UCBRURAL interface allows the user to input the minimum sight distance required for safe passing to occur (e.g., based on MUTCD criteria). In addition, no-passing zones are also established in sections with adequate sight distance for passing but located between no-passing sections separated by a small, user-specified distance, thus limiting the minimum length of a passing zone; the MUTCD uses a 122-m (400-ft) separation for this purpose.

4.3.16 Improvement H7—Expand Ability to Change Default Input Parameters

When the TWOPAS model was first incorporated into the UCBRURAL interface during a previous project, users were provided with the ability to specify values for certain TWOPAS input variables, but others were set to default values within the interface. As part of this improvement, these variables were moved to an external file so that users have the capability to modify the values of these variables. Furthermore, a number of parameters set to default values within TWOPAS were moved to this external files as well so that their values can be changed by users. The average user of TWOPAS and the UCBRURAL interface will have no need to change these variables, but this improvement provides the capability for more experienced users to exercise particular features of the model (e.g., simulation with nonstandard vehicle performance characteristics or specification of regions within which particular truck types travel at crawl speeds on downgrades). Safeguards have been built into the UCBRURAL interface to assure that TWOPAS uses the changed parameter values from the external file only when the user specifically elects to do so.

4.3.17 Improvement H8—Vary Random Number Seeds

The UCBRURAL interface originally allowed the user to enter only one random number seed from which the interface generated the five 8-digit random number seeds

required by TWOPAS. It must be noted that each sequence of generated pseudo-random numbers depends on its random number seed.

Each of the five random number seeds is used to generate a particular traffic characteristic such as entering headways, vehicle types, and desired speeds of entering vehicles for a particular direction of travel. In order to eliminate random number seeds which do not produce well-behaved random numbers and also to vary one traffic characteristic while keeping the others constant, the user needs to be able to directly enter all five random number seeds from within the interface. This allows the user to create “clone” runs in which identical traffic streams operate over roadways with different geometrics.

The UCBRURAL interface has been modified so that the user now has two options for specifying the random number seeds. The first option is as before: the user can direct the interface to generate the five random number seeds that are used by TWOPAS from one user-supplied random number seed. The second option allows the user to enter all five random number seeds when a run is initiated.

4.3.18 Improvement H10—Specify Direction 2 Locations in Direction 1 Coordinates

During the simulation process, TWOPAS collects information at observation stations and over intervals. The user specifies the location of the observation stations and two intervals in input to the UCBRURAL interface. The interface as it was originally developed for TRARR, displayed the schematic of the roadway on the screen with Direction 1 going from left to right with the leftmost coordinate labeled as zero; it displayed Direction 2 from right to left with the rightmost coordinate labeled as zero. The locations of the observation information in Direction 1 were specified by the user in Direction 1 coordinates and the locations of the observation information in Direction 2 were given in Direction 2 coordinates. However, this dual coordinate system was not appropriate for TWOPAS which expects all coordinates to be entered in Direction 1 coordinates. Although the interface converted the Direction 2 coordinates to Direction 1 coordinates before writing them to the TWOPAS input file, its requirement that Direction 2 locations be entered in Direction 2 coordinates proved confusing to many users of the program. To eliminate this confusion, the UCBRURAL interface has been modified so that all locations are entered in Direction 1 coordinates.

4.3.19 Improvement I1—Provide Additional Output Information on Graphs

The purpose of this modification was to review the various graphs produced by the UCBRURAL interface, identify those graphs that were most important to update, and make the necessary modifications. Four graphs of input data are now available for printing from the interface: Sight Distance, Reduced Speed Zones, Horizontal Curve Radii, and Percent

Grade. Three graphs of output data are available for display on the screen for review by the interface user. These are: Mean Speed, Percent Following, and Number of Passes. Four graphs of output data are now available in the interface: Mean Speed, Percent Following, Number of Passes, and Percent Grade. While percent grade is actually an input variable, not an output variable, its inclusion on printed output graphs is useful for comparative purposes. Comparison graphs from two simulation runs can be printed for one user-specified variable chosen from among three options: Mean Speed, Percent Following, and Number of Passes.

4.3.20 Improvement J3—Print Files from the Interface

The UCBRURAL interface for TWOPAS was originally developed to produce graphical output on the screen and to print graphical output. There was no provision to print either the long TWOPAS output or the one- to two-page TWOSUM output from within the interface. This required the user to save the current data files and exit the UCBRURAL interface each time there was a need to print one of these files. After printing the output files, if the user re-entered the interface, the data files had to be re-loaded back into the program. To make printing these files easier, a method for printing an output text file from within the interface has been developed and implemented for printing the TWOSUM output. This capability could easily be applied to other currently existing files such as the TWOPAS input file and TWOPAS multiple run batch file (both of which are created by the interface) and the complete TWOPAS output file, but such improvements have not been implemented.

4.3.21 Improvement K1—Capability to Make Multiple Runs

A capability has been added to the UCBRURAL interface to make multiple TWOPAS runs in batch mode for the same roadway and observation files, but with varying traffic inputs and run specifications. The progress of a TWOPAS multiple run can be followed on the screen in a one-line window that displays the run number and the simulation time of that run. The results of a set of multiple runs can be imported into any standard computer spreadsheet, such as Microsoft Excel, which makes comparison and analysis of the simulation results very efficient. The ability to make multiple runs was helpful in testing program modifications, in the calibration and validation process, and in development of the revised HCM Chapter.

4.3.22 Improvement K5—Update On-Line Help

The UCBRURAL interface provides extensive on-line help. Much of this on-line help is specific to any currently highlighted menu item or to any current window (i.e., it is context sensitive). The on-line help features of the UCBRURAL interface have been updated to incorporate each of the improvements that have been implemented.

4.3.23 Improvement N1—Vertical Curves at Changes of Grade

The TWOPAS model capability of incorporating vertical curves in profile data was not implemented in the UCBRURAL interface when TWOPAS was originally included in the interface in a previous project. However, vertical curves were needed for the automatic sight distance calculation added to the interface in Improvement H5. Therefore, the interface was modified to allow the user to specify vertical curves and to assume vertical curve geometrics by default when the user does not enter a vertical curve. The vertical alignment of the road is specified by entering the grade over each road section for Direction 1. When computing the sight distance, the interface will automatically insert vertical curves at grade discontinuities if the vertical curves have not been entered by the user. The vertical curves are calculated based on the standard equation ($L = K/A$), where L is the length of the curve, A the percent change in grade, and K the rate of vertical curvature. The value of K used in the UCBRURAL interface is set at 200. An added benefit of this modification is that the user can now specify a vertical curves simply by entering a code “99” between two grades and such vertical curves are processed by the interface and added to the grade regions that are passed to the TWOPAS model.

4.3.24 Improvement N2—Correct Speeds When More Than One Vehicle Category is Present

During the process of modifying TWOPAS, an error was found in the processing of desired speed distributions. It was found that, when two or more vehicle categories (i.e., passenger cars, trucks and RVs) with different specified desired speed distributions were used, the program did not generate desired speed distributions consistent with the values specified in input. This problem was not apparent when only one vehicle category was specified or if two or more vehicle categories were used, but with the same desired speed distribution. It was found that a needed COMMON block was missing from one subroutine of the program. This error was corrected and TWOPAS now assigns desired speeds as intended.

4.3.25 Improvement N4—Allow Two User-Defined Data Collection Intervals

The original TWOPAS program allowed the user to define up to 10 non-overlapping data collection intervals per direction of travel. However, the UCBRURAL interface limited the user to defining only one interval per direction. It is the output data from this interval that the interface uses to produce the profile graphs of Mean Speed and Percent Following. TWOPAS users potentially need traffic performance measures not only for the overall interval (from first to last observation station), but also for other subintervals (in order to see, for example, what happens around passing lanes). Thus, the ability to collect data for user-specified intervals which overlap the overall interval was added to the model.

4.3.26 Improvement N5—Remove TRARR from the Interface

The UCBRURAL interface was originally developed for use with the TRARR model. The TWOPAS model was later incorporated into the same interface. With this version of the UCBRURAL interface, the user was able to enter data and then run either the TRARR or TWOPAS model. An interface from which either model could be run proved to be a very useful tool and was used extensively in the early part of this research in comparing the two models. After the TWOPAS model was selected for use in this work, every effort was made to retain the TRARR model in the interface. However, with the inclusion of several of the improvements discussed above, the specifications for accommodating the two models were no longer compatible. Because modifications to the interface to retain compatibility with TRARR were not part of the current project, the research team reluctantly concluded that the TRARR model had to be removed from the current UCBRURAL interface. TRARR-related program logic and all references to TRARR on data screens and on-line help screens in the interface have been removed.

CALIBRATION AND VALIDATION OF THE TWOPAS MODEL

The TWOPAS model was calibrated and validated as part of the research. Calibration involved making a series of TWOPAS runs while systematically varying factors such as entering traffic volume, percent heavy vehicles, and percent grade to verify that the traffic performance measures provided by the model—particularly percent following and average travel speed—respond to the changes in reasonable fashion.

The validation activities involved comparison of the TWOPAS outputs with actual field data. Two sites have been used for validation activities. These are Site CA02 in the San Francisco Bay area of California and a site on the Trans Canada Highway in British Columbia for which data were provided by Prof. John Morrall of the University of Calgary. The California site was chosen because it has the highest peak volumes found for any two-lane highway site for which data are available. This site also has several sharp horizontal curves with restricted speeds. The Trans Canada Highway site was chosen because it has a variety of terrain and a variety of passing and climbing lanes.

For Site CA02, TWOPAS was used to simulate traffic conditions corresponding to ten 15-min periods for which traffic data were available. These 15-min periods were selected to cover a range of flow rates from 960 to 3,300 veh/h for both directions of travel combined. While the field measured values were based on 15-min observations, the simulation model was run for 60-min periods to increase the size of the sample on which the observed performance measures were based. In addition, three replicate runs with the simulation model were made for each set of conditions investigated; the replicate runs used different random number seeds, which create different sequences of drivers and vehicles using the same geometric alignment. Simulation runs were made for a 7.2 km (4.5-mi) site length, but comparisons with field data were made for the portion of the site where field data were

collected [two sets of traffic counters in both directions of travel approximately 1.6-km (1 mi) apart near the eastern end of the site].

Table 22 summarizes the comparison between the simulation and field results for the 10 observation periods that were studied. For both the eastbound and westbound directions on the site, the table shows the simulated values of speed and percent following (based on the averages of the three replicate simulation runs), and the corresponding field measured values. The table shows that, on the average, the simulation model predicted speeds within 8 percent for the eastbound direction and with 7 percent for the westbound direction. Percent following was predicted within 7 percent for the eastbound direction and within 5 percent for the westbound direction. This level of agreement between simulation and field data appears satisfactory.

The second validation effort addressed a 19-km (12-mi) section of the Trans Canada Highway in British Columbia. Field data were available for four stations in the westbound direction (WB1 through WB4) and three stations in the eastbound direction (EB1 through EB3). Simulation runs were made for the conditions corresponding to ten 15-min periods with flow rates in the range of 500 to 800 veh/h for both directions of travel combined. The one-way flow rates for the study periods ranged from 100 to 600 veh/h. This site placed a greater challenge on the predictive ability of the models. Rather than predicting the speed and percent following for a 1.6-km (1-mi) section as was done for Site CA02, we attempted to predict these measures at particular spots where traffic counters had been placed in the field. The results for average spot speed shown in Table 23 and corresponding results for spot percent following are shown in Table 24. Station WB3 is omitted from the tables because only limited field data were available from the traffic counter at this location.

Table 23 shows that spot speeds at the six individual traffic count stations differ by averages of 8.0, 8.1, 4.3, 7.9, 4.9, and 5.4 percent. These differences appear to be within satisfactory limits.

Table 24 shows that the simulated values of spot percent platooning vary more from the field data than do spot speeds. The average differences between the simulated and field-measured values of spot platooning are 8.3, 6.9, 1.7, 7.0, 4.0, and 3.9 percent. This level of agreement appears reasonably good, but it is helped because sometimes the simulation model provides results that are higher than the field data and sometimes it provides results that are lower. When the differences are expressed as absolute values on a percentage basis, the simulated and field values of spot platooning differ by 14.1, 11.2, 15.9, 19.6, 10.2, and 18.1 percent. The research team had hoped for closer agreement than this. The absolute differences in percent following in Table 24 (except at Station EB1) are not much larger than those in Table 22, but they are larger when expressed as absolute percentage differences because the flow rates (and, therefore, the observed percent following) are much lower in Table 24. Furthermore, as stated above, the use of spot

Table 22. Comparison of TWOPAS and Field Values for Mean Speed and Percent Following for Site CA02

Obs No.	Flow (vph)			EB speed (mph)				WB speed (mph)			
	EB	WB	Combined	Simulation	Field	Diff	% Diff	Simulation	Field	Diff	% Diff
92	1656	1252	2908	41.7	44.0	2.3	5.1	45.2	50.2	5.0	10.0
96	1076	888	1964	43.8	47.1	3.3	6.9	47.1	52.0	4.9	9.3
101	800	740	1540	44.7	48.7	4.0	8.1	47.9	51.3	3.4	6.6
107	984	876	1860	44.1	47.9	3.8	7.8	47.1	51.1	4.0	7.7
156	1460	1924	3384	42.6	47.4	4.8	10.0	44.1	44.8	0.7	1.5
223	624	344	968	45.5	49.2	3.7	7.5	50.5	46.5	-4.1	8.7
252	1308	1680	2988	43.0	47.6	4.6	9.6	43.4	43.2	-0.2	0.5
340	1292	964	2256	42.9	45.2	2.3	5.1	46.8	48.6	1.8	3.7
342	1460	1156	2616	42.6	47.8	5.2	10.8	45.8	49.4	3.6	7.3
399	608	736	1344	46.2	50.3	4.1	8.1	48.2	54.3	6.1	11.2
AVERAGES						3.8	7.9			2.5	6.7

Obs No.	Flow (vph)			EB percent following				WB percent following			
	EB	WB	Combined	Simulation	Field	Diff	% Diff	Simulation	Field	Diff	% Diff
92	1656	1252	2908	97.2	96.4	-0.8	0.8	88.8	89.3	0.5	0.6
96	1076	888	1964	92.1	88.1	-4.0	4.5	79.8	79.7	-0.1	0.1
101	800	740	1540	88.0	81.3	-6.8	8.3	74.5	77.0	2.5	3.2
107	984	876	1860	90.6	85.5	-5.1	6.0	79.4	76.7	-2.8	3.6
156	1460	1924	3384	95.3	92.2	-3.2	3.4	93.8	98.3	4.5	4.5
223	624	344	968	83.0	72.3	-10.7	14.8	52.0	62.5	10.5	16.8
252	1308	1680	2988	94.2	91.5	-2.7	3.0	95.5	87.2	-8.4	9.6
340	1292	964	2256	93.8	91.0	-2.9	3.1	82.2	84.2	2.0	2.4
342	1460	1156	2616	95.2	91.7	-3.5	3.8	87.1	87.7	0.6	0.7
399	608	736	1344	82.3	71.3	-11.0	15.4	74.2	74.7	0.5	0.7
AVERAGES						-5.1	6.3			1.0	4.2

NOTE: Percent difference is the absolute value of the difference expressed as a percentage of the field measured value; EB = Eastbound; WB = Westbound.

measures rather than section measures and the use of only one simulation run (rather than the average of several replicates) makes it much more difficult to get good agreement.

The research team is generally satisfied with the validation results obtained and it was concluded that the revised TWOPAS model was suitable for use in development of HCM operational analysis procedures.

Table 23. Comparison of TWOPAS and Field Values for Mean Spot Speed for the Trans Canada Highway Site
Mean Spot Speeds (km/h)—Westbound

Obs No.	WB Flow (vph)	Station WB1				Station WB2				Station WB4			
		Simulation	Field	Diff	% Diff	Simulation	Field	Diff	% Diff	Simulation	Field	Diff	% Diff
1	232	98.3	97.5	-0.8	0.8	92.5	83.3	-9.2	11.0	104.4	96.6	-7.8	8.1
2	188	102.0	106.4	4.4	4.1	95.3	86.8	-8.5	9.8	106.4	105.1	-1.3	1.2
3	147	99.6	109.6	10.0	9.1	95.3	89.3	-6.0	6.7	105.4	111.1	5.7	5.1
4	225	98.6	108.0	9.4	8.7	93.0	84.3	-8.7	10.3	105.4	106.4	1.0	0.9
5	372	95.9	104.1	8.2	7.9	89.2	83.2	-6.0	7.2	102.2	96.1	-6.1	6.3
6	379	94.8	102.2	7.4	7.2	88.8	81.8	-7.0	8.6	102.2			
7	471	91.6	100.4	8.8	8.8	86.3	79.4	-6.9	8.7	99.8			
8	448	93.1	105.7	12.6	11.9	87.7	82.9	-4.8	5.8	99.8			
9	342	96.5	106.4	9.9	9.3	96.5	87.7	-8.8	10.0	102.7			
10	466	92.9	105.6	12.7	12.0	87.2	84.5	-2.7	3.2	99.1			
AVERAGES				8.3	8.0			-6.9	8.1			-1.7	4.3

Mean Spot Speeds (km/h)—Eastbound

Obs No.	EB Flow (vph)	Station EB1				Station EB2				Station EB3			
		Simulation	Field	Diff	% Diff	Simulation	Field	Diff	% Diff	Simulation	Field	Diff	% Diff
1	433	83.4	89.7	6.3	7.0	100.4	105.5	5.1	4.8	95.7	92.9	-2.8	3.0
2	448	83.5	95.9	12.4	12.9	100.3	105.4	5.1	4.8	96.3	95.1	-1.2	1.2
3	573	82.2	95.2	13.0	13.7	92.1	103.9	11.8	11.4	92.3	93.6	1.3	1.4
4	308	85.5	99.7	14.2	14.2	101.9	112.0	10.1	9.0	98.0	102.3	4.3	4.2
5	189	90.4	92.0	1.6	1.7	105.7	103.4	-2.3	2.2	100.7	87.0	-13.7	15.7
6	327	84.5	94.8	10.3	10.9	99.5	104.6	5.1	4.9	95.8	95.3	-0.4	0.5
7	105	96.2	94.1	-2.1	2.2	107.7	104.1	-3.6	3.5	102.5	92.4	-10.1	10.9
8	248	88.4	87.7	-0.7	0.8	104.8	106.0	1.2	1.1	100.1	92.9	-7.2	7.7
9	165	93.7	96.7	3.0	3.1	105.9	110.1	4.2	3.8	101.7	93.9	-7.8	8.3
10	303	83.5	95.8	12.3	12.8	100.6	103.8	3.2	3.1	97.1	96.0	-1.1	1.1
AVERAGES				7.0	7.9			4.0	4.9			-3.9	5.4

NOTE: Percent difference is the absolute value of the difference expressed as a percentage of the field measured value.

Table 24. Comparison of TWOPAS and Field Values for Spot Percent Following for the Trans Canada Highway Site
Percent Following—Westbound

Obs No.	WB Flow (vph)	Station WB1				Station WB2				Station WB4			
		Simulation	Field	Diff	% Diff	Simulation	Field	Diff	% Diff	Simulation	Field	Diff	% Diff
1	232	59.5	50.0	-9.5	19.0	48.2	38.3	-9.9	25.8	41.9	41.4	-0.5	1.2
2	188	44.3	50.0	5.7	11.4	34.7	35.0	0.3	0.9	30.1	37.3	7.2	19.3
3	147	42.0	48.8	6.8	13.9	29.5	29.4	-0.1	0.3	23.8	37.1	13.3	35.8
4	225	56.0	45.5	-10.5	23.1	47.6	44.1	-3.5	7.9	34.9	32.7	-2.2	6.7
5	372	72.0	70.0	-2.0	2.9	58.9	62.5	3.6	5.8	52.3	62.4	10.1	16.2
6	379	73.6	59.2	-14.4	24.3	61.8	55.3	-6.5	11.8	52.4			
7	471	77.2	74.6	-2.6	3.5	65.8	63.4	-2.4	3.8	61.0			
8	448	76.9	68.1	-8.8	12.9	66.7	56.4	-10.3	18.3	63.5			
9	342	68.2	60.5	-7.7	12.7	68.2	51.8	-16.4	31.7	53.5			
10	466	79.0	67.2	-11.8	17.6	66.6	63.2	-3.4	5.4	65.9			
AVERAGES				-5.5	14.1							5.6	15.9

Percent Following—Eastbound

Obs No.	EB Flow (vph)	Station EB1				Station EB2				Station EB3			
		Simulation	Field	Diff	% Diff	Simulation	Field	Diff	% Diff	Simulation	Field	Diff	% Diff
1	433	79.0	71.0	-8.0	11.3	62.9	70.1	7.2	10.3	59.2	55.9	-3.3	5.9
2	448	80.8	72.6	-8.2	11.3	67.6	71.5	3.9	5.5	62.6	56.4	-6.2	11.0
3	573	86.1	70.2	-15.9	22.6	77.9	69.9	-8.0	11.4	72.0	64.7	-7.3	11.3
4	308	73.8	58.1	-15.7	27.0	53.7	58.7	5.0	8.5	48.4	51.2	2.8	5.5
5	189	57.7	51.1	-6.6	12.9	38.2	42.2	4.0	9.5	31.0	32.0	1.0	3.1
6	327	71.9	63.2	-8.7	13.8	57.9	56.3	-1.6	2.8	52.8	42.3	-10.5	-24.8
7	105	41.0	28.6	-12.4	43.4	25.0	35.7	10.7	30.0	21.6	17.4	-4.2	-24.1
8	248	65.2	63.9	-1.3	2.0	44.0	56.3	12.3	21.8	35.2	36.1	0.9	2.5
9	165	50.9	43.2	-7.7	17.8	37.1	36.6	-0.5	1.4	31.7	20.5	-11.2	-54.6
10	303	72.2	54.1	-18.1	33.5	57.7	58.4	0.7	1.2	52.6	38.2	-14.4	-37.7
AVERAGES				-10.3	19.6							-5.2	-10.2

NOTE: Percent difference is the absolute value of the difference expressed as a percentage of the field measured value.

CHAPTER 5.

OPERATIONAL ANALYSIS PROCEDURES FOR TWO-LANE HIGHWAYS

This chapter describes the development of revised operational analysis procedures for two-lane highways. These procedures were developed for incorporation in the HCM2000. The operational analysis procedures are completely presented and explained in Appendix D which presents the portion of HCM2000 Chapter 12 (Highway Concepts) related to two-lane highways and in Appendix E and HCM2000 Chapter 20 (Two-Lane Highways). The following discussion explains the development of these procedures.

This chapter begins with a discussion of the conceptual framework for the HCM2000, which has changed from previous HCM editions. The chapter then discusses key aspects of the procedures including fundamental speed-flow and percent time spent following-flow relationships, capacity, free-flow speed, and demand flow rates. Next, the chapter presents an overview of the specific operational analysis procedures for two-way segments, directional segments, directional segments with passing lanes in level and rolling terrain, and directional segments with climbing lanes and upgrades. The chapter concludes with a discussion of other relevant issues including performance measures available from the procedures, analysis of proposed two-lane to four-lane widening projects, level of service (LOS) assessment for directional two-lane facilities, LOS assessment for uninterrupted flow facilities and corridors containing two-lane highways, design and operational treatments, and planning applications.

CONCEPTUAL FRAMEWORK

This section discusses the conceptual framework for the revised operational analysis procedures for two-lane highways. Aspects of the conceptual framework addressed are service measures, classes of two-lane highways, LOS thresholds, base conditions, and applications.

5.1.1 Service Measures

A key decision in the development of the revised operational analysis procedures was the selection of the measures used to define LOS for two-lane highways. The operational analysis procedures in the 1985 HCM used two measures to define LOS: percent time delay for general terrain segments and average upgrade speed for specific upgrades. These measures have been defined in Chapter 2 of this report.

Limitations of Current HCM Service Measures

Several key limitations of the service measures in the 1985 HCM procedures were identified in the findings of the HCM user survey and in comments received by the HCQS two-lane roads subcommittee. The limitations included:

- HCM users were confused about the meaning of the term percent time delay
- HCM users were concerned that, because the general terrain segment and specific grade procedures used different LOS measures, they can provide different LOS assessments for similar conditions
- HCM users were concerned about the lack of an effect of design speed on LOS

In response to these user concerns, the research team decided that:

- If percent time delay were retained as a service measure, it should be renamed or better explained
- There should not be more than one set of LOS criteria applicable to locations on the same roadway
- LOS should be sensitive to design speed for at least some facility types

HCM User Assessment of Candidate Service Measures

In the user survey, current HCM users were asked to assess potential service measures for the HCM2000. The results, presented in Tables 14 and 15, indicate that the first choice of users was the combination of percent time delay and average travel speed, while the second choice was delay rate. Lower rankings were given to average travel speed by itself, percent time delay by itself, density, and passing ratio. All of the candidate service measures were assessed before a decision was made.

Advantages and Disadvantages of Candidate Service Measures

The combination of percent time delay and average travel speed—rated as first choice by HCM users—is conceptually appealing because it has the potential to combine into one analysis procedure the best features of the 1965 HCM procedures (based on speed) and the best features of the 1985 HCM procedures (based on percent time delay for general terrain segments). This approach also combines the service measures used for general terrain segments and for specific upgrades in the 1985 HCM procedures. This approach brings speed into the LOS definition, so it makes LOS sensitive to the design speed of a roadway, as desired by users. A disadvantage of a combined service measure is that separate

adjustments, such as passenger-car equivalents (PCEs) for heavy vehicles, are needed for percent time delay and average travel speed.

Delay rate was given careful consideration as a potential service measure because of its high ranking by HCM users. Delay rate is defined in units of s/veh-mi or s/veh-km, and it represents the degree to which drivers are forced to travel at speeds less than their desired speed. Delay rate includes the effects of speed reductions by motorists due to restrictive geometric features (e.g., horizontal curves and performance-limiting grades) and due to other traffic. Indeed, one of the appealing features of delay rate is that procedures can be developed to obtain separate estimates of geometric and traffic delay, which can provide insight into the causes of delay on a given facility. Delay also has a direct economic interpretation and can be used in economic studies if the monetary value of a veh-s of delay can be established.

The primary drawback of delay rate as a service measure is that there is no good method by which to measure it in the field. Delay is defined by the difference between the actual travel speeds of drivers and their desired speeds. The desired speeds of drivers whose actual speeds are measured in the field under congested conditions can never be known; they can only be estimated by measuring the speeds of other drivers under free-flow conditions. Thus, delay rate appears very useful as a traffic performance measure in computer simulation models, where an explicit desired speed is assigned to each driver, but it is not appropriate for field application and would be hard to explain to the public.

Percent time delay and average travel speed used separately were each highly rated as potential service measures. However, the drawbacks of the 1965 and 1985 HCM procedures cannot be overcome if either of these measures is used separately.

Density is the number of vehicles present per unit length of roadway lane (i.e., veh/lane-km or veh/lane-mi). Density is intuitively logical as a service measure for two-lane highways because it is the service measure that is used for the other types of uninterrupted flow facilities—freeways and multilane highways. However, density was found to be an inappropriate service measure for two-lane highways because, given the platooned nature of traffic, density is much less evenly distributed on a two-lane highway than on a freeway or multilane highway. Percent time delay does a much better job of representing density; percent time delay is the percentage of their total travel time that drivers spend traveling in locally high-density conditions.

Another candidate service measure, the passing ratio, is defined as the actual number of passing maneuvers to the desired number of passing maneuvers (or the total number of passing maneuvers that would occur on a two-lane highway with continuous passing lanes and with vertical and horizontal curvature similar to the two-lane highway in question). Morrall and Werner (16, 38) found that passing ratio has greater sensitivity to flow rate than percent time delay and recommended passing ratio as a service measure in a unified traffic flow theory model for two-lane highways based on the balance between passing

supply and demand. However, passing ratio was not rated high by HCM users as a service measure and it is not possible to measure passing ratio directly in the field.

Recommended Service Measures

Based on a review of the advantages and disadvantages of candidate service measures discussed above, the research team concluded that the most appropriate service measure for two-lane highways is the combination of percent time delay and average travel speed.

In response to HCM user confusion over the meaning of the term percent time delay, the research team concluded that the meaning of this term would be clearer if it were renamed *percent time spent following* (PTSF). PTSF is defined as the average percentage of their travel time that vehicles on a given roadway segment must travel in platoons behind slower vehicles due to inability to pass during some designated time interval. PTSF represents freedom to maneuver and the comfort and convenience of travel on a two-lane highway which are two important aspects of the LOS concept as explained in the HCM. While PTSF is difficult to measure directly in the field, it can be estimated as the percentage of vehicles traveling at headways of 3 s or less. The 1985 HCM suggested a threshold of 5 s or less for estimating percent time delay in the field. The reason for the change from 5 to 3 s is discussed below.

The other component of the combined service measure, *average travel speed* (ATS), is defined as the length of the roadway segment under consideration divided by the average total travel time for all vehicles to traverse that segment during some designated time interval. ATS reflects the mobility function of a two-lane highway. Speed, as represented by ATS, is another important part of the LOS definition. ATS can be estimated in the field by travel time studies or, less accurately, by measurement of spot speeds. A subsequent section shows that ATS was found to be appropriate in defining LOS for one class of two-lane highways but not for others.

The definitions presented above indicate that both PTSF and ATS are space-averaged measures (i.e., measured over a section of roadway), although both can be estimated from related spot measures. When applied to two-way roadway segments, PTSF and ATS are based on the sum of the travel times for both directions of travel combined. When applied to directional segments, travel times are considered only for the specified direction of travel. For application in capacity and LOS analysis, the time period used to define both PTSF and ATS is the highest-volume 15-min period within the peak hour.

Field Estimation of Percent Time Spent Following

The 1985 HCM stated that percent time delay can be estimated in the field based on spot platooning for a headway of 5 s. However, percent time delay in the current chapter

and PTSF in the revised chapter are actually based not on spot platooning, but on a section measure derived from the TWOPAS simulation model.

In TWOPAS, each vehicle is assigned one of six states depending upon its situation in traffic at any point in time. The six vehicle states used in TWOPAS are:

- 1—Free vehicle, unimpeded by others
- 2—Overtaking a leader, but still 2.4 m/s (8 ft/s) faster than the leader
- 3—Following a leader
- 4—Close following with interest in and capability of passing
- 5—Passing another vehicle
- 6—Aborting a pass

PTSF is determined with the TWOPAS model as the percentage of total travel time that vehicles spend in TWOPAS States 2 through 6 (i.e., not in State 1). This percentage is labeled in the model output as AVG TM DLYST1.

TWOPAS was used to compare three different forms of platooning-based traffic performance measures. These measures were:

- PTSF based on percentage of time spent in TWOPAS States 2 through 6
- PTSF based on percentage of time spent following at various headways from 2 to 6 s
- Average spot platooning based on various headways from 2 to 4 s

The comparison was conducted for two-way flow rates of 800, 1200, and 1600 veh/h with a 50/50 directional split and 5 percent trucks. Three replicate runs with different random number seeds were made for each level of flow rate. The results of the comparison are shown in Table 25. These results indicated that PTSF based on TWOPAS States 2 through 6 agrees very closely with both PTSF in platoons based on a 3-s headway and average spot platooning for a 3-s headway. Therefore, it was determined that the revised HCM chapter should recommend a 3-s headway, rather than a 5-s headway, for estimating PTSF in the field.

Table 25. Comparison of Percent Time Spent Following as Determined with the TWOPAS Model to Various Headway Criteria

Flow rate (veh/h)	Percent time spent following						Average spot platooning		
	TWOPAS State 2-6	Based on headway of:					Based on headway of:		
		2 s	3 s	4 s	5 s	6 s	2 s	3 s	4 s
800	51.3	34.4	53.5	61.9	64.9	66.7	33.6	52.4	60.6
	50.0	32.5	50.8	59.6	61.9	64.0	31.5	50.1	58.3
	50.5	32.9	51.4	60.4	62.7	64.7	32.2	50.2	58.9
1,200	64.5	43.7	64.8	73.5	75.7	77.1	42.7	63.9	72.4
	65.8	42.8	65.8	74.1	76.1	77.6	41.4	64.0	72.5
	66.8	44.2	66.8	76.0	78.0	79.5	43.5	66.0	74.7
1,600	74.1	49.4	73.0	82.4	84.0	85.2	48.7	72.4	81.6
	73.6	48.6	72.6	82.1	84.0	85.2	47.6	71.7	81.0
	74.9	49.0	73.8	83.9	85.4	86.4	48.2	72.9	83.0
	Avg Diff	-21.6	0.1	9.1	11.2	12.8	-22.5	-0.9	7.9

Note: A 50/50 directional split of traffic and 5 percent trucks were assumed for all simulation runs.

5.1.2 Classes of Two-Lane Highways

A major issue in the development of the revised operational analysis procedure was the role to be played by design speed which represents the character of the geometric design elements and, particularly, the horizontal curves on a roadway segment. Users of the existing HCM Chapter 8 identified the lack of an effect of design speed on LOS as a major shortcoming of the chapter. However, the HCM user survey also showed no general agreement about the role of speed in LOS. It was found that the needs of HCM users could best be served by designating two classes of two-lane highways on which the importance of speed in defining LOS differs.

Question 14 in the HCM user survey (see Appendices B and C) highlights the differences in HCM user attitudes toward the role of design speed in defining LOS. Users were asked to consider a winding, rural two-lane highway containing an extended series of horizontal curves with lower design speeds (e.g., 50 to 55 km/h or 30 to 35 mph) and long no-passing zones, but having relatively low traffic volumes. HCM users were asked whether such a facility should have its quality of service classified as:

- A high LOS (e.g., LOS A or B) because there is little operational congestion, or
- A low LOS (e.g., LOS D or E) because drivers are forced to travel at much lower speeds and are unable to pass other vehicles due to long no-passing zones

HCM users were nearly equally divided between these alternatives, making it difficult to judge whether speed should or should not be a criterion in determining LOS.

After careful investigation of the issue, and discussions with HCM users and HCQS committee members, the research team came to the conclusion that users who answered Question 14 with different answers were envisioning different classes of two-lane highways. On major intercity highways, motorists expect to travel at relatively high speed and, when they cannot, this should be reflected in a poor LOS. However, on many scenic or recreational routes, motorists do not necessarily expect to travel at higher speeds

To implement this concept, two classes of two-lane roads were defined as follows:

- Class I highways are two-lane highways on which motorists expect to travel at relatively high speeds. Two-lane highways that function as major intercity routes, primary arterials connecting major traffic generators, daily commuter routes, or as primary links in state and national highway networks are generally assigned to Class I. Class I highways generally serve long-distance trips or provide connecting links between facilities that serve long-distance trips.
- Class II highways are two-lane highways on which motorists do not necessarily expect to travel at high speeds. Two-lane highways that function as access routes to Class I facilities, serve as scenic or recreational routes that are not primary

arterials, or pass through rugged terrain are generally assigned to Class II. Class II facilities generally serve short trips, the beginning and ending portion of longer trips, or trips for which sightseeing activities play a significant role in route choice.

The classes of two-lane roads are closely related to their functional classification since most arterials would generally be considered Class I facilities and most collectors and local roads would generally be considered Class II facilities. However, the primary factor in determining the appropriate classification of a facility for operational analysis is the HCM user's assessment of motorist expectations for that facility, which may differ from its functional classification. For example, an intercity arterial route that passes through rugged mountainous terrain might be classified as a Class II facility if motorists recognize that the terrain makes it infeasible to provide a high-speed route in that corridor.

5.1.3 Level of Service Thresholds

Separate LOS thresholds were developed for Class I and II highways. For Class I highways, on which motorists expect to travel at high speed, LOS is defined by threshold values of both PTSF and ATS. For Class II highways, on which motorists do not necessarily expect to travel at high speeds, LOS is defined based on PTSF alone.

Tables 26 and 27 present the LOS recommended LOS criteria for Class I and II highways, respectively. The PTSF thresholds in Table 52 have generally been increased by the addition of 5 percent to the percent time delay thresholds used to define LOS in the existing HCM Chapter 8. This change was made in response to HCM users who commented that, in the HCM Chapter 8 procedures, reduced levels of service were reached at flow rates that appeared too low. A further increase of 5 percent was made in the PTSF thresholds for Class II highways in Table 53, in comparison to the LOS thresholds for Class I facilities in Table 52, on the grounds that motorists expectations of quality of service would be lower on Class II facilities than on Class I facilities.

Figure 29 provides a graphical illustration of the LOS thresholds for Class I highways. The figure illustrates that, for any given LOS to apply, two criteria must be met: a PTSF criterion and an ATS criterion. The LOS for a Class I highway can be determined by finding the region in Figure 29 within which the applicable combination of PTSF and ATS falls.

5.1.4 Base Conditions

The operational analysis procedures for two-lane highways begin by characterizing the values of the service measures, PTSF and ATS, for a set of base conditions which represent the absence of restrictive geometric, traffic, or environmental factors. The operational analysis methodology provides specific factors to adjust for the effects of geometric,

Table 26. Level-of-Service Criteria for Two-lane Highways in Class I

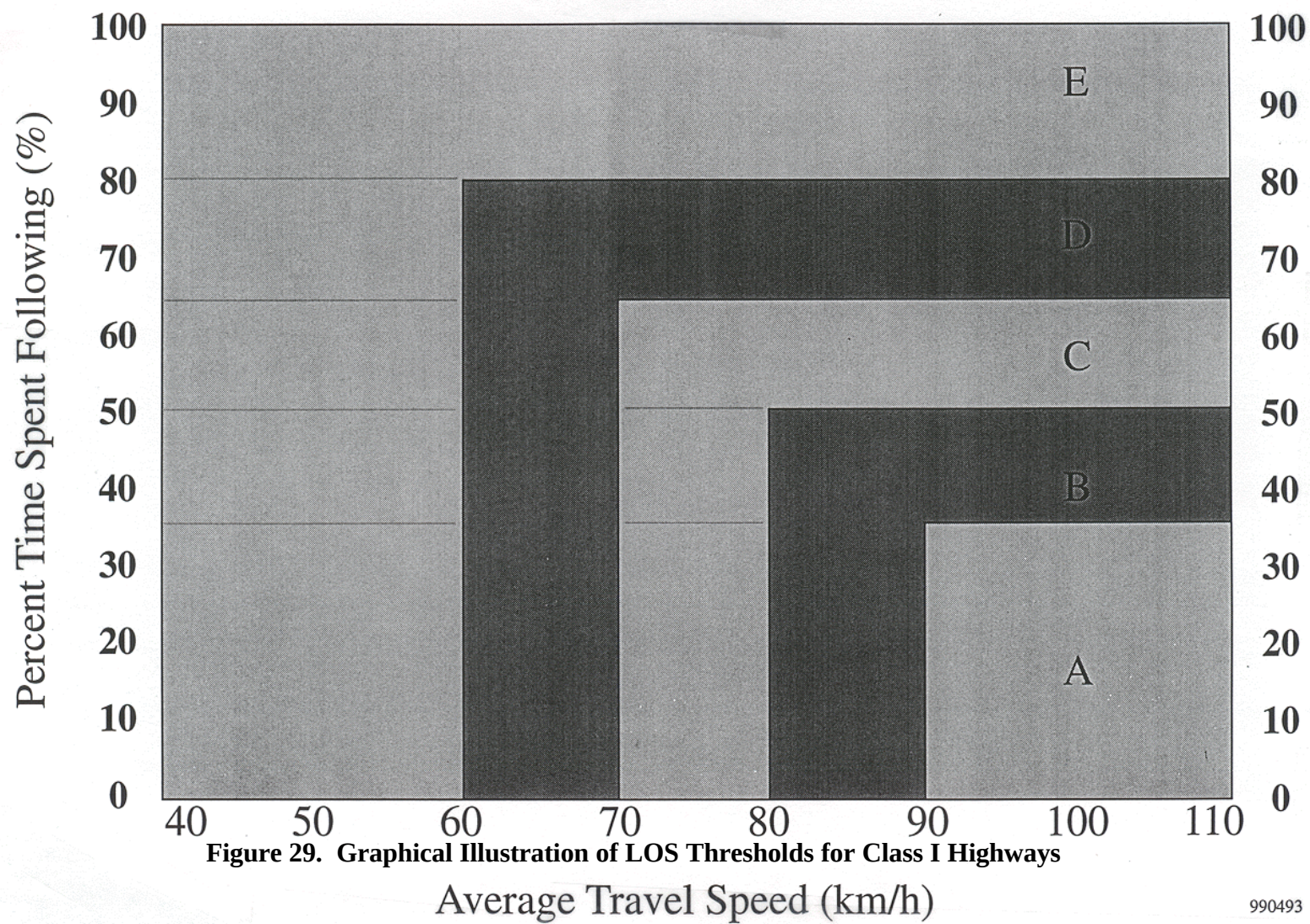
Level of service	Percent time spent following	Average travel speed (km/h)
A	≤ 35	> 90
B	$> 35-50$	$> 80-90$
C	$> 50-65$	$> 70-80$
D	$> 65-80$	$> 60-70$
E	> 80	≤ 60

Note: Level-of-service F applies whenever the demand flow rate exceeds the segment capacity.

Table 27. Level-of-Service Criteria for Two-lane Highways in Class II

Level of service	Percent time spent following
A	≤ 40
B	$> 40-55$
C	$> 55-70$
D	$> 70-85$
E	> 85

Note: Level-of-service F applies whenever the demand flow rate exceeds the segment capacity.



traffic, or environmental conditions that are more restrictive than the base conditions. The base conditions for a two-lane highway include:

- Lane widths greater than or equal to 3.6 m
- Clear shoulders wider than or equal to 1.8 m
- No no-passing zones on the highway
- All passenger cars in the traffic stream (no heavy vehicles)
- No impediments to through traffic due to traffic control and turning vehicles, and
- Level terrain

For analysis of two-way traffic flow (i.e., analysis of both directions of travel combined), a 50/50 directional split of traffic is also considered part of the definition of base conditions.

5.1.5 Operational Analysis Procedures

The operational analysis procedures for two-lane roads consist of four specific applications:

- Analysis of two-way segments
- Analysis of directional segments
- Analysis of directional segments containing passing lanes in level and rolling terrain
- Analysis of directional segments containing climbing lanes on upgrades

Figure 30 illustrates a structure that is common to all four applications of the analysis procedure. The figure shows that separate estimates of ATS and PTSF are made and those estimates are then used in determining the LOS.

The analysis of two-way segments is the traditional method of analyzing operational conditions on two-lane highways and is analogous to the general terrain segment procedure in the existing HCM Chapter 8. This approach uses two-way demand volumes and directional split as input to the operational analysis procedures and estimates traffic performance measures for both directions of travel combined.

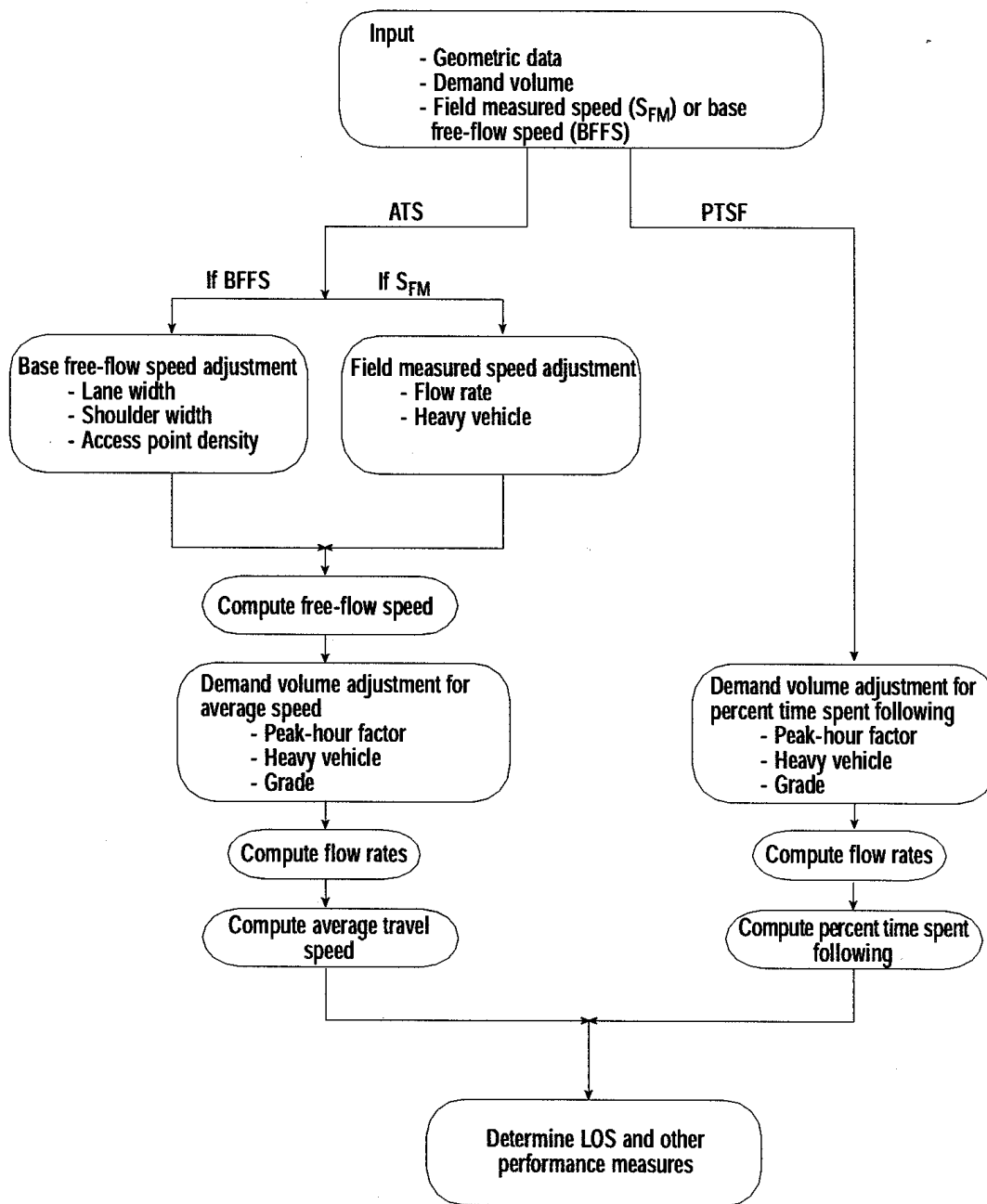


Figure 30. Flow Diagram of Two-Lane Highway Operational Analysis Methodology

A new feature of the revised chapter is the provision of an operational analysis procedure for directional segments. A directional segment is one direction of travel of a two-lane highway. This procedure allows each direction of travel on a two-lane highway to be analyzed separately, rather than combining them together as in the two-way segment procedure. Analysis of directional segments may provide greater insight into the operations on a two-lane highway than is possible if both directions of travel are combined. Although the segments in opposite directions of travel may be analyzed separately, the PTSF and ATS in one directional travel are influenced by the traffic volume in the other direction of travel. Unlike multilane highways or freeways, this interdependence of one direction on another is inherent in two-lane highway traffic operations. The directional segment procedure provides performance measures that can be combined with performance measures for multilane highways and freeways which, by their nature, are always analyzed on a directional basis.

The effect of grades and terrain on two-lane highway operations are represented by two factors—the grade adjustment factor and the heavy vehicle adjustment factor—that are defined later in this chapter. Both the two-way and directional segment analysis procedures address extended roadway segments in level and rolling terrain. Roadway segments on specific upgrades and specific downgrades are addressed only by the directional segment procedure, since grade effects are directional in nature. Level terrain generally includes grades of no more than 2 percent. Rolling terrain generally includes grades of short or medium length of no more than 4 percent. Roadway segments with substantial lengths of grade over 4 percent should be analyzed with the specific upgrade and specific downgrade procedures for directional segments. Any upgrade with a grade of 3 percent or more and a length of 0.4 km or more *may* be analyzed as a specific upgrade. Any upgrade with a grade of 3 percent or more and a length of 1.0 km or more *must* be analyzed as a specific upgrade. The directional segment in the opposite direction to a specific upgrade *must* be analyzed as a specific downgrade. Operations on specific downgrades differ from operations in level terrain only where truck types are present that, to avoid loss of control due to brake failure on the downgrade, must operate at speeds lower than they would in level terrain. Mountainous terrain is not considered in the two-way or directional segment procedures. Grades in mountainous terrain are so highly variable that they are better addressed with the specific upgrade and downgrade procedures. However, illustrative tables of ADT and flow rate thresholds for each LOS mountainous terrain, in comparison to level and rolling terrain, are provided later in this chapter of the report.

The directional segment procedure also provides an opportunity to evaluate the operational effects of passing lanes in level and rolling terrain and climbing lanes on upgrades which are inherently directional in nature. Both procedures require the analyst first to analyze the roadway as a directional segment without the presence of the passing or climbing lane. Then, the operational benefits of the added lane can be assessed.

The operational analysis procedure for a directional segment containing a passing lane in level or rolling terrain evaluates the effect of the passing lane on PTSF and ATS both within the passing lane itself and downstream of the passing lane. The procedure is intended to evaluate the situation in which the analysis segment includes the entire downstream

effective length of the passing lane. However, an alternative procedure is provided for the situation in which the analysis segment ends within the downstream effective length of the passing lane (e.g., because of the presence of a small town, a major intersection, or a change in demand volume).

The operational analysis procedure for directional segments containing climbing lanes on upgrades is similar to the procedure for passing lanes in level and rolling terrain except that the climbing lane procedure addresses only the effects of the climbing lane on PTSF and ATS within the climbing lane and does not address the effect of the climbing lane on the operation of the downstream roadway. Further research is needed to develop a full analysis procedure for portions of the analysis segment downstream of the climbing lane.

Each of the operational analysis procedures is presented later in this chapter of the report. Prior to the presentation of the operational analysis procedures themselves, several key elements common to all of the operational analysis procedures are reviewed.

FUNDAMENTAL RELATIONSHIPS

This section presents fundamental speed-flow and ATS-flow relationships form the basis of the operational analysis procedure.

5.2.1 Speed-Flow Relationships

The speed-flow relationships for two-lane highways developed in the research are similar in shape to the speed-flow curves presented in previous HCM editions (see Figures 1 and 2). However, a key difference from the 1985 HCM is the introduction of the concept of free-flow speed to characterize the quality of geometric design for a facility. The free-flow speed is defined as the average speed of traffic under low-volume conditions and is intended to represent the travel speeds that motorists choose when not impeded by other traffic. Free-flow speed is influenced by the alignment of the road (particularly horizontal curves) and by the roadway cross section (particularly lane and shoulder widths). In the revised operational analysis procedures, speed-flow relationships are represented by a family of parallel curves for differing free-flow speeds rather than a single speed-flow relationship.

While the concept of a family of speed-flow curves for different free-flow speeds did not appear in the 1985 HCM, there is ample precedent for the concept. The other uninterrupted flow procedures in the HCM—multilane highways and freeways—use the concept of free-flow speed and families of parallel speed-flow curves. A similar concept was used in the 1965 HCM based on average highway speed (the weighted average of design speeds within a segment) rather than free-flow speed (see Figure 1). Free-flow speed appeared more appropriate than design speed for use in this context because free-flow speed is based on actual operating conditions, while design speed is selected by a highway agency and may or may not bear a direct relationship to actual operating conditions.

Figure 31 illustrates some example scatter plots of speed-flow data from the field sites evaluated during the research. All of these plots represent two-way speed and flow data. After the speed data from field studies were used in calibrating and validating the TWOPAS simulation model, results from simulation runs were used in developing the speed-flow curves for base conditions. The measure of speed obtained from the model was ATS and the flow rates were the actual simulated flow rates, not the input values of flow rate which were slightly different. Speed and flow-rates were obtained for both a two-way flows and for each direction of travel separately. Based on the speed-flow plots in Figures 1, 2, and 31, and trial analysis of several alternatives, it was decided that the most appropriate functional form for speed-flow curves was a straight line for which the y-intercept represents the free-flow speed and a negative slope represents the decrease of speed with increasing flow rate. No functional form could be found which fit the available data better than a straight line.

The relationship between ATS and two-way flow rate was investigated by analysis of TWOPAS simulation results. Simulation runs were made for all combinations of the following conditions:

- 16 levels of two-way flow rate (200 to 3,200 veh/h in steps of 200 veh/h)
- 6 free flow speeds (60 to 110 km/h in steps of km/h)

For each condition studies, a total of 5 replicate simulation runs were made, so the overall total number of runs was $16 \times 6 \times 5$, or 480 simulation runs. Each of these runs was for the base conditions for two-way flow defined earlier in this chapter.

Regression analyses of ATS versus two-way flow rate were first performed for the data for each individual free-flow speed. In each case, the regression line was forced to have a y-intercept equal to the free-flow speed. The slopes of these regression lines ranged from -0.0081 to -0.0117 (km/h)/(pc/h). The R^2 values for these regression relationships ranged from 0.77 to 0.87, indicating that in these relationships, flow rate explains between 77 and 87 percent of the variation in free-flow speed. To obtain an overall common slope for all free-flow speeds combined, the data were translated by adding to each ATS value the difference between 110 km/h and the free-flow speed for that observation. These translated data provided the following regression relationship:

$$\text{ATS} = \text{FFS} - 0.0097 V \quad (3)$$

where:

ATS = average travel speed for both directions of travel combined (km/h)

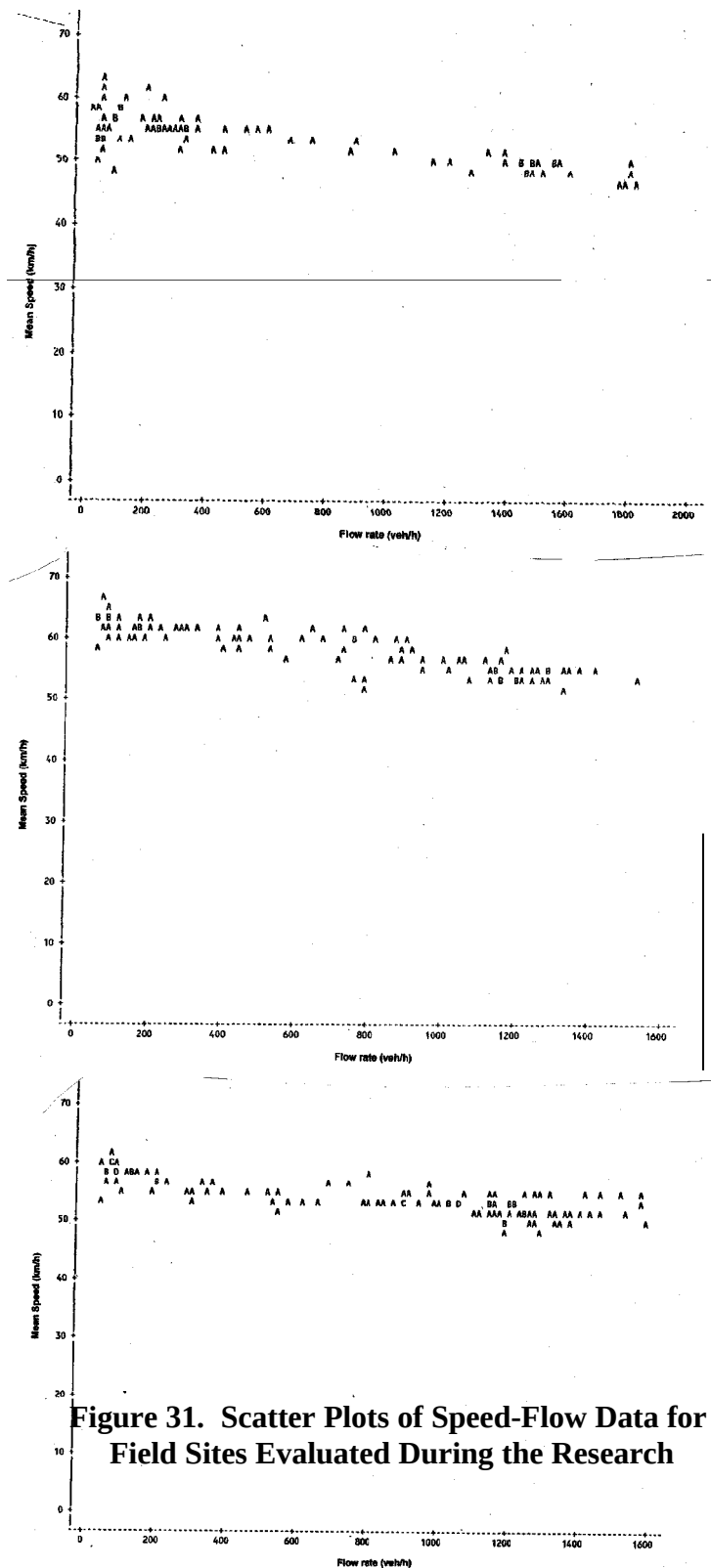


Figure 31. Scatter Plots of Speed-Flow Data for Field Sites Evaluated During the Research

FFS = free-flow speed (km/h)
V = passenger-car equivalent flow rate (pc/h)

This regression relationship for all free-flow speeds combined did not fit the data as well as the separate relationships for the individual free-flow speeds; R^2 for the combined relationship was 0.74.

A similar analysis was conducted to establish a relationship between ATS and directional flow rates. Simulation runs were made for all combinations of the following conditions:

- 10 directional flow rates (100, 200, 400, 600, 800, 1,000, 1,200, 1,400, 1,600, and 1,700 pc/h)
- 10 opposing flow rates (same as the directional flow rates)
- 2 free-flow speeds (80 and 110 km/h)

For each condition studies, a total of 5 replicate simulation runs were made, so the overall total number of runs made was $10 \times 10 \times 2 \times 5$, or 1,000 simulation runs. A regression analysis comparable to Equation (3), but including both the direction and opposing flow rates was developed as:

$$ATD_d = FFS_d - 0.1249 V' - 0.1247 V'' \quad (4)$$

where:

ATS_d = directional average travel speed (km/h)
 FFS_d = directional free-flow speed (km/h)
 V' = directional passenger-car equivalent flow rate (pc/h)
 V'' = passenger car equivalent flow rate for the opposing direction (pc/h)

This equation has an R^2 value of 0.588, indicating that flow rate explains approximately 59 percent of the variation in average travel speed.

The coefficients for same direction and opposing direction flow rates in Equation (4) were so similar that it was decided to use a common slope of -0.0125 (km/h)/(pc/h) for analysis of directional segments. Furthermore, to assure that the two-way and directional segment procedures provide comparable speed predictions, it was decided to use a slope of -0.0125 (km/h)/(pc/h) for the two-way segment procedure as well, rather than the slope of -0.0097 (km/h)/(pc/h) shown in Equation (3).

Figure 32 presents the speed-flow relationship used in the two-way segment and directional segment procedures. In the two-way segment procedure, the predicted ATS for any flow rate is determined by choosing a speed-flow line with a y-intercept equal to the free-flow speed of the segment (interpolation between the curves shown in the figure is

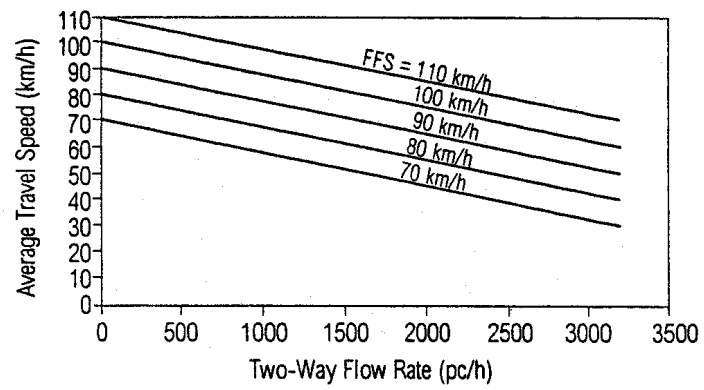


Figure 32. Speed-Flow Relationships Used in the Two-Way and Directional Segment Procedures

acceptable) and finding the speed on that line corresponding to the actual flow rate on the segment. This is equivalent to the following relationship:

$$ATS = FFS - 0.0125 v_p \quad (5)$$

where:

$$v_p = \text{passenger-car equivalent flow rate for peak 15-min period (pc/h)}$$

For a directional analysis, one chooses the y-intercept based on free-flow speed, moves to a lower free-flow speed line by adjusting for the effect of the directional flow rate for the opposing direction, and then moves along that speed-flow line to find the speed the speed corresponding to the direction flow rate. This is equivalent to the following sequence of equations:

$$S' = FFS_d - 0.0125 v_o \quad (6)$$

$$ATS_d = S' - 0.0125 v_d \quad (7)$$

where:

$$\begin{aligned} S' &= \text{average travel speed in the analysis direction after adjusting for} \\ &\quad \text{opposing traffic volume (km/h)} \\ v_d &= \text{passenger-car equivalent flow rate for the peak 15-min period in the} \\ &\quad \text{analysis direction (pc/h)} \\ v_o &= \text{passenger-car equivalent flow rate for the peak 15-min period in the} \\ &\quad \text{opposing direction (pc/h)} \end{aligned}$$

Equations (6) and (7) can be combined to obtain:

$$ATS_d = FFS_d - 0.125 (V_d + V_o) \quad (8)$$

which is the form of the speed-flow relationship for directional segments which is actually used in the HCM operational analysis procedures. The application of Figure 32 in the two-way and directional segment procedures is discussed further in a later portion of this chapter.

5.2.2 Percent Time Spent Following-Flow Relationships

Figure 33 illustrates a scatter plot of the PTSF-flow relationship from field data for one site evaluated in the research. This plot shows a characteristic shape that is similar to the percent time delay-flow curve used in the 1985 HCM, as shown in Figure 2.

Following calibration and validation of the TWOPAS model, simulation results were used to develop PTSF-flow relationships for base conditions. The data used to develop these relationships were the results of the same set of TWOPAS simulation model runs used to develop Figure 32. Based on a review of the field data and the model results, a number of functional forms for the model were considered including polynomial and reciprocal functions. However, none of these functional forms had an appropriate shape that both passed through the origin and asymptotically approached a PTSF of 100 percent. An exponential function that does have these features was found to be the most appropriate functional form to describe the PTSF-flow relationship for two-way segments:

$$\text{PTSF} = 100[1 - e^{-av_p}] \quad (9)$$

where:

PTSF = percent time spent following for both directions of travel combined
a = parameter to be determined

The model in this form that best fits the data for two-way segments is:

$$\text{PTSF} = 100[1 - e^{-0.000879v_p}] \quad (10)$$

This model for two-way segments, illustrated in Figure 34, is nearly identical to the percent time delay-flow relationship used in the 1985 HCM illustrated in Figure 2.

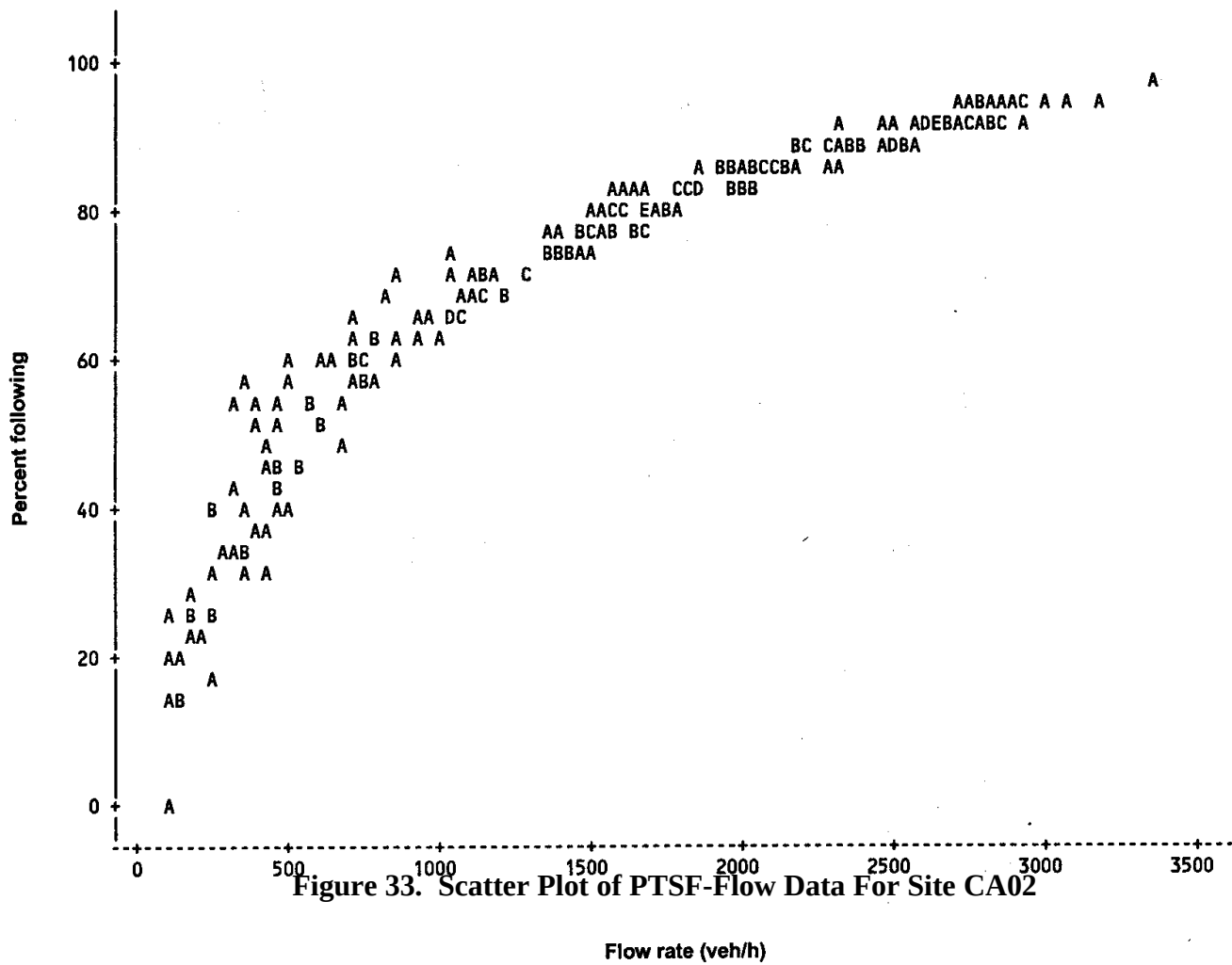
For directional segments, the shape of the relationship between PTSF and directional flow rate was found to vary as a function of the opposing-direction flow rate. A variation of Equation (9), with an added parameter to allow changes in shape, was found to provide the best representation for these data:

$$\text{PTSF}_d = 100[1 - e^{-av_d^b}] \quad (11)$$

where:

PTSF_d = percent time spent following for the analysis direction
a, b = parameters to be determined

Despite the added parameter, models in this form retain the property of passing through the origin and approaching a PTSF of 100 percent asymptotically. Figure 35 presents the PTSF-flow rate relationships for directional segments in the functional form shown in Equation (11). Table 28 presents the parameter values used in Equation (11), as a function of opposing-direction flow rate, to obtain the curves shown in Figure 35.



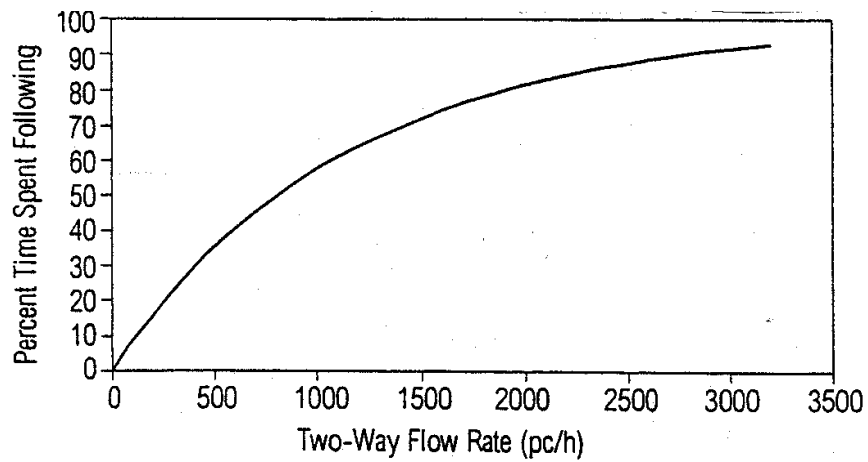


Figure 34. PTSF-Flow Relationship Used in the Two-Way Segment Procedure

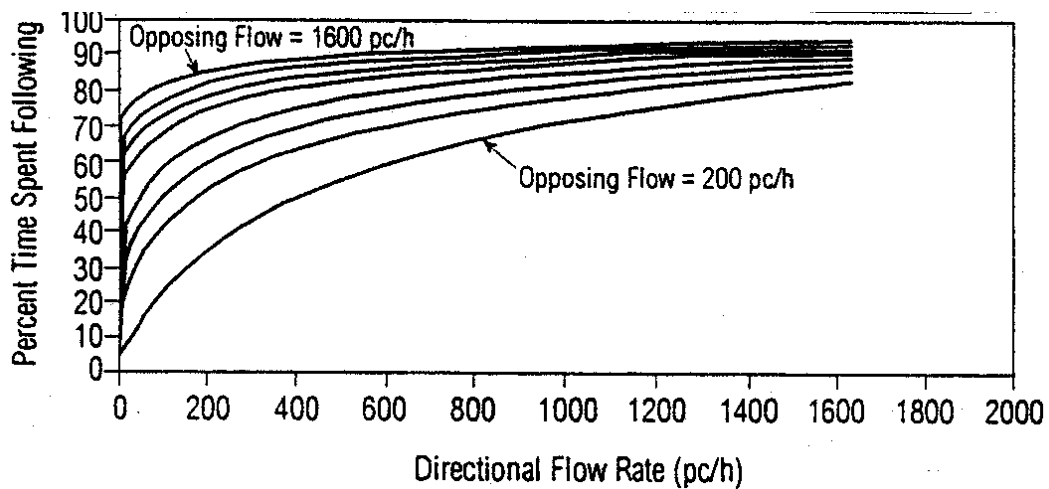


Figure 35. PTSF-Flow Relationships Used in the Directional Segment Procedure

Table 28. Values of Coefficients used in Estimating Percent Time Spent Following for Directional Segments

Opposing demand flow rate V_o (pc/h)	a	b
200	-0.013	0.668
400	-0.057	0.479
600	-0.100	0.413
800	-0.173	0.349
1000	-0.320	0.276
1200	-0.430	0.242
1400	-0.522	0.225
1600	-0.665	0.199

The application of Figures 34 and 35 in the two-way and directional segment procedures is discussed further in later portion of this chapter.

CAPACITY

The 1985 HCM estimated the capacity of a two-lane highway for both directions of travel combined as ranging from 2,000 to 2,800 pc/h as a function of directional split. Data collected as part of this research and other data from highway agencies suggest that the capacity of a two-lane road can be higher than the upper limit of 2,800 pc/h used in the 1985 HCM.

A capacity value cited in the HCM should not represent the highest flow rate ever observed on a given facility type but, rather, should represent a value that can reasonably be expected to be served on most facilities under base conditions when demand is sufficient. Capacity conditions on two-lane roads are very difficult to observe because very few two-lane highways operate at or near capacity. Two-lane highways with demand volumes sufficient to approach or exceed capacity are rarely observed. Usually, a two-lane highway with high volume is widened to four lanes long before the demand approaches capacity. Thus, the capacity of two-lane highways must generally be inferred by observing the highest volume roadways that have not reached capacity, rather than by studying capacity conditions themselves.

Field data for Site CA02 collected as part of this research showed that two-way flow rates as high as 3,200 pc/h occur regularly at this site. Table 29 presents the data for the highest-volume 15-min periods recorded at this site.

Table 29. Traffic Data for Highest Flow Periods at Site CA02

Date	Time ^a	Flow rate		Percent flow in peak direction	Percent trucks	Mean speed ^c (mph)
		vph	pcph ^b			
11/13/96	1615	2,732	2,808	50.4	2.8	48.1
11/13/96	1645	2,836	2,908	59.7	2.5	48.1
11/13/96	1700	2,880	2,940	53.3	2.1	47.6
11/13/96	1715	3,108	3,196	55.6	2.8	38.8
11/13/96	1730	2,880	2,952	57.2	2.5	44.6
11/15/96	0715	2,772	2,884	59.5	4.0	44.3
11/15/96	0730	2,796	2,924	59.2	4.6	42.2
11/15/96	0745	2,904	3,016	57.0	3.8	45.2
6/2/97	1700	2,832	2,928	52.7	3.4	45.9
6/2/97	1715	3,352	3,460	57.4	3.2	45.1
6/3/97	0715	2,756	2,848	59.8	3.3	44.8
6/3/97	0730	2,736	2,812	51.4	2.8	45.1
6/3/97	0745	2,720	2,808	54.4	3.2	43.8
6/3/97	1645	2,816	2,888	51.7	2.6	46.6
6/3/97	1700	2,780	2,896	54.8	4.2	46.3
6/3/97	1715	3,064	3,100	54.8	1.2	47.9
6/3/97	1730	2,788	2,864	53.1	2.7	43.5
6/4/97	1645	2,764	2,868	51.8	3.8	47.7
6/4/97	1700	2,868	2,964	53.4	3.4	46.8
6/4/97	1715	3,172	3,232	52.6	1.9	46.8
6/4/97	1730	2,796	2,852	54.3	2.0	48.2
6/4/97	1745	2,860	2,980	53.4	4.2	47.8
6/5/97	0715	2,920	3,068	59.9	5.1	45.8
6/5/97	0730	2,900	2,996	57.5	3.3	45.5
6/5/97	0745	2,996	3,140	55.8	4.8	46.0
6/5/97	1645	2,840	2,912	53.7	2.5	48.0
6/5/97	1700	2,872	2,952	52.9	2.8	46.7
6/5/97	1715	2,928	3,000	54.8	2.5	47.7
6/5/97	1730	2,932	3,000	53.1	2.3	47.4
6/5/97	1745	2,736	2,816	54.8	2.9	47.9
6/6/97	0715	2,816	2,916	55.5	3.6	47.9
6/6/97	0730	2,928	3,032	55.5	3.6	47.5

^a Start time of 15-min period based on 24-hr clock.

^b Based on assumed passenger car equivalent value of 2.0 for trucks.

^c Mean spot speed for both directions of travel combined.

Table 30 is taken from HCM2000 Chapter 8 (Traffic Characteristics). The table shows maximum reported volumes on rural two-lane highways in the U.S. and Canada. Note that the volumes in the table are in veh/h and, thus, have not been adjusted for any heavy vehicles that might be present in the traffic stream. Flow rates adjusted to incorporate the passenger-car equivalents (PCEs) of heavy vehicles would necessarily be higher than those shown in the table. The final entry in the upper portion of Table 30 is Site CA02 discussed above, based on the field data obtained in this study.

Furthermore, HCM2000 Chapter 8 states that observations on two-lane, two-way rural highways in Europe and other parts of the world have been reported at even higher volumes. Volumes of more than 2,700 veh/h have been observed in Denmark, more than 2,800 veh/h in France, more than 3,000 veh/h in Japan, and more than 2,450 veh/h in Norway. Some of these volumes have contained significant numbers of trucks, in some cases as high as 30 percent of the traffic stream.

Based on all of the available data, the research team concluded that the most appropriate estimate of capacity for two-way flow on a two-lane highway is 3,200 pc/h. The data for Site CA02 demonstrate that the capacity for a two-lane highway must be at least 3,200 pc/h. Furthermore, this appears to be a conservative choice based on the data for several of the sites shown in Table 30 which can be inferred to exceed 3,200 pc/h if adjustments for heavy vehicles are made.

A review of directional flow rates on higher-volume two-lane highways led to a choice of 1,700 veh/h as the capacity for one direction of travel by itself. While directional flow rates above 1,700 pc/h were observed in some cases, these did not appear sufficient to justify the value of 2,000 pc/h used in the 1985 HCM.

The revised capacity values of 3,200 pc/h for two-way flow and 1,700 pc/h for directional flow indicate that capacity is much less influenced by directional split than was suggested by the 1985 HCM. If there were no influence of directional split, the capacity for two-way flow would be twice the capacity for directional flow, or 3,400 pc/h. The recommended two-way capacity value is less than twice the directional capacity value, and this 200 pc/h difference between 3,200 and 3,400 pc/h represents the influence of directional split on capacity.

In the operational analysis procedure for two-lane highways, LOS F applied whenever the demand flow rate exceeds capacity. Both the two-way and directional flow rates and capacities must be compared. Other than this criterion for defining LOS F, capacity has no direct role in determining LOS.

The HCM chapters for other uninterrupted flow facilities (freeways and multilane highways) show capacity varying with free-flow speed. There are little to no field data that demonstrate such a variation for freeways and multilane highways, but a variation of capacity with free-flow was introduced because the service measure for these facility types is density and, if capacity is decreased with free-flow speed, then capacity can be represented on a speed-flow plot as a line of constant density. For two-lane highways, there are no field data indicating that capacity increases with free-flow speed. Since density is not the service measure for two-lane highways, there is also no theoretical reason

Table 30. Reported Maximum Volumes on Selected Two-Lane Rural Highways

Location	Total volume (veh/h)	Peak direction volume (veh/h)	Non-peak direction volume (veh/h)
Highways			
Madera-Olsen Rd., Simi Valley, California	3,107	1,651	1,456
Madera-Olsen Rd., Simi Valley, California	3,027	1,839	1,188
Hwy. 1, Banff, Alberta, Canada	2,450	—	—
Hwy. 35/115, Kirby, Ontario, Canada	2,250	—	—
Wasatch Blvd., Salt Lake City, Utah	2,198	1,504	694
Hwy. 35, Kirby, Ontario, Canada	2,050	—	—
U.S. 50, Lake Tahoe, California	1,796	1,386	410
NJ 50, Cape May Co., New Jersey	1,714	1,445	269
Hwy. 1, Banff-Yoho, Alberta-British Columbia, Canada	1,517	—	—
Hwy. 4, Contra Costa County, California	3,350	1,920	1,430
Bridges/Tunnels			
U.S. 158, Nags Head, North Carolina	3,195	—	—
Midtown Tunnel, Norfolk/Portsmouth, Virginia	2,920	1,827	1,093
Sagamore Bridge, Hudson, New Hampshire	2,701	—	—
TH 15, St. Cloud, Minnesota	2,242	1,146	1,096
Underwood Bridge, Hampton, New Hampshire	1,960	1,041	919
Staley Viaduct, Decatur, Illinois	1,919	971	948

Source: HCQS Survey and Federal Highway Administration

to suppose that capacity should decrease along a line of constant density. Therefore, given a lack of evidence to the contrary, the recommended capacity values for two-lane highways are assumed to be unaffected by free-flow speed.

FREE-FLOW SPEED

As explained above, the quality of geometric design and speed environment on a two-lane highway is represented in the operational analysis procedure by the free-flow speed of traffic. When no restrictive geometrics are present and adjacent development is minimal, free-flow traffic speeds on a two-lane highway may approach or exceed 110 km/h. However, sharp horizontal curves, narrow lanes and shoulders, and adjacent development may lead motorists to travel at substantially lower speeds.

Two general methods can be used to determine the free-flow speed for a two-lane highway: (a) field measurement and (b) estimation with guidelines provided in the operational analysis procedure. The field measurement procedure is provided for those who prefer to gather these data directly or to utilize measurements from an existing speed monitoring program. However, field measurements are not necessary to perform an operational analysis. The free-flow speed can be estimated with guidelines based on field data and user knowledge of conditions on the highway segment in question.

5.4.1 Field Measurement of Free-Flow Speed

The free-flow speed of a highway can be determined directly from a speed study conducted in the field. If field measured data are used, no subsequent adjustments are made to free-flow speed. The speed study should be conducted at a representative location within the highway segment being evaluated; for example, a site on an upgrade should not be selected within a segment that is generally level. Any speed measurement technique that has been found acceptable for other types of traffic engineering speed studies may be used. It is recommended that the field study be conducted in periods of low traffic flow (up to a two-way flow of 200 pc/h), in contrast to free-flow speeds for freeways and multilane highways that can be conducted at flow rates up to 1,400 pc/h. This difference arises from the differences between the shape of the basic speed-flow curves for two-lane highways and the analogous curves for multilane highways and freeways. Off-peak hours are generally good times to observe low flow rates. The speed study should measure the speeds of all vehicles or a systematic sample (e.g., every 10th vehicle). The speed study should not only measure speeds for unimpeded vehicles but should also include a representative sample of impeded vehicles. A sample of at least 100 vehicle speeds should be obtained. Further guidance on the conduct of speed studies is found in standard traffic engineering publications such as the *Manual of Traffic Engineering Studies* (39).

If the speed study must be conducted at a two-way flow rate of more than 200 pc/h, the free-flow speed can be found by using the speed-flow relationships in Figure 32, assuming that data on traffic volumes were taken at the same time. The free-flow speed can be computed based on field data as:

$$FFS = S_{FM} + 0.0125 \frac{v_f}{f_{HV}} \quad (12)$$

where:

FFS	=	estimated free-flow speed (km/h)
S_{FM}	=	mean speed of traffic measured in the field (km/h)
v_f	=	observed flow rate for the period when field data were obtained (veh/h)
f_{HV}	=	heavy-vehicle adjustment factor [defined later in this chapter]

If field measurement of the roadway under study is not feasible, data taken at a similar facility may be used. The surrogate roadway should be similar with respect to variables that are identified in this chapter as affecting free-flow speed.

Highway agencies with ongoing speed-monitoring programs or with existing speed data on file may prefer to use those data rather than conduct a new speed study or use an indirect speed estimation technique. Such data can be used directly if collected in accordance with the procedures presented above.

Estimation from Free-Flow Speed for Base Conditions

The free-flow speed can be estimated indirectly when field data are not available. The free-flow speed is estimated as:

$$FFS = BFFS - f_{LS} - f_A \quad (13)$$

where:

FFS	=	estimated free-flow speed (km/h)
BFFS	=	estimated free-flow speed (km/h) for base conditions
f_{LS}	=	adjustment for lane width and shoulder width (see below)
f_A	=	adjustment for access points (see below)

Adjustment for Lane and Shoulder Width: The first adjustment that is used to modify the estimated free-flow speed relates to the effects of lane and shoulder widths. Base conditions for a two-lane highway consist of 3.6-m lane widths and 1.8 shoulder widths. Table 57 presents the adjustment to modify the estimated free-flow speed for narrower lanes and shoulders. The data in Table 31 indicate, for example, that a two-lane highway with 3.3-m lanes and full shoulders has free-flow speeds that are 0.7 km/h lower than a highway with ideal lane and shoulder widths. Similarly, a two-lane highway with 3.6-m lanes and 0.6-m shoulders has free-flow speed that are 4.2 km/h less than a highway with ideal lane and shoulder width. Exhibit 20-4 in Appendix E has been adapted from Table 31.

Table 31. Reduction in Free-Flow or Desired Speed Due to Lane Width and Shoulder Width (f_{LS})

Lane width (m)	Reduction in free-flow speed (km/h)			
	Shoulder width (m)			
	0.0	0.6	1.2	1.8
2.7	10.3	7.7	5.6	3.5
3.0	8.5	5.9	3.8	1.7
3.3	7.5	4.9	2.8	0.7
3.6	6.8	4.2	2.1	0.0

The data in Table 31 were derived from a field study of the traffic operational effects of shoulder width transitions and an engineering evaluation of lane width effects. Traffic operational field data on the effects of shoulder width were collected upstream and downstream of shoulder width transitions on two-lane highways in California and Missouri. Table 32 summarizes the field data that were collected. The data in Table 32 were used to develop a regression relationship between shoulder width reduction and speed reduction with a slope of 3.78 (km/h)/m. This regression relationship had an R^2 value of 0.89 indicating that shoulder width explains 89 percent of the variation in speed reduction. The shoulder width effect shown in Table 33 was established from this regression relationship.

No suitable field sites could be found for evaluation of lane width transitions in a manner comparable to the evaluation of shoulder width transitions discussed above. Therefore, the lane width effect shown in Table 34 was derived from the lane width factors used in the 1985 HCM procedure for two-lane highways. The values in Table 34 were found to be quite close to the lane width effect reported in Australian work by McLean (40).

Table 31 is simply an additive combination of Tables 33 and 34. No interaction between lane and shoulder width is assumed in their effect on speed because there is no evidence for (or against) the existence of such an interaction.

Adjustment Factors for Access Point Density: Table 35 presents the adjustment for various levels of access-point density. The data indicate that each access point per kilometer decreases the estimated free-flow speed by 0.417 km/h. The access point density is found by dividing the total number of intersections and driveways within the roadway segment (including access points on both sides of the roadway) by the length of the segment in kilometers. An intersection or driveway should only be included by the analyst if it is considered to have influence on traffic flow. Access points that are unnoticed by the driver or those with little activity should not be included.

Table 32. Observed Speed Differences at Lane and Shoulder Width Transitions

Site	Number of 15-min periods	Wider cross section				Narrower cross section				Speed difference		Number of vehicle speeds measured ^c
		Lane width (m)	Shoulder width (m)	Mean speed (km/h) ^a	85th percentile speed (km/h) ^b	Lane width (m)	Shoulder width (m)	Mean speed (km/h)	85th percentile speed (km/h)	Mean speed (km/h)	85th percentile speed (km/h)	
CA01 ^d	15	3.6	2.4	98.0	106.8	3.6	1.2	95.1	103.3	2.9	3.5	1,467
CA03 ^d	25	3.8	2.4	93.8	103.1	3.5	0.6	88.8	99.4	5.0	3.7	1,156
MO01 (SB) ^d	24	3.5	2.6	90.4	100.9	3.5	0.2	83.5	91.6	6.9	9.3	1,293
MO01 (NB) ^e	24	3.5	2.6	90.6	101.2	3.5	0.2	85.1	94.8	5.5	6.4	1,061

^a Average of 15-min mean speeds.

^b Average of 15-min 85th percentile speeds.

^c Number of vehicles measured at furthest upstream location.

^d Speed data for traffic moving from the wider to the narrower cross section.

^e Speed data for traffic moving from the narrower to the wider cross section.

Table 33. Reduction in Mean Speed Due to Narrow Shoulders

Shoulder width (m)	Reduction in mean speed (km/h)
1.8	0.0
1.2	2.1
0.6	4.2
0.0	6.8

Table 34. Reduction in Mean Speed Due to Narrow Lanes

Lane width (m)	Reduction in mean speed (km/h)
3.6	0.0
3.3	0.7
3.0	1.7
2.7	3.5

Table 35. Adjustments for Access Point Density

Access points per km	Reduction in free-flow speed (km/h)
0	0.0
6	2.5
12	5.0
18	7.5
24 or more	10.0

Table 35 is based on a comparison of speeds and access point densities for four sites in the Florida Keys (site FL01 through FL04). Two of the four sites are relatively underdeveloped with about 0.5 access points per km. The other two sites are more heavily developed with an access point density of about 16 access points per km. A regression analysis of the speed and access point data for these sites shows a slope of 0.44 (km/h)/(access point/km). The R^2 value for this regression relationship is 0.58 which indicates that the access point density explains 58 percent of the variability in speed. This relatively low R^2 value is not surprising given the limited sample size (only 4 sites) and the number of other factors that affect speed.

The slope of 0.44 (km/h)/(access point/km) is so close to the slope of 0.417 (km/h)/(access point/km) used in the operational analysis procedure for multilane highways that a decision was reached to use that same effect for both multilane and two-lane highways. Therefore, both Table 30 and Exhibit 20-5 in the HCM 2000 chapter on two-lane highways are based on a slope of 0.417 (km/h)/(access point/km) and are identical to the comparable table that appears in the multilane chapter of the HCM 2000.

Other Considerations in Estimating Free-Flow Speed: If a segment contains sharp horizontal curves with design speeds substantially below the rest of the segment, it may be desirable to determine the free-flow speed separately for curves and tangents and compute a weighted average free-flow speed for the segment as a whole.

The data on which the free-flow speed relationships developed for the HCM chapter are based include both commuter and non-commuter traffic conditions. No significant differences between the two were detected. However, it is frequently held that commuters or other frequent users of a highway will use the facility more efficiently than do recreational users or other occasional drivers. If the effect of a particular driver population is a concern, the analyst is encouraged to measure free-flow speed in the field. If field measurements cannot be made, the analyst may select a free-flow speed that reflects the anticipated effect of the particular driver population. Care should be taken not to underestimate the base free-flow speed of a highway by overstating the effect of a given driver population.

DEMAND FLOW RATES

Three adjustments must be made to hourly demand volumes, whether based on traffic counts or estimates, to arrive at the equivalent passenger-car flow rate used in LOS analysis. These adjustments are the peak-hour factor, the grade adjustment factor, and the heavy-vehicle adjustment factor. These adjustments are applied in the following manner for a two-way segment:

$$V_p = \frac{V}{(PHF)(f_G)(f_{HV})} \quad (14)$$

where:

v_p	=	passenger-car equivalent flow rate for the peak 15-min period (pc/h)
V	=	demand volume for the full peak hour (veh/h)
PHF	=	peak-hour factor
f_G	=	grade adjustment factor
f_{HV}	=	heavy-vehicle adjustment factor

The same adjustments can be applied in a similar manner for the analysis direction and the opposing direction of a directional segment. Each of these adjustments used in obtaining demand flow rates is discussed below.

5.5.1 Peak Hour Factor

The peak hour factor (PHF) represents the temporal variation in traffic flow within the hour. All HCM analyses are based on demand volumes for a peak 15-minute period within the hour of interest, which is usually the peak hour. To utilize the operational analysis procedures, full hour demand volumes must be converted to flow rates for the peak 15 min, as shown in Equation (14).

Where the PHF can be determined from local field data, this should be done. Where field data are not available, the PHF can generally be estimated as 0.88 in rural areas and 0.92 in urban areas.

5.5.2 Grade Adjustment Factor

The grade adjustment factor, f_G , is used to account for the effect of the terrain on ATS and PTSF. The grade adjustment factor represents the difference in ATS or PTSF between a traffic stream composed of 100 percent passenger cars in level terrain and a traffic stream with the same volume in rolling terrain or on a specific grade.

An empirical approach was taken to the quantification of f_G . In this empirical approach, the values of the factors were based directly on the shapes of the fundamental speed-flow and PTSF-flow curves shown in Figures 32 and 34. In general, f_G is computed as the ratio of the flow rate for a specified traffic stream of 100 percent passenger cars in level terrain to the higher flow rate in level terrain that would result in the same value of ATS or PTSF observed for that original specified traffic stream in non-level terrain or on a specific grade. The values of f_G for ATS and PTSF, respectively, are computed as shown in the following equations:

$$f_G = v/[v + (ATS_{pc/grade} - ATS_{pc/level})/-0.0125] \quad (15)$$

$$f_G = v/[v + (PTSF_{pc/grade} - PTSF_{pc/level})/(0.0879e^{-0.000879v})] \quad (16)$$

where:

v	=	flow rate (veh/h)
$ATS_{pc/level}$	=	average travel speed for a flow composed of 100 percent passenger cars in level terrain (km/h)
$ATS_{pc/grade}$	=	average travel speed for a flow composed of 100 percent passenger cars in non-level terrain or on a specific grade (km/h)
$PTSF_{pc/level}$	=	percent time spent following for a flow composed of 100 percent passenger cars in level terrain
$PTSF_{pc/grade}$	=	percent time spent following for a flow composed of 100 percent passenger cars in non-level terrain or on a specific grade

Equation (15) incorporates the slope of the speed-flow relationship $[-0.0125 \text{ (km/h)/(veh/h)}]$ shown in Figure 32, which does not vary with flow rate. Equation (16) incorporates the slope of the PTSF-flow relationship as represented by the first derivative of the PTSF-flow curve shown in Figure 34 evaluated at the flow rate being analyzed.

The values of f_G used in the HCM2000 operational analysis procedure were determined from the results of simulation runs with the TWOPAS model. Simulation runs were made for traffic streams composed of 100 percent passenger cars with a 50/50 directional distribution of traffic. The conditions that were varied in this evaluation were:

- 42 types of terrain (level, rolling, and 40 combinations of specific percentages and lengths of grade)
- 3 traffic volume levels (400, 800, and 1,600 veh/h for both directions of travel combined)

For level and rolling terrain a 10 km (6.2 mi) roadway segment was simulated. For specific grades, the percent grades considered were 3, 4, 5, 6, and 7 percent and the lengths of grades considered were 0.4, 0.8, 1.2, 1.6, 2.4, 3.2, 4.8, and 6.4 km (0.25, 0.50, 0.75, 1.0, 1.5, 2.0, 3.0, and 4.0 mi). The length of roadway simulated for each specific grade was the length of the grade plus a buffer length of 1.0 km (0.6 mi) at each end. For each combination investigated, a total of five replicate simulation runs with different random number seeds were made. Thus, the grade adjustment factors are based on a total of 630 simulation runs (42 x 3 x 5).

Table 36 shows the derivation of the grade adjustment factors for both ATS and PTSF for level and rolling terrain. Tables 37 and 38 show comparable data for grade adjustment factors for 40 combinations of specific grades (5 levels of percent grade x 8 levels of length of grade) for ATS and PTSF, respectively. Most simulation results obtained from the TWOPAS model could be used directly, but the simulation results were, in some cases, found to be highly variable. The tables indicate that values of f_G for ATS generally decrease with increasing percent grade, decrease with increasing length of grade, and increase with increasing flow rate. By contrast, values of f_G for PTSF increase with increasing percent grade, increase with increasing length of grade, and decrease with

increasing flow rate. Where found to be necessary, selected simulation results were discarded and replaced by interpolated values of ATS or PTSF from simulation runs in the neighboring cells to assure that values of f_G increased or decreased monotonically in the expected direction throughout the entire table.

Table 36. Computation of Grade Adjustment Factor (f_G) for Level and Rolling Terrain

Two-way flow rate (pc/h)	Directional flow rate (pc/h)	ATS (km/h)			PTSF		
		(pc/level)	pc/grade)	f _G	pc/level	pc/grade	f _G
Level Terrain							
0 - 600	0 - 300	101.6	101.6	1.00	29.6	29.6	1.00
> 600 - 1,200	> 300 - 600	94.1	94.1	1.00	60.6	60.6	1.00
> 1,200	> 600	86.4	86.4	1.00	86.4	86.4	1.00
Rolling Terrain							
0 - 600	0 - 300	101.6	99.6	0.71	29.6	37.2	0.77
> 600 - 1,200	> 300 - 600	94.1	93.4	0.93	60.6	62.7	0.94
> 1,200	> 600	86.4	86.1	0.99	82.0	84.1	1.00

Note: Values of f_G are computed with Equation (15) for ATS and with Equation (16) for PTSF.

Table 37. Computation of Grade Adjustment Factor (f_G) for Average Travel Speed on Specific Grades

Grade (%)	Length (km)	0 - 300 pc/h			> 300 - 600 pc/h			> 600 pc/h		
		ATS (km/h)		f_G	ATS (km/h)		f_G	ATS (km/h)		f_G
		pc/level	pc/grade		pc/level	pc/grade		pc/level	pc/grade	
3	0.4	101.6	100.4	0.81	94.1	94.1	1.00	86.4	86.4	1.00
	0.8	101.6	100.3	0.79	94.1	94.1	1.00	86.4	86.4	1.00
	1.2	101.6	100.1	0.77	94.1	94.1	1.00	86.4	86.4	1.00
	1.6	101.6	100.0	0.76	94.1	94.1	1.00	86.4	86.4	1.00
	2.4	101.6	99.9	0.75	94.1	94.0	0.99	86.4	86.4	1.00
	3.2	101.6	99.9	0.75	94.1	93.8	0.97	86.4	86.3	1.00
	4.8	101.6	99.9	0.75	94.1	93.6	0.95	86.4	85.8	0.97
	6.4	101.6	99.9	0.75	94.1	93.5	0.94	86.4	85.4	0.95
4	0.4	101.6	100.3	0.79	94.1	94.1	1.00	86.4	86.4	1.00
	0.8	101.6	100.0	0.76	94.1	94.1	1.00	86.4	86.4	1.00
	1.2	101.6	99.7	0.72	94.1	94.1	1.00	86.4	86.4	1.00
	1.6	101.6	99.4	0.69	94.1	93.3	0.93	86.4	86.4	1.00
	2.4	101.6	99.2	0.68	94.1	93.2	0.92	86.4	86.4	1.00
	3.2	101.6	99.0	0.66	94.1	93.1	0.91	86.4	86.3	1.00
	4.8	101.6	99.0	0.65	94.1	93.1	0.91	86.4	85.6	0.96
	6.4	101.6	98.9	0.65	94.1	93.0	0.90	86.4	85.5	0.96
5	0.4	101.6	99.9	0.75	94.1	94.1	1.00	86.4	86.4	1.00
	0.8	101.6	98.9	0.65	94.1	93.4	0.93	86.4	86.4	1.00
	1.2	101.6	98.3	0.60	94.1	92.9	0.89	86.4	86.4	1.00
	1.6	101.6	98.1	0.59	94.1	92.8	0.89	86.4	86.4	1.00
	2.4	101.6	97.8	0.57	94.1	92.5	0.86	86.4	86.2	0.99
	3.2	101.6	97.7	0.56	94.1	92.4	0.85	86.4	86.0	0.98
	4.8	101.6	97.6	0.56	94.1	92.2	0.84	86.4	85.7	0.97
	6.4	101.6	97.5	0.55	94.1	91.9	0.82	86.4	84.9	0.93
6	0.4	101.6	98.7	0.63	94.1	93.1	0.91	86.4	86.4	1.00
	0.8	101.6	97.8	0.57	94.1	92.3	0.85	86.4	86.1	0.99
	1.2	101.6	96.9	0.52	94.1	92.0	0.83	86.4	85.8	0.97
	1.6	101.6	96.8	0.51	94.1	91.4	0.79	86.4	85.7	0.97
	2.4	101.6	96.3	0.49	94.1	91.3	0.78	86.4	85.3	0.95
	3.2	101.6	96.1	0.48	94.1	91.2	0.78	86.4	85.2	0.94
	4.8	101.6	95.8	0.46	94.1	91.0	0.76	86.4	85.0	0.93
	6.4	101.6	95.6	0.45	94.1	90.9	0.76	86.4	84.9	0.93
7	0.4	101.6	98.1	0.59	94.1	92.5	0.86	86.4	86.0	0.98
	0.8	101.6	96.1	0.48	94.1	91.0	0.76	86.4	85.1	0.94
	1.2	101.6	95.3	0.44	94.1	90.5	0.74	86.4	84.5	0.91
	1.6	101.6	94.5	0.41	94.1	89.9	0.70	86.4	84.4	0.91
	2.4	101.6	94.2	0.40	94.1	89.2	0.67	86.4	84.3	0.91
	3.2	101.6	93.9	0.39	94.1	89.1	0.67	86.4	83.8	0.89
	4.8	101.6	93.7	0.39	94.1	89.0	0.66	86.4	83.6	0.88
	6.4	101.6	93.4	0.38	94.1	88.9	0.66	86.4	83.4	0.87

Note: The flow rates used in this table are for the analysis direction of a directional segment. Values of f_G are computed with Equation (15).

Table 38. Computation of Grade Adjustment Factor (f_G) for Percent Time Spent Following on Specific Grades

Grade (%)	Length (km)	0 - 300 pc/h			> 300 - 600 pc/h			> 1,200 veh/h		
		PTSF		f_G	PTSF		f_G	PTSF		f_G
		pc/level	pc/grade		pc/level	pc/grade		pc/level	pc/grade	
3	0.4	28.4	28.4	1.00	54.2	57.4	0.92	76.4	79.2	0.92
	0.8	28.4	28.4	1.00	54.5	57.1	0.93	76.9	79.3	0.93
	1.2	28.4	28.4	1.00	54.9	57.5	0.93	77.3	79.7	0.93
	1.6	28.4	28.4	1.00	55.3	57.9	0.93	77.7	80.1	0.93
	2.4	28.4	28.4	1.00	56.1	58.5	0.94	78.5	80.6	0.94
	3.2	28.4	28.4	1.00	56.9	58.9	0.95	79.3	81.0	0.95
	4.8	28.4	28.4	1.00	58.3	59.2	0.97	80.5	82.0	0.96
	6.4	28.4	28.4	1.00	59.5	59.5	1.00	81.6	82.6	0.97
4	0.4	28.4	28.4	1.00	54.2	56.5	0.94	76.4	79.2	0.92
	0.8	28.4	28.4	1.00	54.5	55.5	0.97	76.9	78.2	0.96
	1.2	28.4	28.4	1.00	54.9	55.9	0.97	77.3	78.6	0.96
	1.6	28.4	28.4	1.00	55.3	56.7	0.96	77.7	78.6	0.97
	2.4	28.4	28.4	1.00	56.1	57.4	0.97	78.5	79.6	0.97
	3.2	28.4	28.4	1.00	56.9	57.6	0.98	79.3	79.6	0.99
	4.8	28.4	28.4	1.00	58.3	58.3	1.00	80.5	80.5	1.00
	6.4	28.4	28.4	1.00	59.5	59.5	1.00	81.6	81.6	1.00
5	0.4	28.4	28.4	1.00	54.2	54.2	1.00	76.4	77.6	0.97
	0.8	28.4	28.4	1.00	54.5	54.5	1.00	76.9	76.9	1.00
	1.2	28.4	28.4	1.00	54.9	54.9	1.00	77.3	77.3	1.00
	1.6	28.4	28.4	1.00	55.3	55.3	1.00	77.7	77.7	1.00
	2.4	28.4	28.4	1.00	56.1	56.1	1.00	78.5	78.5	1.00
	3.2	28.4	28.4	1.00	56.9	56.9	1.00	79.3	79.3	1.00
	4.8	28.4	28.4	1.00	58.3	58.3	1.00	80.5	80.5	1.00
	6.4	28.4	28.4	1.00	59.5	59.5	1.00	81.6	81.6	1.00
6	0.4	28.4	28.4	1.00	54.2	54.2	1.00	76.4	76.4	1.00
	0.8	28.4	28.4	1.00	54.5	54.5	1.00	76.9	76.9	1.00
	1.2	28.4	28.4	1.00	54.9	54.9	1.00	77.3	77.3	1.00
	1.6	28.4	28.4	1.00	55.3	55.3	1.00	77.7	77.7	1.00
	2.4	28.4	28.4	1.00	56.1	56.1	1.00	78.5	78.5	1.00
	3.2	28.4	28.4	1.00	56.9	56.9	1.00	79.3	79.3	1.00
	4.8	28.4	28.4	1.00	58.3	58.3	1.00	80.5	80.5	1.00
	6.4	28.4	28.4	1.00	59.5	59.5	1.00	81.6	81.6	1.00
7	0.4	28.4	28.4	1.00	54.2	54.2	1.00	76.4	76.4	1.00
	0.8	28.4	28.4	1.00	54.5	54.5	1.00	76.9	76.9	1.00
	1.2	28.4	28.4	1.00	54.9	54.9	1.00	77.3	77.3	1.00
	1.6	28.4	28.4	1.00	55.3	55.3	1.00	77.7	77.7	1.00
	2.4	28.4	28.4	1.00	56.1	56.1	1.00	78.5	78.5	1.00
	3.2	28.4	28.4	1.00	56.9	56.9	1.00	79.3	79.3	1.00
	4.8	28.4	28.4	1.00	58.3	58.3	1.00	80.5	80.5	1.00
	6.4	28.4	28.4	1.00	59.5	59.5	1.00	81.6	81.6	1.00

Note: The flow rates used in this table are for the analysis direction of a directional segment. Values of f_g are computed with Equation (16).

The values of f_G derived in Tables 36, 37, and 38 appear in Exhibits 20-7, 20-8, 20-13, and 20-14 in the revised HCM chapter presented in Appendix E of this report.

5.5.3 Heavy Vehicle Adjustment Factor

The heavy vehicle adjustment factor, f_{HV} , represents the effect of heavy vehicles present in the traffic stream. The base conditions for two-lane highway analysis consist of a traffic stream composed solely of passenger cars, but rarely will such an ideal traffic stream exist. Therefore, traffic flow rates (initially in veh/h) must be adjusted to an equivalent flow rate expressed in passenger cars per hour (pc/h). This adjustment is accomplished through application of the factor, f_{HV} .

Adjustment for the presence of heavy vehicles in the traffic stream applies to two-types of vehicles: trucks and RVs. Buses are not be treated as a separate type of heavy vehicle, but are included in the truck population. The value of f_{HV} is determined in two steps. First, passenger car equivalents (PCEs) for trucks (E_T) and RVs (E_R) are found for prevailing operating conditions. Second, these values of E_T and E_R are used to compute the value of f_{HV} as follows:

$$f_{HV} = 1/[1 + P_T(E_T - 1) + P_R(E_R - 1)] \quad (17)$$

where:

f_{HV}	=	heavy vehicle adjustment factor
P_T	=	proportion of trucks in the traffic stream, expressed as a decimal
P_R	=	proportion of RVs in the traffic stream, expressed as a decimal
E_T	=	passenger-car equivalent for trucks
E_R	=	passenger-car equivalent for RVs

The values of E_T and E_R used in the HCM2000 operational analysis procedure are based on the results of simulation runs with the TWOPAS model. The values of E_T for ATS and PTSF, respectively, are computed as shown in the following equations using data for a traffic stream containing a specified percentage of trucks:

$$E_T = \frac{P_T(v/f_G) + ((ATS_{tr/grade} - ATS_{pc/grade})/-0.0125)}{P_T(v/f_G)} \quad (18)$$

$$E_T = \frac{P_T(v/f_G) + ((PTSF_{tr/grade} - PTSF_{pc/grade})/(0.879e^{-0.000879v/f_G}))}{P_T(v/f_G)} \quad (19)$$

where:

$$\begin{aligned} \text{ATS}_{\text{tr/grade}} &= \text{average travel speed for the mixed flow in non-level terrain or on a} \\ &\quad \text{specific grade (km/h)} \\ \text{PTSF}_{\text{tr/grade}} &= \text{percent time spent following for the mixed flow in non-level terrain} \\ &\quad \text{or on a specific grade} \end{aligned}$$

Equations (18) and (19) implement an empirical approach to quantifying E_T and E_R . In this empirical approach, the values of the factors were based directly on the shapes of the fundamental speed-flow and PTSF-flow curves shown in Figures 32 and 34. Equation (18) incorporates the slope of the speed-flow relationship $[(-0.0125 \text{ (km/h)/(veh/h)})]$ shown in Figure 32, which does not vary with flow rate. Equation (19) incorporates the slope of the PTSF-flow relationship as represented by the first derivative of the PTSF-flow curve shown in Figure 34 evaluated the flow rate being analyzed.

The vehicle mix used to determine E_T consisted of 14 percent trucks and 86 percent passenger cars was selected because the HCM suggests a truck proportion of 14 percent as a default value for rural two-lane highways. E_R is computed in a manner analogous to Equations (18) and (19) for a traffic stream containing 14 percent RVs and 86 percent passenger cars.

Simulation runs were made for the same set of conditions evaluated above for determining the grade adjustment factor, but in addition to the runs for traffic streams composed entirely of passenger cars, equivalent runs were also made for two other vehicle mixes (one containing 14 percent trucks and one containing 14 percent RVs). Thus, a total of 1,890 simulation runs were used in determining the PCE values for heavy vehicles (42 types of terrain x 3 volume levels x 3 vehicle mixes x 5 replicates). In the existing HCM Chapter 8, values of E_T and E_R varied by LOS for general terrain segments and did not vary by LOS for specific grades. In the revised HCM chapter, based on the simulation runs described above, E_T and E_R vary not as a function of LOS, but explicitly as a function of traffic volume.

Table 39 shows the derivation of the truck PCE factors (E_T) for both ATS and PTSF for level and rolling terrain. Comparable data for RV factors (E_R) are shown in Table 40. Tables 41 and 42 show comparable data for truck PCE factors (E_T) for 40 combinations of specific grades (5 levels of percent grade x 8 levels of length of grade) for ATS and PTSF, respectively. Comparable data for RV factors (E_R) are shown in Tables 43 and 44. Most simulation results obtained from the TWOPAS model could be used directly, but the simulation results were, in some cases, found to be highly variable. Where found to be necessary, selected results were discarded and replaced by interpolated values of ATS and PTSF from simulation runs in neighboring cells to assure that values of E_T and E_R increased monotonically with increasing percent grade and with increasing length of grade. The values of E_T and E_R derived in Tables 39 through 44 appear in Exhibits 20-9, 20-10, and 20-15 through 20-17 in the revised HCM chapter presented in Appendix E of this report.

TWOPAS simulation model results suggest that E_T and E_R vary not only with terrain

Table 39. Passenger-Car Equivalents for Trucks (E_T) in Level and Rolling Terrain

Two-way flow rate (pc/h)	Directional flow rate (pc/h)	ATS (km/h)			PTSF		
		(pc/grade)	pc/level)	E _T	pc/level	pc/grade	E _T
Level Terrain							
0 - 600	0 - 300	101.6	101.1	1.7	29.6	29.8	1.1
> 600 - 1,200	> 300 - 600	94.1	93.9	1.2	60.6	60.9	1.1
> 1,200	> 600	86.4	86.0	1.1	82.0	82.0	1.0
Rolling Terrain							
0 - 600	0 - 300	99.6	98.1	2.5	37.2	40.3	1.8
> 600 - 1,200	> 300 - 600	93.4	92.1	1.9	62.7	65.4	1.5
> 1,200	> 600	86.1	84.8	1.5	84.1	84.1	1.0

Note: Values of E_T are computed with Equation (18) for ATS and Equation (19) for PTSF.

Table 40. Passenger-Car Equivalents for RVs (E_R) in Level and Rolling Terrain

Two-way flow rate (pc/h)	Directional flow rate (pc/h)	ATS (km/h)			PTSF		
		pc/level	pc/grade	f _G	pc/level	pc/grade	f _G
Level Terrain							
0 - 600	0 - 300	101.6	101.6	1.0	29.6	29.6	1.0
> 600 - 1,200	> 300 - 600	94.1	94.1	1.0	60.6	60.6	1.0
> 1,200	> 600	86.4	86.4	1.0	82.0	82.0	1.0
Rolling Terrain							
0 - 600	0 - 300	99.6	99.5	1.1	37.2	37.2	1.0
> 600 - 1,200	> 300 - 600	93.4	93.3	1.1	62.7	62.7	1.0
> 1,200	> 600	86.1	85.9	1.1	84.1	84.1	1.0

Note: Values of E_R are computed by substituting rv/grade for tr/grade in Equation (18) for ATS and in Equation (19) for PTSF.

Table 41. Computation of Passenger Car Equivalents for Trucks (E_T) for Average Travel Speed on Specific Grades

Grade (%)	Length (km)	0 - 300 pc/h			> 300 - 600 pc/h			> 600 pc/h		
		ATS (km/h)			ATS (km/h)			ATS (km/h)		
		pc/grade	tr/grade	E_T	pc/grade	tr/grade	E_T	pc/grade	tr/grade	E_T
3	0.4	100.4	99.1	2.5	94.1	92.9	1.9	86.4	85.1	1.5
	0.8	100.3	98.0	3.5	94.1	91.6	2.8	86.4	82.9	2.3
	1.2	100.1	96.9	4.5	94.1	90.0	3.9	86.4	81.0	2.9
	1.6	100.0	96.2	5.1	94.1	89.1	4.6	86.4	79.4	3.5
	2.4	99.9	95.1	6.1	94.0	87.7	5.5	86.4	77.6	4.1
	3.2	99.9	94.2	7.1	93.8	86.7	5.9	86.3	75.9	4.7
	4.8	99.9	93.1	8.2	93.6	85.2	6.7	85.8	73.4	5.3
	6.4	99.9	92.3	9.1	93.5	83.8	7.5	85.4	71.7	5.7
4	0.4	100.3	98.0	3.6	94.1	92.1	2.4	86.4	83.9	1.9
	0.8	100.0	95.9	5.4	94.1	89.0	4.6	86.4	79.7	3.4
	1.2	99.7	94.5	6.4	94.1	86.3	6.6	86.4	76.3	4.6
	1.6	99.4	92.6	7.7	93.3	84.4	6.9	86.4	72.8	5.9
	2.4	99.2	90.5	9.4	93.2	82.1	8.3	86.4	69.2	7.1
	3.2	99.0	89.2	10.2	93.1	79.9	9.6	86.3	66.3	8.1
	4.8	99.0	87.9	11.3	93.1	77.6	11.0	85.6	62.7	8.9
	6.4	98.9	86.7	12.3	93.0	76.0	11.9	85.5	60.0	9.7
5	0.4	99.9	96.9	4.2	94.1	90.3	3.7	86.4	81.9	2.6
	0.8	98.9	93.5	6.0	93.4	85.9	6.0	86.4	74.8	5.1
	1.2	98.3	90.7	7.5	92.9	82.7	7.5	86.4	68.2	7.5
	1.6	98.1	88.4	9.2	92.8	80.1	9.0	86.4	64.3	8.9
	2.4	97.8	86.0	10.6	92.5	77.0	10.5	86.2	60.0	10.3
	3.2	97.7	84.2	11.8	92.4	74.8	11.7	86.0	56.6	11.3
	4.8	97.6	81.6	13.7	92.2	71.3	13.5	85.7	52.6	12.4
	6.4	97.5	79.3	15.3	91.9	68.0	15.0	84.9	50.2	12.5
6	0.4	98.7	94.6	4.7	93.1	88.4	4.1	86.4	79.5	3.5
	0.8	97.8	90.2	7.2	92.3	82.1	7.2	86.1	68.5	7.2
	1.2	96.9	85.9	9.1	92.0	78.2	9.1	85.8	62.3	9.1
	1.6	96.8	84.0	10.3	91.4	74.8	10.3	85.7	59.0	10.2
	2.4	96.3	80.6	11.9	91.3	72.0	11.8	85.3	53.7	11.7
	3.2	96.1	78.7	12.8	91.2	70.1	12.7	85.2	50.8	12.6
	4.8	95.8	75.5	14.4	91.0	66.6	14.3	85.0	45.4	14.2
	6.4	95.6	73.5	15.4	90.9	64.7	15.2	84.9	42.7	15.0
7	0.4	98.1	93.2	5.1	92.5	86.3	4.8	86.0	75.8	4.6
	0.8	96.1	86.1	7.8	91.0	78.5	7.8	85.1	64.8	7.8
	1.2	95.3	81.4	9.8	90.5	73.8	9.8	84.5	57.5	9.8
	1.6	94.5	78.5	10.4	89.9	71.2	10.4	84.4	55.7	10.3
	2.4	94.2	75.1	12.0	89.2	66.4	11.9	84.3	51.0	11.8
	3.2	93.9	72.8	12.9	89.1	64.4	12.8	83.8	46.8	12.7
	4.8	93.7	69.3	14.5	89.0	60.7	14.4	83.6	41.1	14.3
	6.4	93.4	66.8	15.4	88.9	58.5	15.3	83.4	37.8	15.2

Note: The flow rates used in this table are for the analysis direction of a directional segment. Values of E_T are computed with Equation (18).

Table 42. Computation of Passenger-Car Equivalents for Trucks (E_T) for Percent Time Spent Following on Specific Grades

Grade (%)	Length (km)	0 - 300 pc/h			> 300 - 600 pc/h			> 1,200 veh/h		
		PTSF		E_T	PTSF		E_T	PTSF		E_T
		pc/grade	tr/grade		pc/grade	tr/grade		pc/grade	tr/grade	
3	0.4	28.4	28.4	1.0	57.4	57.4	1.0	79.0	79.0	1.0
	0.8	28.4	28.4	1.0	57.1	57.1	1.0	79.1	79.1	1.0
	1.2	28.4	28.4	1.0	57.5	57.5	1.0	79.8	79.8	1.0
	1.6	28.4	28.4	1.0	57.9	57.9	1.0	79.6	79.6	1.0
	2.4	28.4	28.4	1.0	58.5	58.5	1.0	79.9	79.9	1.0
	3.2	28.4	28.4	1.0	58.9	58.9	1.0	81.0	81.0	1.0
	4.8	28.4	29.7	1.4	59.2	59.2	1.0	82.0	82.0	1.0
	6.4	28.4	30.3	1.5	59.5	59.5	1.0	82.6	82.6	1.0
4	0.4	28.4	28.4	1.0	56.5	56.5	1.0	79.2	79.2	1.0
	0.8	28.4	28.4	1.0	55.5	55.5	1.0	78.2	78.2	1.0
	1.2	28.4	28.4	1.0	55.9	55.9	1.0	78.6	78.6	1.0
	1.6	28.4	28.4	1.0	56.7	56.7	1.0	78.6	78.6	1.0
	2.4	28.4	28.7	1.1	57.4	57.4	1.0	79.6	79.6	1.0
	3.2	28.4	28.4	1.0	57.6	57.6	1.0	79.6	79.6	1.0
	4.8	28.4	30.7	1.7	58.9	58.9	1.1	80.5	81.5	1.2
	6.4	28.4	32.0	2.0	61.8	61.8	1.5	81.6	83.5	1.4
5	0.4	28.4	28.4	1.0	54.2	54.2	1.0	77.6	77.6	1.0
	0.8	28.4	28.4	1.0	54.5	54.5	1.0	76.9	76.9	1.0
	1.2	28.4	28.4	1.0	54.9	54.9	1.0	77.3	77.3	1.0
	1.6	28.4	28.4	1.0	55.3	55.3	1.0	77.7	77.7	1.0
	2.4	28.4	28.5	1.0	57.1	57.1	1.2	78.5	79.5	1.2
	3.2	28.4	30.4	1.6	58.3	58.3	1.3	79.3	81.5	1.5
	4.8	28.4	33.0	2.3	62.5	62.5	1.9	80.5	84.1	1.7
	6.4	28.4	36.4	3.3	65.0	65.0	2.1	81.6	85.6	1.8
6	0.4	28.4	28.4	1.0	54.2	54.2	1.0	76.4	76.4	1.0
	0.8	28.4	28.4	1.0	54.5	54.5	1.0	76.9	76.9	1.0
	1.2	28.4	28.4	1.0	54.9	54.9	1.0	77.3	77.5	1.0
	1.6	28.4	28.4	1.0	56.2	56.2	1.2	77.7	78.9	1.2
	2.4	28.4	30.3	1.5	59.0	59.0	1.6	78.5	81.3	1.6
	3.2	28.4	31.5	1.9	61.2	61.2	1.9	79.3	83.3	1.8
	4.8	28.4	36.2	3.3	65.5	65.5	2.5	80.5	85.4	2.0
	6.4	28.4	39.9	4.3	69.9	69.9	3.1	81.6	86.6	2.0
7	0.4	28.4	28.4	1.0	54.2	54.2	1.0	76.4	76.4	1.0
	0.8	28.4	28.4	1.0	54.5	54.5	1.0	76.9	76.9	1.0
	1.2	28.4	28.4	1.0	54.9	54.9	1.0	77.3	78.9	1.3
	1.6	28.4	29.3	1.3	57.4	57.4	1.4	77.7	80.7	1.6
	2.4	28.4	32.2	2.1	60.9	60.9	2.0	78.5	83.2	2.0
	3.2	28.4	34.7	2.8	64.3	64.3	2.5	79.3	84.6	2.1
	4.8	28.4	38.7	4.0	68.4	68.4	3.1	80.5	86.6	2.3
	6.4	28.4	41.7	4.8	71.8	71.8	3.5	81.6	87.5	2.2

Note: The flow rates used in this table are for the analysis direction of a directional segment. Values of E_T are computed with Equation (19).

Table 43. Computation of Passenger-Car Equivalents for RVs (E_R) for Average Travel Speed on Specific Grades

Grade (%)	Length (km)	0 - 300 pc/h			> 300 - 600 pc/h			> 600 pc/h		
		ATS (km/h)		E_R	ATS (km/h)		E_R	ATS (km/h)		E_R
		pc/grade	rv/grade		pc/grade	rv/grade		pc/grade	rv/grade	
3	0.4	100.4	100.3	1.1	94.1	94.1	1.0	86.4	86.4	1.0
	0.8	100.3	100.1	1.2	94.1	94.1	1.0	86.4	86.4	1.0
	1.2	100.1	99.9	1.2	94.1	94.1	1.0	86.4	86.4	1.0
	1.6	100.0	99.7	1.3	94.1	94.1	1.0	86.4	86.4	1.0
	2.4	99.9	99.6	1.4	94.0	94.0	1.0	86.4	86.4	1.0
	3.2	99.9	99.5	1.4	93.8	93.8	1.0	86.3	86.3	1.0
	4.8	99.9	99.5	1.5	93.6	93.6	1.0	85.8	85.8	1.0
	6.4	99.9	99.4	1.5	93.5	93.5	1.0	85.4	85.4	1.0
4	0.4	100.3	100.1	1.3	94.1	94.1	1.0	86.4	86.4	1.0
	0.8	100.0	99.7	1.3	94.1	94.1	1.0	86.4	86.4	1.0
	1.2	99.7	99.4	1.3	94.1	94.1	1.0	86.4	86.4	1.0
	1.6	99.4	99.0	1.4	93.3	93.3	1.0	86.4	86.4	1.0
	2.4	99.2	98.8	1.4	93.2	93.2	1.0	86.4	86.4	1.0
	3.2	99.0	98.6	1.4	93.1	93.1	1.0	86.3	86.3	1.0
	4.8	99.0	98.5	1.4	93.1	93.1	1.0	85.6	85.6	1.0
	6.4	98.9	98.4	1.5	93.0	93.0	1.0	85.5	85.5	1.0
5	0.4	99.9	99.4	1.5	94.1	94.1	1.0	86.4	86.4	1.0
	0.8	98.9	98.4	1.5	93.4	93.4	1.0	86.4	86.4	1.0
	1.2	98.3	97.7	1.5	92.9	92.9	1.0	86.4	86.4	1.0
	1.6	98.1	97.5	1.5	92.8	92.8	1.0	86.4	86.4	1.0
	2.4	97.8	97.1	1.5	92.5	92.5	1.0	86.2	86.2	1.0
	3.2	97.7	97.0	1.6	92.4	92.4	1.0	86.0	86.0	1.0
	4.8	97.6	96.8	1.6	92.2	92.2	1.0	85.7	85.7	1.0
	6.4	97.5	96.7	1.6	91.9	91.9	1.0	84.9	84.9	1.0
6	0.4	98.7	98.1	1.5	93.1	93.1	1.0	86.4	86.4	1.0
	0.8	97.8	97.1	1.5	92.3	92.3	1.0	86.1	86.1	1.0
	1.2	96.9	96.2	1.5	92.0	92.0	1.0	85.8	85.8	1.0
	1.6	96.8	96.0	1.6	91.4	91.4	1.0	85.7	85.7	1.0
	2.4	96.3	95.5	1.6	91.3	91.3	1.0	85.3	85.3	1.0
	3.2	96.1	95.2	1.6	91.2	91.2	1.0	85.2	85.2	1.0
	4.8	95.8	94.9	1.6	91.0	90.8	1.2	85.0	85.0	1.0
	6.4	95.6	94.6	1.6	90.9	90.3	1.5	84.9	84.7	1.2
7	0.4	98.1	97.4	1.6	92.5	92.5	1.0	86.0	86.0	1.0
	0.8	96.1	95.2	1.6	91.0	91.0	1.0	85.1	85.1	1.0
	1.2	95.3	94.4	1.6	90.5	90.5	1.0	84.5	84.5	1.0
	1.6	94.5	93.5	1.6	89.9	89.9	1.0	84.4	84.4	1.0
	2.4	94.2	93.2	1.6	89.2	89.2	1.0	84.3	84.3	1.0
	3.2	93.9	92.8	1.6	89.1	89.1	1.0	83.8	83.8	1.0
	4.8	93.7	92.6	1.6	89.0	88.6	1.3	83.6	83.3	1.3
	6.4	93.4	92.2	1.6	88.9	88.2	1.5	83.4	83.0	1.4

Note: The flow rates used in this table are for the analysis direction of a directional segment. Values of E_R are computed with Equation (18) using rv/grade in place of tr/grade.

Table 44. Computation of Passenger-Car Equivalents for RVs (E_R) for Percent Time Spent Following on Specific Grades

Grade (%)	Length (km)	0 - 300 pc/h			> 300 - 600 pc/h			> 600 pc/h		
		PTSF		E_R	PTSF		E_R	PTSF		E_R
		pc/grade	rv/grade		pc/grade	rv/grade		pc/grade	rv/grade	
3	0.4	28.4	28.4	1.0	57.4	57.4	1.0	79.0	79.0	1.0
	0.8	28.4	28.4	1.0	57.1	57.1	1.0	79.1	79.1	1.0
	1.2	28.4	28.4	1.0	57.5	57.5	1.0	79.8	79.8	1.0
	1.6	28.4	28.4	1.0	57.9	57.9	1.0	79.6	79.6	1.0
	2.4	28.4	28.4	1.0	58.5	58.5	1.0	79.9	79.9	1.0
	3.2	28.4	28.4	1.0	57.7	57.7	1.0	81.0	81.0	1.0
	4.8	28.4	28.4	1.0	59.2	59.2	1.0	82.0	82.0	1.0
	6.4	28.4	28.4	1.0	59.5	59.5	1.0	82.6	82.6	1.0
4	0.4	28.4	28.4	1.0	56.5	56.5	1.0	79.2	79.2	1.0
	0.8	28.4	28.4	1.0	55.5	55.5	1.0	78.2	78.2	1.0
	1.2	28.4	28.4	1.0	55.9	55.9	1.0	78.6	78.6	1.0
	1.6	28.4	28.4	1.0	57.1	57.1	1.0	78.6	78.6	1.0
	2.4	28.4	28.4	1.0	56.1	56.1	1.0	79.6	79.6	1.0
	3.2	28.4	28.4	1.0	57.6	57.6	1.0	79.6	79.6	1.0
	4.8	28.4	28.4	1.0	58.3	58.3	1.0	80.5	80.5	1.0
	6.4	28.4	28.4	1.0	59.5	59.5	1.0	81.6	81.6	1.0
5	0.4	28.4	28.4	1.0	54.2	54.2	1.0	77.6	77.6	1.0
	0.8	28.4	28.4	1.0	54.5	54.5	1.0	76.9	76.9	1.0
	1.2	28.4	28.4	1.0	54.9	54.9	1.0	77.3	77.3	1.0
	1.6	28.4	28.4	1.0	55.3	55.3	1.0	77.7	77.7	1.0
	2.4	28.4	28.4	1.0	56.1	56.1	1.0	78.5	78.5	1.0
	3.2	28.4	28.4	1.0	56.9	56.9	1.0	79.3	79.3	1.0
	4.8	28.4	28.4	1.0	58.3	58.3	1.0	80.5	80.5	1.0
	6.4	28.4	28.4	1.0	59.5	59.5	1.0	81.6	81.6	1.0
6	0.4	28.4	28.4	1.0	54.2	54.2	1.0	76.4	76.4	1.0
	0.8	28.4	28.4	1.0	54.5	54.5	1.0	76.9	76.9	1.0
	1.2	28.4	28.4	1.0	54.9	54.9	1.0	77.3	77.3	1.0
	1.6	28.4	28.4	1.0	55.3	55.3	1.0	77.7	77.7	1.0
	2.4	28.4	28.4	1.0	56.1	56.1	1.0	78.5	78.5	1.0
	3.2	28.4	28.4	1.0	56.9	56.9	1.0	79.3	79.3	1.0
	4.8	28.4	28.4	1.0	58.3	58.3	1.0	80.5	80.5	1.0
	6.4	28.4	28.4	1.0	59.5	59.5	1.0	81.6	81.6	1.0
7	0.4	28.4	28.4	1.0	54.2	54.2	1.0	76.4	76.4	1.0
	0.8	28.4	28.4	1.0	54.5	54.5	1.0	76.9	76.9	1.0
	1.2	28.4	28.4	1.0	54.9	54.9	1.0	77.3	77.3	1.0
	1.6	28.4	28.4	1.0	55.3	55.3	1.0	77.7	77.7	1.0
	2.4	28.4	28.4	1.0	56.1	56.1	1.0	78.5	78.5	1.0
	3.2	28.4	28.4	1.0	56.9	56.9	1.0	79.3	79.3	1.0
	4.8	28.4	28.4	1.0	58.3	58.3	1.0	80.5	80.5	1.0
	6.4	28.4	28.4	1.0	59.5	59.5	1.0	81.6	81.6	1.0

Note: The flow rates used in this table are for the analysis direction of a directional segment. Values of E_R are computed with Equation (19) using rv/grade in place of tr/grade.

and traffic volume, but also with the percentage of trucks or RVs in the traffic stream. This makes conceptual sense because it has long been known that the first few heavy vehicles introduced into a traffic stream have much more effect on traffic performance measures like ATS and PTSF than the last few. However, no consistent pattern was found for the variation of E_T and E_R with vehicle mix and, therefore, a decision was reached not to incorporate the vehicle-mix effect in the operational analysis procedures. The role of vehicle mix in PCEs may be complex and may involve interactions with other variables such as terrain and traffic volume. The development of a better understanding of these vehicle-mix effects, leading to their incorporation in a future HCM operational analysis procedure, is a recommended topic for future research.

OPERATIONAL ANALYSIS PROCEDURE FOR TWO-WAY SEGMENTS

HCM2000 Chapter 20, presented in Appendix E, presents an operational analysis procedure for two-way segments. Two-way segments include both directions of travel on a two-lane highway. The two-way segment procedure is applied to extended segments of two-lane highway that are typically at least 3 km (1.8 mi) in length. The two-way segment procedure incorporates the conceptual approach and revised factors discussed earlier in this chapter of the report.

The operational analysis procedure for two-way segments has five steps:

- Determination of free-flow speed
- Determination of demand flow rate
- Determination of average travel speed
- Determination of percent time spent following
- Determination of level of service

Each of these steps is discussed below. The procedure is presented in detail in Appendix E.

5.6.1 Determination of Free-Flow Speed

The determination of free-flow speed has been addressed earlier in this chapter. Free-flow speed should either be measured in the field or estimated with Equation (57) or (58).

5.6.2 Determination of Demand Flow Rate

The demand flow rate for a two-way segment should be determined with Equation (59).

5.6.3 Determination of Average Travel Speed

In the two-way segment procedure, ATS is determined with the following equation:

where:

ATS = average travel speed for both directions of travel combined (km/h)

$$ATS = FFS - 0.0125 v_p - f_{np} \quad (20)$$

f_{np} = adjustment factor for percentage of no-passing zones (see Exhibit 20-11 in Appendix E)

This equation implements the speed-flow relationship in Figure 32. It also incorporates an adjustment factor (f_{np}) for the effect of no-passing zones on ATS. The values of f_{np} presented in Exhibit 20-11 in Appendix E are based directly on simulation results obtained with the TWOPAS model. Each entry in Exhibit 20-11 represents the difference between the average result of five simulation runs for the specified percentage of no-passing zones and the average of five otherwise identical simulation runs for 0 percent no-passing zones.

ATS for two-way segments was found to be not sensitive to the directional distribution of traffic. Therefore, no directional distribution factor is included in Equation (20).

5.6.4 Determination of Percent Time Spent Following

In the two-way segment procedure, PTSF is determined with the following sequence of equations:

$$BPTSF = 100 [1 - e^{-0.000879 v_p}] \quad (21)$$

$$PTSF = BPTSF + f_{d/np} \quad (22)$$

where:

BPTSF = base percent time spent following for both directions of travel combined

$f_{d/np}$ = adjustment for the combined effect on percent time spent following of directional distribution of traffic and percent no-passing zones (see

Exhibit 20-12 in Appendix E)

Equation (21) implements the PTSF-flow relationship in Figure 34. Equation (22) incorporates an adjustment factor ($f_{d/np}$) for the combined effect of directional distribution of traffic and no-passing zones on PTSF. The values of $f_{d/np}$ presented in Exhibit 20-12 in Appendix E are based directly on simulation results obtained with the TWOPAS model. Each entry in Exhibit 20-12 represents the difference between the average of five simulation runs for the specified directional split and percentage of no-passing zones and the average of five otherwise identical simulation runs for a 50/50 directional split and 0 percent no-passing zones.

It should be noted that, while ATS was found to be independent of the directional distribution of traffic, PTSF is influenced by the directional distribution. Therefore, the directional distribution of traffic considered in determining the appropriate value of $f_{d/np}$.

5.6.5 Determination of LOS

The LOS for a Class I facility is determined as a function of both ATS and PTSF from the criteria in Table 26 or Figure 29, which are equivalent. Use of Figure 29 is recommended, because the figure makes clear that both the ATS and PTSF criteria must be met in order for any given LOS to apply. The LOS for a Class II highway is determined from PTSF alone using the criteria in Table 27.

If the demand flow rate exceeds 3,200 pc/h for both directions of travel combined, or 1,700 pc/h for either direction individually, then the two-lane highway is oversaturated and LOS F applies.

OPERATIONAL ANALYSIS PROCEDURE FOR DIRECTIONAL SEGMENTS

HCM2000 Chapter 20, presented in Appendix E, presents an operational analysis procedure for directional segments. Directional segments are defined as sections of two-lane highway including only one direction of travel. Either direction of travel may be analyzed on any two-lane highways, but the LOS in one direction of travel is dependent on operational conditions not only for that direction of travel, but for the opposing direction of travel as well. The directional segment procedure is applied to extended segments of two-lane highway that are typically at least 3 km (1.8 mi) in length or to specific upgrades and downgrades. The directional segment procedure incorporates the conceptual approach and revised factors discussed earlier in this chapter of the report.

The operational analysis procedure for directional segments incorporates the following five steps:

- Determination of free-flow speed

- Determination of demand flow rate
- Determination of ATS
- Determination of PTSF
- Determination of LOS

The approach to each step is described below. The procedure is presented in detail in Appendix E.

5.7.1 Determination of Free-Flow Speed

The determination of free-flow speed has been addressed earlier in this chapter. Free-flow speed should either be measured in the field or estimated with Equation (57) or (58).

5.7.2 Determination of Demand Flow Rate

The demand flow rate for a directional segment should be determined by applying Equation (14) to the demand volume, vehicle mix, and terrain for the analysis direction. The demand flow rate for the opposing direction is also needed for the procedure and is obtained by applying Equation (14) to the demand volume, vehicle mix, and terrain for the opposing direction. The demand volume and vehicle mix for the opposing directions of travel are likely to differ. The terrain in the opposing directions of travel are generally the same for extended segments; for specific grades, if one direction of travel is analyzed as a specific upgrade, the other direction of travel must be analyzed as a specific downgrade.

The value of f_G used to determine the demand flow rate for a directional segment in level and rolling terrain is determined from Exhibits 20-7 and 20-8 in Appendix E. The value of f_{HV} for directional segments in level and rolling terrain is determined with Equation (17) and the appropriate values of E_T and E_R from Exhibits 20-9 and 20-10.

For specific upgrades, the value of f_G is determined from Exhibits 20-13 and 20-14 in Appendix E, while the value of f_{HV} is determined with Equation (17) and Exhibits 20-15 through 20-17.

For most specific downgrades, the value of f_G is 1.0, and the value of f_{HV} is determined in the same manner as for level terrain. Some specific downgrades are long and steep enough that some heavy vehicles are forced to travel at crawl speeds to avoid loss of control on the grade. Slow-moving vehicles of this type are likely to impede other vehicles and will decrease ATS and increase PTSF. Where this occurs, the value of f_{HV} used to determine ATS should be based on the following equation rather than Equation (17):

$$f_{HV} = \frac{1}{1 + P_{TC} P_T (E_{TC} - 1) + (1 - P_{TC}) P_T (E_T - 1) + P_R (E_R - 1)} \quad (23)$$

where:

- P_{TC} = proportion of all trucks in the traffic stream that use crawl speeds on the specific downgrade if interest, expressed as a decimal
- E_{TC} = passenger-car equivalent for trucks that use crawl speeds (see Exhibit 20-18 in Appendix E)

Equation (23) uses PCEs for the subset of the truck population using crawl speeds presented in Exhibit 20-18 in Appendix E. These PCEs are based on comparisons of the results of TWOPAS runs with and without the presence on the downgrades of trucks using crawl speeds. Exhibit 20-18 shows that the PCEs of trucks operating at crawl speeds vary with both the traffic volume level and the difference between the free-flow speed of traffic and the truck crawl speed. Where more specific data are not available, the percentage of trucks that use crawl speeds (P_{TC}) can be estimated as equal to the percentage of all trucks that are tractor-trailer combinations.

5.7.3 Determination of Average Travel Speed

In the directional segment procedure, ATS is determined with the following equation:

$$ATS_d = FFS_d - 0.0125(v_d + v_o) - f_{np} \quad (24)$$

where:

- ATS_d = average travel speed in the analysis direction (km/h)
- FFS_d = free-flow speed in the analysis direction (km/h)
- v_d = passenger-car equivalent flow rate for the peak 15-minute period in the analysis direction (pc/h)
- v_o = passenger-car equivalent flow rate for the peak 15-minute period in the opposing direction (pc/h)
- f_{np} = adjustment for percentage of no-passing zones in the analysis direction (see Exhibit 20-19 in Appendix E)

This equation is analogous to Equation (20) and implements the speed-flow relationship in Figure 32. The separate terms for flow rates in the analysis direction and the opposing direction have a common slope, as discussed above. Equation (24) also incorporates an adjustment factor (f_{np}) for the effect of no-passing zones on ATS. The values of f_{np} used in Equation (24) differ from those used in for two-way segments in Equation (20). The values of f_{np} for directional segments presented in Exhibit 20-19 are based directly on simulation

results obtained with the TWOPAS model. Each entry in Exhibit 20-19 represents the difference between the average result of five simulation runs for the specified percentage of no-passing zones and the average of five otherwise identical simulation runs for 0 percent no-passing zones.

ATS for directional segments was found to be sensitive to the opposing-direction flow rate, but not to the directional distribution of traffic per se.

5.7.4 Determination of Percent Time Spent Following

In the directional segment procedure, PTSF is determined with the following sequence of equations:

$$\text{BPTSF}_d = 100[1 - e^{-a v_d^b}] \quad (25)$$

$$\text{PTSF}_d = \text{BPTSF}_d - f_{np} \quad (26)$$

where:

PTSF_d = percent time spent following in the direction analyzed.
 BPTSF_d = base percent time spent following in the direction analyzed, and
 f_{np} = adjustment factor for percentage of no-passing zones in the analysis direction (see Exhibit 20-20)

Equation (25) implements the PTSF-flow relationship in Figure 35. Equation (26) incorporates an adjustment factor (f_{np}) for the effect of no-passing zones on PTSF. The values of f_{np} presented in Exhibit 20-19 in Appendix E are based directly on simulation results obtained with the TWOPAS model. Each entry in Exhibit 20-20 represents the difference between the average result of five simulation runs for the specified percentage of no-passing zones and the average of five otherwise identical simulation runs for 0 percent no-passing zones.

5.7.5 Determination of LOS

The LOS for a directional segment of a Class I facility is determined as a function of both ATS and PTSF from the criteria in Table 26 or Figure 29, which are equivalent. Use of Figure 29 is recommended, because the figure makes clear that both the ATS and PTSF criteria must be met in order for any given LOS to apply. The LOS for a Class II highway is determined from PTSF alone using the criteria in Table 27.

If the demand flow rate exceeds 3,200 pc/h for both directions of travel combined, or 1,700 pc/h for either direction individually, then the two-lane highway is oversaturated and LOS F applies.

OPERATIONAL ANALYSIS PROCEDURES FOR DIRECTIONAL SEGMENTS CONTAINING PASSING LANES IN LEVEL AND ROLLING TERRAIN

HCM2000 Chapter 20, presented in Appendix E, also presents an operational analysis procedure for directional segments containing passing lanes in level and rolling terrain. A passing lane is an added lane provided in one direction of travel on a two-lane highway to increase the availability of passing opportunities. The addition of a passing opportunity to a two-lane highway provides a three-lane cross section with two lanes in one direction of travel and one lane in the other direction of travel. Traffic in the opposing direction of travel to a passing lane may, depending upon local practice, be prohibited from passing or may, as shown in Figure 36, be permitted to pass where adequate sight distance is available. Passing lanes can also be provided in both directions of travel at the same location resulting in a short section of four-lane undivided highway with improved passing opportunities for both directions of travel.

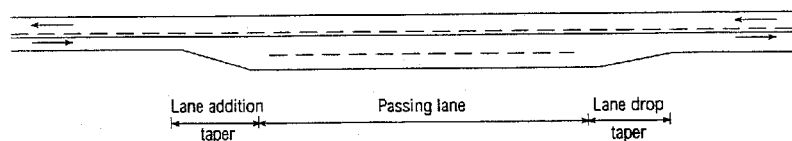


Figure 36. Plan View of a Typical Passing Lane

The operational analysis procedure for passing lanes provides estimates of the effect of the passing lane on PTSF and ATS. The procedure address passing lanes in rolling terrain, but does not address added lanes in mountainous terrain or on specific upgrades. An operational analysis procedures for climbing lanes on upgrades is described later in this chapter of the report.

The operational analysis procedure for directional segments containing passing lanes incorporates the following five steps:

- Application of the directional segment procedure without the passing lane in place
- Division of the segment into regions
- Determination of PTSF
- Determination of ATS

- Determination of LOS

The approach to each step is described below. The procedure is presented in detail in Appendix E.

5.8.1 Application of the Directional Segment Procedure Without the Passing Lane in Place

The first step in a passing lane analysis is to apply the operational analysis procedure for the directional segment without the passing lane in place. The directional segment evaluated for a passing lane should be in level or rolling terrain. The directional segment may contain a portion of two-lane roadway upstream of the passing lane, must contain the entire passing lane and may contain a portion of the two-lane roadway downstream of the passing lane. The results of the initial application of the directional segment procedure are estimates of PTSF and ATS for the normal two-lane cross-section.

5.8.2 Division of the Segment into Regions

The next step is to divide the analysis segments into four regions. These regions are:

- upstream of the passing lane
- within the passing lane
- downstream of the passing lane but within its effective length
- downstream of a passing lane but beyond its effective length

These four lengths must, by definition, sum to the total length of the analysis segment. The analysis segments and their length will differ for estimation of PTSF and ATS because the downstream lengths for these measures differ, as shown in Table 45.

The length of the segment upstream of the passing lane (L_u) and the length of the passing lane itself (L_{pl}) are readily determined when the location (or proposed location) of the passing lane is known. The length of the downstream highway segment within the effective length of the passing lane (L_{de}) is determined from Table 45. Passing lanes generally decrease PTSF and decrease ATS on the roadway downstream of the passing lane; Figure 37 illustrates the effect of a passing lane on PTSF. The values in Table 45 are based on the results of TWOPAS simulation runs performed as part of the research comparing the variation of PTSF and ATS with and without the presence of a passing lane. The values represent the downstream distance required for PTSF or ATS with a passing lane in place to return to the value it would have had if there were no passing lane present.

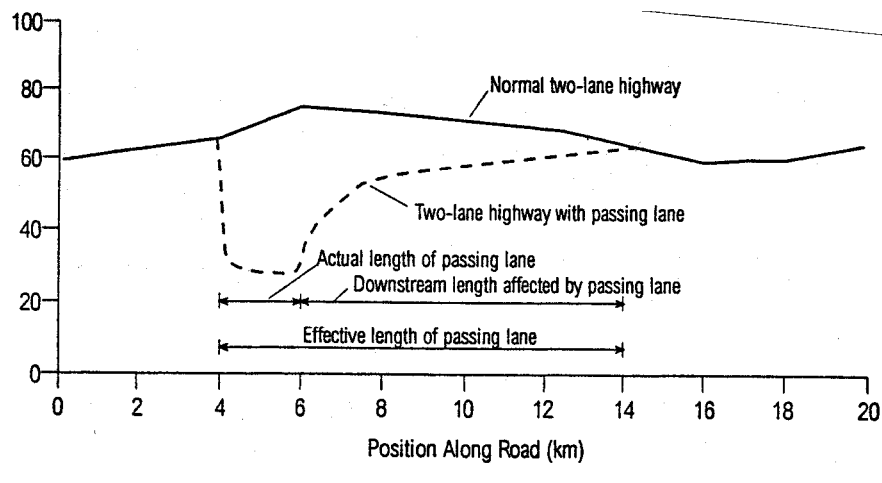


Figure 37. Example of the Operational Effect of a Passing Lane on Percent Time Spent Following

Table 45. Downstream Length of Roadway Affected by Passing Lanes on Directional Segments in Level and Rolling Terrain

Directional flow rate (pc/h)	Downstream length of roadway affected (km)	
	Percent time spent following	Average travel speed
200 or less	20.9	2.8
400	13.0	2.8
700	9.1	2.8
1,000 or more	5.8	2.8

Once the lengths L_u , L_{pl} , and L_{de} are known, the length of the analysis segment downstream of the passing lane and beyond its effective length (L_d) can be determined by subtraction as:

$$L_d = L_t - L_u - L_{pl} - L_{de} \quad (27)$$

where:

- L_d = length of two-lane highway downstream of the passing lane and beyond its effective length (km)
- L_t = total length of analysis segment (km)
- L_u = length of two-lane highway upstream of the passing lane (km)
- L_{pl} = length of the passing lane including tapers (km)
- L_{de} = downstream length of two-lane highway within the effective length of the passing lane (km) (see Table 45)

The passing lane procedure has been calibrated for passing lanes whose lengths are within the optimal ranges shown in Table 46. Passing lanes substantially shorter or longer than the optimum may provide less operational benefit than predicted by the procedure. The optimal passing lane lengths in Table 46 were developed in research by Harwood and St. John (41).

Determination of Percent Time Spent Following

Figure 38 illustrates the approach used to estimate PTSF with a passing lane in place. PTSF within lengths L_u and L_d is assumed to be equal to $PTSF_d$, as predicted by the directional segment procedure. Within the passing lane, PTSF is generally equal to 58 to 62 percent of its upstream value; this effect varies as a function of flow rate as shown in Table 47. Table 47 was derived from TWOPAS simulation runs for passing lanes on two-lane highways with a range of flow rates. Within the downstream effective length of the passing lane, PTSF is assumed to increase linearly with distance from the within-passing-lane value to the normal upstream value. These assumptions result in the following equation for estimating PTSF for the analysis segment as a whole with the passing lane in place:

Table 46. Optimal Lengths of Passing Lanes

Directional flow rate (pc/h)	Optimal passing lane length (km)
100	≤ 0.80
200	$> 0.80-1.20$
300	$> 1.20-1.60$
700 or more	$> 1.60-3.20$

Table 47. Factors (f_{pl}) for Estimation of Average Travel Speed and Percent Time Spent Following Within a Passing Lane

Direction flow rate (pc/h)	Average travel speed	Percent time spent following
0.300	1.08	0.58
$> 300-600$	1.10	0.61
> 600	1.11	0.62

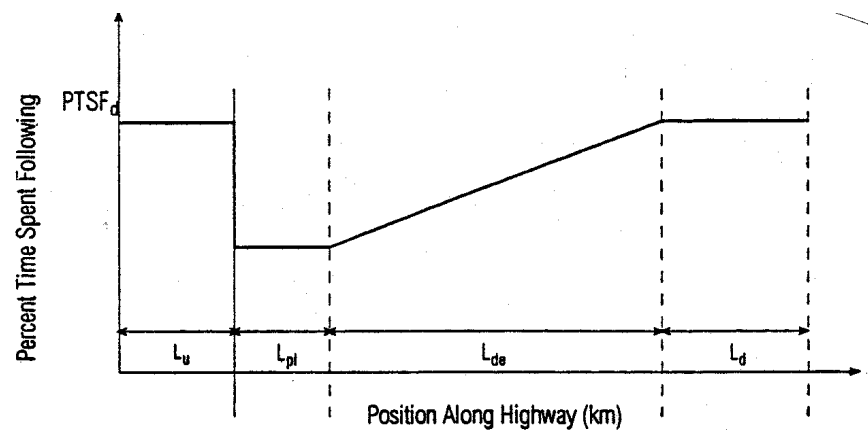


Figure 38. Effect of a Passing Lane on Percent Time Spent Following as Represented in the Operational Analysis Methodology

$$PTSF_{pl} = \frac{PTSF_d \left[L_u + L_d + f_{pl} L_{pl} + \left(\frac{1 + f_{pl}}{2} \right) L_{de} \right]}{L_t} \quad (28)$$

where:

- $PTSF_{pl}$ = percent time spent following for the entire segment including the passing lane
 $PTSF_d$ = percent time spent following for the entire segment without the passing lane
 f_{pl} = factor for the effect of a passing lane on percent time spent following (see Exhibit 20-24 in Appendix E)

If the analysis section is truncated by the presence of a town, a major intersection, or a change in demand volume before the full downstream effective length of the passing lane has been reached, then the distance L_d is not used and the actual downstream length within the analysis segment, L'_{de} , is less than the value of L_{de} shown in Table 45. In this case, Equation (73) should be replaced by Equation (74):

$$PTSF_{pl} = \frac{PTSF_d \left[L_u + f_{pl} L_{pl} + f_{pl} + \left(\frac{1 - f_{pl}}{2} \right) \left(\frac{L'_{de}}{L_{de}} \right) \right]}{L_t} \quad (29)$$

where:

- L'_{de} = actual distance from end of passing lane to end of analysis segment (km). L'_{de} must be less than or equal to the value of L_{de} from Exhibit 20-23 in Appendix E

5.8.3 Determination of Average Travel Speed

ATS with the passing lane in place is determined with a similar approach to PTSF except that ATS is increased, rather than decreased, by the presence of the passing lane. Figure 39 illustrates this approach. ATS within lengths L_u and L_d is assumed to be equal to ATS_d , as predicted by the directional segment procedure. Within the passing lane, ATS is generally equal to 8 to 11 percent higher than its upstream value; this effect varies as a function of flow rate as shown in Table 47. Within the downstream effective length of the passing lane, ATS is assumed to increase linearly with distance from the within-passing-lane value to the normal upstream value. These assumptions result in the following equation for estimating ATS for the analysis segment as a whole with the passing lane in place:

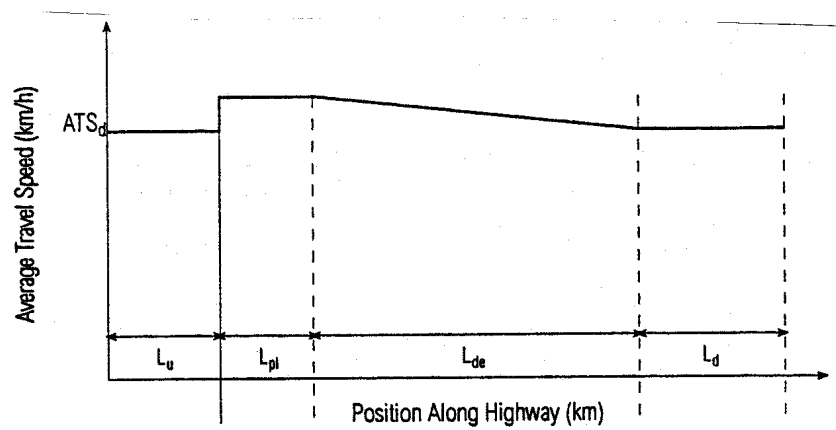


Figure 39. Effect of Passing Lane on Average Travel Speed as Represented in the Operational Analysis Methodology

$$ATS_{pl} = \frac{ATS_d L_t}{L_u + L_d + \frac{L_{pl}}{f_{pl}} + \frac{2L_{de}}{1 + f_{pl}}} \quad (30)$$

where:

- ATS_{pl} = average travel speed for the entire segment including the passing lane (km/h)
- ATS_d = average travel speed for the entire segment without the passing lane (km/h)
- f_{pl} = factor for the effect of a passing lane on average travel speed (see Exhibit 20-24 in Appendix E)

Because of the nature of section-averaged speeds, Equation (30) represents the harmonic mean, rather than the arithmetic mean, of ATS.

If the analysis section is truncated by the presence of a town, a major intersection, or a change in demand volume before the full downstream effective length of the passing lane has been reached, then the distance L_d is not used and the actual downstream length within the analysis segment, L'_{de} , is less than the value of L_{de} shown in Table 45. In this case, Equation (30) should be replaced by Equation (31):

$$ATS_{pl} = \frac{ATS_d L_t}{L_u + \frac{L_{pl}}{f_{pl}} + \frac{2L'_{de}}{\left(1 + f_{pl} + (f_{pl} - 1) \frac{L_{de} - L'_{de}}{L_{de}}\right)}} \quad (31)$$

Determination of Level of Service

The LOS for a directional segment containing a passing lane is determined in a manner identical to a directional segment without a passing lane except that $PTSF_{pl}$ and ATS_{pl} are used in place of $PTSF_d$ and ATS_d .

5.8.4 Passing Lane Effect on Opposing Traffic

Where a passing lane is provided on a directional segment, the directional analysis for the opposing direction of travel should be revised if the installation of the passing lane changes the percent no-passing zones for the opposing direction travel. This will occur, for example, if a highway agency routinely prohibits passing in the opposing direction of travel to a passing lane. However, where passing is permitted in the opposing direction of travel to a passing lane, there may be very little effect on the level of service for the opposing direction of travel.

Where passing lanes are provided in both directions of travel, the operational analyses for the two directions of travel can proceed independently unless the addition of the passing lane in one direction of travel substantially changes the percentage of no-passing zones outside the passing lane in the other direction of travel.

OPERATIONAL ANALYSIS PROCEDURE FOR DIRECTIONAL SEGMENTS CONTAINING CLIMBING LANES ON UPGRADES

The operational analysis procedure for a directional segment containing a climbing lane on an upgrade is the same as the procedure for passing lanes with the following exceptions:

- if the climbing lane procedure is to be applied, the directional segment containing the climbing lane must be analyzed as a specific upgrade
- the climbing lane procedure can be used to estimate PTSF and ATS on the grade itself, but not beyond the top of the grade. Therefore, L_{de} and L_d are generally equal to zero unless the climbing lane ends before the top of the grade. In this case, Equations (29) and (31) for a truncated downstream effective length will generally apply.
- PTSF and ATS for a climbing lane are based on the factors in Table 48 rather than the factors for passing lanes shown in Table 47. Table 48 was derived from TWOPAS runs for upgrades with and without climbing lanes.

Table 48. Factors (f_{pl}) for Estimation of Average Travel Speed and Percent Time Spent Following within a Climbing Lane

Directional flow rate (pc/h)	Average travel speed	Percent time spent following
0-300	1.02	0.20
> 300-600	1.07	0.21
> 600	1.14	0.23

OTHER TRAFFIC PERFORMANCE MEASURES

In addition to PTSF and ATS, each of the operational analysis procedures can estimate a number of additional traffic performance measures including volume-to-capacity ratio, total travel during the peak 15-minute period, total travel during the peak hour, and total travel time during the peak 15-min period. The relationships for determining each of these additional performance measures are presented below.

The volume-to-capacity ratio for an extended two-way segment can be computed as:

$$v/c = \frac{V_p}{c} \quad (32)$$

where:

v/c = volume-to-capacity ratio
 c = two-way segment capacity (normally 3,200 pc/h for a two-way segment and 1,700 pc/h for a directional segment)

The total travel on the two-way or directional segment during the peak 15-min period is:

$$VKT_{15} = 0.25 \left(\frac{V}{PHF} \right) L_t \quad (33)$$

where:

VKT_{15} = total travel on the analysis segment during the peak 15-min period (veh-km)
 L_t = total length of the analysis segment (km)

The total travel on a two-way or directional segment during the peak hour is:

$$VKT_{60} = VL_t \quad (34)$$

where:

VKT_{60} = total travel on the analysis segment during the peak hour (veh-km)

The total travel time during the peak 15-min period is:

$$TT_{15} = \frac{VKT_{15}}{ATS} \quad (35)$$

where:

TT_{15} = total travel time for all vehicles on the analysis segment during the peak 15-min period (veh-h)

OTHER ELEMENTS OF THE REVISED HCM CHAPTERS

This final section of Chapter 5 summarizes several other important elements of the revised chapters prepared for the HCM2000, as shown in Appendices D and E. These include worksheets, examples, planning applications, design and operational treatments, LOS assessment for facilities containing two-lane highway segments.

5.11.1 Worksheets

HCM2000 Chapter 20 presents three worksheets that can be used for manual application of the operational analysis procedures. These worksheets address the applications of the operational analysis procedures:

- two-way segments
- directional segments
- directional segments with passing lanes

While most HCM users will undoubtedly use computer software to perform operational analyses, the procedures are simple enough that they can be applied manually. The worksheets are presented at the end of Appendix E of this report.

5.11.2 Examples

HCM2000 Chapter 20 presents four examples to illustrate the operational analysis procedures. These examples address:

- analysis of a two-way segment on a Class I highway
- analysis of a two-way segment on a Class II highway
- analysis of a directional segment on a Class I highway
- analysis of a directional segment containing a passing lane on a Class I highway

It was originally planned to include an example illustrating the application of the HCM to a proposed two-lane to four-lane widening project. However, this example provided results that were rather obvious and it was therefore dropped. Virtually any volume being served by an existing two-lane highway, even a volume resulting in LOS E, can be served at LOS A or B on a four-lane highway, based on the multilane highway operational analysis procedures in HCM2000 Chapter 21. Therefore, the assessment of whether to widen a highway from two to four lanes hinges on the assessment of whether the current or projected future LOS of the two-lane facility is acceptable. If the LOS for the two-lane facility is considered acceptable by the responsible highway agency, then widening is not needed. If the LOS of the two-lane highway can be made acceptable by addition of passing and /or climbing lanes, either as a substitute for widening to four lanes

or as an interim measure, then such an option can be considered. If the LOS for the two-lane facility is not acceptable, and cannot be made acceptable through addition of passing and/or climbing lanes, then widening to four lanes is justified on the basis of traffic operations. Of course, many factors in addition to traffic operations (including safety, right-of-way availability, environmental factors, and construction cost) must be considered in evaluating a proposed project involving widening to four lanes.

5.11.3 Planning Applications

Planning applications of the operational analysis procedure are illustrated by Tables 49 and 50. Table 49 presents the service flow rates representing to LOS thresholds for the base conditions for two-way segments. Table 50 presents these LOS thresholds expressed as annual average daily traffic volumes (AADTs) based on the assumption that the design hour volume represents 10 percent of the AADT. These tables can be used by planners to anticipate the traffic operational conditions likely to prevail under various levels of demand volume. It should be kept in mind, however, that the threshold values in the table represent base conditions and, that, lower LOS thresholds will apply when there are heavy vehicles in the traffic stream, when the terrain is other than level, when the directional distribution of traffic is other than 50/50, and when there are no-passing zones within the analysis segment.

Design and Operational Treatments

The appendix to HCM2000 Chapter 20 describes five design and operational treatments with the potential to improve traffic operations on two-lane highways that are not addressed by the operational analysis procedures. These are:

- turnouts (9, 42, 43, 44)
- shoulder use (9, 42)
- wide cross sections (45)
- intersection turn lanes (46, 47, 48)
- two-way left-turn lanes (9, 42)

The appendix for HCM2000 Chapter 20 describes these treatments and explains what is known about their operational effectiveness. However, a judgement was made by the research team that the current state-of-knowledge about the operational effects of these treatments was insufficient to include their effects in the operational analysis procedures. Further research about the operational effects of these treatments is encouraged.

Table 49. Maximum Two-Way Service Volume (pc/h) Versus Level of Service for a Class I Two-Lane Rural Highway

Free-flow speed (km/h)	Level of service				
	A	B	C	D	E
Level terrain					
110	260	490	900	1,570	2,680
100	260	490	900	1,570	2,680
90	N/A	N/A	900	1,570	2,680
80	N/A	N/A	490	1,420	2,680
70	N/A	N/A	N/A	490	2,680
Rolling terrain					
110	130	260	710	1,490	2,500
100	130	260	710	1,490	2,500
90	N/A	260	710	1,490	2,500
80	N/A	N/A	280	1,100	2,500
70	N/A	N/A	N/A	280	2,500
Mountainous terrain					
110	40	160	310	510	1,410
100	40	160	310	510	1,410
90	N/A	160	310	510	1,410
80	N/A	N/A	180	510	1,410
70	N/A	N/A	N/A	180	1,410

Note: assumes 60/40 directional split; 20, 40, and 60 percent no-passing zones for level, rolling, and mountainous terrain, respectively; 14% trucks; and 4% RVs.

N/A = not achievable for given condition.

Table 50. Maximum AADT (veh/day) Versus Level of Service for a Class I Two-Lane Rural Highway

Free-flow speed (km/h)	Level of service				
	A	B	C	D	E
Level terrain					
110	2,600	4,900	9,000	15,700	26,800
100	2600	4,900	9,000	15,700	26,800
90	N/A	4,900	9,000	15,700	26,800
80	N/A	N/A	4,900	14,200	26,800
70	N/A	N/A	N/A	4,900	26,800
Rolling terrain					
110	1,300	2,600	7,100	14,900	25,000
100	1,300	2,600	7,100	14,900	25,000
90	N/A	2,600	7,100	14,900	25,000
80	N/A	N/A	2,800	11,000	25,000
70	N/A	N/A	N/A	2800	25,000
Mountainous terrain					
110	400	1,600	3,100	5,100	14,100
100	400	1,600	3,100	5,100	14100
90	N/A	1,600	3,100	5,100	14,100
80	N/A	N/A	1,800	5,100	14,100
70	N/A	N/A	N/A	1,800	14,100

Note: assumes design hour factor (K) = 0.10; 60/40 directional split; 20, 40, and 60 percent no-passing zones for level, rolling, and mountainous terrain, respectively; 14% trucks; and 4% RVs.

N/A = not achievable for given condition.

Level-of-Service Assessment for Directional Facilities Containing Two-lane Highway Segments

HCM2000 Chapter 20 describes procedures for assessing traffic operations on directional facilities containing two-lane highway segments. For facilities composed entirely of two-lane highway segments, LOS can be assessed by combining the PTSF and ATS for the individual segments as shown in Equations 20-23 and 20-24 in Appendix E and applying the LOS criteria in Exhibits 20-3 and 20-4. Facilities or corridors composed of two-lane highway segments, plus multilane highway or basic freeway segments, can be assessed by combining the individual ATS estimates for the segments and applying the LOS criteria in HCM2000 Chapter 29. These procedures address undersaturated conditions only. HCM2000 Chapter 22 presents a procedure for assessing freeway facilities under both undersaturated and oversaturated conditions. However, these procedures have not been extended to include multilane or two-lane highway segments. Research to develop a procedure for assessing all uninterrupted facility types in a single analysis is recommended.

CHAPTER 6.

CONCLUSIONS AND RECOMMENDATIONS

This chapter presents the conclusions of the research and recommendations for future work. Based on a review of the existing operational analysis procedures for two-lane highways, it was concluded that:

- The procedures of existing HCM Chapter 8 (1,2), while providing a useful operational analysis tool that has been in regular use for nearly 15 years, have several limitations that warrant improvement of the procedures these limitations include:
 - confusion about the meaning of the term percent time delay
 - use of different service measures in the existing general terrain segment and specific grade procedures, which can provide in different LOS assessments for similar conditions
 - LOS thresholds that result in LOS D and E being reached at volume levels that appear, to some HCM users, to be unreasonably low
 - the lack of an effect of roadway design speed or free-flow speed on LOS
 - the lack of a procedure for evaluating the traffic operational effects of passing and climbing lanes
 - the lack of an operational analysis procedure for long, steep downgrades
 - the 5.0-s headway criterion for estimating percent time delay in the field, which is disputed by some users and misunderstood by others

To meet the needs of HCM users, a revised operational analysis procedure that addresses these limitations has been developed.

- The TWOPAS model, as improved in this research, provides a valid and useful tool for computer simulation of traffic operations on two-lane highways. The UCBRURAL interface provides a user-friendly environment for applying the TWOPAS model.
- The most appropriate service measure for two-way and directional roadway segments on two-lane highways is the combination of percent time spent following and average travel speed. As applied in the operational analysis procedure, any given roadway segment must satisfy both an upper limit on percent time spent following and a lower limit on average travel speed to qualify for any given level of service.

- Percent time spent following is the new recommended term for the measure known as percent time delay in the previous HCM procedure. Percent time spent following can be estimated in the field as the percent of vehicles traveling at headways of 3 s or less at a representative point on the roadway segment.
- Two-lane highways can be classified for operational analysis based on whether motorists expect to travel at relatively high speeds on that roadway or in that corridor. Two-lane highways on which drivers expect to travel at relatively high speeds, such as major intercity routes, primary arterials connecting major traffic generators, daily commuter routes, or primary links in state and national highway networks are generally assigned to Class I. Two-lane highways that function as access routes to Class I facilities, serve as scenic or recreational routes that are not primary arterials, or pass through rugged terrain are generally assigned to Class II. The level of service for Class I highways is defined in terms of both percent time spent following and average travel speed. The level of service for Class II highways is defined in terms of just percent time spent following.
- Revised procedures for estimating percent time spent following and average travel speeds on two-lane highways under various flow rates have been developed and are presented in Chapter 5 of this report. These procedures include estimates of free-flow speeds, speed-flow relationships, percent time spent following-flow relationships, grade and heavy vehicle effects, and effects of no-passing zones and directional distribution of traffic developed from TWOPAS simulation results. The estimates of percent time spent following and average travel speed provided by these procedures can be used in determining the level of service for two-lane highways.
- The improved operational analysis procedures developed in this research are recommended for incorporation in the HCM2000. Draft chapters for incorporation in the HCM2000 are presented in Appendices D and E.

The research highlighted the need for future research in a number of areas:

- The concept of passenger-car equivalents for heavy vehicles needs a thorough review. In particular, it appears that the passenger car equivalents for heavy vehicles vary with the proportion of heavy vehicles in the traffic stream. However, this effect has not been consistently observed and quantified for inclusion in an operational analysis procedure.
- Only limited field data are available for climbing lanes and downgrade crawl regions. The climbing lane and downgrade analysis procedures presented in this report are based on simulation, and represent a good first step, but further research to refine these procedures is needed. In particular, the climbing lane procedure needs to be extended to include the traffic operational performance of the roadway downstream of the climbing lane. It would be desirable, in the downgrade procedure, to provide more guidance on which trucks are likely to use crawl speeds on particular downgrades.

- The operational analysis procedures identify five design and operational treatments that are known to improve traffic operations on two-lane highways but whose traffic operational effects have not been quantified. These are:
 - turnouts
 - shoulder use
 - wide cross sections
 - intersection turn lanes
 - two-way left-turn lanes

It is recommended that the operational effects of these treatments be quantified in future research and that those effects be incorporated in future HCM procedures.

- It would be desirable to develop and test a general procedure for combining the analysis of all types of uninterrupted flow facilities that may exist in a highway corridor. HCM2000 Chapter 22 on freeway facilities has broken important ground in integrating the analysis of basic freeway segments, ramps and ramp junctions, and weaving areas. Research is recommended to extend this methodology to include directional segments of multilane and two-lane highways.
- The TWOPAS model, which was used in development of the operational analysis procedure presented here, was developed based on traffic operational field data collected in the 1970s and 1980s. As part of this research, the vehicle population used in input has been updated to represent the late 1990s and the rest of the model has been calibrated and validated using recently collected field data. One area that has not been fully addressed is the simulation of passing maneuvers made using the opposing traffic lane. Passing maneuvers in TWOPAS have been modeled based on field data collected in the 1970s. It is not clear what changes in motorist passing behavior may have taken place in the intervening years, but it is recommended that current passing behavior be documented and that TWOPAS be updated to the extent found to be necessary.
- The research completed initial development of program logic to incorporate the traffic operational effect of individual at-grade intersections and driveways into TWOPAS. It is recommended that development and testing of this logic be completed and that it be fully incorporated in TWOPAS.

REFERENCES

1. Bureau of Public Roads, *Highway Capacity Manual*, Washington, DC, 1950.
2. Highway Research Board, "Highway Capacity Manual—1965," *HRB Special Report 87* (1965).
3. Transportation Research Board, "Highway Capacity Manual," *Special Report 209* (1985).
4. Transportation Research Board, "Highway Capacity Manual," *Special Report 209*, Third Edition (1994).
5. Messer, C. J., "Two-Lane, Two-Way Rural Highway Capacity," Final Report of NCHRP Project 3-28A, Texas Transportation Institute (February 1983).
6. Werner, A., and J. F. Morrall, "The Effect of Recreational Vehicles on Highway Capacity" presented at the annual conference of the Roads and Transportation Association of Canada, Toronto, Ontario, September 1974.
7. Werner, A., and J. F. Morrall, "Passenger Car Equivalencies of Trucks, Buses, and Recreational Vehicles for Two-Lane Rural Highways," *Transportation Research Record* 615, 1976.
8. Cunagin, W. D., and C. J. Messer, "Passenger Car Equivalents for Rural Highways," Report No. FHWA/RD-82/132, Federal Highway Administration, December 1982.
9. Harwood, D. W., and C. J. Hoban, "Low-Cost Methods for Improving Traffic Operations on Two-Lane Highways—Informational Guide," FHWA Report No. FHWA-IP-87-2, Federal Highway Administration (January 1987).
10. Transportation Research Board, "Research Needs for Capacity and Level of Service Analysis of Two-Lane Highways," Committee on Highway Capacity and Quality of Service, Subcommittee on Two-Lane Roads, unpublished (1993).
11. McLean, J., "Two-Lane Road Capacity 1985: Should We Acquiesce?," *Australian Road Research Board Proceedings* (1986).
12. Hoban, C. J., "Planning and Design of Rural Roads: Observations on a Year in America," Research Report AAR No. 140, Australian Road Research Board, December 1986.
13. Guell, D. L., and M. R. Virkler, "Capacity Analysis of Two-Lane Highways," *Transportation Research Record* 1194, Transportation Research Board (1988).
14. Krumins, I., "Two-Lane Highway Capacity and Level of Service Research Project: Phase III Final Report," Transportation Association of Canada, 1991 (including

- Technical Supplement 1, “Data Collection Sites,” and Technical Supplement 2, “Traffic and Driver Perception Data”).
15. Johnson, G. P., “1985 Highway Capacity Manual Misses Mark on Rural Highway Capacity,” Proceedings of the 65th Annual Meeting, Institute of Transportation Engineers, 1995.
 16. Morrall, J. F., and A. Werner, “Measuring Level of Service of Two-Lane Highways By Overtakings,” *Transportation Research Board 1287*, Transportation Research Board (1990).
 17. DeArazoza, R. D., and D. S. McLeod, “Methodology to Assess Level of Service on US-1 in the Florida Keys,” *Transportation Research Record 1398*, Transportation Research Board (1993).
 18. Botha, J. L., E. C. Sullivan, and X. Zeng, “Level of Service of Two-Lane Rural Highways with Design Speeds Less Than 60 mph,” Report No. CA/SJSU-1-RR-92-1, California Department of Transportation, June 1992.
 19. Botha, J. L., E. C. Sullivan, and X. Zeng, “Level of Service of Two-Lane Rural Highways with Low Design Speeds,” *Transportation Research Record 1457*, Transportation Research Board, 1994.
 20. Cassidy, M. J., and A. D. May, “The 1985 Highway Capacity Manual: Supplemental Report for California Conditions,” Research Report No. UCB-ITS-RR-88-1, University of California–Berkeley, February 1988.
 21. Cassidy, M. J., and A. D. May, “Research Problem Statements for 1985 Highway Capacity Manual,” Research Report No. UCB-ITS-RR-88-2, University of California–Berkeley, February 1988.
 22. Archilla, A. R., “Test and Evaluation of the TWOPAS Rural Traffic Simulation Model,” Federal Highway Administration, January 1996 (draft).
 23. Crowley, K. W., R. S. Hostetter, and G. W. Dauber, “Capacity Adjustments for Lane Width and lateral Clearance on Two-Lane, Two-Way Highways,” First Draft Final Report of Contract No. DTFH61-88-C-00063, Institute for Research, State College, PA, January 1989.
 24. Archilla, A. R., and J. F. Morrall, “Traffic Simulation on Two-Lane Highway Downgrades,” *Road and Transport Research*, Vol. 4, No.3, 1995.
 25. Werner, A., and J. Morrall, “Canadian Survey of Two-Lane Highway Problems and Planning Practices,” Final Report of Project 92-39, Transportation Association of Canada (May 1994).
 26. Harwood, D. W., and A. D. St. John, “Operational Effectiveness of Passing Lanes on Two-Lane Highways,” Report No. FHWA/RD-86/196, Federal Highway

- Administration, April 1986.
27. St. John, A. D., and D. W. Harwood, "TWOPAS Users Guide," Federal Highway Administration, May 1986.
 28. St. John, A. D., and D. W. Harwood, "TWOPAS Programmers Guide," Federal Highway Administration, July 1986.
 29. Robinson, G. K., "A Model for Simulating Traffic on Rural Roads," Technical Manual ATM No. 10, Australian Road Research Board, 1980.
 30. Hoban, C. J., R. J. Shepherd, and G. J. Fawcett, "A Model for Simulating Traffic on Two-Lane Rural Roads: User Guide and Manual for TRARR Version 3.2," Australian Road Research Board, 1991.
 31. St. John, A. D., and D. R. Kobett, "Grade Effects on Traffic Flow Stability and Capacity," *NCHRP Report 185*, Transportation Research Board, 1978.
 32. St. John, A. D., and W. D. Glauz, "Implications of Light-Weight, Low-Powered Vehicles in the Traffic Stream," Final Report of Contract No. DOT-FH-11-9434, Midwest Research Institute, October 1981.
 33. Leiman, L., A. R. Archilla, and A. D. May, "TWOPAS Model Improvements," Task 6 Working Paper, NCHRP Project 3-55(3), University of California, Berkeley, California, July 1998.
 34. May, A. D., "Traffic Performance and Design of Passing Lanes," *Transportation Research Record 1303*, Transportation Research Board, 1991.
 35. Wardrop, J. G., "Some Theoretical Aspects of Road Traffic Research," *Proceedings of the Institution of Civil Engineers*, Part II, 1952.
 36. Davis, S. C., *Transportation Energy Databook*, Edition 17, Report No. ORNL-6919, Center for Transportation Analysis, Oak Ridge National Laboratory, August 1997.
 37. Davis, S. C., *Transportation Energy Databook*, Edition 18, Report No. ORNL-6941, Center for Transportation Analysis, Oak Ridge National Laboratory, September 1998.
 38. Werner, A., and J. F. Morrall, "A Unified Traffic Flow Theory Model for Two-Lane Highways," *Transportation Forum*, Vols. 1-3, December 1984.
 39. Institute of Transportation Engineers, *Manual of Traffic Engineering Studies*.

40. McLean, J., "Traffic Capacity Values for Two-Lane Road Stereotypes in NIMPAC-Type Models," Working Document No. WD R98/004, ARRB Transport Research Ltd., Vermont South, Victoria, Australia, February 1998.
41. Harwood, D. W., and A. D. St. John, "Operational Effectiveness of Passing Lanes on Two-Lane Highways," Report No. FHWA/RD-86/195, June 1986.
42. Harwood, D. W., and A. D. St. John, "Passing Lanes and Other Operational Improvements on Two-Lane Highways," Report No. FHWA/RD-85/028, Federal Highway Administration, July 1984.
43. Rooney, F. D., *Turnouts - Summary Report: Traffic Operational Report No. 1*, California Department of Transportation, November 1976.
44. Rooney, F. D., *Turnouts: Traffic Operational Report No. 2*, California Department of Transportation, November 1976.
45. Bergh, T., *2+1 roads with Cable Barrier for Improved Traffic Safety*, in Proceedings of the Third International symposium on Highway Capacity, road Directorate, Ministry of Transport - Denmark, Transportation Research Board, June 1998.
46. Harmelink, M. D., *Volume Warrants for Left-Turn Storage Lanes at Unsignalized Grade Intersections*, Highway Research Record 211, Highway Research Board, 1967.
47. Hoban, C. J., *Model for Predicting Turning and Platooning Delays*, Internal Report AIR 359-16, Australian road Research Board. 1986.
48. Sebastian, O. L., and Pusey, R. S., *Paved-Shoulder Left-Turn By-Pass Lanes: A Report on the Delaware Experience*, Delaware Department of Transportation, October 1982.
49. Normann, O. K., "Preliminary Results of Highway Capacity Studies," *Public Roads*, Vol. 19, No. 12, 1939.
50. Normann, O. K., "Results of Highway Capacity Studies," *Public Roads*, Vol. 23, No. 4 (1942).
51. Galarraga, J. J., and F. Barsky, *Cálculo de automóviles equivalentes para camiones en pendientes específicas*, Instituto Superior de Ingeniería del Transporte, Facultad de Cs. Exactas, Físicas y Naturales—Universidad Nacional de Córdoba, Argentina (cited in Reference 21).