NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM REPORT **133**

PROCEDURES FOR ESTIMATING HIGHWAY USER COSTS, AIR POLLUTION, AND NOISE EFFECTS

HIGHWAY RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING

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PROCEDURES FOR ESTIMATING HIGHWAY USER COSTS, AIR POLLUTION, AND NOISE EFFECTS

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AREAS OF INTEREST:
- TRANSPORTATION ECONOMICS
- HIGHWAY DESIGN
- ROAD USER CHARACTERISTICS
- URBAN COMMUNITY VALUES
- URBAN TRANSPORTATION SYSTEMS

HIGHWAY RESEARCH BOARD
DIVISION OF ENGINEERING NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING 1972

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Highway Research Board of the National Academy of Sciences-National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as: it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state, and local governmental agencies, universities, and industry; its relationship to its parent organization, the National Academy of Sciences, a private, nonprofit institution, is an insurance of objectvity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway departments and by committees of AASHO. Each year, specific areas of research needs to be included in the program are proposed to the Academy and the Board by the American Association of State Highway Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are responsibilities of the Academy and its Highway Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

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The study reported herein was undertaken under the aegis of the National Academy of Sciences—National Research Council. The National Cooperative Highway Research Program, under which this study was made, is conducted by the Highway Research Board with the express approval of the Governing Board of the NRC. Such approval indicated that the Board considered that the problems studied in this program are of national significance; that solution of the problems requires scientific or technical competence, and that the resources of NRC are particularly suitable for the oversight of these studies. The institutional responsibilities of the NRC are discharged in the following manner: each specific problem, before it is accepted for study in the program, is approved as appropriate for the NRC by the NCHRP Program Advisory Committee and the Chairman of the Division of Engineering of the National Research Council.

Topics for synthesis are selected and defined by an advisory committee that monitors the work and reviews the final report. Members of the advisory committees are appointed by the Chairman of the Division of Engineering of the National Research Council. They are selected for their individual scholarly competence and judgment, with due consideration for the balance and breadth of disciplines. Responsibility for the definition of this study and for the publication of this report rests with the advisory committee.

Although reports in this category are not submitted for approval to the Academy membership nor to the Council, each report is reviewed and processed according to procedures established and monitored by the Academy's Report Review Committee. Such reviews are intended to determine *inter alia*, whether the major questions and relevant points of view have been addressed, and whether the reported findings, conclusions and recommendations arose from the available data and information. Distribution of the report is permitted only after satisfactory completion of this review process.

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FOREWORD

By Staff

Highway Research Board

This report is recommended to highway engineers, planners, and policymakers responsible for the the planning and evaluation of highway programs. It contains information on procedures that can be used to select level of traffic service on the basis of user costs and related consequences of air and noise pollution.

Recommended techniques for conducting comprehensive economic analyses of planned highway projects are often slow, cumbersome, or of questionable validity. In addition to the need for simplified and authoritative procedures, a need exists for an economic analysis supplement to the *Highway Capacity Manual* utilizing the manual's definitions of highway types, levels of highway service, and other key concepts. The *Highway Capacity Manual* describes six levels of service for each of five types of highway facilities and provides detailed procedures for determining levels of service under various conditions. Up to now, however, these levels of service have not been quantified with respect to user costs and related consequences.

The objectives of this project were to evaluate data related to user costs on various highway facilities under different levels of service, volumes, and other conditions, and to develop a methodology for relating these variables to user costs and to air pollution and noise effects. The research was to include sensitivity analyses to identify the highway design and situation variables that have major impacts on output variables and therefore should be included in results presented to highway decision makers.

The research team from Stanford Research Institute compiled and updated motor vehicle running cost data for use in calculating relative road user costs at different levels of highway service and as affected by details of geometric design and traffic performance. By use of Appendix A of the *Highway Capacity Manual,* relationships were derived for peak-hour volume per lane as a function of AADT per lane pair. Queuing was analyzed based on the shock wave method for uninterrupted flow and the deterministic method for interrupted flow. A methodology for estimating vehicle emissions was developed based on prototypical vehicle results and projected trends in pollution control standards.

Highway engineers will find this report of special value in helping to determine the economic benefits to the highway user of alternative facility proposals.

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PROCEDURES FOR ESTIMATING HIGHWAY USER COSTS, AIR POLLUTION, AND NOISE EFFECTS

SUMMARY

The text of this report constitutes a manual of procedures for estimating road user costs, the imputed value of travel time, and air and noise pollution associated with existing or proposed highway facilities under different levels of traffic. The report is intended as an economic supplement to the "Highway Capacity Manual—1965" (*1*); hence, it uses the HCM's definitions of highway types, levels of highway service, and other key concepts.

Chapters Two through Six of the report constitute the findings of the study, in the form of detailed procedures for analysis organized according to the steps indicated as follows under each chapter heading.

Chapter Two: Assumptions and Project Description:

1. Select economy study features and variables.
2. Identify and describe projects and project alternatives to be considered (Worksheet 1).
3. Define relevant highway network and sections.

Chapter Three: Travel Time and Running Costs:

4. For uninterrupted flow sections of each alternative: define capacity; estimate peak and off-peak hourly traffic demand; and compute demand/capacity ratios for years 1 and 20 or other representative years (Worksheet 2). For signalized intersections: go to Worksheet 5 after estimating traffic demand.
5. Analyze congested traffic (queueing) for sections and intersections in which demand exceeds capacity (Worksheets 2A and 5A).
6. Enter charts and tables to obtain unit vehicle running costs and travel time as a function of the volume to capacity ratio (Worksheets 3 and 3A).
7. Using other charts and tables, add the effects of grades and curves and stops (Worksheets 3 and 4).
8. Multiply all resulting unit values (user cost and time) by distance (the roadway section length) and number of vehicles to obtain annual running cost and travel time per section for peak and off-peak periods (Worksheets 3 again).

Chapter Four: Air Pollution and Noise Effects:

9. Estimate air pollution effects (Worksheet 6).
10. Calculate and describe noise impacts (Worksheet 7).

Chapter Five: Accident Costs and Consumers' Surplus:

11. Estimate accident costs (Worksheet 8).
12. Obtain average trip time and costs for all parallel sections of each alternative, and multiply time and cost savings by average traffic volume to find the

benefits (change in consumers' surplus, or total savings in travel time value and user costs) for each alternative in years 1 and 20 (Worksheet 9).

Chapter Six: Summary and Interpretation:

13. Obtain present worths of savings in user costs and passenger-car travel time value for each alternative, and calculate the incremental benefit/cost ratio or other economic indexes (Worksheet 10).

14. For highway improvement programming or comparison of several projects: rank project alternatives in order of decreasing incremental benefit/cost ratios, and apply budget limitation to cumulative investment cost of projects to determine the last project to be included in the economically optimal set (Worksheets 11 and 12).

Figure 2 shows the sequence of the worksheets; illustrative problems appear on the worksheets provided in the text of the report. Additional problems and some shortcuts are given in Chapter Seven, "Applications." Appendices A through D provide reference charts on running time and road user cost factors, an account of the research approach (including the empirical basis of the suggested time and cost factors and procedures), and blank worksheets.

CHAPTER ONE

INTRODUCTION AND RESEARCH APPROACH

PURPOSE AND SCOPE OF REPORT

The "Highway Capacity Manual—1965" (*1*; referred to herein as HCM) describes six levels of service for each of five types of highway facilities. It also provides detailed procedures for determination of levels of service under various conditions. At present, however, these levels of service have not been quantified with respect to user costs and related consequences. A need exists for such information that could be used by engineers, planners, and policy makers to select levels of traffic service for highway improvements on the basis of user costs and their components—running costs and accident costs—as well as travel time, air pollution, and noise effects.

The objectives of this study were to evaluate various highway facilities under different levels of service and other conditions, and to develop a methodology that would relate these variables to user costs, travel time, air pollution, and noise effects. The methodology was to be presented in tables and graphs designed to minimize the amount of additional calculations needed to use the results.

This report is in the form of a procedural manual to meet these objectives. The procedures are summarized in the Summary. The possibility of deriving final results directly from look-up procedures on a series of charts was explored, but this approach would have required thousands of charts for the necessary degree of flexibility and accuracy instead of the dozens of charts that are now used in conjunction with several worksheets. Also, the sensitivity procedures envisioned in the original objectives of the project, to identify variables that should be included, turned out not to be necessary in view of the desirability of a flexible procedure permitting inclusion of all relevant variables.

By way of comparison or contrast with other references covering some of the same material, this report updates and extends the AASHO's *Road User Benefit Analyses for Highway Improvements* (*2*) and *A Manual for Conducting Highway Economy Studies* (*3*). Further, it provides detailed procedures and worksheets for solving many of the types of problems dealt with in *Economic Analysis for Highways* (*4*), in "Strategies for the Evaluation of Alternative Transportation Plans" (*5*), and in "Summary and Evaluation of Economic Consequences of Highway Improvements" (*6*). In addition, it gives procedures for economic analysis of some problems (such as congested traffic flow, the HCM's level of service F) that are not covered elsewhere because most guidelines for highway economy studies are based on estimates of AADT (average annual

daily traffic). Congested traffic flow can be analyzed only by using hourly or shorter time periods, and this report does use hourly periods.

The preceding references should be consulted for the theoretical foundations of this report, which is restricted to the presentation of procedures for highway economy studies (including air and noise pollution calculations). Voluminous information that is readily available elsewhere has not been reproduced here; users are therefore advised to have at hand the reports of Winfrey (*4*) and Gordon et al. (*25*), as well as the HCM (*1*).

The report is primarily for use in comparing proposals for highway improvement where level of service is a factor. Such proposals can include the following types of projects:

1. Construction of new freeways, expressways, and roads without access control, to supplement or replace existing roads.
2. Widening of existing roads or reconstruction to higher geometric standards.
3. Straightening or eliminating curves.
4. Grade reductions and passing lanes (on grades or mountainous roads).
5. Highway programming, considering various levels and mixes of highway improvement proposals.

To avoid unduly increasing the complexity and bulk of the report, matters such as improved traffic signal control, intersection improvements, weaving sections, curve design, and railroad/highway grade separations are not covered. However, if these matters can be translated (through use of the HCM or traffic engineering references) into influences on highway capacity or on traffic speed and delay, their effects on user costs can be estimated by the procedures in this report. For example, the effect of left turns on delay can be accounted for by their effect on intersection capacity as computed by the methods of the HCM. Decreases in capacity due to left turns will cause increase in average delays to traffic. Similarly, reductions in the capacity of freeways due to weaving sections will result in decreased speed and increased travel time for the average vehicle.

The report is intended for studies in the United States and in other countries having well-developed economies and road systems, and does not cover problems that are encountered only in developing countries. It therefore focuses on the effects of highway construction on users and operators of highways and does not include indirect effects, such as changes in land use, land values, or economic growth. This emphasis is consistent with current economic theory, which concludes that in developed countries substantially all net changes in real income and economic welfare caused by highway improvements are covered by the resulting savings in transportation costs. Information about community impacts is also frequently of crucial importance; however, community effects other than air and noise pollution are outside the scope of this report (see Refs. *6, 7, 8, 9, 10,* and *11* for suggestions on evaluating community impacts of transportation systems).

Figure 1 shows the coverage of the report in relation to the total highway design and evaluation process.

RESEARCH APPROACH

The conduct of this study followed the general sequence described in the following and discussed further in Appendix B (technical terms are defined in the next section):

1. Development of a methodology for expressing the relationships between traffic volume, user costs, and air and noise pollution effects on traffic on different types of highways.
2. Updating and refinement of unit vehicle operating cost tables. Briefly, the researchers began with the tables developed in the early 1960's by Winfrey (*4*) for automobiles and four truck types (out of which two representative truck types were selected) because of their completeness and internal consistency; modified the cost factors to reflect selected findings by Claffey (*12*), notably fuel consumption and, for speed changes, tire costs as well; and updated the costs to 1970 price levels.
3. Collection of speed profile data for use in deriving the costs of traffic speed changes as a function of volume/capacity ratio; derivation of functions for the average values of such costs; and addition of speed change costs to the cost of operation at uniform speeds on level tangents.
4. Development of procedures for analysis of level of service F, involving congestion and queueing.
5. Refinement and adaptation of relationships between traffic volume, air and noise pollution effects, and accident costs, for different types of facilities.
6. Development and use of computer programs to calculate the relationships between traffic volume and different variables and to analyze the sensitivity of the relationships.
7. Assembly of the components into a draft report; development of sample problems; and review of the procedure and sample problems by selected state highway agencies for clarity, simplicity, and practicality.

DEFINITIONS

The definitions that follow include the principal technical terms used herein. Readers can consult the HCM (*1*, Chap. Two), from which most of these definitions were derived, for other terms of possible interest. The only departure from HCM terminology in this report is the use of "average speed" in place of the HCM's "average over-all traffic speed."

Capacity. The HCM defines capacity as the maximum number of vehicles that have a reasonable expectation of passing over a given section of a lane or a roadway in one direction (or in both directions for a two-lane or a three-lane highway) during a given time period under prevailing roadway and traffic conditions. In the absence of a time modifier, capacity is an hourly volume.

Passenger Car. A motor vehicle with seating capacity up to nine persons, including for capacity and economy study purposes taxicabs, station wagons, and two-axle, four-tired pickups, panels, and light trucks.

Truck. A motor vehicle having dual tires on one or more axles, or having more than two axles, designed for cargo rather than passengers. Trucks are represented in this report by two types—single-unit trucks and 3-S2 truck

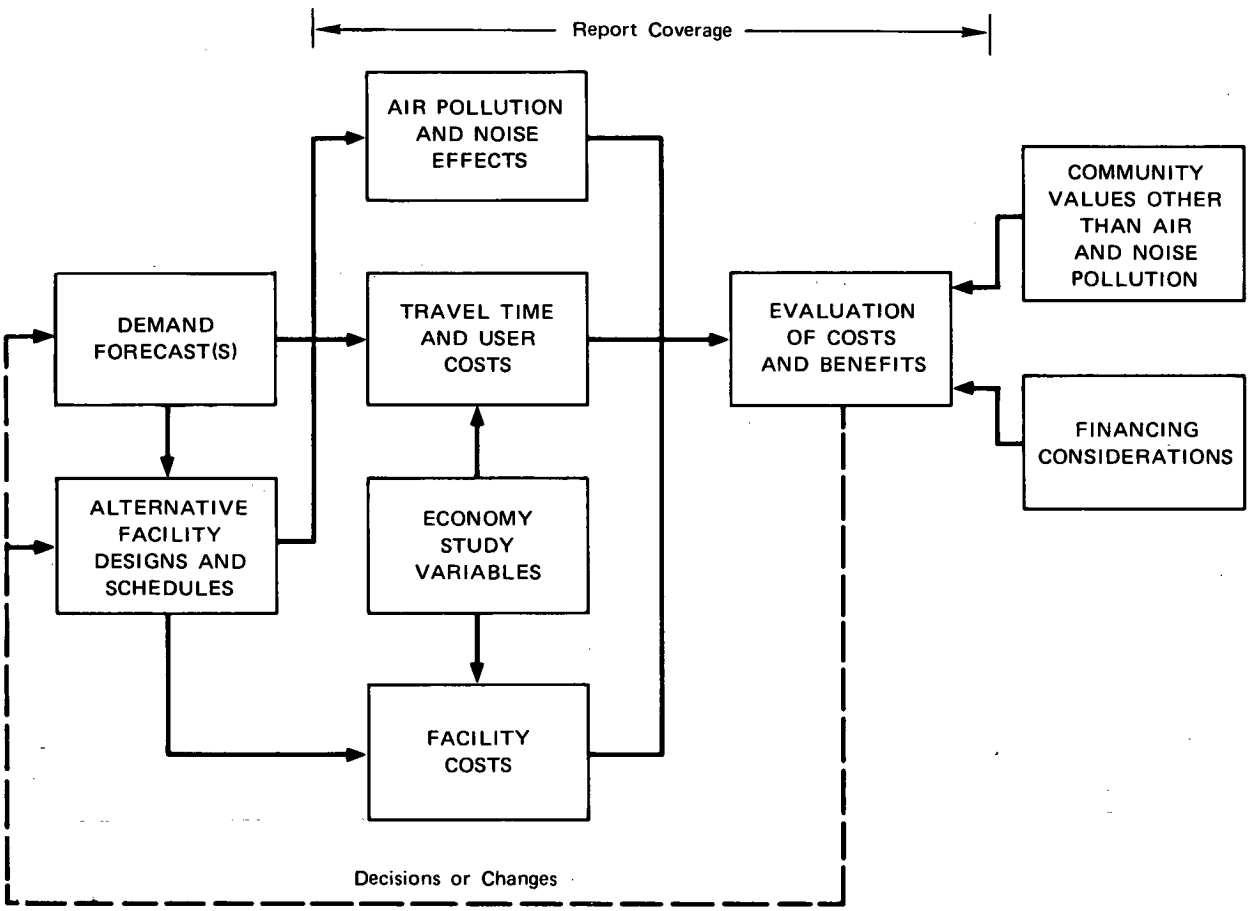


Figure 1. Report coverage.

combinations—as defined in the introduction to Appendix A. Buses, defined as vehicles with seating capacities of ten or more passengers, should be counted as single-unit trucks for capacity and economy study purposes.

Average Speed. This term is employed as shorthand for "average over-all traffic speed" as defined in the HCM; that is, "the summation of distances traveled by all vehicles or a specified class of vehicles over a given section of highway during a specified period of time, divided by the summation of over-all travel times."

Running Speed. The speed over a specified section of highway, being the distance divided by running time (the time the vehicle is in motion). Average running speed is the same as average speed if there are no stops; otherwise, it is higher. Both average speed and average running speed include the effects of speed changes (variations from constant speed).

Average Tangent Speed. The average speed of traffic on the tangent portions of a road under given traffic conditions, free from influence by curves, signals, or STOP signs.

Design Speed. A speed selected for purposes of design and correlation of those features of a highway, such as curvature, superelevation, and sight distance, on which the safe operation of vehicles depends.

Operating Speed. The highest over-all safe speed at which a driver can travel on a given highway under favorable weather conditions and under prevailing traffic conditions.

Highway Types. Descriptions of the five major highway types for which speed/volume relationships are given in the HCM are as follows. The first three types generally provide "uninterrupted flow" and the last two types provide "interrupted flow," according to HCM definitions (i.e., an "interruption" is a cause external to the traffic stream, such as a traffic signal or STOP sign).

1. *Freeways and other expressways* (or Freeways and expressways). Expressways are divided arterial highways for through traffic with full or partial control of access and generally with grade separations at major intersections; freeways are expressways with full control of access.

2. *Multilane rural highways.* Roads with four or more lanes, without significant access control features and with low enough adjacent development to permit speed limits of greater than 40 mph * and signalization (signals or STOP signs) with average spacings of more than 1 mile.

* This 40-mph dividing line is specified in the HCM (*1*, p. 263). In some other parts of the HCM a 35-mph speed limit is specified as the dividing line between arterials and highway types 2 and 3 (multilane rural highways and two-lane highways), but 40 mph appears to be a more representative speed. New or improved arterials with 45-mph speed limits are not uncommon.

3. *Two-lane highways.* Roads with two lanes, speed limits generally greater than 40 mph in both directions, and average signal spacing more than 1 mile. They may have varying degrees of access control.

4. *Arterials, or arterial streets, or urban and suburban arterials.* Major streets and highways outside the central business district having *either* speed limits of 40 mph or less, *or* average signal spacing of 1 mile or less.

5. *Downtown streets.* Central business district streets for which through traffic is often of secondary importance to facilitating access to abutting property by cars, buses, trucks, and pedestrians.

Peak-Hour Factor. A ratio of the volume occurring during the peak hour to the maximum rate of flow during a given time period within the peak hour. It is a measure of peaking characteristics, whose maximum attainable value is 1. The term must be qualified by a specified short period within the hour; this is usually 5 or 6 min for freeway operation and 15 min for intersection operation. For example, "a peak-hour factor of 0.80 based on a 5-min rate of flow."

Average Annual Daily Traffic. The total yearly volume divided by the number of days in the year, commonly abbreviated as AADT.

K *and* D *Factors.* K is the percentage of AADT represented by two-way traffic volume in the design hour, and D is the percentage of design hour traffic in the heaviest direction of flow. Then the one-way design hour volume is derived as follows: One-way $DHV = (K)(D)$ (two-way AADT in design year). The product of K and D thus gives a single factor for applying to AADT estimates in the design year to obtain the DHV.

Values for K and D tend to decrease as traffic volume approaches highway capacity in more hours of the year; hence, typical values for rural areas are higher than for urban areas. Illustrative values for K, D, and KD in the 30th highest hour of the year—sometimes used as the design hour—are as follows. See the HCM (*1*, p. 40) for further details.

AREA	K (%)	D (%)	KD (%)
Rural	13	65	8.5
Suburban	11	60	6.6
Urban	9	55	5.0

Level of Service. A qualitative measure of the freedom of a flow of traffic from constraints, interruptions, or other inconveniences, relative to the best possible conditions for a given type of highway facility. Other things being equal, the level of service for a given highway is an inverse function of the hourly traffic volume or the traffic density at the time of observation. See the HCM (*1*, pp. 7-8, 78-87) for added discussion.

The HCM defines six alphabetically designated level of service categories in terms of speed or volume/capacity ratios for each of the five preceding highway types, ranging from level A, free flow, to level F, forced flow. For the first four highway types, level of service is defined in terms both of speed and of the volume/capacity (v/c) ratio of traffic; for downtown streets, only the speed-based definition is used. The speed-based definition of level of service categories is given by the HCM in terms of operating speed (highest attainable safe speed) for the first three highway types, because operating speed coincides with design speed at zero volume and thus serves as the principal speed constraint or determinant. Level of service is given in terms of average speed, however, for the last two highway types (arterials and downtown streets). Table 1 gives, first, the speed-based levels of service by highway type, both as given by the HCM and with operating speed for the first three highway types translated to average speed. The conversion to average speed is for consistency with the present report as well as with arterial and downtown street speed-based service levels.* Next, Table 1 gives the v/c ratio-based levels of service by highway type as given by the HCM.

Table 1 is used by entering the table at the estimated operating speed or average speed, as indicated for given highway types in the top section of the table, to find the level of service implied by the estimated speed. Alternatively, for facilities of 70-mph design speed, the lower section of the table can be entered at the estimated v/c ratio to find the implied level of service (for design speeds of less than 70 mph, the speed-based definition of level of service, indicated by the top section of the table, must be used).

Users of this report also should be aware of the following characteristics of the HCM's level of service concept:

1. Levels of service based on v/c ratios are not meaningful for arterials and city streets because their capacity is typically constrained by signalized intersections, not by the mid-block or intersignal sections; hence, no entries are given for these two highway types in the lower section of Table 1. The HCM (Table 10.13) does give illustrative v/c ratios for arterials, but they vary according to the effectiveness of signal progression and load factor.

2. Shifts in the designated level of service can represent either very small or fairly large changes in speed or v/c ratio. For example, a shift from level of service C to D for freeways and expressways can represent a change of anywhere from 1 mph (50 to 49 mph) to 14 mph (54 to 40 mph). Thus, level of service is at best a rough index of traffic characteristics. The solution used by many highway departments is to design to the *lower limit* of service levels C or D; but these also could be designated as the upper limits of service levels D or E.

3. Because the level of service categories are represented by specific and unchangeable numbers defined by the high-

* The HCM generally gives operating speed in its tables and charts, but the researchers have converted operating speed to average speed throughout the present report to (1) provide a single speed more representative of the speeds of all vehicles; (2) make speed immediately translatable into travel time; and (3) enable easy confirmation of speed predictions by observed traffic behavior (either through timing cars that "float" with the traffic, or through timing and averaging the speeds of all vehicles). A formula developed by Makigami et al. (*13*), based in turn on AASHO procedures, was used for converting operating speed to average running speed. The formula is $AS = OS - \left(\frac{DS}{10} 1 - \frac{v}{c}\right)$, in which AS is average running speed, DS is design speed or speed limit for the facility type, v is volume, and c is capacity (both v and c are specified in vehicles per lane per hour). The formula results in a curve that starts at $DS/10$ mph below the operating speed when $v = 0$, and coincides with operating speed when $v = c$.

TABLE 1

SPEED RANGES AND VOLUME/CAPACITY RATIOS BY LEVEL OF SERVICE AND HIGHWAY TYPE

	OPERATING SPEED (MPH)			AVERAGE SPEED (MPH)				
LEVEL OF SERVICE	FREE-WAYS AND EXRESS-WAYS	MULTI-LANE RURAL HIGHWAYS	TWO-LANE HIGHWAYS	FREE-WAYS AND EXPRESS-WAYS [a]	MULTI-LANE RURAL HIGH-WAYS [a]	TWO-LANE HIGH-WAYS [a]	ARTERIALS	DOWN-TOWN STREETS
A Free flow	≥60	≥60	≥60	≥56	≥55	≥54	≥30	≥25
B Stable flow (upper speed range)	55-59	55-59	50-59	52-55	51-54	47-53	25-29	20-24
C Stable flow	50-54	45-54	40-49	48-51	43-50	38-46	20-24	15-19
D Approaching unstable flow	40-49	35-44	35-39	40-47	35-42	34-37	15-19	10-14
E Unstable flow (capacity)	30-39 [b]	30-34 [b]	30-34 [b]	30-39 [b]	30-34 [b]	30-33 [b]	about 15	10
F Forced flow	<30 [b]	<30 [b]	<30 [b]	<30 [b]	<30 [b]	<30 [b]	<15	Stop and go

	VOLUME/CAPACITY RATIO FOR 70-MPH DESIGN SPEED				
	FREEWAYS AND EXPRESSWAYS			MULTI-LANE RURAL HIGHWAYS	TWO-LANE HIGHWAYS
LEVEL OF SERVICE	4-LANE	6-LANE	8-LANE		
A Free flow	≤0.35	≤0.40	≤0.42	≤0.30	≤0.20
B Stable flow (upper speed range)	0.36-0.50	0.41-0.58	0.43-0.63	0.31-0.50	0.21-0.45
C Stable flow	0.51, to 0.75×PHF	0.59, to 0.80×PHF	0.64, to 0.83×PHF	0.51-0.75	0.46-0.70
D Approaching unstable flow	0.75 ×PHF to 0.90×PHF	0.81×PHF to 0.90×PHF	0.84×PHF to 0.90×PHF	0.76-0.90	0.71-0.85
E Unstable flow (capacity)	0.91×PHF to 1.00	0.91×PHF to 1.00	0.91×PHF to 1.00	0.91-1.00	0.85-1.00

[a] The curves for 70-mph design speeds, from HCM Figs. 9.1, 10.1, and 10.2a, where used for deriving v/c in the formula to convert operating speed to average speed (see text). For freeways, the six-lane curve was used. The two-lane highways have 100 percent passing sight distance over 1,500 ft.

[b] An operating speed of 30 mph, denoted as "approximate," is given by HCM Tables 9.1, 10.1, and 10.7 for the boundary between levels of service E and F. This boundary is equivalent to a v/c ratio of 1.0, or capacity. Note, however, that the following capacity speeds are given by the *curves* in HCM Figs. 9.1, 10.1, and 10.2:

DESIGN SPEED	FREEWAYS AND EXPRESSWAYS	MULTILANE RURAL HIGHWAYS	TWO-LANE HIGHWAYS
70 mph			
6 and 8 lane	35	n.a.	n.a.
4 lane	30	30	n.a.
2 lane	n.a.	n.a.	30
60 mph	30	30	28.5
50 mph	28	27	27

These values, although approximate, reflect in general the observed tendency for the speed at which maximum capacity is attained to decrease as the design speed of the facility decreases.

Note: PHF = peak hour factor; n.a. = not applicable.

Source: HCM (*1*).

est facility type (the speeds associated with highways of 70-mph design speed), they are of limited use in describing the *relative* service at different v/c ratios that is offered by highways of lower design speed (e.g., an urban freeway of 50-mph design speed ranges entirely between levels of service D and E; it therefore cannot, by definition, attain service levels A, B, or C, even at low volumes).

4. Level of service is an hourly concept that would have no meaning as a composite rating for highway economy studies, in which user time and costs are summed over all highway sections for all peak and off-peak hours of two widely separated years.

Because of these level-of-service characteristics, the researchers have taken the approach in graphs herein of showing the entire continuous range of user costs and other

consequences associated with different v/c ratios for each type of highway (rather than, for example, associating average estimated consequences with each discrete level of service). Readers are then free to consult Table 1 for translation of resulting speeds or v/c ratios into level of service, and Worksheet 3 (Fig. 11) provides space for entering the level of service designation for each highway section in the peak and off-peak periods.

Service Volume. The maximum number of vehicles that can pass over a given section of a lane or roadway in one direction on multilane highways (or in both directions on a two- or three-lane highway) during 1 hr while operating conditions are maintained corresponding to the selected or specified level of service.

The procedure given in the HCM for computing service volume for capacity is:

$$SV = 2{,}000\ N\ W\ T\ (v/c) \qquad (1)$$

in which

SV = service volume (total for one direction);
N = number of lanes in one direction;
W = adjustment for lane width and lateral clearance (from HCM tables; shoulder adjustment may also be necessary);
T = truck factor (from HCM tables); and
v/c = volume to capacity ratio.

This is an extremely versatile formula. For example, solving for the v/c ratio permits finding operating or average speed and the associated level of service. The needed number of lanes can also be calculated by setting SV equal to the design hour volume (see "K and D Factors") at the designated level of service, which determines the v/c ratio, and solving for N. Finally, the formula can be reduced to the formula for capacity ($c = 2{,}000\ N\ W\ T$) by letting SV = capacity and $v/c = 1.0$.

Incremental Cost. The net change in dollar costs directly attributable to a given decision or proposal compared with some other alternative (including the existing situation or "do-nothing" alternative). This definition includes any resulting cost reduction, or negative costs. The only costs that are relevant to a given proposal are incremental future costs, in contrast to sunk costs of the past.

Transportation cost. The sum of highway investment cost and road user costs, the value of travel time, and annual highway maintenance cost. These terms are defined as follows.

1. *Highway Investment Cost.* Total investment required to prepare a highway improvement for service, including engineering design and supervision, right-of-way acquisition, construction, signals and signs, and landscaping.

2. *User Costs (or Road User Costs).* The sum of motor vehicle running cost, truck time cost, and traffic accident cost.

   a. *Motor vehicle running cost.* The mileage-dependent cost of running automobiles, trucks, and other motor vehicles on the highway, including the cost of fuel, tires, engine oil, maintenance, and a portion of depreciation. Operating and ownership costs that do not vary with mileage are excluded from running cost; e.g., license and parking fees, insurance premiums, the time-dependent portion of depreciation, and any costs of off-highway use.
   b. *Truck time cost.* The direct, time-dependent (rather than mileage-dependent) cost for running trucks and buses on the highway.
   c. *Accident cost.* The cost attributable to motor vehicle traffic accidents, usually estimated by multiplying estimated accident rates by average unit accident costs.

3. *Value of Travel Time.* The product of passenger-car travel time multiplied by the average unit value of time.

   a. *Passenger-car travel time.* The total vehicle-hours of time traveled by passenger cars, estimated from average speed, distance of travel, and amount of traffic.
   b. *Unit value of time.* The value attributed to 1 hr of passenger-car travel time.

4. *Highway Maintenance Cost.* The cost of keeping a highway and its appurtenances in serviceable condition. (Operating costs, such as for traffic control and lighting, should be included with maintenance costs if operating costs differ appreciably between two highway alternatives.)

Present Worth. The present amount that is equivalent to specified amounts of money or time in different time periods, at a given discount rate. Two related considerations underlie the need for computing present worths: (1) the fact that money has a time value or capital cost, due to its productiveness and scarcity (see the following definition of discount rate), and (2) the need in an economy study for comparing or summing outlays or savings of money or time in different time periods.

Discount Rate. A percentage figure representing the opportunity cost of capital for an investment, used for discounting future costs and benefits for a project to an equivalent present worth or to equivalent annual costs.

Study Period (or Analysis Period). The time period chosen for summation of incremental costs in an economic analysis. The final year of construction is designated year 0 (zero), and subsequent years are designated year 1, year 2, etc. (Projects involving stage construction that extends over more than four or five years should either be divided into separate projects for the separable stages, or use the final year of construction for the first major stage as year 0, with subsequent capital outlays being discounted to their present worth in year 0.)

Residual or Salvage Value. The value of an investment or capital outlay remaining at the end of the study period.

Budget Planning Period. The years during which all projects being considered in a given highway construction program are grouped for the purpose of deriving an optimal set of projects. For example, if the budget planning period is FY '75-'78, all projects being considered for construction during that period would be analyzed together.

Project. Any relatively independent component of a proposed highway improvement. By this definition, independent links of a large improvement proposal can be

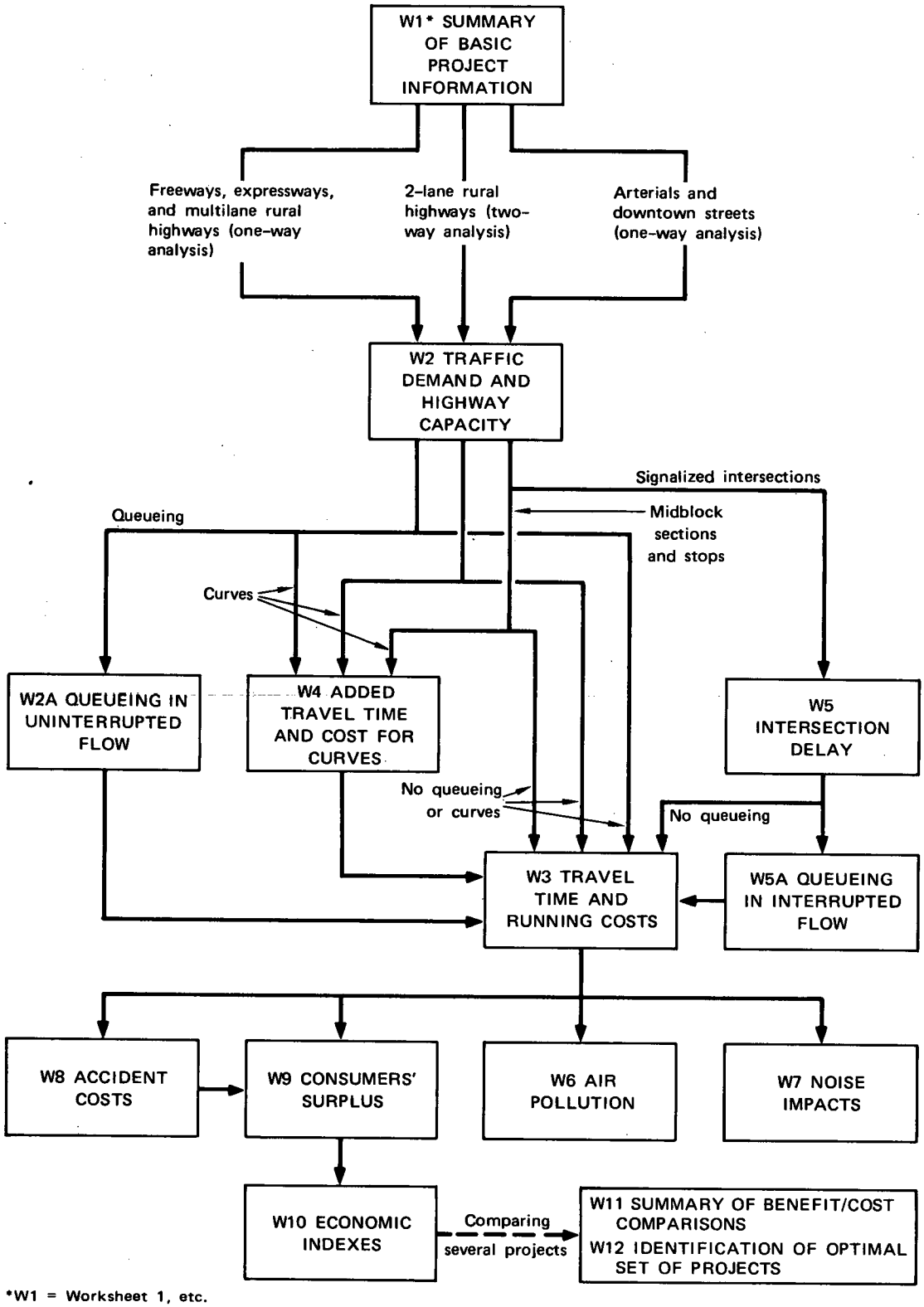


Figure 2. Organization of worksheets.

evaluated separately, and individual contracts or work orders usually can be considered as separate projects. However, highly interdependent sections of a highway improvement (such as a bridge and its approach roads) ordinarily should be considered as one project for economy study purposes.

Project alternatives are any variations to the basic project plan that (1) involve significantly different investment

costs, (2) result in significantly different levels of service or demand by highway users, or (3) incorporate different route locations or other distinctive design features.

ORGANIZATION OF WORKSHEETS

The 12 worksheets used in the report provide for analysis of all types of highways, but the worksheets used for a particular highway will vary according to its type and alignment and the presence of traffic signals or queueing. Figure 2 summarizes the procedural variations for different highways and conditions. These variations are described further in the introduction to Chapter Three as they relate to the procedure from Worksheet 2 through to the end of Worksheet 3, where the greatest number of variations is possible.

CHAPTER TWO

ASSUMPTIONS AND PROJECT DESCRIPTION

ECONOMY STUDY VARIABLES

General features and numerical variables of the economy study can be selected by users of this report from a wide variety of possibilities, and each user should make his own selection in harmony with local conditions and policies. However, to illustrate the use of the report, economy study features and variables have been incorporated in the procedures that would, in the opinion of the researchers, be generally reasonable and appropriate for current use. These features and variables are summarized as follows. See Klein (*14*, Chap. VI) for a detailed rationale.

1. Future costs and benefits are discounted to their present worth at the rate of 8 percent, compounded annually. This discount rate is based on the estimated opportunity cost of capital to the taxpayer; i.e., the estimated average market rate of return that would be achieved if more highway investment funds were left in private hands rather than being paid to the government in taxes. A range of 6 percent to 10 percent is common in current economy studies of public projects; for example, the U.S. Office of Management and Budget recommends a 10 percent discount rate for Federal Government economy studies (see ref. *15*). The possible effects of uniform future price increases—inflation—should be ignored. See Lee and Grant (*16*) for details.

2. A study period of 25 years is used, which is somewhat longer than the typical 20-year traffic forecast. The suggested study years (periods for which traffic demand and economy study data are estimated) are year 20, corresponding to the usual design year, and year 1. Other years can readily be added or substituted, if desired. Methods for extrapolating the results of individual study year calculations to the entire study period are embodied in Worksheet 10 (Fig. 31).

3. Residual value is estimated as the full cost of land for right-of-way (excluding legal costs, moving and demolition expenses, etc.) plus 50 percent of the cost of earthwork, grading, and structures, due to the permanence or long life of these features.

4. A travel time value of $3.00 per passenger-car hour is used, based on a range of $2.00 to $2.50 for the value of time per person. The rate of $3.00 per vehicle assumes an average vehicle occupancy of 1.5 persons per vehicle at a $2.00 value of time (more typical of rural areas) down to 1.2 persons per vehicle at the $2.50 value of time (more typical of urban areas, to reflect higher income levels and a higher proportion of work trips).

Thomas and Thompson (*17*) have shown that travel time values vary considerably with trip purposes and duration, geographic locations, and the traveler's income level; the range of $2.00 to $2.50 per person per hour was selected as generally representative of average figures. Truck time costs of $5.00 per vehicle-hour are used, based on updating and averaging the costs suggested by Adkins et al. (*18*).

5. The basic economic indexes provided for are benefit/cost ratios, both excluding and including the value of passenger-car travel time. Benefit/cost ratios are calculated by including user benefits, highway maintenance costs, and residual value in the numerator of the ratio. The internal rate of return and equivalent uniform annual cost also can be calculated if desired. The consumers' surplus approach is used for estimating benefits in order to provide for generated or induced traffic.

IDENTIFICATION AND DESCRIPTION OF PROJECTS AND ALTERNATIVES

The next step is to identify and describe all highway improvement projects and alternatives to be included in the analysis. The types of projects appropriate for analysis are listed in Chapter One.

Worksheet 1 (Fig. 3) * is provided for entering the basic descriptive information regarding each project alter-

* An example of each worksheet appears with the instructions for its use; copies of blank worksheets are provided in Appendix D.

Worksheet 1

SUMMARY OF BASIC PROJECT INFORMATION

Project No. _____ Construction Period _____ Highway Type(s) _____

1. System, route, and location _____

2. Type of improvement _____

3. Description of alternative plans

4. Alternative					
5. Highway investment cost, thousands of dollars					
5.1 Land for right-of-way					
5.2 Earthwork, grading, and structures					
5.3 All other					
5.4 Total					
6. Annual maintenance cost, thousands of dollars					
7. Length, miles					
8.					
9.					
10.					
11.					
12.					

Figure 3. Worksheet 1 example.

native. The worksheet is designed with the most complex projects in mind, such as construction of new roads, and can be adapted or shortened for other types of projects.

Instructions for completing Worksheet 1 follow.

Heading: Enter the project number, the years during which construction is planned or anticipated (e.g., FY '75-'76), and the highway type or types according to the HCM classification scheme described in Chapter One.

Item 1: Specify the highway system, route, and geographical location (e.g., FAP state route 81, sections ab and bc, from 6.1 to 9.1 miles southeast of Sampton).

Item 2: Describe briefly the type of improvement contemplated, based preferably on some standard list of project types, and specify whether location is in a rural, urban, or suburban area (for example, widening of existing road, rural location).

Item 3: Describe briefly the alternative project plans in order of increasing initial cost; for example:

1. Widen to four lanes with no median.
2. Widen to four lanes with 20-ft median.

Item 4: Enter the cost designation for each of the alternatives being compared. The designation 0 (zero) in the first column is suggested for the existing or "do-nothing" alternative, with successive alternatives being numbered 1, 2, 3, etc., in ascending order of initial cost, in the succeeding columns. For projects requiring more than the available columns to identify all of the alternatives, columns can be added to the right side of this or other worksheets as needed, or additional worksheets can be used.

Items 5 and 6: Enter the total investment cost of the improvement and the annual maintenance cost, in thousands of dollars. (Changes in annual maintenance costs over time should be reflected in a single equivalent annual maintenance cost figure.)

Item 7: Enter the total length of each alternative, in miles, to the nearest hundredth of a mile.

Remaining items: Blank items are provided for additional data that may be of interest for certain projects; for example:

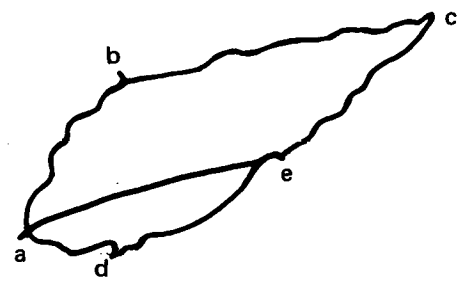
1. Total rise and fall, feet.
2. Maximum gradient, percent.
3. Maximum curvature, degrees.
4. Type or timing of signal control system.
5. Turn counts at intersections or interchanges.
6. Per-mile frequency counts of:
   Stop signals.
   STOP signs.
   Cross streets.
   Commercial and residential driveways.
   Freeway on- and off-ramps.

Worksheet 2 (Fig. 6) provides more detailed information and estimates regarding separable sections (if any) of each alternative listed on Worksheet 1, as the basis for calculations regarding user costs and other consequences. The next part of this report contains guidelines for identifying the appropriate network and sections for each alternative.

DEFINITIONS OF HIGHWAY NETWORK AND SECTIONS

Any roads, regardless of location (including feeder roads, crossroads, and bypassed road sections), for which travel time or user costs will be significantly affected by the project, should be a part of the total highway network considered in the analysis. The different sections of different alternatives in a given highway network can be identified by combinations of letters (such as ab, bc), following the alternative code. For example, consider the network shown in Figure 4. Assuming that ab and ec are long, continuous, uphill grades and that all other road sections are level to light rolling, the following six road sections should be identified: 0ab, 0bc; 1ae, 1ec; 2ade, 2ec.

Because section ec is common to alternatives 1 and 2, road user costs for ec would need to be computed only once if 1ec and 2ec are essentially identical with respect to road geometry and traffic volume.



Alternative 0 (existing road): abc

Alternative 1 (lower cost): aec

Alternative 2 (higher cost): adec

Figure 4. Illustrative highway network.

The foregoing example assumes that alternatives 1 or 2 will replace the existing road. However, if those alternatives would supplement rather than replace the existing road, and if enough traffic would be shifted to the new routes to decrease travel time or user costs on the existing road, the traffic on abc should be included in all alternatives, as follows:

Alternative 0: 0ab, 0bc.
Alternative 1: 1ae, 1ec, 1ab, 1bc.
Alternative 2: 2ade, 2ec, 2ab, 2bc.

The foregoing nomenclature lends itself to a further refinement—that of designating the direction of traffic flow in order to treat each direction as a discrete section (this convention is suggested as the basic option in subsequent worksheets, except for two-lane roads). The number of sections in the preceding example would then double, as follows:

Alternative 0: 0ab, 0ba; 0bc, 0cb.
Alternative 1: 1ae, 1ea; 1ec, 1ce; 1ab, 1ba; 1bc, 1cb.
Alternative 2: 2ade, 2eda; 2ec, 2ce; 2ab, 2ba; 2bc, 2cb.

Because most of the subsequent analysis is at the level of road sections, it is desirable to keep the number of sections to a minimum. Accuracy does require that individual sections have reasonably similar geometric characteristics and traffic conditions, but judgment will be needed as to what is "reasonably similar." The following general guidelines are therefore suggested in defining sections:

1. Conditions that usually require the creation of separate sections include:
   a. The occurrence of different road types (such as freeways and two-lane roads).
   b. Appreciable variation (say, more than 20 percent) in computed service v/c ratios, caused by changes in either traffic or capacity.
   c. Potential bottlenecks (excess of demand over ca-

pacity) that could cause congested flow on adjacent sections.

Because the last two conditions may not become apparent until Worksheet 2 is partially completed, the creation of more sections than was originally anticipated may be necessary at that time.

2. Long sections combining shorter tangent sections with different road characteristics can be used by applying the average vehicle speed and cost per mile to the total vehicle-miles of traffic flow over the entire section, if v/c ratios and speed do not vary too much. Experience should indicate what is "too much" variation, based on the relative flatness and linearity of most v/c-cost and v/c-speed curves in Appendix A over most of their length. As a rough guide, the highest v/c ratio on sections to be averaged should be less than 0.9 and should not exceed the lowest v/c ratio by more than 0.4. In cases of doubt, shorter sections should be used.

3. Grades and curves often can be collected into composite grades or curves over a single composite section; in any case, only those grades or curves that are changed by road improvements or which experience traffic differences between alternatives are crucial. (Light rolling or curving roads can be regarded as level and straight if their gradients and curvature are not significantly affected by the highway improvements. Again, experience will be needed to make such judgments.)

4. Signalized intersections are treated separately in this report (see Worksheets 5 and 5A), and their effects are added to the effects of midblock or uninterrupted flow conditions (either separately or in groups). Therefore, new arterial and street sections do not need to be created at each intersection.

5. The most general case for the procedures to be discussed in ensuing chapters allows for separate directional analysis; i.e., dividing each highway section into two directions of traffic flow and analyzing each direction separately. In the case where traffic conditions are substantially the same in both directions over the period of a day and where the elements of highway design are similar in both directions, the general procedure can be modified by listing and analyzing only one direction of flow and multiplying these results by 2 as provided on Worksheets 6 and 9. This is because user costs determined for the traffic flowing in one direction under the conditions described previously can be assumed to be one-half the total cost associated with the total traffic flowing in both directions. This approach is discussed more fully in Chapter Three.

CHAPTER THREE

TRAVEL TIME AND RUNNING COSTS

This chapter describes the preparation of a number of worksheets for use in calculating travel time and running costs for representative years and for representative types of vehicles. The sequence of computations is outlined in the Summary. This introduction further explains the sequence of computations and other general information needed to complete the worksheets.

Freeways, other expressways, and multilane and two-lane highways can be considered generally to provide uninterrupted traffic flow. For road sections of this type, calculation of the volume/capacity ratio forms the basis of further cost computations. The midblock portions of arterials and downtown streets can also, for analysis purposes, be considered as uninterrupted flow, but capacity and v/c ratio estimates are not needed for such roads because stops and signalized intersections are a more important constraint than midblock capacity. Signalized intersections are therefore analyzed on a special worksheet, where the degree of saturation, corresponding to the v/c ratio on uninterrupted flow sections, is calculated. An associated assumption is that queuing situations, if any, will be caused by and attributed to the intersections rather than the midblocks. Midblocks therefore can be considered as the free-flowing sections of interrupted-flow facilities.

The sequence of procedures following determination of the v/c ratio for uninterrupted flow sections or the degree of saturation for signalized intersections runs to special worksheets for analysis of service level F, involving congested or forced flow and queuing, whenever demand exceeds capacity. Peak and off-peak time periods are employed throughout the analysis instead of AADT, in order to provide a sufficiently fine-grained time span for prediction of queuing effects.

In terms of individual worksheets, the calculations in this chapter center around Worksheet 3 (Travel Time and Running Costs). Worksheet 2 provides input to Worksheet 3 in terms of traffic demand estimation, and, where necessary, calculation of roadway capacity and relevant v/c ratios. Worksheet 4 provides a means for calculating the added time and costs associated with curves, which can then be entered on Worksheet 3. Worksheet 5 allows for the calculation of time delays at intersections which then become an input to Worksheet 3. Finally, Worksheets 2A and 5A provide for the evaluation of the

effects of level of service F associated with uninterrupted and interrupted flow, respectively. The results of these worksheets then become an input to Worksheet 3 for sections operating in level of service F.

Although the worksheets may appear involved, familiarity will enable them to be completed rapidly. Of necessity, the worksheets have been flexibly designed to accommodate a variety of problems. In many cases, however, parts or all of some worksheets can be skipped. For example, in cases not involving level of service F, Worksheets 2A and 5A are omitted.

The worksheets include algebraic operators for performing computations, to save looking up written procedures each time. These operators, shown in parentheses in the stub column, follow the convention of underlining any numerical constants (e.g., 1,000) and not underlining numbers that refer to other lines of the worksheets (e.g., W2, 9.1 indicates Worksheet 2, line 9.1). Also, information on the worksheets that is independently estimated or given (as opposed to factors that are obtained from graphs or tables, or numbers that are calculated from other data) is identified on the worksheet by a dot (●) in front of the item. If large numbers of projects are to be analyzed, it may be economical to program the computations for a computer, using the dot-preceded information as inputs.

TRAFFIC DEMAND AND HIGHWAY CAPACITY

Worksheet 2 facilitates the estimation of traffic demand for each analysis section and the calculation of capacity for highway facilities characterized by uninterrupted flow. The v/c ratio summarizes the relationship between the estimated traffic demand and the elements of highway design on uninterrupted flow sections, and so will form the basis for further computations for such sections. As explained in the HCM, the v/c ratio is also a determinant of level of service for uninterrupted flow facilities. However, the use of v/c ratios in Worksheet 2 (as opposed to a single level of service designation) allows one to define a level of service over a continuous range so that conditions can be portrayed more accurately within a given level of service.

Worksheet 2 and most subsequent worksheets include an example that will be used to illustrate the procedures for completion of the worksheets. For completeness, the example is concerned primarily with an uninterrupted flow facility. Where useful, comments and examples are provided with regard to the application of the procedures to interrupted flow facilities.

The example, identified as Project 410 and shown in Figure 5, assumes an existing 70-mph design speed freeway that has been divided into three parts for analysis. For simplicity, no on- or off-ramps are assumed on the entire section of the freeway. Sections ab and cd are each 5 miles in length and are three lanes in each direction; section bc is 2 miles in length and has only two lanes in each direction. Note that sections bc and cb are designated as bottleneck sections, whereas the other sections are designated upstream or downstream sections, depending on the direction of traffic flow. Other features of Figure 5 are discussed later in this chapter.

In this example it is desired to evaluate the redesign of section bc to a six-lane freeway. This is designated as alternative 1. The existing or "do-nothing" alternative is designated as alternative 0. More information regarding these alternatives is given in the dot-preceded items of Worksheet 2 and other worksheets of this chapter.

Instructions for completing Worksheet 2 (Fig. 6) follow.

Item 1: Enter the identification of each individual section to be analyzed. As noted previously, in the common case where traffic demand and roadway design features are substantially the same in both directions, it is possible to analyze a single direction and multiply the results by 2. This includes cases where the characteristics of peak traffic flow are similar for each direction but the peak flow occurs during different times of the day. Because this is assumed to hold true for Project 410, the entries for the example on Worksheet 2 are for the easterly direction of traffic only. In the case where this is not so, each direction must be analyzed separately.

Single-direction analysis also is feasible for two-lane highways, except that the peak-period demand/capacity (d/c) ratio (item 13) must be calculated on the basis of two-way capacity less the estimated volume in the non-peak direction. This step provides for consistency with HCM treatment of capacity for two-lane highways on a two-way basis.

To detect the physical interrelationships between sections (especially the effects of bottleneck sections on upstream sections) of a given alternative, it is best to place physically adjacent sections in adjacent columns of Worksheet 2. The analysis can then proceed section by section for the entire route and, where appropriate, in one direction at a time. In other words, each line of Worksheet 2 for each section under study should be completed before proceeding to the next line.

Note that three sections have been designated for alternative 0 (the existing or "do-nothing" case) and only two for alternative 1. The reason is that section 1bc with six lanes can be assumed to have the same characteristics as section 1ab and thus can be combined with it.

Item 2: Indicate type of facility being considered (e.g., freeway, suburban arterial). If the entry is the same for each section, a check or similar mark will suffice.

Item 3: Indicate the weighted average of the design speeds within the highway section or, in the case of arterials or downtown streets, the weighted average speed limit on the road section.

Item 4: Enter the total length of each section, in miles, to the nearest hundredth of a mile.

Items 5 and 6: Indicate for each section the number of traffic lanes in one direction and the average width of each lane in feet.

Item 7: Enter the percent gradient in each section. In the example, it is assumed that the 3 percent grade will not significantly affect (in either direction) the vehicle speeds on this section. Hence, the notation ± is used in the example to indicate that the facility is being analyzed only in one direction for doubling later. Then, in Worksheet 3, plus and minus gradient cost factors can be averaged. Similarly, if a road section consists of mod-

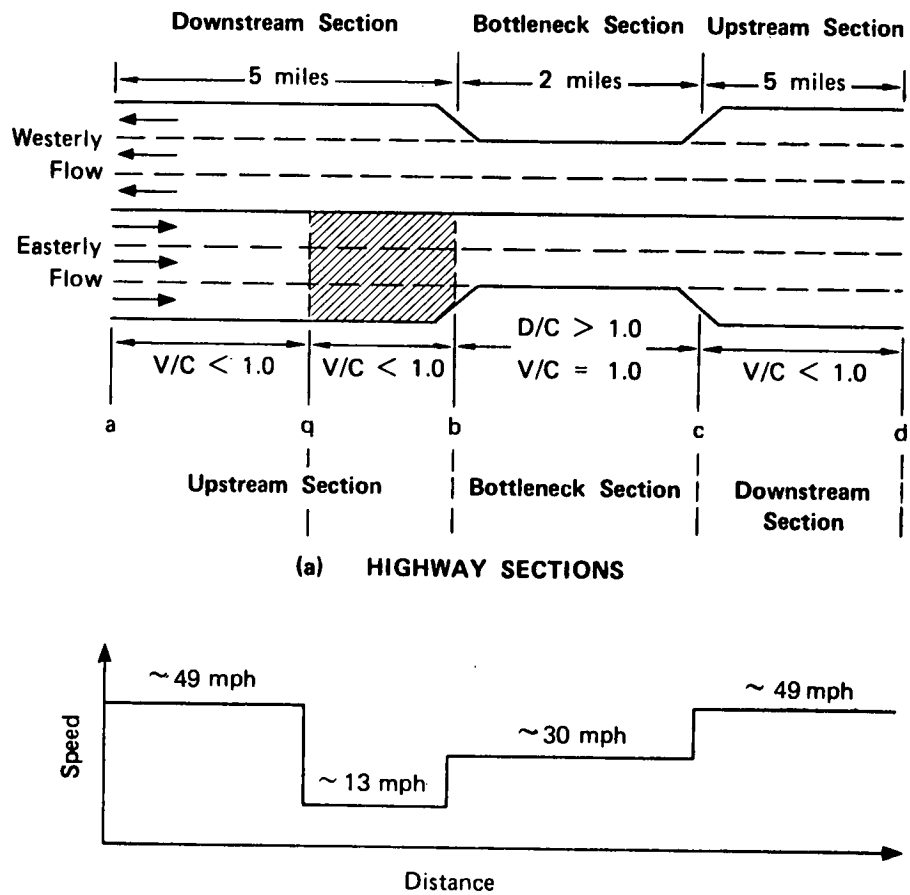


Figure 5. Schematic of Project 410 under queuing conditions.

erately rolling terrain, the averaging of plus and minus cost factors can be used. In general, the costs associated with uphill and downhill grades of the same magnitude do not "cancel" one another and thus the average cost factor will not be zero.

Note that the suggested "shortcut" averaging method cannot be used when uphill and downhill speeds or costs will be affected differently by different alternatives. In such cases (e.g., when grades exceed 3 percent or trucks exceed 5 percent of total vehicles, and one or more improvement alternatives involve changes in gradients or roadway width on grades), separate analysis for each direction of travel should be considered.

Item 8: Other factors (except percentage of trucks, which is entered later) needed to describe each section in question can be entered in the spaces provided. Because item 12 requires that section capacities be computed, the items entered here should include those that are relevant to such computations (e.g., passing sight distance or lateral clearance).

Item 9: Enter two-way estimated AADT on lines 9.1 and 9.2. Then, divide these figures by item 5 to find, for lines 9.3 and 9.4, the AADT per lane pair for each representative study year for each section.

Item 10: For this item and each of the remaining items in Worksheet 2, estimates relating to both peak and off-peak traffic conditions are specified. The reason for this is that use of a single average value to represent traffic demand often masks the actual diurnal pattern of such demand and prevents proper analysis of peak period congestion travel time and costs. Separation of daily traffic into peak and off-peak periods more adequately reflects this pattern and thus enables more realistically computed user costs and other consequences.

In item 10, for each representative year, enter the estimated one-way AHT (average hourly traffic demand) for both peak and off-peak periods in vehicles per hour. The AHT values should be average figures that, when multiplied respectively by the number of peak and off-peak hours per year, equal the total one-way annual traffic multiplied by 365. The number of hours per day for which the demand is expected to approximate the corresponding hourly traffic should be noted in the space provided. The footnote for this item should be used for denoting assumptions about numbers of peak and off-peak hours expected in a year (an added footnote may be necessary if the number of peak period hours changes between year 1 and year 20).

Worksheet 2

TRAFFIC DEMAND AND HIGHWAY CAPACITY

Project Number 410

• 1.	Section	0ab		0bc		0cd		1ac		1cd	
• 2.	Type of facility	Freeway		↘		↘		↘		↘	
• 3.	Design speed (mph)	70		↘		↘		↘		↘	
• 4.	Length, miles	5.0		2.0		5.0		7.0		5.0	
• 5.	Number of traffic lanes in one direction	3		2		3		3		3	
• 6.	Lane width, feet	12		↘		↘		↘		↘	
• 7.	Grade, percent	0		0		±3		0		±3	
• 8.	Other factors affecting capacity										
	8.1										
	8.2										
	8.3										
9.	AADT, vehicles per day										
	9.1 Total, Year 1	36000		36000		36000		36000		36000	
	9.2 Total, Year 20	78000		78000		78000		78000		78000	
	9.3 Per lane pair, Year 1 (9.1 ÷ 5.)	12000		18000		12000		12000		12000	
	9.4 Per lane pair, Year 20 (9.2 ÷ 5)	26000		39000		26000		26000		26000	
10.	Estimated one-way AHT (two-way AHT for 2-lane roads), vehicles per hour (9.3 and 9.4 to Figure 7 values x 5.)	Peak	Off-Peak	Peak	Off-Peak	Peak	Off-Peak	Peak	Off-Peak	Peak	Off-Peak
	Duration, Hrs/day* Year / Peak / Off-Peak										
	10.1 1 / 1.69 / 16.31	2280	867	2280	867	2280	867	2280	867	2280	867
	10.2 20 / 1.69 / 16.31	4200	1956	4200	1956	4200	1956	4200	1956	4200	1956
11.	Percent Trucks										
	11.1 Single unit										
	11.2 Truck combinations										
	11.3 Total, or average trucks	3%	5%	3%	5%	3%	5%	3%	5%	3%	5%
12.	One-way section capacity (two-way for 2-lane roads), vehicles per hour	5820	5700	3880	3800	5280	4920	5820	5700	5280	4920
13.	Demand/Capacity Ratio (10.1 and 10.2 ÷ 12)										
	13.1 Year 1	.39	.15	.58	.22	.43	.18	.39	.15	.48	.20
	13.2 Year 20	.72	.34	1.08	.51	.80	.40	.72	.34	.80	.40

* Based on 2 hours per weekday and 2 per weekend (618 per year) for one-way peak period AHT, and 16 hours per weekday plus 34 per weekend (5952 per year) for one-way off-peak AHT.

Figure 6. Worksheet 2 example.

To represent or model all daily traffic as within the two representative types of hourly period (peak and off-peak), it is more accurate to add the traffic in minimum traffic hours to off-peak traffic; hence, to denote the minimum traffic hours as having zero traffic. The total peak and off-peak hours will then add up to less than the total hours in a year. Accordingly, a model day of 18 hr is suggested, with the 6 hrs from 12 M to 6 AM added to off-peak traffic.

As an example of this suggestion, if 2 *one-way* peak hours per weekday and 2 per weekend of the same magnitude are assumed together with 16 off-peak hours per weekday and 34 per weekend, total annual traffic would be as follows:

ITEM	ONE-WAY ANALYSIS		TWO-WAY ANALYSIS
2 peak hours × 253 weekdays	506		
2 peak hours × 56 weekends (taking 8 holidays as equivalent to 4 weekends)	112		
Total peak hours per year		618	1,236
16 off-peak hours × 253 weekdays	4,048		
34 off-peak hours × 56 weekends	1,904		
Total off-peak hours		5,952	5,334
Total hours in model (18 × 365)		6,570	6,570

Note that the 2 peak hours per day and per weekend in this example are, in effect, doubled when a one-way section analysis is eventually multiplied by 2, making a total of 4 peak hours in the day. As the right-hand column of the example indicates, the increase in two-way peak hours is offset by a decrease in two-way off-peak hours.

It is assumed that users of this report can determine or estimate average hourly peak-period traffic volume on existing highways in year 1 and on proposed highways in years 1 and 20 from direct observations of comparable roads and from use of K and D factors and the formula for finding service volume (see definitions of these concepts in Chapter One). For estimating hourly peak-period volume on existing roads in year 20, however, predictions must be made that may run beyond current experience in the geographical area under study. Figure 7 has been prepared to facilitate such estimates; it also may be used to derive or confirm hourly peak period volumes in year 1.

The upper lines in Figure 7 are regressions based on actual hourly vs AADT traffic observations, from the HCM (*1*, App. A). They show the relationship between *AADT per lane pair and average 30th and 100th hour traffic volume per lane.* The product of K and D factors implied by these curves also is indicated at selected points along the curves.

The procedure for estimating peak period hourly demand volumes from Figure 7 is as follows: (1) enter Figure 7 with AADT per lane pair (item 9.3 or 9.4); (2) obtain the hourly volume per lane; (3) multiply this figure by the number of lanes in one direction (item 5); and (4) enter the result in the appropriate space under item 12. To find off-peak period hourly demand volume: (1) multiply the peak period hourly volume by the number of peak hours in the year; (2) subtract the result from total yearly traffic in one direction (item 9.1 or 9.2 × 365/2); and (3) divide the result by the total number of off-peak hours in the year. The peak and off-peak volumes are entered on lines 10.1 and 10.2 of Worksheet 2 for years 1 and 20, respectively.

The off-peak volume calculation is incorporated in the lower curves of Figure 7 for cases in which daily one-way peaks occur at the rate of 1, 2, or 3 hr a day for 365 days a year (or less regular peak hours can be converted to an average daily rate). The lower curves are based on an average of the upper curves, and will serve for approximating average off-peak hourly volume within ±3 percent for an 18-hr model day. This volume is then multiplied by the number of lanes, item 5, to arrive at an off-peak hourly volume for line 10.1 or 10.2.

An important aspect of this procedure is that Figure 7 cannot be used for bottleneck sections because it will never result in more peak period hourly traffic than 2,000 vph (which serves in effect as an upper asymptote for the graph). One way of obtaining the correct demand volume for bottleneck sections is to use the estimated peak period hourly demand in the *upstream section* as derived from Figure 7. This method was followed in the problem illustrated on Worksheet 2. Whatever method is employed, users should note that peak period hourly volume, not AADT, is the key variable in this procedure.

Some further considerations entailed in estimating peak period traffic volume and duration follow.

1. When weekends present no peak hours or only peak hours of about the same magnitude as weekday peak hours, the 36 hr in a model weekend should be added to weekday peak and off-peak periods. For such situations, the 100th highest hour curve seems to be a good average of the peak volumes in the 253 weekdays of the year. For a daily peak of 1 hr, the 100th hour is a close average. As the number of daily peaking hours increases, some higher highest hour (i.e., 200th) might in theory seem to be a more appropriate average. However, inspection of HCM curves ranking hours of the year in declining order of traffic volume shows that 100th-hour traffic is not usually

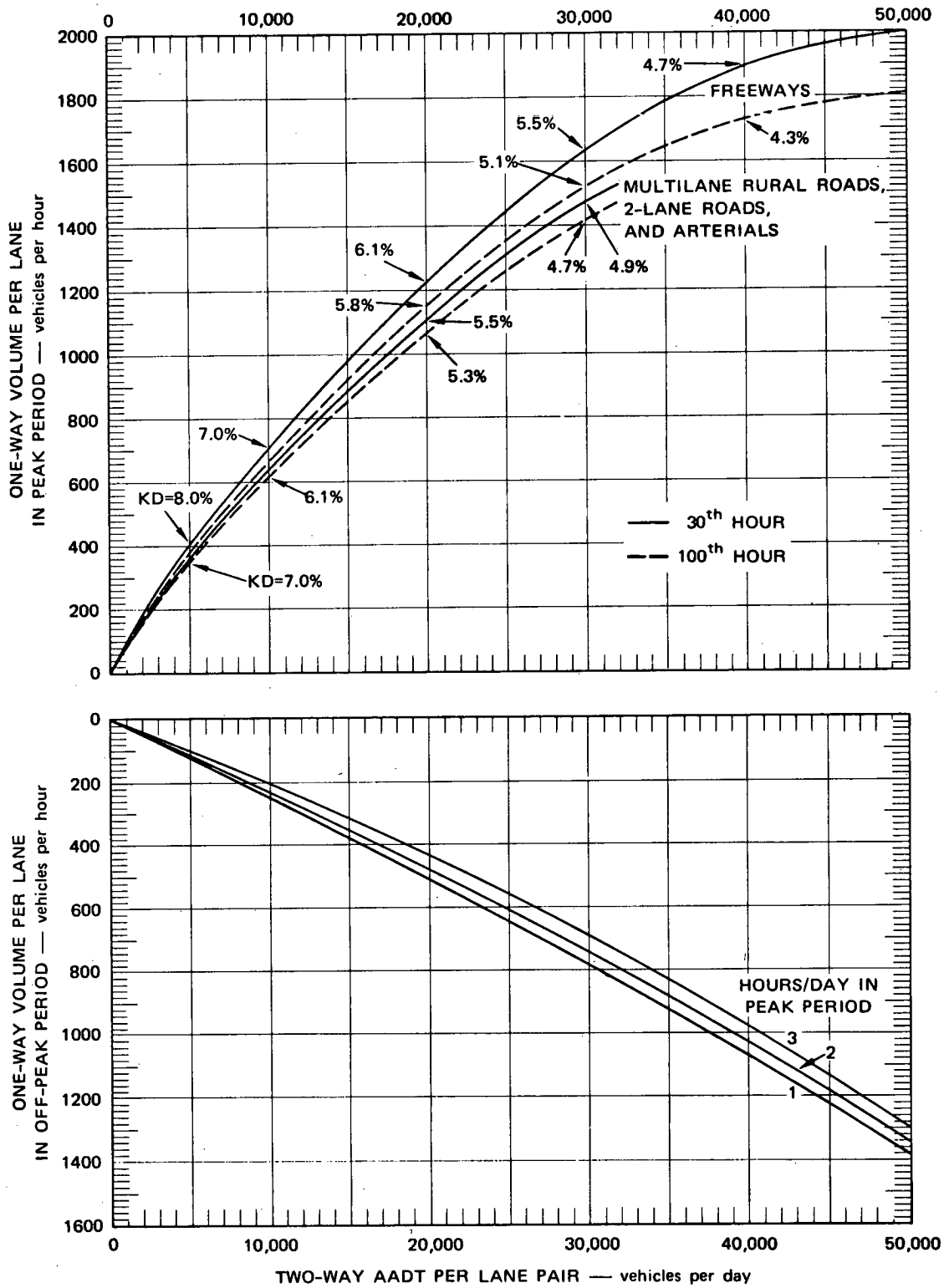


Figure 7. Conversion of AADT per lane pair to hourly peak period and off-peak period traffic per lane.

far above 200th-hour traffic and hence constitutes a good general average.

2. When weekends present peaks estimated as higher than weekday ones, there are two possible ways of doing the calculations: (1) weekends can be analyzed separately and the results can be added to those of weekdays. The weekend peaks will be better represented by the 30th highest hour curve, particularly if their occurrence does

not sum up to a large number of peak hours per year. (2) Grouping together all peak hours (for weekdays and weekends) and using the 100th-hour volume will simplify the calculations at the cost of somewhat underestimating the peak volume costs on weekends, particularly for high v/c ratio highways.

3. The length of the peak period can be expected to increase with the magnitude of the peak period hourly volume, and hence should not be a completely independent estimate. However, the researchers know of no studies or data correlating these two estimates. In the absence of better information, report users may wish to assume, as the researchers have in the problems for Chapter Seven, that the maximum observed daily volume for each road type occurs with a one-way peak period of 2½ hr (or 5 hr for the two-way peak), diminishing linearly to zero as AADT declines to zero.

Item 11: Enter the estimated percentage of trucks in peak and off-peak periods for each section. Provision is made for single-unit trucks and truck combinations to represent the total range of truck types, or alternatively (in line 11.3) for "average trucks" if a single composite truck count is used.

Table 2 gives the representative distributions of traffic by vehicle class and highway type. See the distinction between passenger cars and trucks in the definitions of Chapter One for guidelines in counting types of vehicles. Also, note that the worksheets and the running cost data given for trucks in Appendix A are based on two of the vehicles described by Winfrey (*4*, p. 334): single-unit trucks correspond to the 12,000-lb gross weight gasoline-powered dual-rear-tire vehicle; and truck combinations correspond to the 50,000-lb gross weight 3-S2 diesel-powered vehicle.

Item 12: This item concerns estimating highway capacity, leading to calculation in item 13 of d/c ratios for relevant road sections. For interrupted flow sections such as arterials and downtown streets that contain controlled intersections, however, such calculations are not necessary and these two items may be skipped. The reason is that for sections of this type the ratio of demand to capacity has little relevance in defining level of service and does not provide a basis for deriving running cost factors later (the average speed through midblock subsections of these roadways, to be entered on Worksheet 3, provides the basis for such factors). Hence, for further analysis of interrupted flow sections, it is recommended that users proceed to Worksheet 3, unless there is a question about even the ability of midblock sections to handle projected demand.

For uninterrupted flow sections, enter for item 12 the one-way hourly highway capacity for each section (for two-way roads, find two-way capacity/2). If truck percentages differ significantly between peak and off-peak periods, two separate capacities should be computed. The general equation for freeway capacity as given in the HCM can be expressed as

$$C = 2{,}000\ N\ W\ T \qquad (2)$$

in which

N = number of lanes;
W = width factor; and
T = truck factor.

Derivation of these variables is explained in the HCM. For the example used here, $W = 1.00$, and values for the truck factors, T, are given as follows for the relevant situations.

	VALUE OF T, BY GRADE	
TRUCKS (%)	0%	3%
3	0.97	0.88
5	0.95	0.82

The truck factors for the 0 percent or level grade are taken directly from the HCM (*1*, p. 257, Table 9.3). The factors corresponding to the 3 percent grade are an average of uphill and level factors, reflecting the fact that an uphill grade is involved in one direction of traffic and a downhill grade in the other. The uphill factor is obtained from the HCM (*1*, Tables 9.4, 9.6), and the downhill direction corresponds, for capacity purposes, to a level grade. The results are shown in the foregoing tabulation, and the computed capacities are shown on Worksheet 2 (Fig. 6).

This method of combining the effect of grades in opposite directions, although approximate, helps to reduce computations by analyzing the section for a single direction and doubling the resulting costs. The method is acceptably accurate when the associated d/c ratios are not close to or greater than 1.0, and when truck percentages are not too high (e.g., less than 10 percent). Otherwise, computations should be carried out separately for both directions of travel.

Item 13: Enter the ratio of demand volume to section capacity for each representative year for both peak and off-peak hours for each section under consideration (item 10/item 12). The d/c ratio will be identical to the HCM's

TABLE 2

REPRESENTATIVE PERCENTAGE DISTRIBUTIONS OF TRAFFIC BY VEHICLE CLASS AND HIGHWAY TYPE

	DISTRIBUTION OF TRAFFIC (%)			
		TRUCKS		
HIGHWAY TYPE	PASS. CAR [a]	SINGLE-UNIT [b]	COMBINATION	TOTAL
Freeways:				
Rural	86.5	3.6	9.9	13.5
Urban	95.4	1.9	2.7	4.6
Multilane rural highways	85.9	7.4	6.7	14.1
Two-lane rural highways	86.0	9.0	5.0	14.0
Arterials	92.6	4.7	2.7	7.4

[a] Includes passenger cars, motorcycles, and panel and pickup trucks.
[b] Includes buses.
Sources: Winfrey (*4*, p. 429); Claffey (*12*, p. 41).

service v/c ratio, except in the case where service level F is indicated and queuing is encountered.

To find the correct d/c ratio for the peak direction on *two-lane highways:* (1) estimate hourly traffic in the non-peak direction during the peak period (average hourly off-peak traffic, from item 10, may be used in the absence of a better estimate); (2) subtract this number from two-way capacity for the highway to obtain the capacity available for the peak direction of traffic flow; and (3) divide the result into hourly peak period volume. As an example, let

Peak period traffic in off-peak direction	= 800 vph
Two-way capacity	= 2,000 vph
Peak period traffic in peak direction	= 1,440 vph

Then:

$$v/c \text{ ratio for peak direction} = \frac{1{,}440}{2{,}000-800} = \frac{1{,}440}{1{,}200} = 1.2$$

Note in this example that the off-peak v/c ratio will have been computed as 0.8 (from 800/1,000). The accuracy of this estimate is demonstrated by its equivalence to the two-way off-peak volume (1,600) divided by the two-way capacity (2,000).

When $d/c > 1.0$, a potential queuing situation can be anticipated, because demand volume exceeds section capacity. To highlight this situation and to indicate that further action is necessary, calculated d/c values greater than 1.0 should be circled on Worksheet 2.

In the case where queuing situations are *not* indicated, the analyst may proceed directly to Worksheet 3. In the case where potential queuing situations are indicated, however, the following section of the text should be consulted. In that section, the problem of queuing under conditions of uninterrupted flow is discussed and procedures for dealing with the problem are presented.

QUEUING IN UNINTERRUPTED FLOW

According to the HCM, level of service F describes a forced-flow condition in which the highway acts as a storage for vehicles backing up from a downstream bottleneck. In other words, physical lines of waiting vehicles (i.e., queues) occur upstream of the bottleneck section. Such bottlenecks may result from irregularly occurring situations such as stalled vehicles or traffic accidents, or from regularly occurring traffic demand that exceeds capacity for a given bottleneck section of roadway. In either case, however, the net effect is that demand for highway service exceeds the roadway's capacity to furnish it.

The costs to the highway user are greatly increased when there is queuing, and elimination of such conditions will provide large cost savings to the user. The decision to perform queuing analysis can be based on the need to justify a construction project, or the need to understand driving conditions. However, evaluation of the consequences of level of service F is usually a tedious process. Effects of queues on the bottleneck, upstream, and downstream sections of the roadway must be considered, and these effects change over time. Possible closure of on-and off-ramps by queues is another effect to reckon with. Moreover, road users may choose to bypass queues (in time or space) and highway agencies may wish to consider solutions to obvious queuing situations even before evaluating their costs. For these reasons, it is strongly recommended that *other possibilities be considered first* before going ahead with analysis of queuing when the estimated service volume in a given situation exceeds roadway design capacity.

Such possibilities should include the reconsideration of demand estimates or design parameters to determine whether queuing is actually a problem. In other words, the analyst should make sure that *estimated demand is realistic* and *design capacity cannot or should not be increased.* If queuing is then no longer a problem, the standard (nonqueuing) evaluation procedures of this manual can be used; otherwise, the ramifications of queuing will have to be considered. Selected demand and design considerations are explained further in the following.

1. If unacceptable congestion is projected, the first step probably should be to review traffic demand estimates. One possibility would simply be to reduce demand projections. Original demand estimates might, on review, look unrealistically high. For example, demand estimates may be somewhat contingent on the construction of a new facility. If the new facility were not built, estimated demand on an existing facility may not develop to the extent originally estimated. On the other hand, review of the situation might reveal that adjacent upstream roadway capacities are such that they cannot accommodate projected demand for the road section under study. Demand estimates would have to be reconsidered in light of such factors.

2. If it is believed that demand projections cannot be reduced—at least to a point where projected demand is less than capacity—another possibility exists. It might be determined that excess (above capacity) demand would divert, either over space or time. For example, drivers encountering excessive congestion may take alternative (parallel) routes, thus reducing effective demand for the roadway under consideration. Such a process is called redistribution of demand over space, and all traffic affected by this redistribution should be included in the analysis.

Another means of lowering peak period demand is for many vehicles to delay their trips so as to drive during the off-peak period. This process is called redistribution of demand over time and can also greatly affect peak period demand volume. Such a result can be reflected in the present procedure by increasing the estimated length of the peak period.

3. Finally, if demand projections cannot reasonably be altered, the identification of possible queuing situations may be an indication that the design capacity of the highway section in question needs to be increased. This often will be true in the case of new construction or planned improvements—especially where avoidance of or relief from con-

gestion is one of the original objectives. Increasing the capacity of the proposed roadway section to a point equal to or above estimated peak service volume would then make it possible to use the standard procedures of this manual. Such a solution is not feasible, of course, in the case of the "do-nothing" alternative.

Methods for Quantifying Queuing

For cases where the foregoing considerations do not alleviate the problem of congested flow, methods are presented here for quantifying the situation. Before describing Worksheet 2A, in which these methods are incorporated, a brief conceptual discussion of the problem is presented.

In the example used up to this point (see Fig. 5), section 0bc is identified as a potential bottleneck in year 20 (and possibly earlier as well) by the fact that the associated d/c ratio is greater than 1.0. *Because section 0bc is the bottleneck, queuing will occur on the upstream section 0ab.* Similarly, for the westerly direction, queuing will occur on upstream section 0dc when and if demand exceeds the bottleneck capacity. If it is assumed (as in this example) that the situation with regard to the westerly direction will be the same but reversed (e.g., a morning peak in one direction and an afternoon peak in the other), then doubling of single direction results is acceptable. However, if directional conditions are significantly different, separate analyses will be necessary.

This example indicates the importance of properly dividing alternatives into sections and of taking a network viewpoint so that potential bottlenecks can be identified. If section bc was not separately identified, the procedure might associate an erroneous v/c ratio with section abc.

In the Worksheet 2 example of easterly traffic flow from Figure 5, peak traffic *demand* in year 20 is a constant 4,200 vph; upstream *capacity* (section ab) is 5,820 vph; and downstream *capacity* (section bc) is 3,880 vph. The downstream subsection 0bc thus becomes a bottleneck, which causes queuing of vehicles in excess of its capacity and backs them into the upstream subsection. Flow in the bottleneck section is reduced to its capacity (by definition, it cannot be larger) and thus the v/c ratio for the bottleneck section (section bc, Fig. 5) is $v/c_{bc} = 1.0$. Peak period traffic flow in section cd also is reduced by the bottleneck.

The congested flow in the upstream subsection can be described conceptually (modeled) in the following manner. As a result of the downstream bottleneck, physical queues will form, resulting in congested flow on some or all of the upstream section.* Although the distance of the backup of traffic flow will change over time, the average queue length can be used as the measure of the extent of congested flow. The other important quantitative variable is the time required to dissipate the queue after cessation of peak demand. Because peak demand exceeds capacity, it is necessary that excess demand be "worked off" over a period of time longer than the duration of peak period demand. It is asumed that this excess demand will be worked off or dissipated during part of the off-peak period, leading in effect to prolonging the peak period for sections qb, bc, and cd. The time required to dissipate the queue during the off-peak period is known as dissipation time.

The foregoing discussion assumes for simplicity that speed changes from free flow to congested flow and back are instantaneous, and the speed profile shown on the bottom of Figure 5 represents typical average resulting speeds over the entire easterly section during the period of peak flow.

The rate of flow leaving the upstream section is 3,880 vph, because this must be the same as that entering and transversing the downstream section; hence the service v/c ratio of the upstream subsection on which there is congested flow is $v/c_{qb} = (3{,}880/5{,}820) = 0.67$. Note that this v/c ratio is for level of service F, forced flow, as defined by the HCM. In the upstream section in which there is not yet congestion the v/c ratio is $v/c_{aq} = 0.72$, and traffic conditions are free flowing.

The average speed and running costs associated with traffic flows at the v/c ratios of 1.00 and 0.72 can be obtained from the graphs in Appendix A. The average speed associated with the level of service F v/c ratio of 0.67 should be obtained from Figure 9. Running costs as a function of v/c ratios for level of service F are shown in Figure 13.

Instructions for Worksheet 2A

Worksheet 2A first requires subdivision of the upstream section into separate "free-flowing" and queuing subsections (aq and qb, respectively, in Fig. 5). The worksheet then entails procedures for computing queue length and dissipation time, based on the shock-wave approach to queuing described in Appendix B and Ref. *19*. Results from this worksheet will be transferred to Worksheet 3 and used for the calculation of revised v/c ratios and peak and off-peak period durations for the sections affected by queuing.

Instructions for completing Worksheet 2A (Fig. 8) follow; the numerical example begun on Worksheet 2 is continued on the sample of Worksheet 2A.

Heading: Enter the project to which this worksheet applies and identify the bottleneck section of the roadway and the section immediately upstream of that section. Identify the year for which the worksheet is being completed and the time of day when the peak period volume occurs.

Item 1: Enter from Worksheet 2, line 10.1 or 10.2, the demand volume for the bottleneck section of roadway for the appropriate year. Demand volume is defined as the number of vehicles wishing to use a section of roadway during an hour, and thus it may exceed the hourly design capacity of the roadway. The demand volume for the bottleneck is also the demand volume in the section upstream of the bottleneck on this worksheet. (This assumption may sometimes understate the true upstream volume and density, as for instance when an off-ramp that will be blocked by the queue is connected to the upstream section of the freeway. However, some underestimation is acceptable because a lower-than-actual demand upstream

* If physical queues are expected to extend further than the upstream section under study, additional upstream sections should be included in the analysis. In general, if a bottleneck situation is being evaluated it is necessary to evaluate not only the bottleneck section but also all other affected upstream or downstream sections so that the full effect of congestion is taken into account.

Worksheet 2A

QUEUEING IN UNINTERRUPTED FLOW

Project No. 410 Upstream section identification Oab

Year 20 Bottleneck section identification Oac

Time of Day a.m.

	Peak	Off Peak
1. Demand volume for bottleneck (W2, 10.1 or 10.2)	4200 veh/hr	1956 veh/mh
2. Time duration of volume (W2, 10.1 or 10.2)	1.69 hrs	
3. Capacity of upstream section (W2, 12.)	5820 veh/hr	
4. Capacity of bottleneck (W2, 12.)	3880 veh/hr	
5. V/C ratios		
5.1 Noncongested subsection (W2, 13.1 or 13.2)*	.72	
5.2 Congested subsection (4. ÷ 3.)*	.67	
5.3 Bottleneck section (MIN (1.0, 1. ÷ 4.))*	1.00	
6. Rate of queueing (1. - 4. of peak) (a positive value indicates an increasing queue)	+320 veh/hr	-1924 veh/hr
7. Speed of vehicles through each section		
7.1 Upstream section (from Appendix A)*	49 mi/hr	
7.2 Congested subsection (from Figure 9)*	13 mi/hr	
7.3 Bottleneck section (from Appendix A)*	30 mi/hr	
8. Density of vehicles using each section		
8.1 In upstream section (1. ÷ 7.1)	85.7 veh/mi	
8.2 In congested subsection (4 ÷ 7.2)	298.5 veh/mi	
8.3 In bottleneck section (1. ÷ 7.3)	140.0 veh/mi	
9. Change in density in going from the upstream section to the congested section (8.2 - 8.1)	212.8 veh/mi	
10. Average length of queue (congested subsection) (2. x 6. ÷ 9. ÷ 2)*	1.27 mi	
11. Time required during off-peak to dissipate queue, in hours (6. of peak x 2./6. of off peak x -1)*		.28 hrs

* These results are utilized for Worksheet 3, lines 6, 8.1, 4, and 5.

Figure 8. Worksheet 2A example.

of the bottleneck will lead to an underestimation of the queue length and therefore of user costs. Underestimation of user costs for the "do-nothing" alternative, which will be the more typical, is considered "conservative" when evaluating proposed improvement projects, and the assumption helps to keep the queuing calculations from becoming much more complex.)

Item 2: Enter from Worksheet 2, line 10.1 or 10.2, the time duration of the peak period volume for the appropriate year. There is no need for the off-peak duration on this worksheet, because it can be assumed to be long enough to dissipate the queue that builds during the peak period.

Item 3: Enter from Worksheet 2, line 12, the capacity of the upstream section of roadway as estimated from the HCM. Do not include any compensation in the estimation for congestion that might arise due to a bottleneck downstream.

Item 4: Enter from Worksheet 2, line 12, the capacity of the bottleneck section of the roadway as estimated from the HCM.

Item 5: Note that this item relates to three highway sections, of which the first two represent subsections of the upstream section. These two subsections should be clearly identified in item 1 of Worksheet 3 (Fig. 11; in the example, 0aq is the noncongested subsection, and 0qb is the congested subsection).

Line 5.1: Enter from Worksheet 2, line 13.1 or 13.2, the demand v/c ratio of the upstream section of roadway for the peak period. Transfer this v/c ratio to Worksheet 3, line 6 (Fig. 11), for the noncongested upstream subsection.

Line 5.2: Calculate the v/c ratio of the "congested subsection" of the upstream section as indicated and enter in the "peak" column. This congested subsection should be separately identified on Worksheet 3 (Fig. 11) from the noncongested subsection—in the example, 0aq indicates the noncongested subsection and 0qb indicates the congested subsection. Only the peak period value is computed, because the off-peak v/c ratio is the same as that of the upstream section. Enter this value on Worksheet 3, line 6, under the "peak" column for the congested subsection.

Line 5.3: Enter from Worksheet 2, line 13.1 or 13.2, the demand v/c ratio of the bottleneck section (or 1.0 if demand volume exceeds capacity) for the peak period. Enter this v/c ratio on Worksheet 3, line 6, for the bottleneck section.

Item 6: The rate of queuing (positive values indicate queue buildup, in the opposite direction to the traffic flow) is calculated here. This is equivalent to the difference in the arrival rate at the upstream section and the departure rate from the bottleneck section. Note that *peak* capacity flow (item 4) of the bottleneck is used as the departure rate in *both* peak and off-peak calculations for this item.

Line 7.1: Average speed through the upstream section of roadway is derived from the appropriate speed vs v/c graph in Appendix A using v/c ratios given on line 5.1 of the worksheet. (Note that the average speed through the upstream section of roadway is based on a v/c ratio that may be upstream of affected off-ramps. The worksheet assumes that vehicular speed before any off-ramps remains the same after the off-ramp even though the v/c has changed. Again, this assumption is made for the simplicity of the queuing computation.) Enter the peak value under the column for the upstream subsection.

Line 7.2: Average speed through the congested subsection of the upstream section is derived from Figure 9, which gives speed vs v/c ratio at level of service F. Only one speed is required, because the off-peak value is as-

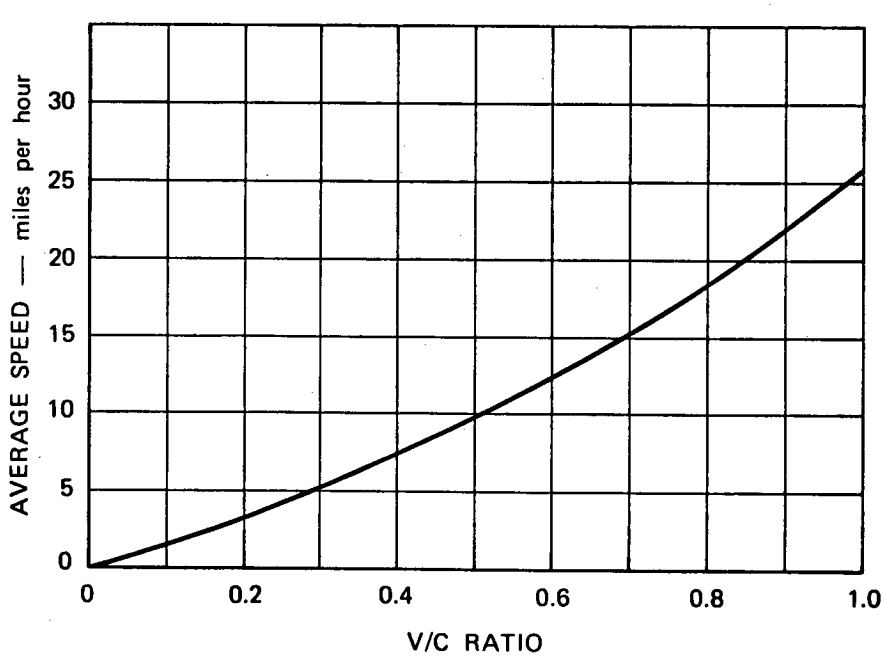


Figure 9. Average speed versus v/c ratio for level of service F.

sumed to be the same as the peak period value while the congestion remains, and is the same as the upstream value after the congestion dissipates. The needed v/c ratio is given on line 5.2 of the worksheet. The speed value also should be entered on line 8.1 of Worksheet 3, under the "peak" column for the congested subsection.

Line 7.3: Average speed through the bottleneck is derived from the appropriate chart in Appendix A (depending on the facility, it will range from 27 to 35 mph during the peak period, for a v/c ratio of 1.0). The v/c ratio for the peak period is given on line 5.1 of the worksheet. The resulting speed value also should be entered on line 8.1 of Worksheet 3 under the column for the bottleneck section.

Item 8: The density of vehicles on a section of roadway is defined as the number of vehicles occupying 1 mile of roadway at any instant of time.

Line 8.1: The density of vehicles in the section upstream from the bottleneck during the peak period is computed as the flow or volume divided by the speed of vehicles in that section of roadway.

Line 8.2: Calculate the density of vehicles in the congested subsection of roadway during the period of congestion.

Line 8.3: Calculate the density of vehicles in the bottleneck section during the peak period.

Item 9: The change in density in going from the upstream section of roadway to the congested section of roadway is calculated for the peak period density.

Item 10: The length of the queue is a function of the rate of queuing (line 6), the change in density at the buildup point (line 9), and the time duration of buildup (line 2). The queue is assumed to reach maximum length at the end of the peak period; the average length during the peak period is half the maximum length. The calculation performed in this item is for average queue length during the peak period. Enter this value for the *congested subsection* on line 4 of Worksheet 3. Subtract this value from the total length of the section (Worksheet 2, line 4) for the *noncongested subsection* on line 4, Worksheet 3.

Item 11: The time required to dissipate the queue is the rate of queue buildup (line 6—for the peak period) divided by the rate of queue dissipation (line 6—for the off-peak period) and multiplied by the time duration of the peak period (line 2). Enter this value on line 11, then add this value to the peak period duration and subtract it from the off-peak period duration in item 10 of Worksheet 2; enter the modified values on line 5, Worksheet 3, for the congested subsection and bottleneck section. Note that no changes are made for the uncongested upstream subsection, because the peak hour duration there is unchanged.

It should be noted that the downstream section of this example, 0cd, also will be affected by the queuing situation. Although the peak demand for this section in year 20 is estimated at 4,200 vph, no more than the capacity of the bottleneck section, or 3,880 vph, will get through. In other words, the peak-hour volume on 0cd equals the "queue throughput" of 3,880 vph. This change should be noted on Worksheet 3. The corresponding d/c ratio and hence v/c ratio also should be revised [from (4,200/5,280) = 0.80 to (3,880/5,280) = 0.73]. In addition, to account for the excess demand that cannot be serviced on section 0cd during the peak period, the length of the peak period on this section should effectively be increased by the same amount (0.36 hr) that the bottleneck and upstream section peak durations are to be increased.

VEHICLE TRAVEL TIME AND RUNNING COSTS

Worksheet 3 (Figs. 10, 11, and 12) enables the calculation of motor vehicle travel time and costs associated with each section of each project alternative. Separate copies of the worksheet should be prepared for each representative year (e.g., year 1 and year 20) and for each type of vehicle analyzed. Three alternatives are shown in the heading for vehicle types; the choice is up to the user, and the following guidelines are offered regarding this choice and its consequences.

1. In many cases, it will be possible to evaluate "all vehicles" (from item 10 of Worksheet 2) without serious errors. This is believed to be the case when (1) trucks constitute 5 percent or less of all vehicles, or (2) grades do not exceed 3 percent, or (3) the highway improvement alternatives do not involve changes of gradient or widening of the roadway (such as adding a truck lane) on grades. The worksheets are set up to incorporate the "all vehicles" approach in calculating the value of truck time savings (see Worksheet 9); in this case, changes in truck running costs are ignored. However, this is of little consequence under any of the three conditions just noted, especially in view of the relatively level or downward-sloping trend of truck running costs with increasing v/c ratios (see Appendix A).

2. Where one of the three conditions just noted is not present, Worksheet 3 can be completed separately for passenger cars and trucks. In this case, the passenger-car curves in Appendix A will be consulted for relevant passenger car time and cost factors. Then the weighted average of single-unit truck and truck combination time and cost factors will be obtained, based on the proportion of each of these types of vehicle in the truck traffic (Worksheet 2, item 11). Thus, a single worksheet can be completed for trucks, through the derivation of these average truck time and cost factors.

The general sequence of computations in Worksheet 3 is to enter for each section identified in Worksheet 2 or 2A the v/c ratio and other pertinent information for peak and off-peak traffic periods; find travel time and operating cost factors for level tangent travel; then (sometimes using other worksheets) add the time and cost factors for stops, signals, curves, and gradients; and, finally, multiply by vehicle-miles traveled to find total travel time and running costs for the sections and year under consideration.. Items 1 through 9 of Worksheet 3 involve special procedures for queuing and bottleneck sections; the procedures in such cases, especially for items 4, 5, and 6, are explained in more detail in connection with their respective sources in Worksheet 2A. Instructions for completing Worksheet 3 (Figs. 10 to 12) follow.

Worksheet 3

TRAVEL TIME AND RUNNING COSTS

Project No. 410 Year 1 X All Vehicles __ Passenger Cars __ Trucks

1. Section	Oab		Obc		Ocd			
2. Facility Type	70mph-FWY-6lns		✓		✓			
3. Grade, percent	0		0		±3			
4. Length, miles (W2, 4., or* W2A, 10.)	5.0		2.0		5.0			
	Peak	Off-Peak	Peak	Off-Peak	Peak	Off-Peak	Peak	Off-Peak
5. Duration of service volume period, hours per day (W2, 10., or* W2A, 11.)	1.69	16.31	1.69	16.31	1.69	16.31		
6. Volume/Capacity (V/C) ratio (W2, 13., or* W2A, 5.)	.39	.15	.58	.22	.43	.18		
7. One-way AHT (average hourly traffic) (W2, 10. or* 6. x W2, 12)	2280	867	2280	867	2280	867		
8. Speed and time:								
8.1 Average tangent speed, mph (6. to Appendix A or Table 3 or* Figure 9)	56.1	60.5	52.3	59.3	55.4	60.0		
8.2 Hptm (Table 4)	17.8	16.5	19.1	16.9	18.4	16.7		
8.3 Hptv (4. x 8.2)	89.0	82.5	38.2	33.8	92.0	83.5		
9. Running cost factor, $ptvm (6. or 8.1 to Appendix A, or* 6. to Figure 13)	49.7	50.1	49.5	49.9	49.7	50.0		
10. Added delay from stops, etc.								
•10.1 Number of signals								
10.2 Average signal stops per vehicle (10.1 x W5, 16.)								
10.3 Added time for signals, hptv (10.1 x W5, 15. x .2778)								
•10.4 Number of stop signs								
10.5 Added hours per 1,000 stops (8.1 to Table 5)								
•10.6 Average hours stopped per 1,000 stop signs (__ sec./veh. x .2778)								
10.7 Total hours per 1,000 stop signs (10.5 + 10.6)								
10.8 Added time due to stop signs, hptv (10.4 x 10.7)								
10.9 Added time due to curves hptv (W4 Recap, 3. x 6.)								
10.10 Total added time, hptv (10.3 + 10.8 + 10.9)								
10.11 Idling time, hptv (10.3 - [10.2 x 10.5] + [10.4 x 10.6])								
11. Added costs from stops, etc.								
11.1 Stopping cost factor, $/1,000 stops (Table 5)								
11.2 Idling cost factor, $/hour (Table 5)								
11.3 Added cost due to stops, $ptv (10.2 + 10.4) x 11.1 + 10.11 x 11.2)								
11.4 Added cost due to curves, $ptv (W4 Recap, 7. + 8.)								
11.5 Grade cost factor, added $ptvm (3. and 9.1 to Appendix A)					-.23	+.41		
12. Time and cost ptv, speed, service level								
12.1 Total hptv (8.3 + 10.10)	89.0	82.5	38.2	33.8	92.0	83.5		
12.2 Total $ptv ([9.+11.5] x 4.+11.3+11.4)	248.5	250.5	99.0	99.8	247.4	252.0		
12.3 Average speed (12.1 ÷ 4., to Table 4)	56.1	60.5	52.3	59.3	55.4	60.0		
12.4 Level of service (6. or 12.3 to Table 1)	A	A	B	A	B	A		
13. Total traffic, millions (5. x 365 days/year x 7. ÷ 10^6)	1.41	5.16	1.41	5.16	1.41	5.16		
14. Total vehicle-miles, millions (4. x 13.)	7.05	25.80	2.82	10.32	7.05	25.80		
15. Travel Time, thousands of hours (12.1 x 13.)	125.49	425.70	53.86	174.41	129.72	430.86		
16. Running cost, thousands of dollars (12.2 x 13.)	350.38	1292.58	139.59	514.97	348.83	1300.32		

hptvm = hours per thousand vehicle-miles

* For queueing, bottleneck, or downstream sections only

Figure 10. Worksheet 3 example.

Worksheet 3

TRAVEL TIME AND RUNNING COSTS

Project No. 410 Year 20 X All Vehicles __ Passenger Cars __ Trucks

1. Section	Oag		Ogb		Obc		Ocd	
2. Facility Type	70mph-FWY-6lns		↘		70mph-FWY-4lns		70mph-FWY-6lns	
3. Grade, percent	0		0		0		±3	
4. Length, miles (W2, 4., or* W2A, 10.)	3.73		1.27		2.0		5.0	
	Peak	Off-Peak	Peak	Off-Peak	Peak	Off-Peak	Peak	Off-Peak
5. Duration of service volume period, hours per day (W2, 10., or* W2A, 11.)	1.69	16.31	1.97	16.03	1.97	16.03	1.97	16.03
6. Volume/Capacity (V/C) ratio (W2, 13., or* W2A, 5.)	.72	.34	.67	.34	1.00	.51	.73	.40
7. One-way AHT (average hourly traffic) (W2, 10. or* 6. x W2, 12)	4200	1956	3880	1956	3880	1956	3880	1956
8. Speed and time:								
8.1 Average tangent speed, mph (6. to Appendix A or Table 3 or* Figure 9)	49	56	13	56	30	52	49	56
8.2 Hptm (Table 4)	20.4	17.8	76.9	17.8	33.3	19.2	20.4	17.8
8.3 Hptv (4. x 8.2)	76.1	64.4	97.7	22.6	66.7	38.4	102.0	89.0
9. Running cost factor, $ptvm (6. or 8.1 to Appendix A, or* 6. to Figure 13)	49.58	49.76	64.0	49.76	60.0	49.29	47.20	48.39
10. Added delay from stops, etc.								
•10.1 Number of signals	—	—	—	—	—	—	—	—
10.2 Average signal stops per vehicle (10.1 x W5, 16.)	—	—	—	—	—	—	—	—
10.3 Added time for signals, hptv (10.1 x W5, 15. x .2778)	—	—	—	—	—	—	—	—
•10.4 Number of stop signs	—	—	—	—	—	—	—	—
10.5 Added hours per 1,000 stops (8.1 to Table 5)	—	—	—	—	—	—	—	—
•10.6 Average hours stopped per 1,000 stop signs (__ sec./veh. x .2778)	—	—	—	—	—	—	—	—
10.7 Total hours per 1,000 stop signs (10.5 + 10.6)	—	—	—	—	—	—	—	—
10.8 Added time due to stop signs, hptv (10.4 x 10.7)	—	—	—	—	—	—	—	—
10.9 Added time due to curves hptv (W4 Recap, 3. x 6.)	—	—	—	—	—	—	—	—
10.10 Total added time, hptv (10.3 + 10.8 + 10.9)	—	—	—	—	—	—	—	—
10.11 Idling time, hptv (10.3 - [10.2 x 10.5] + [10.4 x 10.6])	—	—	—	—	—	—	—	—
11. Added costs from stops, etc.								
11.1 Stopping cost factor, $/1,000 stops (Table 5)	—	—	—	—	—	—	—	—
11.2 Idling cost factor, $/hour (Table 5)	—	—	—	—	—	—	—	—
11.3 Added cost due to stops, $ptv (10.2 + 10.4) x 11.1 + 10.11 x 11.2)	—	—	—	—	—	—	—	—
11.4 Added cost due to curves, $ptv (W4 Recap, 7. + 8.)	—	—	—	—	—	—	—	—
11.5 Grade cost factor, added $ptvm (3. and 8.1 to Appendix A)	—	—	—	—	—	—	-.17	-.11
12. Time and cost ptv, speed, service level								
12.1 Total hptv (8.3 + 10.10)	76.1	64.4	97.7	22.6	66.7	38.4	102.0	89.0
12.2 Total $ptv ([9.+11.5] x 4.+11.3+11.4)	184.9	185.6	81.4	63.2	120.0	98.6	235.2	241.4
12.3 Average speed (12.1 ÷ 4., to Table 4)	49	56	13	56	30	52	49	56
12.4 Level of service (6. or 12.3 to Table 1)	C	A	F	A	E/F	B	C	A
13. Total traffic, millions (5. x 365 days/year x 7. ÷ 10^6)	2.59	11.64	2.79	11.44	2.79	11.44	2.79	11.44
14. Total vehicle-miles, millions (4. x 13.)	9.66	43.42	3.54	14.53	5.58	22.88	13.95	57.20
15. Travel Time, thousands of hours (12.1 x 13.)	197.01	749.62	272.58	258.54	186.09	439.30	284.58	1018.16
16. Running cost, thousands of dollars (12.2 x 13.)	479.89	2160.38	227.11	723.01	334.80	1127.98	656.21	2761.62

hptvm = hours per thousand vehicle-miles

* For queueing, bottleneck, or downstream sections only

Figure 11. Worksheet 3 example (continued).

Worksheet 3

TRAVEL TIME AND RUNNING COSTS

Project No. 410 Year 1 & 20

X All Vehicles __ Passenger Cars __ Trucks

	Year 1				Year 20			
1. Section	1ac		1cd		1ac		1cd	
2. Facility Type	70mph-FWY-6lns		✓		✓		✓	
3. Grade, percent	0		±3		0		±3	
4. Length, miles (W2, 4., or* W2A, 10.)	7.0		5.0		7.0		5.0	
	Peak	Off-Peak	Peak	Off-Peak	Peak	Off-Peak	Peak	Off-Peak
5. Duration of service volume period, hours per day (W2, 10., or* W2A, 11.)	1.69	16.31	1.69	16.31	1.69	16.31	1.69	16.31
6. Volume/Capacity (V/C) ratio (W2, 13., or* W2A, 5.)	.39	.15	.43	.18	.72	.34	.80	.40
7. One-way AHT (average hourly traffic) (W2, 10. or* 6. x W2, 12)	2280	867	2280	867	4200	1956	4200	1956
8. Speed and time:								
8.1 Average tangent speed, mph (6. to Appendix A or Table 3 or* Figure 9)	56.1	60.5	55.4	60.0	49.2	57.1	47.4	55.9
8.2 Hptm (Table 4)	17.8	16.5	18.4	16.7	20.3	17.5	21.1	17.9
8.3 Hptv (4. x 8.2)	124.6	115.5	92.0	83.5	142.1	122.5	105.5	89.5
9. Running cost factor, $ptvm (6. or 8.1 to Appendix A, or* 6. to Figure 13)	49.7	50.1	49.7	50.0	49.6	49.8	49.7	49.7
10. Added delay from stops, etc.								
•10.1 Number of signals	—	—	—	—	—	—	—	—
10.2 Average signal stops per vehicle (10.1 x W5, 16.)	—	—	—	—	—	—	—	—
10.3 Added time for signals, hptv (10.1 x W5, 15. x .2778)	—	—	—	—	—	—	—	—
•10.4 Number of stop signs	—	—	—	—	—	—	—	—
10.5 Added hours per 1,000 stops (8.1 to Table 5)	—	—	—	—	—	—	—	—
•10.6 Average hours stopped per 1,000 stop signs (__ sec./veh. x .2778)	—	—	—	—	—	—	—	—
10.7 Total hours per 1,000 stop signs (10.5 + 10.6)	—	—	—	—	—	—	—	—
10.8 Added time due to stop signs, hptv (10.4 x 10.7)	—	—	—	—	—	—	—	—
10.9 Added time due to curves hptv (W4 Recap, 3. x 6.)	—	—	—	—	—	—	—	—
10.10 Total added time, hptv (10.3 + 10.8 + 10.9)	—	—	—	—	—	—	—	—
10.11 Idling time, hptv (10.3 - [10.2 x 10.5] + [10.4 x 10.6])	—	—	—	—	—	—	—	—
11. Added costs from stops, etc.								
11.1 Stopping cost factor, $/1,000 stops (Table 5)	—	—	—	—	—	—	—	—
11.2 Idling cost factor, $/hour (Table 5)	—	—	—	—	—	—	—	—
11.3 Added cost due to stops, $ptv (10.2 + 10.4) x 11.1 + 10.11 x 11.2)	—	—	—	—	—	—	—	—
11.4 Added cost due to curves, $ptv (W4 Recap, 7. + 8.)	—	—	—	—	—	—	—	—
11.5 Grade cost factor, added $ptvm (3. and 9.1 to Appendix A)	—	—	-.23	+.41	—	—	-.10	-.22
12. Time and cost ptv, speed, service level								
12.1 Total hptv (8.3 + 10.10)	124.6	115.5	92.0	83.5	142.1	122.5	105.5	89.5
12.2 Total $ptv ([9.+11.5] x 4.+11.3+11.4)	347.9	350.7	247.4	252.0	347.2	348.6	248.0	247.4
12.3 Average speed (12.1 ÷ 4., to Table 4)	56.1	60.5	55.4	60.0	49.2	57.1	47.4	55.9
12.4 Level of service (6. or 12.3 to Table 1)	A	A	B	A	C	A	D/C	B/A
13. Total traffic, millions (5. x 365 days/year x 7. ÷ 10^6)	1.41	5.16	1.41	5.16	2.59	11.64	2.59	11.64
14. Total vehicle-miles, millions (4. x 13.)	9.87	36.12	7.05	25.80	18.13	81.48	12.95	58.20
15. Travel Time, thousands of hours (12.1 x 13.)	175.69	595.98	129.72	430.86	368.04	1425.90	273.24	1041.78
16. Running cost, thousands of dollars (12.2 x 13.)	490.54	1809.61	348.83	1300.32	899.25	4057.70	642.32	2879.74

hptvm = hours per thousand vehicle-miles

* For queueing, bottleneck, or downstream sections only

Figure 12. Worksheet 3 example (concluded).

Heading: Note the project number, the representative study year, and the class of vehicle being analyzed.

Item 1: Enter the identification of each individual section to be analyzed. Note that in the case of queuing, separate free-flowing and congested subsections should be designated for the section on which queuing is expected.

Items 2 and 3: Enter facility type and percent grade for each section.

Item 4: Enter the length of each section, in miles, to nearest hundredth of a mile.

For any upstream uninterrupted flow section under queuing conditions, Worksheet 2A already will have determined the length of the section that will have free-flowing conditions—the uncongested subsection—and the length that will be in service level F—the congested subsection.

Item 5: Enter duration of service volume period, in hours, to the nearest hundredth.

In year 20 for the existing facility in the example the duration of the peak period is increased, and the off-peak duration is decreased, for the queuing subsection, the bottleneck section, and the downstream section owing to the results of Worksheet 2A. The downstream section also is affected because the bottleneck effectively reduces the traffic flow volume during the peak period. The excess demand upstream is thus dissipated during part of the off-peak period over the queuing, bottleneck, and downstream sections. Hence, the effective peak period duration is increased over these sections.

Item 6: Enter the v/c ratio for each section from either Worksheet 2 or, where appropriate, from Worksheet 2A. To highlight the situation, circle any v/c ratio associated with level of service F. For reasons just discussed, the peak period v/c ratio for the existing facility downstream section in year 20, section 0cd, is revised downward from the ratio listed on Worksheet 2.

Item 7: Enter the one-way average hourly traffic for each section as estimated on Worksheet 2, or, in the case of queuing, as derived by multiplying the relevant v/c ratio (item 6) by the section capacity (Worksheet 2, item 12). If the analysis deals with passenger-car or truck types rather than with all vehicles, these volume figures will have to be revised by taking into account the proportion of total volume that these vehicle classes represent. For instance, if the analysis were for passenger cars only and passenger cars represented 90 percent of the total volume (as estimated on Worksheet 2), 90 percent of the total traffic volume should be entered here.

Line 8.1: For freeways, multilane rural highways, and two-lane roads, enter the average tangent speed for each section corresponding to the given v/c ratio (for levels of service other than F, see Figs. A-1 through A-12; for level of service F, see Fig. 9). For arterials and city streets, the speed limit or signal timing speed can be used in the absence of a better estimate; the time required for stops is added later.

Table 3 gives some very approximate guidelines for use in estimating downhill truck speeds when better information is not available. The data reflect separate average speeds for sections more and less than 4 miles in length, for all types of trucks. The very slow speeds for high gradients on long grades reflect attempts by truck drivers to reduce the risk of runaways. There could be variations in the length at which very slow downhill truck speeds are used, depending on road curvature and other characteristics of particular grades. Also, some states have maximum downhill truck speed limits that would affect these estimates. Winfrey gives a more detailed discussion of downhill truck speeds, with additional data (*4*, pp. 445-456).

Lines 8.2 and 8.3: To convert average speed into hours per thousand miles (hptm) for line 8.2, use Table 4. Hours per thousand vehicles (hptv, item 8.3) is then obtained by multiplying section length (item 4) by item 8.2.

Item 9: The running costs for operating motor vehicles on level tangents are obtained by entering the appropriate Appendix A charts (or Fig. 13 for service level F) for the v/c ratio in line 6. Figures A-1 through A-12 should

TABLE 3

AVERAGE DOWNHILL TRUCK SPEEDS ON SPECIFIED GRADE LENGTHS

	TRUCK SPEEDS (MPH), BY LENGTH OF GRADE (MILES)	
GRADE (%)	< 4	> 4
0	50	50
1	54	54
2	52	50
3	50	46
4	47	42
5	44	32
6	41	22

Source: California (*20*).

TABLE 4

CONVERSION OF MILES PER HOUR TO HOURS PER THOUSAND MILES

MPH	HPTM	MPH	HPTM	MPH	HPTM	MPH	HPTM
5	200.0	25	40.0	45	22.2	65	15.4
6	166.7	26	38.5	46	21.7	66	15.2
7	142.8	27	37.0	47	21.3	67	14.9
8	125.0	28	35.7	48	20.8	68	14.7
9	111.1	29	34.5	49	20.4	69	14.5
10	100.0	30	33.3	50	20.0	70	14.3
11	90.9	31	32.2	51	19.6	71	14.1
12	83.3	32	31.2	52	19.2	72	13.9
13	76.9	33	30.3	53	18.7	73	13.7
14	71.4	34	29.4	54	18.5	74	13.5
15	66.7	35	28.6	55	18.2	75	13.3
16	62.5	36	27.8	56	17.8	76	13.2
17	58.8	37	27.0	57	17.5	77	13.0
18	55.6	38	26.3	58	17.2	78	12.8
19	52.6	39	25.6	59	16.9	79	12.7
20	50.0	40	25.0	60	16.7	80	12.5
21	47.6	41	24.4	61	16.4		
22	45.4	42	23.8	62	16.1		
23	43.5	43	23.2	63	15.9		
24	41.7	44	22.7	64	15.6		

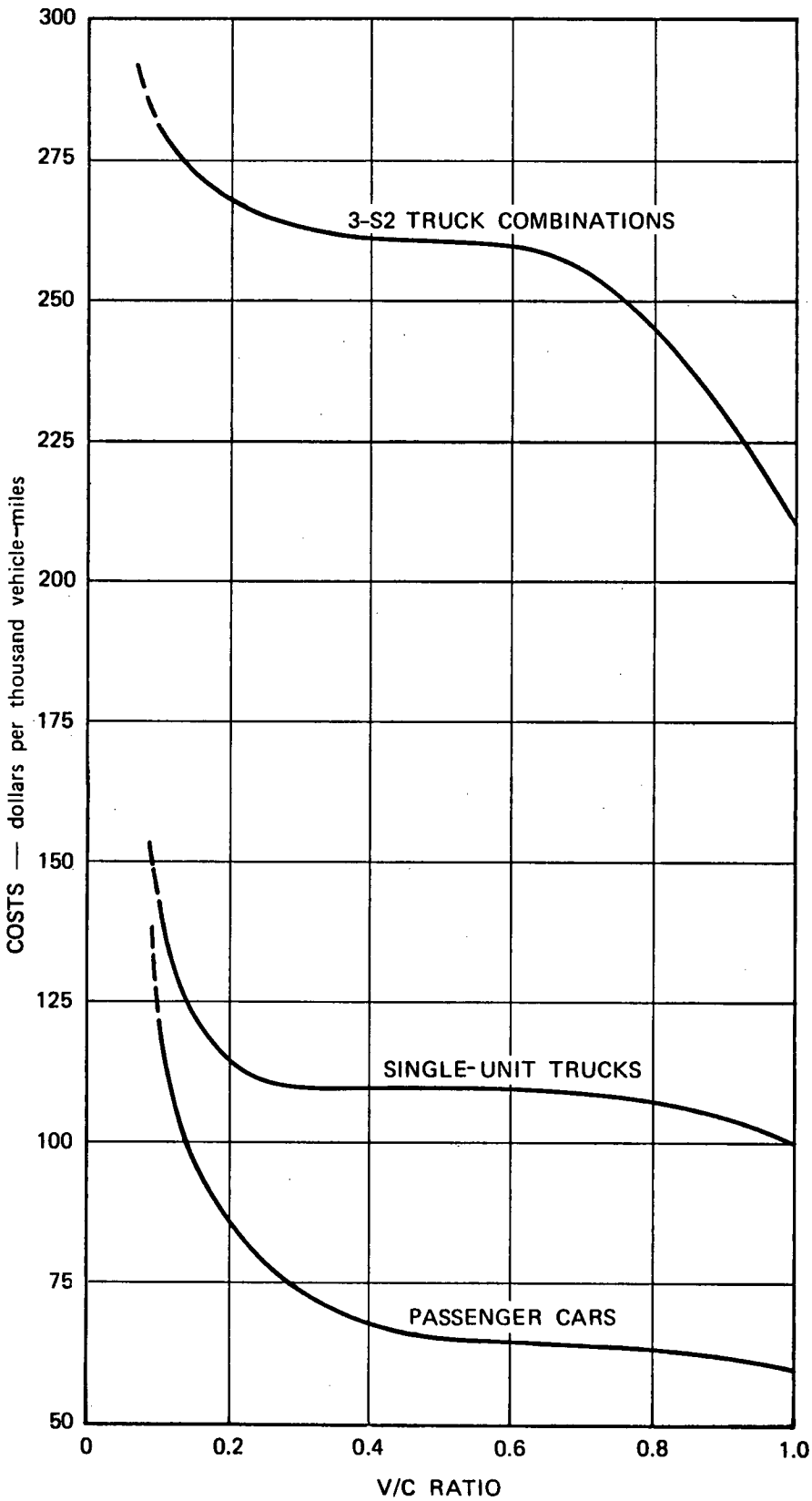


Figure 13. Running costs versus v/c ratio for level of service F.

be used for freeways, multilane rural highways, and two-lane highways. They *include* the cost of typical speed changes under the indicated v/c ratio for each highway type. Figures A-13, A-14, and A-15, which *exclude* speed change costs, should be used for the midblock sections of arterials and city streets; these curves are entered with average tangent speed, item 8.1. (See earlier note for deriving weighted average truck cost factors.) For bottleneck sections under queuing conditions, where $v/c = 1.0$, the Appendix A charts rather than Figure 13 should be used. This suggestion assumes no unaccounted-for bottleneck section turbulence from on- and off-ramps or weaving sections. If such conditions exist, they should be taken into account in determining capacity for the section.

Worksheet 4

ADDED TRAVEL TIME AND RUNNING COST FOR CURVES

Project Number _____ Year ____ ___ All Vehicles ___ Passenger Cars ___ Trucks

• Alternative, Section, & Curvature (degrees)	• Number of Curves	• Average Speed, mph Tangent	Average Speed, mph Curve	hptm Where Curve Speed Dominates	• Curve Mileage	Added Curve Cost Factors, Dollars ptvm	Added Cost per Thousand Speed Change Cycles, Dollars
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
0 hi							
8°	2	42	~~46~~		.20	27	
16°	2	~~42~~	34	29.4	.10	51	2.50
20°	2	~~42~~	31	32.2	.10	57	3.20
0 jk							
4°	2	45	~~62~~		.30	12	
6°	2	45	~~54~~		.20	22	
8°	1	45	~~46~~		.10	36	
1 hi							
3°	2	50	~~70+~~		.40	12	
5°	2	50	~~66~~		.20	25	
2 hi							
3°	4	52	~~70+~~		.80	13	

Recapitulation

	0 hi	0 jk	1 hi	2 hi	
1. Alternative and section	0 hi	0 jk	1 hi	2 hi	
2. Hours ptv where curve speed dominates (Σ col 5 x col 6)	6.16				
3. Total curve mileage where curve speed dominates*	.20				
4. Average hptm on curves (2. ÷ 3.)	30.8				
5. Average hptm on tangents (W3, line 8.2)	23.6				
6. Added hptm for curves (4. − 5.)*	7.2				
7. Added cost for curves, dollars ptv (Σ col 6 x col 7)*	16.20	11.60	9.80	10.40	
8. Added cost for speed change cycles, dollars ptv (Σ col 2 x col 8)*	11.40				

* These results are utilized in Worksheet 3, lines 10.9 and 11.4.

Figure 14. Worksheet 4 example.

Worksheet 5

INTERSECTION DELAY

Project No. Example Intersection Identification _____

Year 20 Time 8-10am

Intersection Approach Identification	(1) YX		(2) XY	
	Peak	Off-Peak	Peak	Off-Peak
1. Demand volume, veh/hr (W2, 10.)	1800	1000	1900	1000
2. Demand volume duration, hrs (W2, 10.)	2	16	2	16
• 3. Saturation flow, veh/hr (S)	3000	3000	3000	3000
• 4. Effective green time of signal, sec (G)	55	40	55	40
• 5. Cycle length of signal, sec (C)	90	60	90	60
6. Green to cycle time ratio (λ) (4. ÷ 5.)	.61	.66	.61	.66
7. Capacity of approach, veh/hr (3. x 6.)	1830	1980	1830	1980
8. Degree of saturation (X) (1. ÷ 7.) (if X is greater than 1, do the queueing worksheet, W5A)	.984	.505	1+	.505
9. Delay per vehicle, sec/veh (7. and 8. to Figure 16)	90	5		5
10. Correction Factor, sec/veh (5. and 6. to Figure 16 insert)	0	0		0
11. Average Delay per vehicle, sec/veh (9. + 10., or enter from W5A)	90	5	173	5
12. Time to dissipate queue (if any) during Off-Peak period, hrs. (W5A, line 12.)				.169
13. Difference in delay between Peak and Off-Peak period, sec/veh (Peak 11. - Off-Peak 11.)				168
14. Increase in average delay due to queueing that extends into Off Peak period, sec/veh (12. ÷ 2. x 13.)				1.848
15. Average delay per vehicle, sec/veh (11. + 14.)*	90	5	173	6.8
16. Proportion of vehicles that were stopped. MIN (1, (1 - 6.) ÷ (1 - 1. ÷ 5.)*	.975	.5	1.0	.5

*These results are utilized for Worksheet 3, lines 10.3 and 10.2

Figure 15. Worksheet 5 example.

Item 10: Where appropriate, enter relevant factors and calculate added time due to the interrupted portions of the roadway (stops or signals) or due to curves. An assumption is made in item 10 that all signal parameters are similar. If differences between signals are significant, the calculations in item 10 should be performed separately for each signal type.

The proportion of vehicles stopped at signalized intersections and added time (added delay per vehicle) at signals (lines 10.2 and 10.3) are obtained from Worksheet 5. The constant 0.2778 in lines 10.3 and 10.6 is used to convert time per vehicle in seconds to hours per thousand vehicles. STOP sign delays are calculated directly in lines 10.4 through 10.8. Table 5 is referred to in line 10.5. Added time due to curves (line 10.9) is entered from Worksheet 4.

Idling time (line 10.11) is the time spent actually stopped at intersections, either signalized or having STOP signs. By subtracting the amount of time due to slowing down at signals (10.2 × 10.5) from the total average delay at signals (10.3), which includes time to slow down, and adding to this result the time stopped at STOP signs (10.4 × 10.6) one can find the idling time. The resulting idling time is a component of total added time (10.10), for use in line 11.3 and later in air pollution calculations on Worksheet 6. No entries are shown in item 10 of Worksheet 3 for the example problem because Project 410 concerns uninterrupted flow on tangent sections. However, illustrative intersection and curve problems are shown on Worksheets 4, 5, and 5A.

Item 11: Where appropriate, enter relevant factors and calculate added running costs due to stops, curves, and gradients. Table 5 includes cost factors for slowing to and accelerating from a stop (entered in line 11.1) and for idling the engine of a stationary vehicle (entered in line 11.2). Curves costs are transferred from Worksheet

TABLE 5

ADDED TIME AND VEHICLE RUNNING COSTS PER 1,000 STOPS, AND IDLING COSTS

	ADDED TIME (HR/1,000 STOPS) (EXCLUDES IDLING TIME)			ADDED COST ($/1,000 STOPS)[a] (EXCLUDES IDLING COST)		
INITIAL SPEED (MPH)	PASS. CAR	SINGLE-UNIT TRUCK	TRUCK COMBINATION	PASS. CAR	SINGLE-UNIT TRUCK	TRUCK COMBINATION
5	1.02	0.73	1.10	0.71	2.43	8.88
7½				1.55	3.92	14.68
10	1.51	1.47	2.27	2.32	5.44	20.35
12½				3.01	7.10	26.89
15	2.00	2.20	3.48	3.98	8.90	34.13
17½				4.72	10.94	40.98
20	2.49	2.93	4.76	5.71	12.71	49.91
22½				6.61	14.91	56.99
25	2.98	3.67	6.10	7.53	16.80	67.37
27½				8.48	18.93	75.03
30	3.46	4.40	7.56	9.48	21.07	86.19
32½				10.60	23.08	93.97
35	3.94	5.13	9.19	11.57	25.44	106.05
37½				12.79	27.42	114.04
40	4.42	5.87	11.09	13.84	29.83	126.63
42½				15.08	31.97	135.02
45	4.90	6.60	13.39	16.30	34.16	147.62
47½				17.59	36.23	156.92
50	5.37	7.33	16.37	18.99	38.33	168.70
52½				20.10	40.40	178.87
55	5.84	8.07	20.72	21.92	42.25	189.54
57½				23.12	44.89	199.81
60	6.31	8.80	27.94	25.13	47.00	209.82
62½				26.62	49.41	
65	6.78	9.53		28.63	51.43	
67½				30.40		
70	7.25			32.46		
72½				34.59		
75	7.71			36.64		
77½				37.02		
80	8.17			41.19		
Idling cost ($/veh-hr.)[b]				0.1819	0.2017	0.2166

[a] Includes fuel, tires, engine oil, maintenance, and depreciation.
[b] Includes fuel, engine oil, maintenance, and depreciation.
Source: Winfrey (*4*).

4. The grade cost factor, denoting excess cost above that of running on a level tangent, is obtained by entering the appropriate chart in Appendix A with level tangent speed and percent grade. In cases where plus-and-minus gradients have been designated in line 3, the plus-and-minus gradient cost factors should be added algebraically and divided by 2 to obtain the average one-way cost factor.

In some situations—generally for lower speeds or where gradients are less than 4 percent—the added factor for plus-and-minus grades will be negative. This situation occurs in the example. For instance, section 0cd in year 20 (Fig. 11) has a ±3 percent grade with an indicated speed of 49 mph. The excess cost for the +3 percent grade (from Appendix A) is $7.14/1,000 vehicle-miles; for −3 percent the factor is −$7.44. The algebraic average of these values is −$0.17, which is the factor entered on the worksheet. Recall that it has been assumed, in this example, that the vehicular speeds for the mix of vehicles under study are substantially the same in each direction so that this shortcut averaging method is applicable.

Lines 12.1 and 12.2: Compute total hours per thousand vehicles (hptv) and total dollars per thousand vehicles by the indicated additions and multiplications.

Line 12.3: Divide the total hptv by the section length and enter Table 4 to obtain an over-all average speed for the section.

Line 12.4: The level of service associated with each section for each demand period should be looked up in Table 1 from the average speed over the section and from the v/c ratio for the section. The lower (if different) of the two levels of service determined by speed and v/c ratio is the appropriate service level to enter here. In borderline cases, two service levels should be shown, as indicated by the entry E/F for the peak period of section 0bc (Fig. 11).

Item 13: Compute total traffic, in millions of vehicles, for each section for each demand period. This is done by multiplying the one-way AHT (item 7) by the daily duration of the service volume period (item 5) and the number of days per year. The result is divided by 10^6. The number of days per year can be assumed to be 365 unless special analyses (e.g., separate weekends and weekdays) are required.

Items 14, 15, and 16: Compute total vehicle-miles, travel time, and running cost for each section as indicated by the instructions on Worksheet 3. These resultant values are to be transferred to Worksheet 9, on which a summary of user costs and travel time is calculated.

ADDED TRAVEL TIME AND COST FOR CURVES

Because the speed and running costs of vehicles on curves vary nonlinearly with the degree of curvaturue, it is necessary to analyze curves on virtually an individual basis whenever a new or improved road will appreciably change road curvature. Worksheet 4 (Fig. 14) facilitates calculation of curve travel time and added cost calculations by grouping all curves of similar curvature on each road section. Data from Worksheet 4 are then used for items 10.9 and 11.4 of Worksheet 3.

Instructions for completing Worksheet 4 follow.

Heading: Separate sheets should be prepared for passenger cars and for trucks, unless the analysis is being conducted for "all vehicles" (see discussion preceding instructions for Worksheet 3).

Column 1: Enter and underline the project alternative and section, followed by a list of curves by approximate degree of curvature. Curves of less than 1° should be ignored. (Note that the example on Worksheet 4 is for a different project than is illustrated on previous worksheets.)

Column 2: List the number of curves of each degree of curvature identified.

Column 3: Enter the average tangent speed for each section from line 8.1 of Worksheet 3.

Column 4: Estimate the average curve speed, for each degree of curvature shown in column 1 (this estimate may be based on the curve design speed for the degree of curvature and superelevation of such curve; see Table 6). Then, compare each curve speed with the average composite tangent speed for the section in column 3, and cross out the higher of the two speeds. When the two speeds are equal, cross out the curve speed. The speed that is not crossed out is referred to hereafter in these instructions as the "dominant" curve speed.

Column 5: Find the hours per thousand vehicle-miles corresponding to each curve where the curve speed dominates, using Table 3 for the conversion, and enter the result in column 5.

Column 6: Enter the total length of curve, for each degree of curvature listed in column 1, to the nearest hundredth of a mile.

Column 7: From the Appendix A chart for curve costs, for the proper vehicle type, obtain the added running cost factor per thousand vehicle-miles for each degree of curvature and enter this value. Interpolation or extrapolation may be necessary for curvatures not specifically charted.

A sharp increase in curve costs as curvature increases is shown in the Appendix A charts. This suggests a need for highway designers to reduce curvatures where possible, and to include sharp curves (such as freeway on- and off-ramps) in economy calculations where their number or degree of curvature will be affected by design alternatives.

Column 8: For curves where the curve speed dominates and is at least 5 mph less than the tangent speed: consult the Appendix A chart for added cost of speed change cycles for the given vehicle type; enter the chart at the tangent speed in column 3; read to the left along the lines on the chart to the curve speed shown in column 4; and read off on the vertical axis the indicated cost factor for column 8. (Interpolation may be necessary for initial speeds in between the lines on the chart.)

Recapitulation: In line 1, enter the same project alternatives and sections shown in column 1.

In line 2, enter the cumulative product of data in columns 5 and 6; i.e., hptm for the lowest degree of curvature where the curve speed dominates times the mileage of that degree of curvature, plus hptm for the next higher degree of curvature where the curve speed dominates times that mileage, and so forth.

Line 3 is the sum of entries in column 6 for curves where the curve speed dominates.

For line 4, enter the quotient of lines 2 and 3.

For line 5, obtain the average hptm on tangents from line 8.2 of Worksheet 3.

For line 6, obtain the excess hptm for curves by subtracting line 5 from line 4.

Lines 7 and 8 are the cumulative products, respectively, of columns 6 and 7, and of columns 2 and 8, for each road section listed.

INTERSECTION DELAY

Intersection delay is often difficult to measure and time consuming to compute by formulas. A graphic method of finding delay is therefore presented in this section, for use in connection with Worksheet 5 (Fig. 15). The basis of the intersection delay graph is Webster's equation for computing delays at signalized intersections (see Appendix B or Ref. *22* for details).

Instructions for completing Worksheet 5 follow. An example is used on the worksheet for illustrative purposes.

Heading: Enter the project number, intersection identification, and intersection approach identification, year, and time. The worksheet has two columns for opposing approaches to the intersection in case the traffic flows are not uniform in opposite directions. A second worksheet is required to compute delay at four approaches of an intersection. Columnar headings under each approach label values that are entered for the peak and off-peak periods.

Item 1: Enter the demand volumes for each approach to the intersection. If the intersection approach is part of a section of roadway, the volume is entered from Worksheet 2, line 10.1 or 10.2. Note that the demand volumes for the example are different for different directions during the peak period. This indicates that separate directional analyses are needed.

Item 2: Enter the time duration of each demand volume from Worksheet 2, line 10.1 or 10.2.

Item 3: Enter the saturation flow (S) for the intersection approach. If the HCM is used, the saturation flow is the approach volume in vehicles per hour of green time that is found for the intersection when the load factor is 1.0 and the appropriate adjustment factors are applied. In the absence of HCM solutions, recommended values for saturation flow are 1,700 to 1,800 vph times the number of approach lanes.

Item 4: Enter the effective green time (G) during one signal cycle. If the HCM is used, effective green time is the actual green time of the signal (G). If the HCM is not used, effective green time is defined as the total available for vehicular movement. (If it is assumed that the part of the yellow interval used for vehicular movement and the time lost while the queue gets in motion are equal, then both methods of defining effective green time are equivalent.)

Item 5: The cycle length (C) of a signal is the total time taken for display of all of the several indications provided by a signal.

Item 6: The green-to-cycle-time ratio (λ) is the same as the G/C ratio of the HCM, and is the ratio of the effective green time to cycle time for an approach.

Item 7: The capacity is the service volume of the approach at a load factor of 1.0. This is also the saturation flow times the green-to-cycle-time ratio.

Item 8: The degree of saturation (χ) is the ratio of demand volume to capacity. This is analogous to the v/c ratio used throughout this text which describes the condition of uninterrupted flow. If $\chi > 1$, the queing worksheet, 5A, must be completed.

Item 9: The delay per vehicle is found from Figure 16. To use this figure enter the lower graph with values of degree of saturation (χ) and capacity. Using the point so located, draw a vertical line to the graph above which intersects the delay curve for the appropriate G/C (λ) ratio. Read the average delay from a horizontal line to the ordinate.

Item 10: The correction factor is read from the small graph at the top of Figure 16. From the signal cycle length on the abscissa, draw a line to the G/C (λ) ratio curve, then draw a horizontal line from this point to the correction factor on the ordinate.

Item 11: The average delay is the sum of lines 9 and 10 if there is no queuing. If there is queuing, enter the value for average delay from Worksheet 5A, line 11.

Item 12: If there is queuing, enter the time to dissipate the queue from Worksheet 5A, line 12. This is the time during the off-peak period in which vehicles experience

TABLE 6

CURVE DESIGN SPEEDS

CURVATURE (°)	DESIGN SPEED (MPH), BY RATE OF SUPERELEVATION (FT/FT) 0.06	0.08	0.10	0.12	CURVE RADIUS (FT)
3	70+	70+	70+	70+	1910
4	62	66	70	70+	1432
5	57	60	62	66	1146
6	53	56	58	60	955
8	46	49	51	52	716
10	42	44	46	48	573
12	39	40	42	44	477
14	36	38	39	40	409
16	34	35	37	38	358
18	33	34	35	36	318
20	31	32	33	34	286
25	28	29	30	31	229
30	25	26	27	28	191

Source: Based on solutions to the following formula, adapted from Olgesby and Hewes (*21*, p. 263, formula 9-6):

$$V = \left[\frac{85{,}950(e+f)}{D}\right]^{\frac{1}{2}}$$

in which

V = design speed (mph);
D = curvature (degrees);
e = superelevation (ft/ft of roadway width); and
f = coefficient of side friction.

The first step in developing the table was to solve the equation for D, with e ranging from 0.06 to 0.12 ft/ft, at the following design speeds and coefficients of side friction (based on maximum values of f recommended by AASHO):

Design speed, mph (V)	30	40	50	60	70
Coefficient (f)	0.16	0.15	0.14	0.13	0.12

The resulting points were then plotted and smooth curves were drawn between them, from which the values in the table for 3° to 20° curves were read off. Table values for 25° and 30° curves were solved individually, with $f = 0.16$.

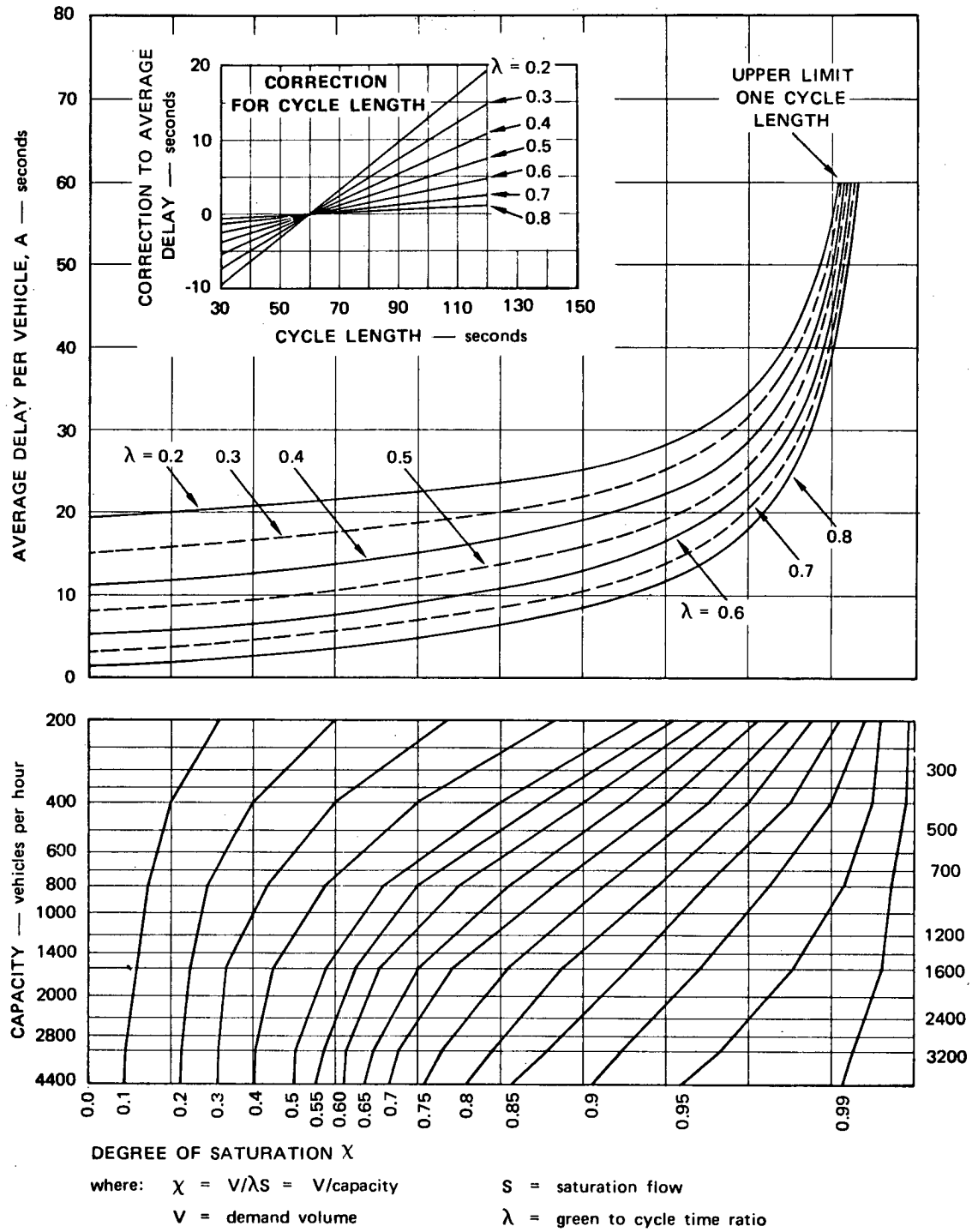


Figure 16. Intersection delay per vehicle.

peak period delays. If there is no queuing, lines 12 through 14 are left blank.

Item 13: If there is queuing, enter the difference in delay experienced by vehicles driving in the peak period and the off-peak period. This is the peak value from line 11 minus the off-peak value from line 11. (The value is always positive.)

Item 14: The queue that builds during the peak period dissipates during the off-peak period. The increase in delay to vehicles is proportional to the time duration of the queue during the off-peak and the time duration of the off-peak. Divide the off-peak queue time (line 12) by the duration of the off-peak (line 2) and multiply by the difference in delay from line 13.

Item 15: Average delay per vehicle is equal to line 11 for the peak period and to line 11 plus line 14 for the off-

peak period. These values are used on Worksheet 3, line 10.3.

Item 16: The proportion of vehicles (E) that stop at least once at the STOP signal was derived by Webster to be:

$$E = (1 - \lambda)/(1 - v/S) \qquad (3)$$

The value of E may not exceed 1. These proportions are used for Worksheet 3, line 10.2.

QUEUING IN INTERRUPTED FLOW

The deterministic queuing method is used to predict queuing under interrupted flow conditions. The method is de-

Worksheet 5A

QUEUEING IN INTERRUPTED FLOW

Project No. Example Intersection Identification _____

Year 20 Time 8-10am Approach Identification XY

1.	Demand Volume for peak hour (W5, 1.)	1900	veh/hr
2.	Demand volume for off-peak hour (W5, 1.)	1000	veh/hr
3.	Capacity of intersection during peak period (W5, 7.)	1830	veh/hr
4.	Time duration of peak period (W5, 2.)	2	hrs
5.	Cycle length of signal during peak period (W5, 5.)	90	sec
6.	Effective green time during peak period (W5, 4.)	55	sec
7.	Speed of vehicles on the approach to the intersection during the peak period (W3, 8.1)	30	mi/hr
8.	Number of lanes of the approach (W2, 5.)	3	lanes
9.	Rate of arrival of vehicles into the intersection queue		
	9.1 Density of vehicles per mile per lane when queued (240 veh/mi/lane assumes 22 ft/veh spacing in the queue)	240	veh/mi/lane
	9.2 Adjusted arrival rate (1. (1 + (1. - 3.) ÷ (8. x 7. x 9.1. - 1.)))	1907	veh/hr
	9.3 Arrival rate to be used (1. or 9.2)	1907	veh/hr
10.	Duration of interruption by signal (5. - 6.)	45	sec
11.	Average delay due to queue (MAX [((4. x 3600 x ((9.3 ÷ 3.) - 1) + 10.) ÷ 2), 5.])*	173	sec/veh
12.	Time to dissipate the queue built up during peak period (4. x (1. - 3.) ÷ (3. - 2.)*	.169	hrs

* These results are utilized for Worksheet 5, lines 11 and 12.

Figure 17. Worksheet 5A example.

scribed in Appendix B and by Newell (*23*). The deterministic method requires fewer computations than the shock-wave method (which was used for the case of uninterrupted flow) and does not assume continuous flow, as does the shock-wave method.

(The simplified method presented here assumes the uniform flow of vehicles rather than random traffic movements and therefore tends to underestimate queue buildup somewhat. Consequently, user costs while driving through the queue and benefit calculations made for cost-benefit analysis of queue-reducing improvements also will tend to be underestimated, or "conservative." Note also that, under this deterministic queuing method, the duration of the peak and off-peak periods is unchanged, in contrast to the shock-wave approach used for the uninterrupted-flow case, where the peak hour is extended by the duration of time required to dissipate a queue. Instead, user time and hence costs during the off-peak period are increased to account for dissipation of a queue that occurred during the peak period. Although the methodologies are different, the result is the same—user costs are increased to account for the dissipation of vehicle queues.)

Instructions for completing Worksheet 5A (Fig. 17) follow. The example is carried over from Worksheet 5.

Heading: Enter the project number, intersection identification, approach identification, and the year and time for which the worksheet is being done.

Item 1: Enter the peak value for volume from Worksheet 5, line 1.

Item 2: Enter the off-peak value for volume from Worksheet 5, line 1.

Item 3: Enter the capacity of the approach during the peak period from Worksheet 5, line 7. Note that this capacity is assumed to exist until the queue is dissipated during the off-peak period. The assumption here is that a vehicle-actuated signal will give a maximum green time to an approach with a queue. If the other signal phases remain unchanged, owing to similar peak demand extensions, the capacity will remain unchanged.

Item 4: Enter the time duration of the peak period from Worksheet 5, line 2.

Item 5: Enter the peak cycle length from Worksheet 5, line 5.

Item 6: Enter the peak effective green time from Worksheet 5, line 4.

Item 7: Enter the peak approach speed from the appropriate section and column of Worksheet 3, line 8.1.

Item 8: Enter the number of approach lanes from the appropriate section and column of Worksheet 2, line 5.

Item 9: This section may be used to compute the rate of vehicle arrivals into the queue during the peak period. The most simple estimate is the demand volume (line 2), which represents the flow at some point upstream of the queue. However, the end of the queue moves upstream, thus increasing the rate that vehicles arrive in the queue. A more accurate estimate, provided for in this item, takes into account the rate the queue builds up as well as the flow at some point upstream.

Line 9.1: Enter the density of vehicles in the queue. If all vehicles are stopped and occupy 22 ft per vehicle, the density is 240 vehicles per mile per lane. Line 9.1 may be left blank if the simple entry is to be made at 9.3.

Line 9.2: Compute the adjusted arrival rate in the queue based on the rate the queue backs up. Line 9.2 may be left blank if the simple entry is to be made at 9.3. This adjustment is based on the fact that the back of the queue is moving upstream. Thus, the relative speed with which arriving vehicles approach the queue is greater than the speed at which they approach the intersection; hence, the arrival rate, which is directly proportional to speed, increases.

Line 9.3: Enter the arrival rate to be used from line 1 or line 9.2.

Item 10: Compute the cycle length minus the effective green time.

Item 11: Compute the average delay to vehicles that drive through the queue, and enter this value also on Worksheet 5, line 11. To be consistent with Webster's equation, this value must be greater than or equal to one signal cycle length when queuing occurs. Hence, this value is the maximum of signal cycle length (item 5) *or* one-half (in order to arrive at an average) the sum of the duration of interruption by the signal (item 10) and the product of the fractional amount that the arrival rate (line 9.3) is in excess of the intersection capacity (item 3) times the duration of the peak period (item 4).

Item 12: Compute the time to dissipate the queue during the off-peak period, and enter this value also on Worksheet 5, line 12. This time is based on off-peak demands, the queue at the end of the peak period, and peak period capacities.

CHAPTER FOUR

AIR POLLUTION AND NOISE EFFECTS

Although the costs of operating vehicles over highway facilities are borne by highway users, who finance most of these facilities through fuel and license taxes, the impact of both facilities and vehicles on the community are somewhat more complex to measure and value than are facility and user costs. High on the list of possible unfavorable consequences to the community through which the facility passes are air pollution and noise. This chapter describes procedures for estimating the magnitude of air pollution and noise effects on the community.

AIR POLLUTION

A number of gases, vapors, and types of particles have been identified as air pollutants. Of these, six pollutants that are generated from vehicular transportation have been classed as among the most important: (1) carbon monoxide (CO), (2) hydrocarbons (HC), (3) oxides of nitrogen (NO_x), (4) smoke and particulate matter, (5) lead, and (6) photochemical smog. With the exception of photochemical smog, these are called primary pollutants, because they are the products emitted from the vehicles; photochemical smog is a result of a reaction in the atmosphere among oxides of nitrogen, hydrocarbon, and other pollutants in the presence of sunlight.

Carbon monoxide and hydrocarbon emissions result from incomplete combustion of fuel during the operation of internal-combustion engines. Hydrocarbon emissions also result from evaporation of fuel from the tank or carburetor and from open ventilation of the engine crankcase on automobiles manufactured before the 1963 model year. Oxides of nitrogen are produced when the oxygen and nitrogen in the air used by the internal-combustion engine combine under the heat and pressure of the combustion process. Smoke particles, primarily carbon, are emitted when combustion of the fuel is incomplete. Other particulate matter in the exhaust may contain lead. Lead-organic compounds are added to some gasoline fuels to improve antiknock qualities.

Although the most effective reduction of motor vehicle air pollution probably will come from modifications to automobile designs or limitations on their use, highway planners can contribute to pollution abatement by designs such that:

1. Emissions per vehicle-mile are reduced, for example, by increasing speeds and reducing the number of speed changes (e.g., through reducing congestion).
2. High concentrations of pollutants are avoided where they will harm persons, property, or agriculture processes.
3. Peak concentrations are reduced by spreading traffic over time.

Air Pollution Measurement

To describe adequately the effects of air pollution, the highway designer needs to know the trade-offs between the usual design parameters such as highway cost, travel time, speed changes, vehicle costs, and related air pollution effects. Ideally, he would like to have a common system of valuation for all these quantities, such as dollars, so that he can present analytical comparisons of alternative highway improvements. If a common system of valuation is lacking, some numerical measure will assist in selection among highway design alternatives.

The quantity of pollutants emitted from the vehicles as they travel over the alternative paths under study by the designer serves as one such quantitative measure. Another measure that might be applied is the prediction of pollution concentration levels at critical locations near the facility under study. Methods for simplified analysis that would predict concentration levels from the amounts of emission and meterological factors are being developed, but at present the procedures are too complex for recommendation as widespread tools. Therefore, it is recommended that emission levels be used as an interim measure of pollution consequences.

More specifically, the hourly emission by type (HC, CO, NO_x) in year 1 is recommended as the primary set of measures. The reasons for this recommendation are that (1) the mix of these pollutants will vary under different conditions, and (2) their impact will be greatest in the first year of operation, because of the increasing effectiveness predicted for vehicle emission control regulations as older, less well-controlled vehicles are scrapped. The pounds of emissions produced by alternative designs can then be evaluated in comparison with current emission levels and pollution concentration near the proposed change.

Worksheet 6 (Fig. 18) and Figures 19 through 23 provide for calculation of the foregoing measures. Figures 19 through 22 show data on a "reference year" automobile (1968). Figure 23 shows factors to convert reference year emissions to average emissions for a given year, taking into account expected future emission standards, vehicle maintenance practices, and the mix of new and old vehicles expected to be on the road each year. Idle emission factors, NO_x emission factors, and single-unit truck emission factors are included in the stub column of Worksheet 6 for items 11.6, 12.6, 13, and 14. The derivation of these figures and factors is described in Appendix B.

Note that the pollution performance of vehicles that is shown in these figures is based on limited data and should be used with the understanding that the design of vehicles to meet the emission requirements specified by the Federal Government is not yet firm. Because the design is not firm, the details of the performance of these vehicles cannot be

Worksheet 6

AIR POLLUTION

Project Number 979 Year 1

1. Section	Ors		Otu		1ru	
2. Facility type	Downtown arterial		Suburban arterial		Urban Freeway	
3. Length, miles (W3, 4.)	1.0		5.0		6.0	
	Peak	Off-Peak	Peak	Off-Peak	Peak	Off-Peak
4. V/C ratio (W3, 6.)	0.6	0.3	0.6	0.3	0.2	0.1
5. Year 1 auto AHT, thousands (W3, 7.)	2.5	1.2	2.5	1.2	2.5	1.2
6. Year 1 auto vehicle miles per hour, thousands (3. x 5.)	2.5	1.2	12.5	6.0	15.0	7.2
7. Percent single unit trucks (W2, 10.1)	1	2	1	2	1	2
8. Average tangent auto speed (W3, 8.1)	30	35	35	35	55	65
9. Number of stops per vehicle (W3, 10.2 + 10.4)	1	1	17	12	1	1
10. Idling time, hptv (W3, 10.11)	1.66	1.66	55.0	33.3	1.66	1.66
11. Reference HC emissions for automobiles						
11.1 Steady speed factor (8. to Fig. 19)	.71	.71	.71	.71	.76	.80
11.2 Speed changes factor (4. to Fig. 22)	.08	.07	.08	.07	.03	.02
11.3 Running HC emissions, pounds per hr. (11.1 + 11.2 x 6.)	2.0	.94	9.88	4.68	11.85	5.90
11.4 HC per 1000 stops, pounds (8. to Fig. 20)	.01	.04	.04	.04	.26	.41
11.5 HC emissions from stops, pounds per hr. (11.4 x 9. x 5.)	.02	.05	1.70	0.57	.65	.49
11.6 HC emissions from idling, pounds per hr. (10. x 5. x 0.0087)	.04	.02	1.30	0.35	.04	.02
11.7 Total reference HC emissions (11.3 + 11. 5 + 11.6)	2.06	1.01	12.88	5.60	12.54	6.41
12. Reference CO emissions for automobiles						
12.1 Steady speed factor (8. to Figure 19)	27	25	25	25	22	21
12.2 Speed changes factor (4. to Fig. 21)	14	9	14	9	12	11
12.3 Running CO emissions, pounds per hr. (12.1 + 12.2 x 6.)	103	41	488	204	510	230
12.4 CO per 1000 stops, pounds (8. to Fig. 20)	14	21	21	21	59	79
12.5 CO emissions from stops, pounds per hr. (12.4 x 9. x 5.)	35	25	893	302	148	95
12.6 CO emissions from idling, pounds per hr. (10. x 5. x 1.19)	5	2	164	48	5	2
12.7 Total reference CO emissions (12.3 + 12.5 + 12.6)	143	68	1545	554	663	327
13. Reference auto NO_x, pounds (6. x 8.81)	22	10	110	53	132	63
14. Reference single unit truck emissions, pounds						
14.1 HC (7. x 11.7 x 0.025)	.05	.05	.31	.28	.31	.32
14.2 CO (7. x 12.7 x 0.025)	4	4	39	28	17	16
14.3 NO_x (7. x 13. x 0.025)	1	1	3	3	3	3
15. Total two-way emissions per hr. in Year 1, (1976) pounds						
15.1 Figure 23 HC & CO factor for Year 1 (1.36 x 2*)						
15.2 Figure 23 NO_x factor for Year 1 (0.81 x 2*)						
15.3 HC (11.7 + 14.1) x (15.1)	5.74	2.88	35.88	15.99	34.95	18.31
15.4 CO (12.7 + 14.2) x (15.1)	400	196	4308	1583	1850	933
15.5 NO_x (13. + 14.3) x (15.2)	37	18	183	91	219	107
	1 vs 0		2 vs 0		3 vs 0	4 vs 0
16. Reduction in emissions from improvement alternatives vs. do-nothing case, pounds per hour						
16.1 HC (15.3, Σ Alternative 0 - Σ Alternative 1, etc.)	7.23					
16.2 CO (15.4, same as 16.1)	3704					
16.3 NO_x (15.5, same as 16.1)	3					

* Omit this factor for any sections that are analyzed separately for each direction of travel.

Figure 18. Worksheet 6 example.

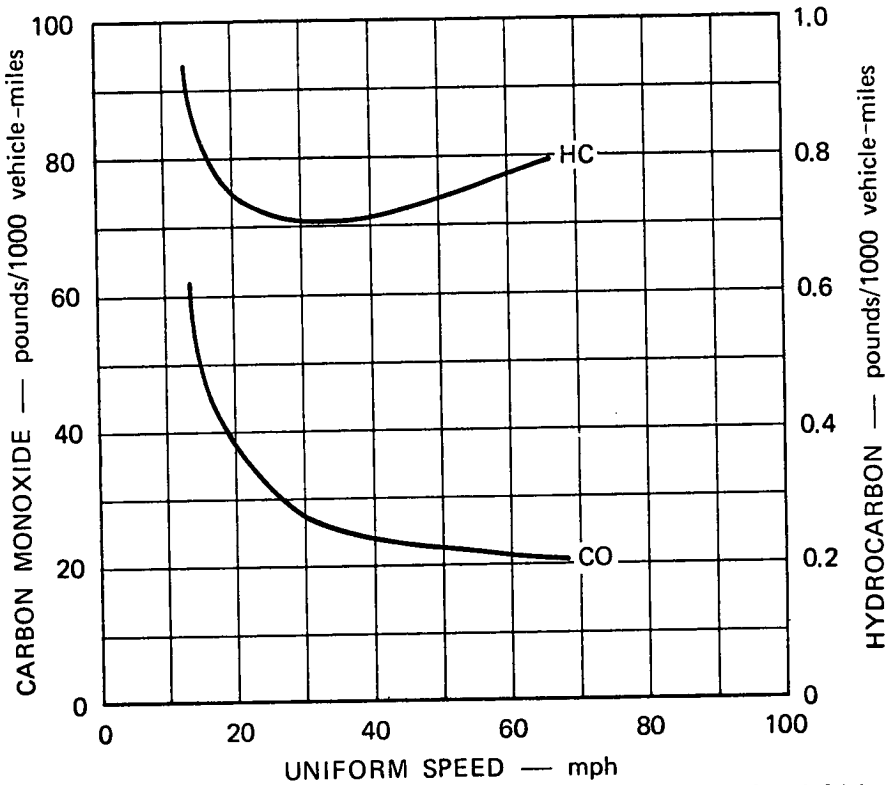


Figure 19. Hydrocarbon and carbon monoxide emissions per 1,000 miles of driving at uniform speed (reference automobile).

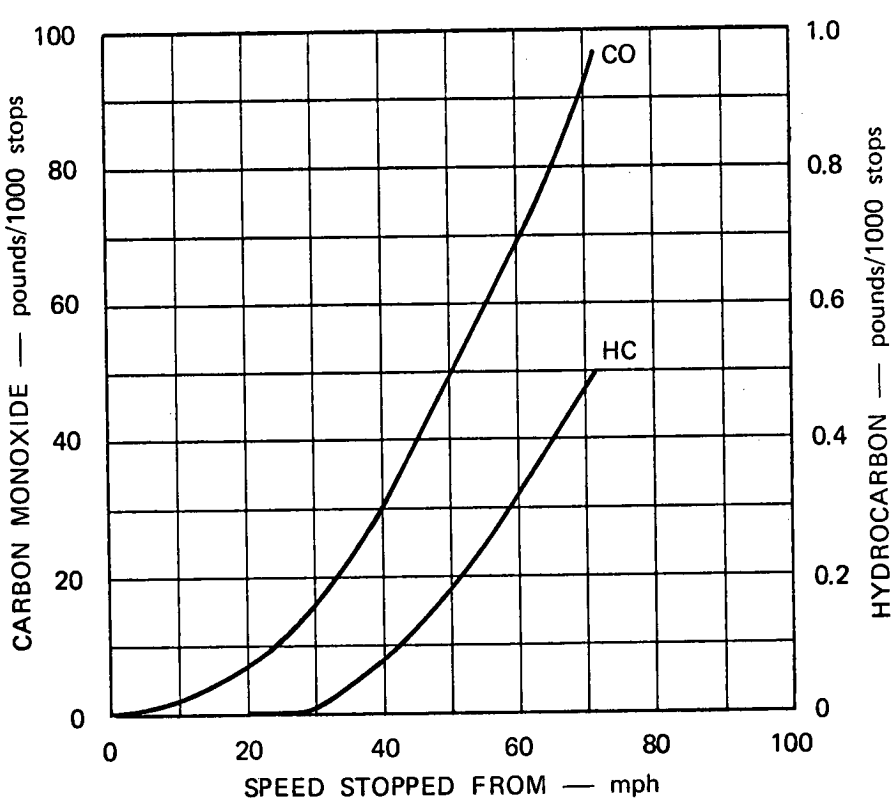


Figure 20. Hydrocarbon and carbon monoxide emissions added per 1,000 stops (reference automobile).

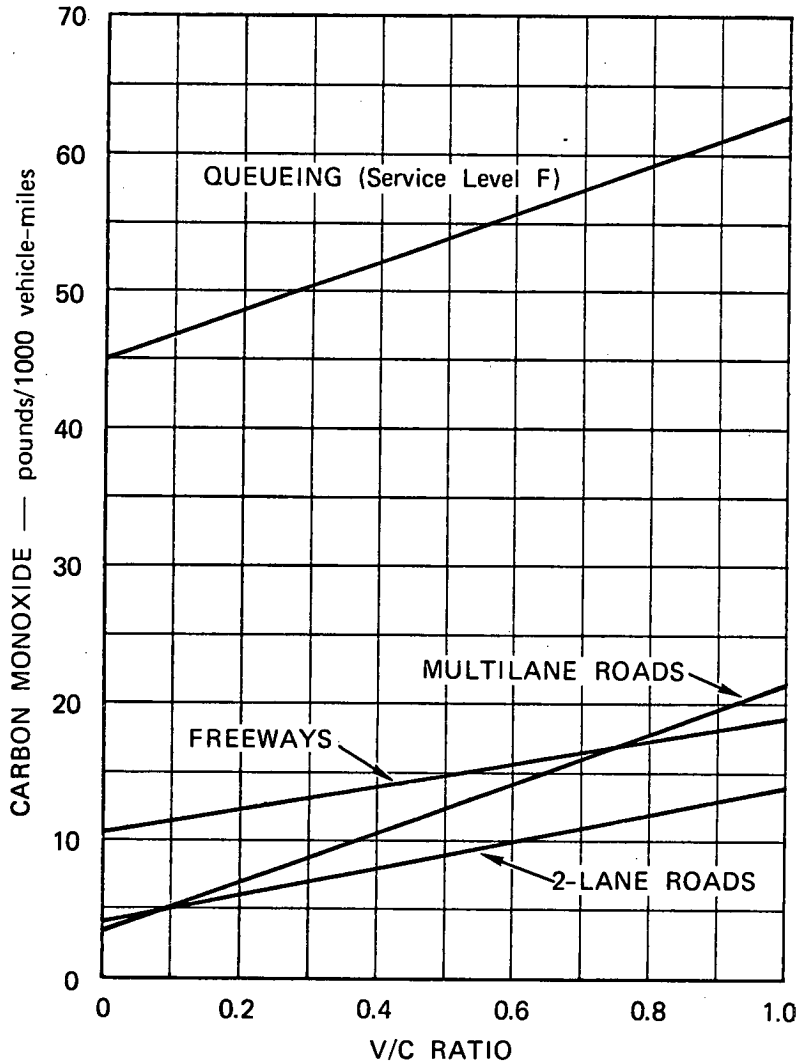


Figure 21. Carbon monoxide emissions added from speed changes per 1,000 vehicle-miles (reference automobile).

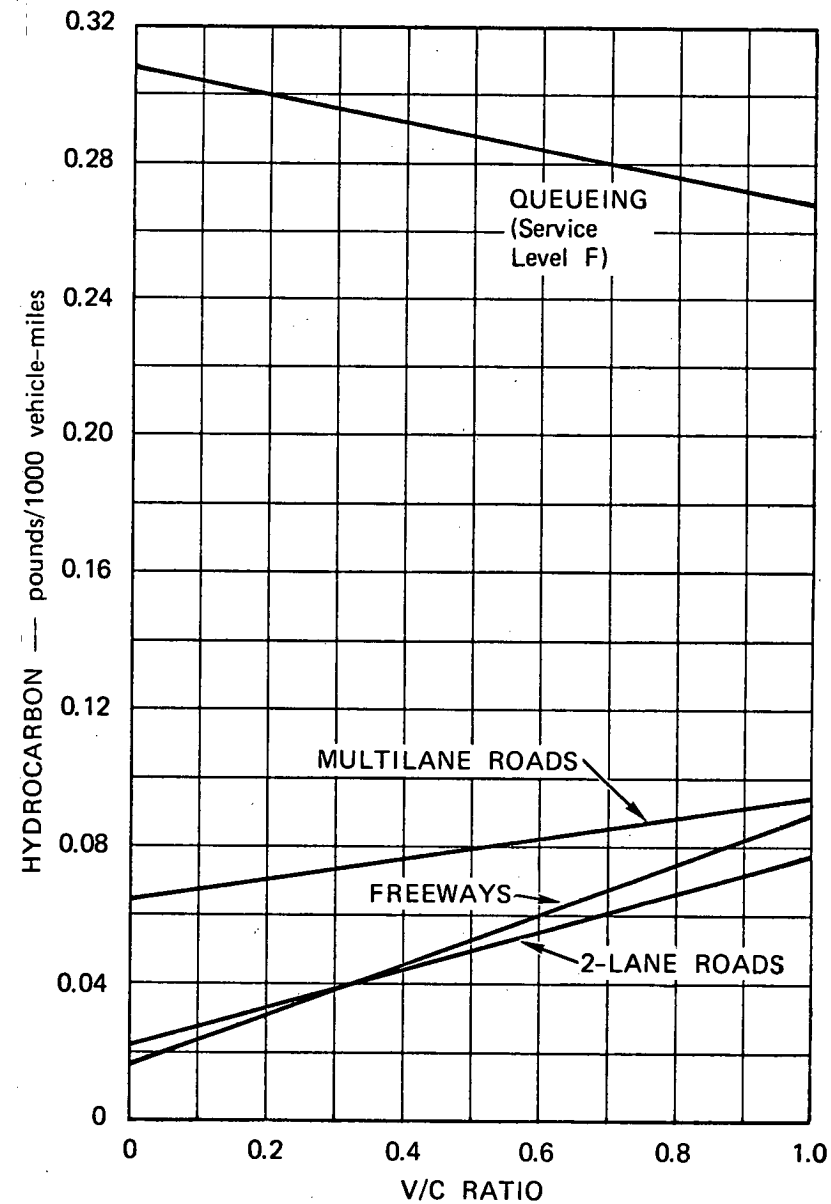


Figure 22. Hydrocarbon emissions added from speed changes per 1,000 vehicle-miles (reference automobile).

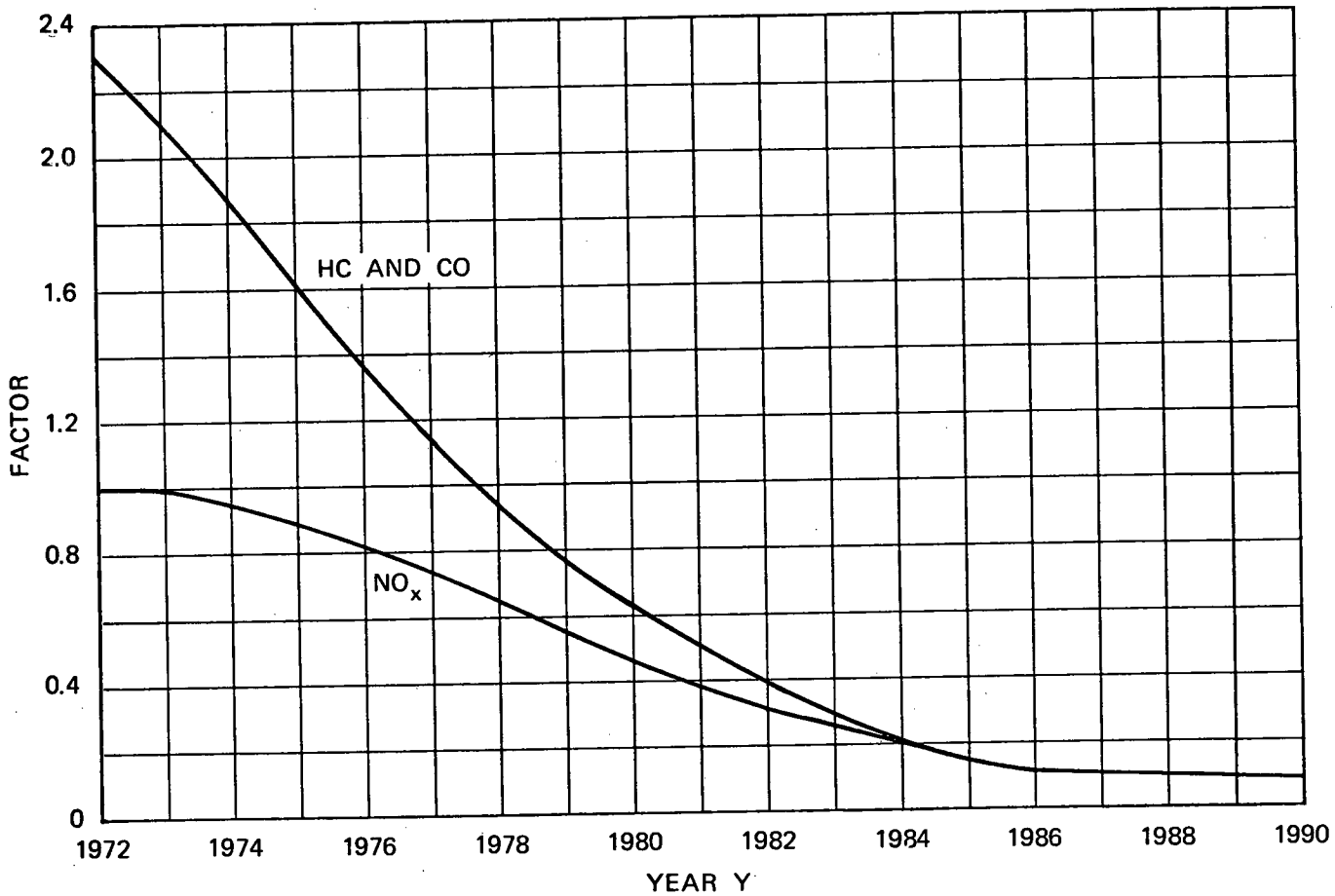


Figure 23. Factor to convert reference year emissions to emissions in year Y.

known. The performance data shown are therefore subject to changes that may result from innovative designs by the automotive industry to meet emission control requirements.

Instructions for completion of Worksheet 6 are provided on the worksheet, with an example to illustrate its use. The example, based on the testing procedure in a federal program to evaluate the emission performance of new automobiles, was developed to show calculations of effects of a wide variety of conditions on vehicle emissions. For this reason it does not use the same projects shown on earlier worksheets.

Lead and Smoke Particulates

As far as can be determined, data on lead and smoke particulate emissions as a function of driving conditions have not been collected, and it is not possible to provide a value for emissions of smoke or other particulates. An approximate value of lead emission can be determined from the fact that average lead content in today's leaded gasoline is about 2 gm per gallon, and that about 80 percent of the lead in the fuel eventually is exhausted from the tailpipe. The remainder of the lead stays in the engine until removed with the oil or the oil filter. If a fuel consumption of 15 miles per gallon is assumed, the lead emissions per vehicle mile would approximate 0.106 gm, or about 2.3×10^{-4} lb per vehicle-mile. Future legislation that would limit the use of tetraethyl lead additives in gasoline (which also facilitates the use of catalytic exhaust control systems to reduce other pollutants) would reduce this value.

NOISE EFFECTS

Noise is unwanted sound, a subjective result of sounds that intrude on or interfere with activities such as conversation, thinking, reading, or sleeping. Sound can exist without people—noise cannot.

Vehicles make sounds during their operation over roadways, from engine and exhaust, tire-roadway interaction, brakes, air disturbance, and chassis or load vibration. The sounds vary with the number and operating conditions of the vehicles, and the directionality and amplitude of the sound vary with highway design features. The highway planner may therefore be concerned with how highway locations and designs influence the vehicle noise perceived by persons living or working nearby.

Some examples of design features that can produce objectionable noises are plus grades, stops, sharp curves or corners, rough open-texture pavements, expansion joints, and at-grade or elevated freeway elevations. Where such

features cannot be avoided or minimized in residential or other noise-sensitive areas, they may at least be compensated for through adequate noise shielding or other protective measures.

Much objectionable highway noise originates with exceptionally noisy vehicles such as motorcycles and trucks. It would be desirable to consider cost and effectiveness trade-offs between (1) better muffler systems and other vehicle noise controls and (2) highway noise control features. In the meantime, the highway designer must cope with present vehicle designs.

The degree of annoyance created by noise—how "unwanted" it is—varies not only with its amplitude but also with its pitch, frequency of occurrence, duration, un-

TABLE 7

ILLUSTRATIVE NOISE LEVELS AND CONSEQUENCES

APPROX. RATIO OF LOUDNESS TO THAT AT REF. LEVEL	dBA	EXAMPLES	PROBABLE PUBLIC REACTION TO PEAK LEVEL NEAR RESIDENCE
×4	90	Average peak noise from diesel trucks at about 35 ft from pavement edge	Local community activity with influential or legal action
			Petition of protest
×2	80		
			Letters of protest
		Downtown traffic in large cities from sidewalk	Complaints likely
			Complaints possible
Reference	70		
		Conversational speech at 3 ft	Complaints rare
×½	60	Quiet residential traffic at 15 ft from pavement edge	Acceptance
×¼	50	Average business office	
		Average residence	
×⅛	40		

Source: Beaton and Bourget (*24*).

expectedness, and relation to background noise levels, as well as with the attitude and economic status of the observers. Of these variables, only amplitude, pitch, and variability are taken into account in the procedures that follow, so the other variables mentioned should be considered by the analyst whenever they differ significantly from average conditions.

Noise Measurement

Noise is commonly measured by the sound pressure level or power level of the sound pressure waves that are carried through air, and by the frequency distribution of the waves that in turn affects the amount of irritation of the hearers. A number of complex measures that relate the amplitudes, frequencies, and variability of the noises have been studied. Sound energy is generally measured in decibels (dB), a unit that corresponds to 10 times the common logarithm of the ratio of energy between two sounds. A change in sound pressure level of 1 dB is barely noticeable to a trained observer; a 3-dB change is easily noticeable to most people; a 10-dB change corresponds approximately to a doubling or a halving of loudness. Sound pressure level is read from a sound level meter, an instrument whose characteristics have been internationally standardized. It has been demonstrated many times that the subjective reaction of humans to noise is reasonably closely correlated with readings on the A-scale (in dBA) of the sound level meter. For this scale, frequencies of greater than 400 Hz are weighted more heavily than those below. This corresponds to the fact that the high-pitched noises are more annoying.

Table 7 gives a scale indicating the probability of public reaction to peak noise near residences and giving some examples of average noise levels. The reference level in Table 7 is 70 dBA, indicated by Beaton and Bourget (*24*) as usually "the maximum limit of exposure in a residential area before public complaint ensues."

Gordon et al. (*25*), in *NCHRP Report 117*, give a comparison of different sound scales, together with detailed procedures for analyzing the variations in traffic noises with traffic parameters, roadway parameters, and location of observers relative to the facility. The procedures permit relating the amount of noise that is heard at any point near the facility to these parameters. The "statistical time distribution" of noise is considered through use of two measures, L_{50} and L_{10}, to indicate levels that are exceeded, respectively, 50 percent and 10 percent of the time. The traffic and roadway parameters used by Gordon et al. (*25*) are generally the same as those used in this report to compute user costs, with the exception that the cross-section and the presence of embankments and depressions or elevations of the roadway relative to the surrounding terrain must be noted for noise evaluation purposes.

A complementary procedure, for tabulating the buildings affected by traffic noise according to the magnitude of the noise effects, is presented in this section. Users therefore will be presumed to have *NCHRP Report 117* (or some similar procedure for estimating traffic noise levels) at hand in the discussion that follows. The *NCHRP Report 117* procedures are not repeated here, as they entail a number of worksheets and reference charts and tables.

Comment on Noise Variability Measurement

One serious potential drawback of the noise criteria in *NCHRP Report 117* that users should be aware of is that noise levels can exceed even the 10 percent standards up to 10 percent of the time *to an unlimited degree*. Thus, typical peak truck noise of 15 dBA greater than average automobile noise levels will *not* exceed the criteria unless the noise occurs more than 10 percent of the time. If the average duration of peak truck noise is about 10 sec, and there are 3,600 sec in an hour, this means that up to 360 sec of truck noise could be tolerated by the design standard. This is 30 trucks per hour, or one every 2 min. It does not require a vivid imagination to guess the consequences of such truck frequencies, or even much lower frequencies, on a nearby conversation or a night's sleep.

Other means of allowing for variability in noise duration have been attempted. The most extreme approach short of counting the highest noise peak is to use the mean of representative peak noise occurrences. This approach is supported by evidence that individuals cannot adapt well to high-level intermittent noises, in comparison with their adaptability to continuous noise sources. No general agreement has yet been reached, however, on this or any other approach to quantifying the annoyance from varying peak noises. Perhaps the only safe assumption is that more research is required to take the effects of peak noise occurrences into account without "overcorrecting" for them. This position is essentially the one taken in *NCHRP Report 117* (see p. 33 concerning research needed on time varying noise). In the meantime, however, the researchers suggest basing noise criteria on the mean of peak noise occurrences, whether in place of or as a supplement to *NCHRP Report 117* procedures. For this purpose, the 70-dBA reference level given in Table 7 is probably a good general guide to objectionable peak daytime, outdoor noise

TABLE 8

EXAMPLE OF RECOMMENDED NOISE LEVELS FOR VARIOUS LAND USES

	RECOMMENDED MAXIMUM MEAN SOUND PRESSURE LEVEL (dBA)		
LAND USE	TIME OF DAY	AT PROPERTY LINE	INSIDE A STRUCTURE
Residential (single and multiple family)	Day	70	65
	Night	65	55[b]
Business, commercial, industrial	All	75	65
Educational institutions	All	70	60
Hospitals, rest homes	Day	60[a]	55
	Night	50[a]	45
Public parks	All	70	55

[a] Expected ambient noise level.
[b] Air-conditioning systems commonly operate at 55 dBA. For non-air-conditioned residential structures, it may be desirable to reduce this value by 5 dBA.
Source: TTI (*26*).

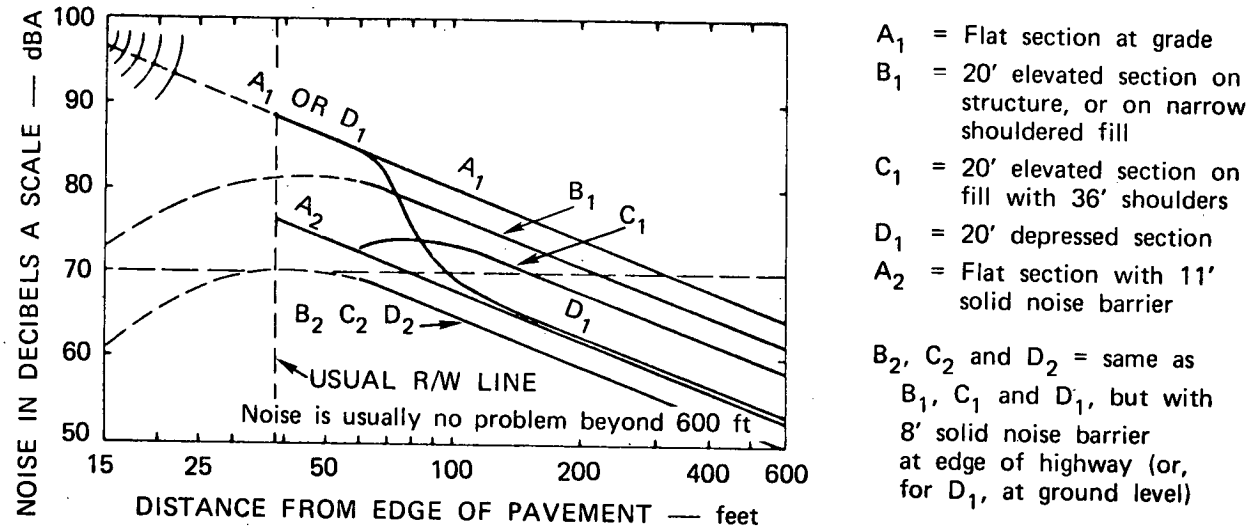


Figure 24. Noise comparison chart for different highway designs. Source: Beaton and Bourget (24).

levels in residential areas. An example of further criteria based on maximum mean noise levels is given in Table 8.

Figure 24 provides an approximate guide to average peak diesel truck noise levels on freeways, using a discrete or point noise source model that results in 6-dB reductions in sound pressure level with a doubling of distance. *NCHRP Report 117* indicated that this approach is appropriate for low-density truck noise. For high-density truck noise (more or less continuous), a line source model would be a better approximation, which would result in a 3-dB reduction per doubling of distance. In such cases, the distance figures on the horizontal axis of Figure 24 should be doubled.

Valuation and Quantification of Noise Measurements

Noise from operation of a highway facility may intrude on persons near the facility in their occupation, living patterns, or recreation. The intrusion may have adverse economic consequences because the location may be rendered less desirable as a place to live or work as a result of the noise, but the relation between the amount of noise and the degree of economic effect has not been well identified. Therefore, it is necessary to produce a statement of environmental impact that will describe and quantify the effect, even though the effect cannot be reduced to a dollar value and included in trade-off analysis with other variables of highway project decisions.

The environmental impact of a change in a highway facility can be measured by the amount of noise that is added or reduced at locations where people are, and it depends on what the people are doing. A measure of the activity is inferred by the type of structure in which the people are located—homes, schools, churches, and offices. Thus, the environmental impact can be quantified by tabulating the number of these kinds of locations that are affected by the proposed highway changes.

There is a strong interaction between land-use planning, land-use controls, the timing of developments alongside highways, and planning for minimizing highway noise impacts. Obviously, if a highway goes through or next to an existing residential development, it will have vastly different impacts than if it traverses vacant land. After the highway is built, the issues of what the land is zoned for and how close noise-sensitive uses, such as single-family residences, are permitted become important. In such cases, consideration of trade-offs between noise-attenuating designs and noise-insensitive land uses may be possible. For example, if land-use plans could limit new residential developments to 500 ft or more from freeway rights-of-way, lower noise control standards could be used.

The procedure for quantifying highway noise impacts consists of three steps: (1) identifying the locations that will be affected by changes in noise level; (2) computation of noise levels at the affected locations; and (3) tabulation of the impacts on Worksheet 7 (Fig. 25). These are discussed in turn in the following.

Identification of Affected Locations

The effects of noise from roadways diminish with increasing distance from the roadway. Some general guidelines for estimating the area affected follow.

1. Intervening structures such as rows of houses provide shielding for structures farther away from the facility, so only the row closest to the facility is usually of interest.
2. Noise levels from highway traffic usually diminish to tolerable levels at distances of approximately 300 to 1,000 ft, so, usually, no location more than 1,000 ft removed is affected.
3. The maximum affected distance depends on the existing level of noise. In quiet rural or suburban locations the distance will approach the 1,000-ft distance, whereas in noisier urban locations the effect probably will not extend more than 300 ft.

Figure 24 provides a comparison of the average peak noise reduction effectiveness of different freeway elevations, with and without barriers, as a more detailed guide to affected distances.

Worksheet 7

NOISE IMPACTS

Project Number _____ Year 1

1. Section (or alternative)	_____					_____					_____				
	Fav-orable			Unfav-orable		Fav-orable			Unfav-orable		Fav-orable			Unfav-orable	
2. Degree of Impact	G	M	N	M	G	G	M	N	M	G	G	M	N	M	G
3. Residential Units (Number)															
3.1 Single-Family															
3.2 Multi-Family															
4. Schools (Number)															
5. Churches (Number)															
6. Hospitals (Number)															
7. Office Space (Square Feet)															
8. Other															
8.1 _____															
8.2 _____															
8.3 _____															
8.4 _____															

Note: G = great impact; M = moderate impact; N = no impact.

Figure 25. Worksheet 7 example.

Computation of Noise Levels at the Affected Locations

With the locations of interest having been identified, the noise levels are computed for these points under each alternative of highway construction that is being studied. The procedure described in *NCHRP Report 117* allows the evaluator to compute the L_{50} and L_{10} noise level at any point, using the traffic volumes and speeds estimated in Chapter Three and information about the roadway cross-section. Average peak noise levels can be estimated from Figure 24, in the absence of more refined procedures.

First-year traffic speeds and volumes should be used in the computation. Because the effect of the noise level changes is most pronounced when there is a rapid change in the noise level, the initial impact is likely to be greater than that produced by subsequent traffic growth.

To reduce computation, locations chosen for computation of the noise level should be as representative as possible of structures of a particular type. That is, the noise levels at two different points that are the same distance from the facility are the same, if the geometry of the road-

way and the traffic flow conditions are the same. Thus, the computation need not be made for every location identified in the preceding step.

Noise level contours may be constructed from a small number of points if the roadway characteristics are reasonably uniform, by using the principles described in the referenced procedure. By interpolating between the points, the effect on each location of interest may be determined.

Tabulation of the Impacts

Worksheet 7 (Fig. 25) provides a means for tabulating the impacts on the affected structures. It shows, for each alternative considered, the number or square feet of locations of interest, classified by their direction and degree of impact. The definition of unfavorable impact is derived from *NCHRP Report 117* (Table B-10). The definition of favorable impact is derived from reasoning that an inverse scale should apply when noise levels from existing facilities are forecast to decline more than 5 dBA. Suggested definitions of impacts in terms of dBA in relation to the criterion level are as follows. (The choice of criteria levels is up to the user; either the values in Table 8, or those in *NCHRP Report 117*, or others may be used.)

Favorable impact:	
Great	More than 10 dbA below criterion.
Moderate	More than 5 dBA to 10 dBA below criterion.
No impact	0 dBA to 5 dBA below criterion.
Unfavorable impact:	
Moderate	More than 0 dBA to 5 dBA above criterion.
Great	More than 5 dBA above criterion.

CHAPTER FIVE

ACCIDENT COSTS AND CONSUMERS' SURPLUS

ACCIDENT COSTS

The procedures suggested in this section for estimating traffic accident costs rely on user-supplied accident rate estimates as a function of vehicle-miles of travel on each section or highway type, in combination with estimated costs per accident. Suggested costs per accident by severity class are given in Table 9. Derivation of the accident cost estimates and procedures is described in Appendix B. Winfrey (*4*) gives a detailed history and discussion of approaches to accident cost estimates for a variety of highway and traffic conditions.

Past studies attempting to associate accident frequency with speed or traffic density have given conflicting results (e.g., see Ref. *27* and *28*); therefore, no provision is made in this report for such estimates.

Instructions for completing Worksheet 8 (Fig. 26) follow. The example shown on the worksheet is for a freeway and an arterial section.

Heading: Enter the project number, the study year, and the vehicle type. Note that "all vehicles" should be the usual choice, even if passenger cars and trucks are analyzed separately in Chapter Three. This is because accident rates and costs for trucks are not readily estimated or accounted for separately. Also, the much smaller incidence of truck accidents decreases the statistical reliability of truck accident rates for a small sample of roads and traffic.

Item 1: Enter the code designation for each project section or alternative. Individual road sections of similar highway facility types can be added together when no distinction can be made between them as to accident rates. Similarly, in the items that follow, the peak and off-peak traffic periods can be added together (as in the example) when no accident rate or unit cost differences are discernible between the two periods.

Item 2: Enter for each alternative the estimated accident rates per million vehicle-miles, by the severity class of the accident (fatal, nonfatal injury, or property damage only). The guidelines below should be followed in making these estimates:

1. Experience has shown that only from 20 to 40 percent of property-damage-only accidents are normally re-

TABLE 9

AVERAGE ACCIDENT COSTS BY SEVERITY CLASS

	COST ($), BY SEVERITY CLASS		
ROAD DESCRIPTION	FATAL	NON-FATAL INJURY	PROPERTY DAMAGE [a]
Urban surface roads	13,800	2,900	270
Rural surface roads and all freeways	18,500	4,500	370

[a] Average costs for all property-damage-only accidents, including unreported accidents; they should be multiplied by 2.5 to produce costs per reported property-damage-only accidents.

Note: These estimates do not imply that injuries or loss of human life can be fully equated with some dollar cost. Dollar costs of accidents represent only the tangible economic consequences of accidents and do not include such intangible effects as physical suffering and family separation. Nor do these accident costs include the public costs of traffic safety programs, driver education, and highway patrols, which presumably would not be affected by small changes in accident rates. These average costs also exclude loss of future earnings in cases of death and permanent disability, a controversial indirect cost that is sometimes included in accident cost estimates.

Source: Developed from estimates of the direct cost of accidents by the 1958 Illinois study (*29*), modified to include estimated auto insurance overhead cost (×1.4), and updated from 1958 to 1970 price level.

Worksheet 8

ACCIDENT COSTS

Project Number 1303 Year 1 X All Vehicles __ Passenger Cars __ Trucks

1. Section or alternative	(ARTERIAL)		(FREEWAY)					
	Peak	Off-Peak	Peak	Off-Peak	Peak	Off-Peak	Peak	Off-Peak
2. Accident rates per MVM								
2.1 Fatal	.040		.020					
2.2 Nonfatal injury	3.31		0.66					
2.3 Property damage*	21.85		2.23					
3. Average cost per accident								
3.1 Fatal	$13,800		$18,500					
3.2 Nonfatal injury	2,900		4,500					
3.3 Property damage	270		370					
4. Accident costs per MVM								
4.1 Fatal (2.1 x 3.1)	552		370					
4.2 Nonfatal injury (2.2 x 3.2)	9599		2970					
4.3 Property damage (2.3 x 3.3)	5900		825					
4.4 Total (4.1 + 4.2 + 4.3)	16,051		4165					
5. Billions of vehicle miles of traffic (W3, 14. x 10^{-3})	0.100		0.100					
6. Total, thousands of dollars (4.4 x 5. x 2†)	3210		833					

* The indicated rates should include unreported accidents (multiply reported property-damage-only accident rate by 2.5 in the absence of a better estimate).

† Omit this factor for sections that are analyzed separately for each direction of traffic.

Figure 26. Worksheet 8 example.

ported to public authorities. To compensate, property-damage-only accident rates should be adjusted: in the absence of more precise information on the degree of under-reporting, multiplying the reported rates by 2.5 is suggested, which assumes a 40 percent reporting level. Note that the illustrative property-damage-only rates in Table 10 already have been adjusted to include unreported accidents.

2. Accident rates for a given road section that are based on the occurrence of less than 50 accidents (of all types) in 1 year cannot be considered statistically reliable. In such cases, the accident rates estimate should be based on either (1) several years of recent accident experience, or (2) average accident rates on similar types of roads or intersections from city, regional, or statewide statistics. Examples of such rates (already adjusted to include estimated unreported property-damage-only accidents) are given in Table 10, together with illustrative cost computations based on the average costs in Table 9. However, these accident rates are generalized, and it is preferable not only to use

local statistics, but to tailor the accident rate estimate to specific highway conditions and hazards when possible.

3. Some discretion is necessary when estimating accident rates for new or radically improved roads in comparison with actual past rates for the existing road. For example, there may be cases where the past accident rates for the existing highway are much *higher* than typical rates for that type of road, owing perhaps to some unsafe feature of the particular section. If this is so, and if the unsafe feature is not to be corrected during construction of the improvement, the accident rate may also be higher than usual for the "improved" road. Alternatively, the past accident rate for the existing road may be *lower* than is typical for roads of its type, and even lower than the typical rate for the new type of road to be constructed. But it would be contrary to common sense to show higher accident rates for a much-improved replacement road. Judgment must therefore be used in deciding whether to accept typical accident rates for a new road, to use a lower or higher rate, or to ignore accident rates and costs altogether for the particular improvement.

TABLE 10

ILLUSTRATIVE TRAFFIC ACCIDENT RATES AND COSTS

ROAD DESCRIPTION	ACCIDENT RATES AND COSTS ($/MILLION VEH-MI)[a] FATAL	NON-FATAL INJURY	PROPERTY DAMAGE[b]	TOTAL
Urban surface roads				
Chicago, 1958:				
Local streets				
Accident rates	0.047	6.05	49.23	55.33
Accident costs	649	17,545	13,292	13,486
% of total costs	2.1	55.7	42.2	100
Urban arterials				
Accident rates	0.040	3.31	21.85	25.20
Accident costs	552	9,599	5,900	16,051
% of total costs	3.4	59.8	36.8	100
California state highways, 1963:				
Undivided, 4 or more lanes				
Accident rates	0.032	2.13	9.40	11.56
Accident costs	442	6,177	2,538	9,157
% of total costs	4.8	67.5	27.7	100
Divided expressways				
Accident rates	0.067	1.49	5.88	7.44
Accident costs	925	4,321	1,588	6,834
% of total costs	13.5	63.3	23.2	100
Rural surface roads				
California state highways, 1963:				
Undivided, 2 lanes				
Accident rates	0.084	1.05	3.23	4.36
Accident costs	1,554	4,725	1,195	7,474
% of total costs	20.8	63.2	15.0	100
Undivided, 4 or more lanes				
Accident rates	0.102	1.45	5.75	7.30
Accident costs	1,887	6,525	2,128	10,540
% of total costs	17.9	61.9	20.2	100
Freeways				
Chicago, 1958, urban:				
Accident rates	0.016	1.28	7.63	8.93
Accident costs	296	5,760	2,823	8,879
% of total costs	3.3	64.9	31.8	100
California, 1963:				
Urban				
Accident rates	0.020	0.66	2.23	2.91
Accident costs	370	2,970	825	4,165
% of total costs	8.9	71.3	19.8	100
Rural				
Accident rates	0.040	0.45	1.25	1.74
Accident costs	740	2,025	462	3,227
% of total costs	22.9	62.8	14.3	100

[a] Accident costs = accident rates × average unit accident costs (Table 9).
[b] Reported property damage accident rates have been multiplied by 2.5 to allow for unreported accidents.

Item 3: Enter the proper set of average costs per accident from the two sets given in Table 9, depending on the type and location of each road alternative (urban or rural surface road or freeway).

Item 4: Obtain and enter the product of items 2 and 3. If separate dollar totals are not needed for each severity class of accident, lines 4.1 through 4.3 can be omitted as long as the cumulative product is shown on line 4.4.

Item 5: Calculate and enter the billions of vehicle-miles of traffic per year, obtained by multiplying line 14 of Worksheet 3 by 10^{-3}.

Item 6: Total accident costs is the product of lines 4.4 and 5 (times 2, if the other direction of traffic will not be separately analyzed). These results will be transferred to Worksheet 9, line 4.

An alternative method of estimating accident costs may be necessary when only data or estimates on the total number of accidents are available, without information on severity rates. The suggested method in such cases is to estimate and multiply by an average cost per accident, but keep in mind that the average costs per accident will (1) vary widely depending on the proportionate distribution of accidents by severity, and (2) be almost doubled if unreported accidents are omitted from the statistics or estimates. Table 11, which gives average cost per accident based on Table 10, illustrates these two phenomena.

SUMMARY OF USER COST AND TIME REDUCTIONS

Worksheet 9 (Fig. 30) provides for a summary of user costs and travel time reductions according to the consumers' surplus approach in order to allow for the contingency of *generated traffic*. Generated or induced traffic is *that portion of predicted trips that could not or would not have been made in the absence of a new highway facility*, including long-run traffic increases due to residential and commercial development that is induced by the new facility (i.e., that would not take place without the new facility). Generated traffic is distinguished from *diverted traffic* that comes from *other less attractive routes within the network under analysis* to travel on a new facility. The former costs of diverted traffic can be estimated and accounted for in the highway network, and normal growth traffic is included for both the "do-nothing" and improvement alternatives,

but the previous travel time and user costs for generated traffic (or the equivalent time and costs of activities for which the travel is being substituted) are not known, and some distribution of previous time and costs for generated traffic must therefore be assumed.

The assumption suggested, and provided for in Worksheet 9, is that the former time and costs for generated traffic are distributed uniformly over the range from old to new travel time and user costs. The suggested assumption is consistent with the method proposed by Mohring and Horowitz (*30*) for estimates of benefits attributable to generated traffic, based on the consumers' surplus formulation of traveler benefits and assuming that the demand curve is linear over the relevant range.

The consumers' surplus formulation is shown in Figure 27, in which

$\bar{P}_0$ = average price (average user costs plus average value of travel time) per trip predicted for a given time period under unimproved conditions (alternative 0);

$\bar{P}_1$ = average price per trip for the same time period under alternative 1;

V_0 = volume of traffic predicted for the same time period under unimproved conditions (alternative 0); and

V_1 = higher volume of traffic for the same time period under alternative 1.

Consumers' surplus is defined as the excess of the price (represented in this case by user costs and travel time value) that consumers would be willing to pay rather than go without a good or service or substitute other goods or services, in contrast to the price actually paid. Hence, in Figure 27, the consumers' surplus at a given price is the total area above that price enclosed by the demand curve up to the point of its intersection with the price axis. Thus, in comparing two prices such as $\bar{P}_0$ and $\bar{P}_1$, it is convenient to refer to differences in consumers' surplus (i.e., the area enclosed by the price axis, the demand curve, and the two unit price lines) as the net benefit.

The consumers' surplus concept usually is applied in the short run, when only the given price changes and other conditions, including the marginal utility of money to consumers, remain unchanged. However, in the long-run conditions of traveler adaptation to a new highway facility, it is believed to be an acceptable simplification to postulate a single long-run demand curve, as in Figure 27, rather than multiple short-run curves. The single curve implies that smaller increments of highway improvements would have demand-generating effects intermediate between the old and the new price for highway service, which is a reasonable assumption. Put another way, the long-run curve corresponds to the locus of points on short-run demand curves for each time period that are actually built (i.e., a short-run curve can only represent demand in relation to short-run variations in highway service levels or prices).

Note that whereas short-run travel demand curves can be observed (within limits) and are relatively inelastic, the long-run demand curve can be estimated or inferred only from short-run curves and is relatively more elastic. The increased elasticity occurs because more variables relating to highway service can be changed in the long run.

TABLE 11

ILLUSTRATIVE AVERAGE COSTS PER ACCIDENT

ROAD DESCRIPTION	AVERAGE COST ($) ALL ACCIDENTS [a]	AVERAGE COST ($) PER REPORTED ACCIDENT
Urban surface roads		
Chicago, 1958:		
Local streets	570	1,200
Urban arterials	640	1,300
California state highways, 1963:		
Undivided, 4 or more lanes	790	1,500
Divided expressways	920	1,700
Rural surface roads		
California state highways, 1963:		
Undivided, 2 lanes	1,710	3,100
Undivided, 4 or more lanes	1,440	2,700
Freeways		
Chicago, 1958, urban	990	2,000
California, 1963:		
Urban	1,430	2,700
Rural	1,850	3,300

[a] Including estimated unreported accidents (see Table 10).

Use of the consumers' surplus approach may be questioned as only approximately representing a complex situation in which the generated traffic may substitute higher over-all transportation costs (although costs per mile are lower) for lower prices of other goods, notably housing. However, the researchers believe there is no other valid way of handling this problem, even in an approximate way.

The increase in consumers' surplus in Figure 27 is equal to Area D ($\bar{P}_0KL\bar{P}_1$). This increase can be defined as the benefits or perceived excess of value (willingness to pay)

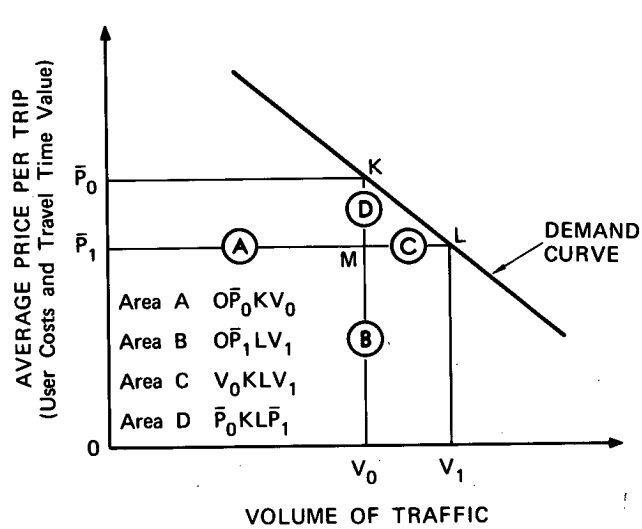


Figure 27. Consumers' surplus method.

over actual user costs accruing to all users of a commodity (highway service, in this case), compared with the excess of value over user costs under previous conditions. Note again that Figure 27 (1) is for a single point or period in time (e.g., year 20 of the study period), and (2) represents the comparison of conditions *with* versus conditions *without* a given highway improvement alternative.

There are basically two methods of calculating Area D: (1) indirectly as the summation of Areas A − B + C, or (2) directly as the difference in traffic volume times average user costs. The two methods are discussed in turn in the following.

INDIRECT METHOD FOR CALCULATING CHANGE IN CONSUMERS' SURPLUS

The indirect method for calculating Area D entails the following steps.

1. Area A $= \bar{P}_0 V_0$ (4)
2. Area B $= \bar{P}_1 V_1$ (5)
3. Area C $= \left(\frac{\bar{P}_0 + \bar{P}_1}{2}\right)\left(V_1 - V_0\right)$ (6)
4. Area D = A − B + C (7)

TABLE 12

CHECK ON ILLUSTRATIVE CONSUMERS' SURPLUS PROBLEM

ASSUMPTION	ALL 1,000 CARS ON 1rr GENERATED	411 GENERATED CARS ON 1rr AND 989 ON 1fw	ALL 1,400 OF THE GENERATED CARS ON 1fw
1. Savings per vehicle-mile for generated traffic (one-half of $\bar{P}_0 - \bar{P}_1$) ($)			
a. On 1rr	0.01	0.01	0.01
b. On 1fw	0.015	0.015	0.015
2. Savings per vehicle for nongenerated traffic ($)			
a. On 1rr	—	0.02	0.02
b. On 1fw	0.03	0.03	0.03
3. Number of vehicles			
a. Generated on 1rr	1,000	411	0
b. Nongenerated on 1rr	—	589	1,000
c. Generated on 1fw	400	989	1,400
d. Nongenerated on 1fw	2,000	1,411	1,000
4. Increase in consumers' surplus (benefits) ($)			
a. For 1rr:			
1a.×3a.	10	4	—
2a.×3b.	—	12	20
b. For 1fw:			
1b.×3c.	6	15	21
2b.×3d.	60	42	30
c. Total (4a.+4b.)	$76	$73	$71

The first two steps are sometimes the only ones used in highway economy studies. Of course, they produce an understatement of benefits without step 3 whenever a highway improvement results in increased travel demand. The understatement of benefits is equal to Area A, and is proportional to the amount of generated traffic.

Direct Method for Calculating Change in Consumers' Surplus

The direct method for calculating Area D is to solve

$$\text{Area D} = (\bar{P}_0 - \bar{P}_1)\,\frac{V_0 + V_1}{2} \qquad (8)$$

This formula is the product of the unit price savings times the average traffic volume, and it is equally applicable to running cost and time reductions, reduced congestion delays, or accident cost savings. The formula is considerably simpler than the four-step indirect method for calculating Area D; its only disadvantage is that total travel costs are not produced in the calculations. However, the "total costs" that can be produced by the indirect method are misleading anyway. The "total costs" produced by that method would consist of Areas A + C (this is in a sense total costs at the new volume, V_1, but at the old price, $\bar{P}_0$) and Area B (which is total costs at the new volume and price, V_1 and $\bar{P}_1$). The difference between these two totals, or (A + C) − B, correctly gives benefits, or increase in consumers' surplus. But a correct result *cannot* be obtained through comparing actual total travel costs for alternatives with different travel volumes; that is, Area A vs Area B. This comparison does provide a simple and correct solution when there is no generated traffic; but the direct consumers' surplus method provides an equally simple and more general solution; therefore, it has been adopted in Worksheet 9.

Example

To illustrate that the consumers' surplus method produces valid results even when applied at the disaggregated level of individual road sections, consider the following simplified data and solutions for an example problem.

Project Description

A new 1-mile four-lane freeway is being built to supplement an existing 1-mile two-lane rural road. A peak-period traffic volume of 2,000 vph is estimated for year 20 without the freeway, or 3,400 vph with the freeway, in which case the freeway would have 2,400 vehicles and the two-lane road would have 1,000 vehicles, or 41.2 percent of total trips. Note that it is not obvious at this point how the generated trips are distributed between the two links.

Figure 28 shows the project. Complete data for the two links are as follows:

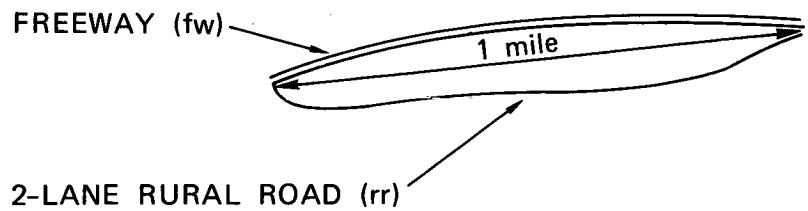


Figure 28. Example project layout.

PROJECT DATA:			
Alternatives	DO NOTHING	BUILD FREEWAY	
Road type	RURAL ROAD	RURAL ROAD	FREEWAY
Section code	0rr	1rr	1fw
P_0, \$/mi (running cost)	0.08		
P_1, \$/mi	—	0.06	0.05
V_0, vph	2,000		
V_1, vph	—	1,000	2,400
Indirect consumers' surplus solution:			
1. Area A $= P_0V_0$	\$160		
2. Area B $= P_1V_1$		\$60	\$120
3. Area C:			
a. $\frac{\bar{P}_0 + \bar{P}_1}{2}$		\$0.0665 *	
b. $V_1 - V_0$		1,400	
c. a. $\times$ b.		\$93	
4. Area D = A − B + C		\$73	
Direct consumers' surplus solution:			
1. Area D:			
a. $\bar{P}_0 - \bar{P}_1$		\$0.027	
b. $\frac{V_0 + V_1}{2}$		2,700	
c. a. $\times$ b.		\$73	

* $\bar{P}_0 = \$0.08$; $\bar{P}_1 = \frac{(\$0.06 \times 1{,}000) + (\$0.05 \times 2{,}400)}{3{,}400} = \frac{\$180}{3{,}400} = \$0.053$; $\frac{\$0.08 + \$0.053}{2} = \$0.0665$.

Results.—The two solutions lead to the same consumers' surplus increase of \$73. Note that these methods presume the allocation of generated traffic in proportion to the final traffic on each section, which is a reasonable assumption. In this problem, this presumes 41.2 percent on each, which amounts to 411 vehicles on the rural road and the balance of 989 on the freeway.

Check on Results.—As a check on the results, the same problem is worked out in Table 12 with different assumptions as to the proportion of traffic on each section presumed to be generated. Note that only the middle assumption, of 411 generated cars on the rural road, produces results consistent with the \$73 resulting from the foregoing calculations.

Origin and Destination Considerations

Note that the preceding problem entails alternative routes in the same direction, and between the same pair of origins and destinations (O&D pairs). When different O&D pairs are included in the problem being analyzed, it is essential to subaggregate the benefits to each O&D pair separately, so that different prices and demands for the same commodity (trip) will, in effect, be compared. This requires the explicit definition of all O&D pairs and corresponding traffic volume assignments. For example, Project 410 (Fig. 5) constitutes the only link between a and d; hence, it can be treated as one section for obtaining the increase in consumers' surplus. Similarly, in the preceding problem, both the two-lane road and the freeway connect the

same O&D pair, so they can (and should) be treated together.

To illustrate a more complex problem, consider the network and AADT estimates shown in Figure 29.

Applying the foregoing principles to this network, the O&D pairs ad and ac would be analyzed separately, with 1,000 vehicles assigned to ab and a total of 4,000 vehicles assigned to ad, 2,000 via the bottom route and 2,000 via the top route. (Average speed and running costs per vehicle would, of course, first need to be calculated separately for sections ab and bc, using their respective total volumes of 3,000 and 1,000 AADT, before dividing the network as indicated for the calculation of consumers' surplus.)

Worksheet 9 Instructions

Worksheet 9 (Fig. 30) incorporates the foregoing approach to consumers' surplus calculations; the relationship between the symbols used in the preceding discussions of consumers' surplus and the items of Worksheet 9 is summarized as follows:

SYMBOL	ITEM
$\bar{P}_0$	Item 6 for alternative 0 sections
$\bar{P}_1$	Item 6 for alternative 1 sections
$\bar{P}_0 - \bar{P}_1$	Item 8
V_0	Item 4 for alternative 0 sections
V_1	Item 4 for alternative 1 sections
$(V_0 - V_1)/2$	Item 9

Instructions for completing Worksheet 9 follow. The example on the worksheet is based on the same Project 410 that is analyzed on Worksheets 2, 2A, and 3.

Item 1: Enter the individual sections to be analyzed, except that abutting sections for which the traffic is identical (or which in total handle all the traffic between an O&D pair) can be combined. Because AADT is the same for all sections of Project 410 in each study year, the complete alternatives are shown in item 1 for each study year.

Note that peak and off-peak period data are combined on Worksheet 9. Separate headings would be needed only for the case where it is desired to find separate total costs or benefits for peak period and off-peak period traffic by carrying the two periods through all the calculations. Note also that sections or alternatives should be listed in *ascending order of construction cost,* from left to right. If separate sections rather than alternatives were being listed—due, say, to an off-ramp at point b in Figure 5—this would require the following type of listing:

0ab 1ab 0bc 1bc

Items 2, 3, and 4: Obtain these data from Worksheets 3 and 8, as indicated.

Item 5: Annual traffic volume is summed only for peak and off-peak hours, as indicated, not over different sections (that would be double counting).

Note: At this point, traffic volumes between alternatives in each year should be compared. In cases where the traffic volume is equal for two alternatives, there is no generated traffic, and *items 6, 8, and 9 can be skipped.* In that event, which holds true for Project 410, calculation of consumers' surplus in lines 10.1, 10.2, and 10.3 is based on lines 2, 3, and 4 of the worksheet, as follows:

1. *Line 10.1:* In line 2, subtract alternative 1 from alternative 0, and multiply the result by 2, unless sections are being analyzed directionally.
2. *Line 10.2:* In line 3, follow the procedure for line 10.1.
3. *Line 10.3:* In line 4, follow the procedure for line 10.1.

Items 6, 8, and 9: For sections or alternatives involving generated traffic (see preceding note), carry out the indicated calculations.

Item 7: Fill in the headings for alternative pairs to be compared. Note that this item must be consistent with item 1; i.e., if individual sections are specified in item 1, item 7 should specify the *comparison* of those sections between two alternatives (e.g., 1ab vs 0ab). In the worksheet example, entire alternatives are specified in item 7 because that is the option used in item 1 also (see item 1 of Worksheet 10 for further comments on specification of alternative pairs in the event of multiple alternatives).

Items 10.1, 10.2, and 10.3: Perform the indicated calculations.

Item 10.4: This item requires assumption of a value of truck time (e.g., $5.00 per hour). Also, if truck percentages differ in the peak and off-peak periods, an average truck percentage should be obtained, weighted by the amount of traffic in the peak and off-peak hours; i.e.:

$$\frac{(\text{Peak \% trucks} \times \text{peak traffic}) + (\text{Off-peak \% trucks} \times \text{off-peak traffic})}{\text{Total traffic}}$$

For Project 410, the weighted average truck percentage is 3.78 percent.

Item 10.5: An hourly value should be assumed for truck time (e.g., $3.00 per vehicle-hour). This figure is used as indicated to calculate the value of passenger-car time reduction, or consumers' surplus. Note in the example problem that running costs increase somewhat between alternatives 0 and 1 (savings are negative), but this increase is more than offset by savings in the value of truck time and passenger-car time.

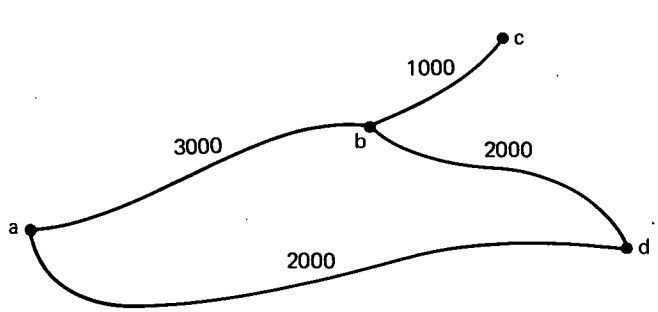


Figure 29. Example network problem.

Worksheet 9

SUMMARY OF USER COST AND TIME REDUCTIONS

Project Number 410 Year 1 & 20 X All Vehicles ___ Passenger Cars ___ Trucks

	Year 1		Year 20	
1. Alternative or section	0	1	0	1
2. Vehicle time, thousands of hours (∑ W3, 15., all sections with same traffic volume)	1340.04	1332.25	3405.88	3108.96
3. Running cost, thousands of dollars (∑ W3, 16., same sections)	3946.67	3949.30	8471.00	8479.01
4. Accident costs, thousands of dollars (∑ W8, 6., same sections)	___	___	___	___
5. Annual traffic volume, millions of vehicles (W3, 13., peak + off-peak)	6.57	6.57	14.23	14.23
6. Average costs and travel time:				
6.1 Vehicle time, hptv (2. ÷ 5.)	___	___	___	___
6.2 Running cost, $ptv (3. ÷ 5.)	___	___	___	___
6.3 Accident costs, $ptv (4. ÷ 5.)	___	___	___	___
7. Alternative pairs		1 vs 0		1 vs 0
8. Reduction in average costs and travel				
8.1 Vehicle time, hptv (6.1, Alternative 0-1, 0-2, etc.)		___		___
8.2 Running cost, $ptv (6.2, Alternative 0-1, 0-2, etc.)		___		___
8.3 Accident cost, $ptv (6.3, Alternative 0-1, etc.)		___		___
9. Average traffic volume, millions of vehicles (5. for alternative 0 + given alternative, ÷ 2)		___		___
10. Time and cost reductions (benefits)				
10.1 Vehicle time, thousands of hours (8.1 x 9. x 2*)		15.51		593.68
10.2 Running cost, thousands of dollars (8.2 x 9. x 2*)		- 5.26		- 15.94
10.3 Accident cost, thousands of dollars (8.3 x 9. x 2*)		___		___
10.4 Value of truck time if trucks are not analyzed separately, thousands of dollars (10.1 x W2, 11.3 x $ 5 /hr)		2.93		112.21
10.5 Value of passenger car time, thousands of dollars ([10.1 x $ 3 /hr] - 10.4)		43.60		1668.83

* Omit this factor for any sections that are analyzed separately for each direction of travel.

Figure 30. Worksheet 9 example.

CHAPTER SIX

SUMMARY AND INTERPRETATION

ECONOMIC INDEXES

Worksheet 10 (Fig. 31) provides for derivation of economic indexes for a project, based on user costs and other data from previous worksheets. These indexes are explained in Appendix C.

Instructions for completing Worksheet 10 follow.

Item 1: Enter the code designations for the pairs of alternatives to be compared, in the following sequence: (1) the code number of the higher-investment-cost alternative, followed by (2) a hyphen or "vs," followed by (3) the code number of the lower-investment-cost alternative.

For projects involving more than one alternative for improving the existing condition, *each* alternative must be compared with favorable alternatives of lower investment cost to obtain indexes for each *increment* of investment cost, as well as for each alternative compared with the existing situation (alternative 0). (Certain comparisons may be eliminated by logic. For example, when alternative 1 and 2 are unfavorable (incremental benefit/cost ratios less than 1.0 *or* less than the ratios for alternatives 3 and 4), alternatives 3 and 4 need not be compared against them.) If the number of alternatives (other than the "do-nothing" case) is designated by N, the maximum number of comparisons to be made is equal to $1 + 2 + 3 + \ldots N$. For example, when $N = 4$, the number of comparisons could be $1 + 2 + 3 + 4$, or 10, identified as follows:

1-0	2-1	3-2	4-3
2-0	3-1	4-2	
3-0	4-1		
4-0			

Items 2 and 3: Calculate and enter the incremental consumers' surplus for each alternative, as indicated.

Item 4: The average annual increase or decrease in the consumers' surplus of user costs and travel time value is obtained as indicated. This presumes a constant annual change in user costs and time value. See Winfrey (*4*) if geometric changes are presumed; i.e., a constant annual *percentage* change.

Item 5: Specify the study period and discount rate (e.g., 25 years at 8 percent), and find the present worth factors, as indicated. Selected factors can be printed on the worksheets if the study period length and the discount rate are uniform for all projects.

Line 5.1 is the uniform series present worth factor, used to convert a constant annual amount to an equivalent present worth.

	UNIFORM SERIES PRESENT WORTH FACTOR (SPW)			
YEARS	6%	7%	8%	10%
20	11.47	10.59	9.82	8.51
21	11.76	10.84	10.02	8.65
22	12.04	11.06	10.20	8.77
23	12.30	11.27	10.37	8.88
24	12.55	11.47	10.53	8.98
25	12.78	11.65	10.67	9.08

Line 5.2 is the factor for converting a uniformly increasing gradient series to an equivalent present worth; it is based on calculating the first gradient at the end of year 1. In other words, the increase is assumed to begin at the end of year 0, consistent with the presumption that traffic increases would begin on opening a highway improvement. See Winfrey (*4*, p. 91).

	GRADIENT SERIES PRESENT WORTH FACTOR (GPW)			
YEARS	6%	7%	8%	10%
20	98.70	88.10	78.91	63.92
21	104.88	93.17	83.08	66.76
22	110.98	98.14	87.13	69.46
23	117.00	102.99	91.04	72.03
24	122.93	107.62	94.83	74.47
25	128.76	112.33	98.48	76.77

Line 5.3 is the single amount present worth factor.

	SINGLE AMOUNT PRESENT WORTH FACTOR (PW)			
YEARS	6%	7%	8%	10%
20	0.3118	0.2584	0.2145	0.1486
21	0.2942	0.2415	0.1987	0.1351
22	0.2775	0.2257	0.1839	0.1228
23	0.2618	0.2109	0.1702	0.1168
24	0.2470	0.1971	0.1577	0.1015
25	0.2330	0.1842	0.1460	0.0923

Line 6: Present worths are computed as indicated. Note that maintenance costs are expressed as a cost reduction; therefore, they are a net benefit when positive.

The worksheet example is not continued beyond calculating the present worth of user costs and travel time value (line 6.6). The results indicate that an investment of up to $8,510,000 plus the present worth of residual value and maintenance cost reductions would be economically justified for Project 410 under the indicated conditions and assumptions. Of this total amount, only $461,000 is due to user cost reductions, and the balance, $8,049,000, is due to passenger-car travel time savings.

Worksheet 10

ECONOMIC INDEXES

Project Number 410 X All Vehicles ___ Passenger Cars ___ Trucks

	All Vehicles	Passenger Cars	Trucks
1. Alternative pairs	1 vs 0		
2. Year 1 cost and time value reductions, thousands of dollars			
2.1 User costs (W9, Σ 10.2 + 10.3 + 10.4 for all sections, both peak and off-peak)	-2.33		
2.2 Value of passenger car travel time (W9, Σ 10.5 for all sections, both peak and off-peak)	43.60		
3. Year 20 cost and time value reductions, thousands of dollars			
3.1 User costs (same as 2.1)	96.27		
3.2 Travel time value (same as 2.2)	1668.83		
4. Average annual increase in benefits			
4.1 User costs ([3.1 - 2.1] ÷ 20)	4.93		
4.2 Value of travel time ([3.2 - 2.2] ÷ 20)	81.26		
5. Present worth factors, for 25 years at 8 %			
5.1 Uniform series 10.67			
5.2 Gradient series 98.48			
5.3 Single payment .1460			
6. Present worths, thousands of dollars			
6.1 User costs (2.1 x 5.1) + (4.1 x 5.2)	461		
6.2 Highway maintenance cost reduction (5.1 x W1, 6.: Alternative 0-1, 0-2, etc.)			
6.3 Residual value (5.1 x [W1, 5.1 + 0.5 x 5.2])			
6.4 Subtotal (6.1 + 6.2 + 6.3)	461		
6.5 Travel time value (2.2 x 5.1) + (4.2 x 5.2)	8049		
6.6 Total (6.4 + 6.5)	8510		
7. Highway investment cost (W1, 5.4, for given alternative)			
8. Benefit/cost ratio			
8.1 Excluding travel time value (6.4 ÷ 7)			
8.2 Including travel time value (6.6 ÷ 7)			
Optional Indexes			
9. Rate of return			
9.1 Excluding travel time value (interest rate at which 6.4 = 7.)			
9.2 Including travel time value (interest rate at which 6.6 = 7.)			
10. Cost of time ([7 - 6.4] x $___ ÷ 6.5)			

Figure 31. Worksheet 10 example.

Items 7 and 8: These are found as indicated. Note that benefit/cost ratios (lines 8.1 and 8.2) are calculated both excluding and including the value of travel time. Appendix C describes some uses, and misuses, of benefit/cost ratios.

Item 9: Enter the internal rate of return for each investment cost increment. Detailed procedures for finding the rate of return are available in engineering economy texts; e.g., Winfrey (*4*).

Item 10: Enter the cost of passenger-car time savings for each increment of highway investment cost, obtained as indicated. Note that negative time reductions for line 10 would indicate an increase in passenger-car travel time for the comparison under consideration. The cost of time concept is explained at the end of Appendix C.

PROJECT RANKING AND SELECTION

If a number of independent projects are being compared, as for example in highway improvement programming and budgeting, it is sound economic practice to rank the projects according to their incremental benefit/cost ratios (or some other economic index) and to select the highest-ranking projects. However, a number of issues arise if this procedure is to be carried out correctly, particularly if some projects involve multiple alternatives. Appendix C provides background information on the alternative approaches to this problem, and conclusions regarding preferred approaches.

The balance of this chapter contains procedures for implementing benefit/cost comparisons of projects involving multiple alternatives. It is assumed in this discussion that the existing or "do-nothing" condition, with which improvement alternatives are compared, is itself desirable; e.g., that a given road proposed for improvement is worth retaining in preference to being abandoned. See Winfrey (*4*, pp. 137-140) for a discussion of this question.

Worksheets 11 and 12 are involved in these procedures; copies of these worksheets appear in Appendix D. Completion of Worksheet 11 is obvious because it requires only the transfer of benefit/cost ratios from lines 8.1 or 8.2 of Worksheet 10, plus calculation of incremental ratios *between alternatives other than doing nothing* (alternative 2 vs 1, 3 vs 2, etc.).

Worksheet 12 is for convenience in listing projects and alternatives selected in the procedure, to find the economically optimal project set for a given budget period. Instructions and an example for completing the worksheet follow.

1. To make the first selection, compare all project alternatives with the existing or "do-nothing" alternative; identify the alternative with the highest benefit/cost ratio; and list this alternative on Worksheet 12, together with its investment cost from line 7 of Worksheet 10. The selection of this alternative should be considered tentative if a higher-investment-cost alternative exists for the given project. In subsequent iterations, it is possible for a higher-investment-cost alternative of the same project to be selected and thus displace the lower-investment-cost alernative from the list. If no higher-investment-cost alternative exists, the selection is, of course, final.

In this first iteration, the project alternative with the highest benefit/cost ratio (compared with the "do-nothing" condition) is the one to be selected in each case. For example, if the following data are assumed:

PROJECT ALTERNATIVES COMPARED	BENEFIT/COST RATIO
1 vs 0	1.27
2 vs 0	1.61
3 vs 0	1.82
4 vs 0	1.69

alternative 3, with a 1.82 benefit/cost ratio, would be the one for initial comparison.

2. For the second iteration, consider the same set of project alternatives considered in the previous step, minus the alternatives selected and any lower-investment-cost alternatives of that project, and identify the next lower benefit/cost ratio.

Note that for any higher-investment-cost alternatives of the project alternative selected in the previous step, it is the benefit/cost ratio for the incremental investment cost (compared with the alternative selected) that is to be considered for the second iteration. For example, if the data given for the four projects in the first iteration are assumed, only the benefit/cost ratio for the extra cost of alternative 4 over the cost of alternative 3 would be considered—i.e., the comparison 4-3, according to the code used in Worksheet 11.

If a higher-investment-cost alternative of the project alternative selected in a previous iteration is selected in step 2, the project alternative selected in step 1 should be identified by an asterisk in column 3 of Worksheet 12 and the investment cost listed in column 5 of Worksheet 12 for the higher-investment-cost alternative should be the incremental investment cost of the alternative compared with the investment cost of the lower-investment-cost alternative previously selected.

3. For the third and subsequent iterations, repeat step 2 until the total investment budget or benefit/cost ratios of less than an acceptable level are reached. (Acceptance of projects with benefit/cost ratios of 1.0 or greater is a common economic criterion.)

4. If the last project to be included (the one with the lowest benefit/cost ratio) exceeds the budget, and management policy is to make the list of budgeted projects equal to or less than the investment budget, another iteration must be attempted to find a lower-cost alternative that can be accomplished within the budget. This step is illustrated in the following example.

Example

As an example of the procedure for selecting successive projects, consider the following assumed data for four separate projects—A, B, C, and D—being considered for inclusion in a given year's highway construction budget of $2,200,000. Each project has mutually exclusive alternatives designated as follows: A1, A2, A3, B1, B2, B3, C1, C2, C3, D1, D2, and D3. The construction costs for each

alternative design, arrayed in order of increasing construction costs within a project, are as follows:

DESIGN ALTERNATIVE	COST ($1,000), BY PROJECT			
	A	B	C	D
1	400	600	900	300
2	550	700	1,100	300
3	600	—	1,300	800

Costs derived from this tabulation are arranged in the following to show the increment of construction cost between pairs of project alternatives. A "do-nothing" alternative, designated by the number 0, is included for each project.

PROJECT ALTERNATIVES COMPARED	COST ($1,000), BY PROJECT			
	A	B	C	D
1 vs 0	400	600	900	100
2 vs 0	550	700	1,100	300
3 vs 0	600	—	1,300	800
2 vs 1	150	100	200	200
3 vs 1	200	—	400	700
3 vs 2	50	—	200	500

The increments of construction cost shown here are assumed to result in the respective incremental benefit/cost ratios shown in the following tabulation, which is in the format used for Worksheet 11.

PROJECT ALTERNATIVES COMPARED	INCREMENTAL BENEFIT/COST RATIO BY PROJECT			
	A	B	C	D
1 vs 0	2.75	1.96	2.18	1.67
2 vs 0	2.52	0.42	2.42	1.51
3 vs 0	1.99	—	0.86	1.42
2 vs 1	1.90	−8.82	3.50	1.43
3 vs 1	0.47	—	−2.10	1.39
3 vs 2	−3.84	—	−7.70	1.40

The remainder of this example follows the analytical process prescribed for Worksheet 12.

The first iteration for selecting the lowest benefit/cost ratio considers all alternatives of a project compared with the respective "do-nothing" alternative (0) of the same project for each of the four projects. The first iteration, therefore, considers only the following results of comparing each project alternative with the "do-nothing" alternative:

PROJECT ALTERNATIVES COMPARED	INCREMENTAL BENEFIT/COST RATIO, BY PROJECT			
	A	B	C	D
1 vs 0	2.75	1.96	2.18	1.67
2 vs 0	2.52	0.42	2.42	1.51
3 vs 0	1.99	—	0.86	1.42

The highest benefit/cost ratio in comparing project alternatives with their respective "do-nothing" alternative (0) is 2.75 for alternative A1. Project alternative A1 would be the first one listed on Worksheet 12. No further use is made of the two other comparisons to the "do-nothing" alternative in Project A (i.e., A2 vs 0, A3 vs 0).

It is the next increment of highway construction cost in Project A that must compete with all other project alternatives in selecting the next lowest cost of time in the second iteration. Therefore, only the comparisons of two higher-construction-cost alternatives (2 and 3) of Project A are considered in subsequent iterations.

The second iteration considers the following comparisons:

PROJECT ALTERNATIVES COMPARED	INCREMENTAL BENEFIT/COST RATIO, BY PROJECT			
	A	B	C	D
1 vs 0	n.a.*	1.96	2.18	1.67
2 vs 0	n.a.	0.42	2.42	1.51
3 vs 0	n.a.	—	0.86	1.42
2 vs. 1	1.90	n.a.	n.a.	n.a.
3 vs 1	0.47	—	n.a.	n.a.

* Comparisons designated n.a. are not applicable.

The iteration reveals that alternative C2 has the highest benefit/cost ratio—2.42; this alternative therefore is listed next on Worksheet 12. Only the next higher construction cost (alternative C3) is considered in subsequent iterations.

The third iteration considers the following comparisons:

PROJECT ALTERNATIVES COMPARED	INCREMENTAL BENEFIT/COST RATIO, BY PROJECT			
	A	B	C	D
1 vs 0	n.a.	1.96	n.a.	1.67
2 vs 0	n.a.	0.42	n.a.	1.51
3 vs 0	n.a.	—	n.a.	1.42
2 vs 1	1.90	n.a.	n.a.	n.a.
3 vs 1	0.47	—	n.a.	n.a.
3 vs 2	n.a.	—	−7.70	n.a.

On the third iteration, alternative B1 has the highest benefit/cost ratio—1.96. It is therefore listed on Worksheet 12.

The fourth iteration considers the following comparisons:

PROJECT ALTERNATIVES COMPARED	INCREMENTAL BENEFIT/COST RATIO, BY PROJECT			
	A	B	C	D
1 vs 0	n.a.	n.a.	n.a.	1.67
2 vs 0	n.a.	n.a.	n.a.	1.51
3 vs 0	n.a.	—	n.a.	1.42
2 vs 1	1.90	−8.82	n.a.	n.a.
3 vs 1	0.47	—	n.a.	n.a.
3 vs 2	n.a.	—	−7.70	n.a.

At the fourth iteration, project alternative A2 qualifies with its incremental benefit/cost ratio of 1.90; therefore, alternative A2 is listed on Worksheet 12 with its incremental investment cost of $150,000. Because incremental investment costs have been used, the total investment cost for alternative A2 is the combination of incremental investments in project alternative comparisons A1 vs 0 and A2 vs A1. Project alternative A1 has now been displaced by the higher total-investment-cost alternative A2, so an asterisk is added to alternative A1 in column 3 of Worksheet 8 to indicate this fact.

Project D did not have an alternative that was selected by this procedure.

The data for the foregoing iterations of the example are summarized as follows, for reference, as they would appear on Worksheet 12.

ITERATION	TENTATIVE SELECTION PROJECT NO.	TENTATIVE SELECTION ALTERNATIVE	BENEFIT/COST RATIO	INCREMENTAL INVESTMENT COST ($000)	CUMULATIVE TOTAL ($000)
(1)	(2)	(3)	(4)	(5)	(6)
1	A	1 *	2.75	400	400
2	C	2	2.42	1,100	1,500
3	B	1	1.96	600	2,100
4	A	2	1.90	150	2,250

* Indicates a project alternative displaced later in the list by one having a higher total investment cost.

The incremental cost of $150,000 for alternative A2 brings the cumulative total in column 5 to $2,250,000, or $50,000 over the budget limit. If management decides either (1) to leave alternative A2 in the budget so that it can at least be started in the budget period, or (2) to expand the budget to cover the additional $50,000 for alternative A2, the analysis would stop at this point. However, if management policy is to make the list of budgeted projects equal to or less than the investment budget, the selection of alternative A2 must be voided, and a fifth iteration must attempt to find a lower-cost alternative that can be accomplished within the budget limitation.

The fifth iteration in this example would take the form of a review of alternatives with next-higher benefit/cost ratios to determine whether they fit within the budget limitation. Note that the next-highest benefit/cost ratio of 1.90 in the fourth iteration is 1.67 for alternative D1. The use of alternative D1 exactly fulfills the budget limitation of $2,200,000. This result is summarized as follows, as it would appear on Worksheet 12.

ITERATION	TENTATIVE SELECTION PROJECT NO.	TENTATIVE SELECTION ALTERNATIVE	BENEFIT/COST RATIO	INCREMENTAL INVESTMENT COST ($000)	CUMULATIVE TOTAL ($000)
(1)	(2)	(3)	(4)	(5)	(6)
1	A	1	2.75	400	400
2	C	2	2.42	1,100	1,500
3	B	1	1.96	600	2,100
~~4~~	~~A~~	~~2~~	~~1.90~~	~~150~~	~~2,250~~
5	D	1	1.67	100	2,200

A situation could occur in which all alternatives remaining after B1 would cause an overrun of the budget. In that situation, B1 would be the last acceptable alternative.

CHAPTER SEVEN

APPLICATIONS

SAMPLE PROBLEM OBJECTIVES AND PROCEDURES

Applications of the procedures in this report to problems of determining running costs, air pollutant emissions, accident and truck time costs, the value of time, and an economically optimum set of projects are illustrated in detail on worksheets in preceding chapters. Instead of providing additional examples of the same type, this chapter develops only annual passenger-car running costs and total user costs (excluding accident costs) for the simplified condition of level tangent highways, over a wide range of AADT, under standardized assumptions regarding the proportion of AADT in the peak hour and other conditions. It was believed that such data, presented in comparative form for a variety of highway facility types, would be useful in illustrating the type of results that obtained with the report. Also, the results can be quickly used on Worksheet 10, in combination with AADT projection, to find the present worth of running costs or used costs for passenger cars under the conditions indicated.

The effects of curves and gradients are not included in the basic problem, nor is queuing, owing to the complexities and variety of possible outcomes entailed. One other constraint used, for uninterrupted traffic flow, was that only 70-mph design speed facilities and only two-lane highways with 100 percent passing sight distance were considered. Because lower design standards and speeds are generally the result of curves or grades, it was believed that the running cost of such facilities could not be simply modeled using level tangent cost factors.

The first step in the analysis was to estimate, for each type of facility examined, a reasonable upper limit on AADT, from AADT records for very high volume roads. The limits used are listed as follows, together with assumed typical truck percentages for each facility type (used later to calculate capacity).

FACILITY TYPE	AADT LIMIT (VPD)	ASSUMED TRUCKS (%)
Freeways and expressways:		
10-lane	250,000	4.6
8-lane	200,000	4.6
6-lane	150,000	7
4-lane	100,000	10
4-lane rural highways	60,000	14
2-lane highways	18,000	14
4-lane arterials (with 0.5 to 4 signals per mile)	30,000	—

Calculations then proceeded as follows:

1. AADT per lane pair was determined; this value was used to enter Figure 7 and determine the associated peak-hour volume.
2. The duration of the one-way peak period in hours was assumed as (2.5) (AADT per lane pair)/(AADT limit per lane pair). Thus, the peak duration is very short for small AADT's and increases up to 2.5 hr at the maximum AADT.
3. Peak and off-peak traffic were assumed to flow over an 18-hr day (see Worksheet 2 instructions for a discussion of this concept). Based on this value plus the AADT, the peak-hour volume, and the duration of the peak period, both the duration of the off-peak period and the off-peak hourly volume were determined.
4. Associated v/c ratios for peak and off-peak periods were determined, assuming the capacity of each highway facility to be 2,000 vph per lane (per both lanes of two-lane highways) but reduced by a truck factor based on the previously indicated percentages for each highway type. (The v/c ratio is not relevant for arterials, so no truck assumption was necessary on such facilities.)
5. Average passenger-car speeds and units running cost factors over the various facilities during the peak and off-peak demand periods were determined by entering the graphs of Appendix A with the v/c ratio (or with an average constant speed, for arterial highways). Unit travel time was obtained from the reciprocal of average speed. A value of passenger-car travel time of $3.00 per vehicle-hour was assumed in the calculations.
6. Total per-mile running cost (or travel time) was obtained as the sum of the products of the unit running cost and time value factors times the hourly volume times the duration of peak or off-peak demand periods.
7. For arterials, the running cost and time delay cost of stopping at signalized intersections were calculated and added to the results.

Sample calculations of the foregoing procedure, including the stated assumptions, are given in Table 13. Calculations are given for both a 70-mph, eight-lane freeway and a 35-mph, four-lane arterial with two signals per mile. A single AADT of 16,000 vpd is used. Calculations over the entire range of AADT's were performed on a computer.

RESULTS

Table 14 gives the full results of the foregoing calculations for annual running costs; Table 15 gives the results for running costs plus the value of travel time. Differences in running cost between uninterrupted flow facility types are small, although arterial types are noticeably higher. Differences of both types are larger when travel time values are included.

TABLE 13

SAMPLE COST AND TIME CALCULATIONS

ITEM	8-LANE, 70-MPH FREEWAY		4-LANE, 35-MPH ARTERIAL [a]	
AADT (vpd)	16,000		16,000	
AADT (veh/lane pair/day)	4,000		8,000	
Assumed capacity (veh/lane/hr)	1,900		N.A.	
ITEM	PEAK	OFF-PEAK	PEAK	OFF-PEAK
Hourly volume/lane	320	109	520	198
Demand period (hr/day, each way)	0.2	17.8	1.33	16.67
v/c ratio or degree of saturation	0.17	0.06	0.43	0.17
Average running speed (mph)	60.6	62.2	35.0	35.0
Hours/1,000 vehicle-miles	16.5	16.1	28.6	28.6
Annual hours/mile due to running at average speed (in one direction)	1,542	45,429	14,425	69,004
Running cost factor, for uninterrupted flow at average speed ($/1,000 veh-mi)	50.14	50.26	49.96	46.96
Annual running cost for running at average speed (in one direction) ($/mi)	4,685	142,063	23,706	113,403
Average delay/vehicle/signal (sec)	—	—	5.96	4.88
Vehicles stopped at signal (%)	—	—	43	34
Annual time delay at signals (in one direction) (hr/mi)	—	—	1,672	6,548
Annual idling time at signals (in one direction) (hr/mi)	—	—	63 [b]	484 [b]
Annual cost of stopping at signals (in one direction) ($/mi)	—	—	5,031	18,960
Annual idling cost (in one direction) ($/mi)	—	—	11	89
Total annual running cost (both directions) ($/mi)	293,496		322,400	
Total annual time cost (both directions) ($/mi)	281,826		549,894	
Total annual user cost ($/mi)	575,322		872,294	

[a] Two signals per mile with 70 percent green/cycle time ratio, 90-sec cycle times.
[b] This is included in annual time delay at signals.
N.A. = Data not available.

The data in Tables 14 and 15 are plotted in Figure 32, on a logarithmic scale in both directions. The AADT limits used are shown below the curves. The closeness of the different facility types in running costs is striking, as is the close approximation to a straight line for both annual running costs and total annual costs. The virtual linearity of the results means that the economy of passenger-car travel on level tangents could be well represented by constant unit costs. For example, all highways with 70-mph design speeds have running costs of about $0.05 per vehicle-mile and total costs of about $0.11 to $0.12 per vehicle-mile, over the daily traffic volumes considered.

A scale at the bottom of Figure 32 shows the level of service range implied by the peak period hourly volume for nonarterial facilities. (Peak-hour factors of 0.77, 0.83, 0.91, and 0.91, respectively, were used for four, six, eight, and ten-lane freeways in deriving this scale.) Note that, for two-lane highways, capacity (the limit of service level E) is reached before the assumed AADT limit. Also, for multilane highways, the AADT limit used for the facility is reached before even the limit of service level D. This result could be avoided by assuming a higher AADT limit, but this assumption rarely would be reached because the congestion and traffic impediments (such as signalized intersections) encountered at such volumes would have long since converted the road to an arterial or caused its replacement or supplementation by a higher-type facility.

It is tempting to think of Figure 32 (or Tables 14 and 15) as a basic type of result that can then easily be modified by using the ratios of truck-to-passenger-car costs, curve cost ratios, and gradient factor ratios to find the resulting annual costs for different traffic mixes and highway geometrics. However, truck costs are not a constant ratio to automobile costs under different conditions, and the unit cost curves for gradients and curves have to be entered with average speed, which is different for peak and off-peak periods. Therefore, the problem would soon break down into returning to the worksheets anyway. The only shortcut that may be feasible is to assemble, as is done in Table 13, the essential ingredients of a given type of problem, leaving out those that are irrelevant to the highway and traffic conditions under study. Computer programs are of great potential assistance in reducing computation time; their development is recommended in Chapter Eight.

TABLE 14

ANNUAL RUNNING COST PER MILE AS A FUNCTION OF AADT, FOR SELECTED HIGHWAY FACILITIES ($1,000)

	Freeways				Multi-	2-Lane	4-Lane Arterials With Signal Density of			
AADT	10-Lane	8-Lane	6-Lane	4-Lane	Lane	Rural	0.5/MI.	1/MI.	2/MI.	4/MI.
1,000	18.4	18.4	18.4	18.4	18.4	18.2	17.8	18.4	19.7	22.3
2,000	36.7	36.7	36.7	36.7	36.7	36.2	35.6	36.9	39.5	44.8
4,000	73.4	73.4	73.4	73.4	73.3	71.7	71.2	73.9	79.2	89.9
6,000	110.0	110.1	110.1	110.0	109.7	107.0	106.9	111.0	119.2	135.5
8,000	146.8	146.8	146.7	146.6	146.1	142.5	142.7	148.2	159.3	181.5
10,000	183.5	183.5	183.5	183.1	182.5	178.3	178.5	185.5	199.7	227.9
12,000	220.2	220.2	219.9	219.5	218.7	214.4	2.413	223.0	240.3	274.9
14,000	256.9	256.8	256.5	255.9	254.9	251.4	250.8	260.6	281.2	322.4
16,000	293.5	293.5	293.0	292.3	290.9	288.0	286.3	298.3	322.4	370.6
18,000	330.2	330.1	329.5	328.6	327.0	324.7	322.4	336.2	364.0	419.4
20,000	366.9	366.7	366.0	364.8	362.9		358.6	374.3	405.9	469.0
25,000	458.4	458.2	457.1	455.2	452.7		449.5	470.6	512.7	597.0
30,000	549.9	549.6	548.0	545.2	542.3		541.4	568.7	623.3	732.4
40,000	732.7	732.2	729.5	724.7	721.8					
50,000	915.2	914.4	910.3	903.7	902.7					
60,000	1097.5	1096.4	1090.9	1082.5	1086.3					
70,000	1279.4	1277.9	1271.2	1261.8						
80,000	1461.2	1459.4	1451.4	1444.4						
90,000	1642.8	1640.6	1631.5	1627.1						
100,000	1824.2	1821.8	1811.8	1810.8						
125,000	2277.3	2273.6	2262.8							
150,000	2729.0	2725.4	2716.8							
175,000	3180.8	3177.5								
200,000	3632.6	3631.2								
225,000	4085.0									
250,000	4539.0									

TABLE 15

ANNUAL RUNNING COST PLUS VALUE OF TRAVEL TIME * AS A FUNCTION OF AADT, FOR SELECTED HIGHWAY FACILITIES ($1,000)

AADT	* * * * FREEWAYS * * * * 10 LANE	8 LANE	6 LANE	4 LANE	MULTI-LANE	2 LANE RURAL	* 4 LANE ARTERIALS, WITH SIGNAL DENSITY OF: * 0.5/MI.	1/MI.	2/MI.	4/MI.
1000	35.8	35.8	35.8	35.8	35.8	36.1	49.7	51.0	53.5	58.6
2000	71.5	71.5	71.6	71.6	71.7	72.9	99.4	102.0	107.2	117.5
4000	143.1	143.2	143.3	143.5	143.8	149.0	199.0	204.2	214.8	235.8
6000	214.8	214.9	215.2	215.6	216.3	229.1	298.6	306.7	322.9	355.2
8000	286.6	286.8	287.2	288.0	289.3	314.2	398.4	409.4	431.5	475.5
10000	358.5	358.7	359.4	360.8	362.7	404.9	498.4	512.5	540.7	597.1
12000	430.4	430.8	431.8	433.8	436.6	504.0	598.4	615.8	650.5	719.9
14000	502.4	503.0	504.4	507.2	511.1	616.0	698.7	719.5	761.0	844.1
16000	574.5	575.3	577.1	581.1	586.1	717.7	799.2	823.6	872.3	969.8
18000	646.8	647.7	650.1	655.3	661.6	823.6	899.8	928.0	984.5	1097.3
20000	719.1	720.2	723.2	729.9	737.8		1000.8	1033.0	1097.5	1226.6
25000	900.3	902.1	907.0	918.5	931.1		1254.3	1297.9	1385.2	1559.8
30000	1082.1	1084.8	1092.2	1110.5	1129.0		1510.1	1567.4	1682.1	1911.4
40000	1447.7	1452.8	1467.1	1505.2	1541.4					
50000	1816.0	1824.8	1848.6	1916.4	1981.1					
60000	2187.6	2201.0	2237.5	2345.6	2456.6					
70000	2562.3	2581.5	2634.6	2816.8						
80000	2940.7	2967.2	3039.7	3373.9						
90000	3322.9	3358.1	3453.5	3890.7						
100000	3709.0	3754.3	3878.5	4410.9						
125000	4692.8	4773.0	5009.7							
150000	5708.3	5835.2	6203.3							
175000	6754.2	6956.5								
200000	7847.1	8104.7								
225000	8981.7									
250000	10130.9									

* At $3.00 per hour.

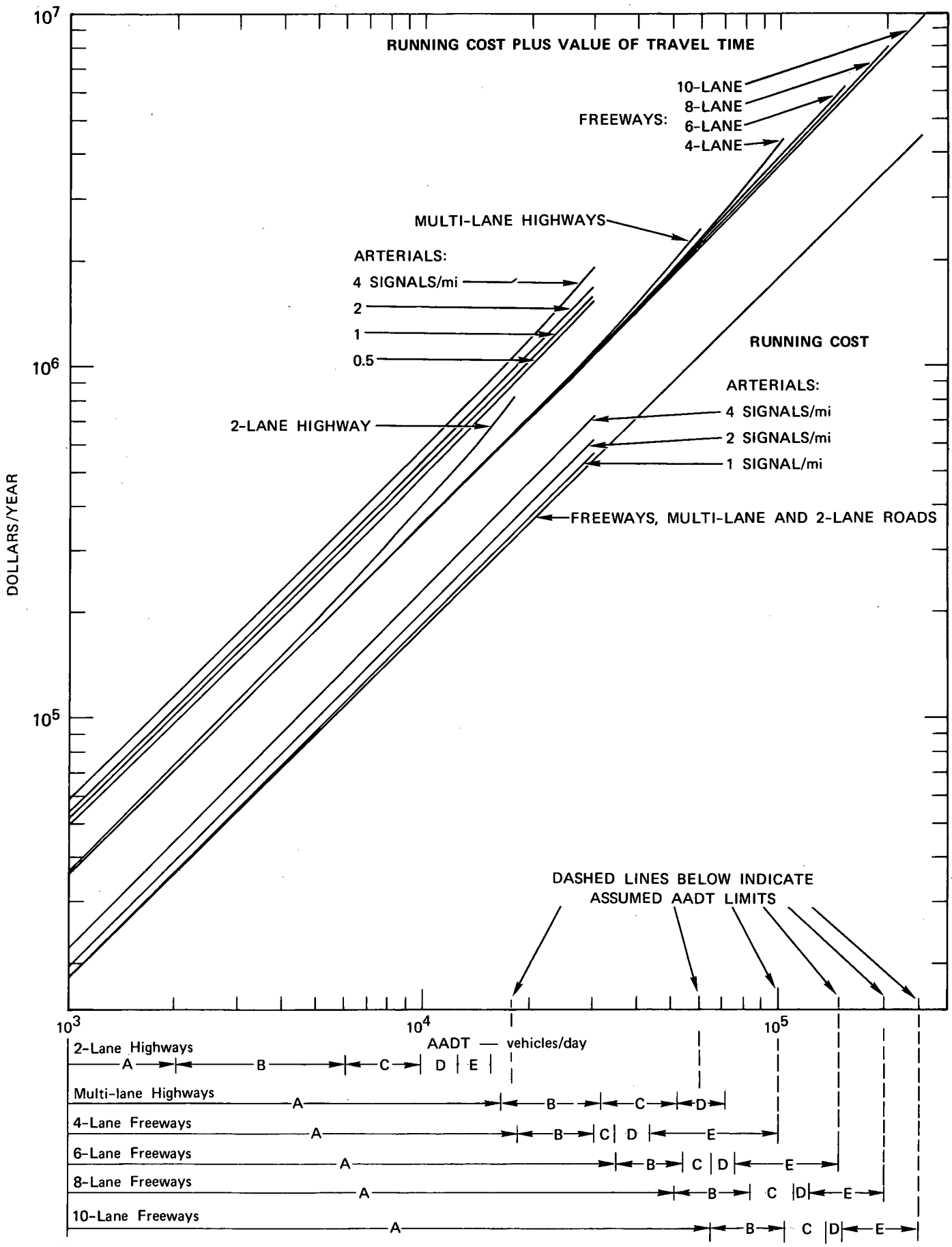


Figure 32. Running costs and value of travel time for passenger cars vs AADT.

CHAPTER EIGHT

SUGGESTIONS FOR FURTHER RESEARCH

The two principal suggestions for further research resulting from this project concern (1) the need to facilitate rapid solution of highway economy study problems through development of computer procedures based on this report, and (2) the need to apply, test, and improve the procedures through use in state and local highway agencies.

Other research needs concern refinement, simplification, or extension of procedures or cost factors that already appear in the report. Complete research recommendations follow, in the form of brief task statements followed by explanatory comments.

1. Preparation of a computer program to carry out the calculations embodied in the report.

Comment: The prospect of computerizing this report is an urgent priority in view of both time savings in project analysis and the possibility of developing further refinements in the manual procedure. Development of such a program is relatively simple, because many of the needed inputs and subroutines already have been prepared in connection with this study. A further possibility and refinement would be to combine such a program with traffic simulation and capacity programs such as those described by May et al. (*31*).

2. Collaboration with state or local highway agencies in using the report for economic, air pollution, and noise analyses, possibly including the publication of results from such applications.

Comment: Although several state highway agencies reviewed a draft of this report during its preparation, actual use is the best way to test and refine its procedures. Moreover, it could accelerate the application of economic analysis at state levels if means were provided for technical assistance to state highway agencies in the use of this report, especially if a computer program were available as an adjunct to it.

3. Development of guidelines, similar to Figure 7, for estimating the number of peak hours per day associated with different levels of ADT per lane.

Comment: The length of the peak period is not usually associated with traffic levels in the 30th or 100th hours in published highway use statistics seen by the researchers. A further search for such correlated data should be made, supplemented by field data if necessary, to develop guidelines for estimating peak period length. This is a key variable in this report, and it must presently be estimated largely by guesswork.

4. Research on improved queuing methodologies.

Comment: The researchers suggest as a research project the derivation of a single method that predicts delay for both nonqueuing and queuing situations. This method would have the advantage of predicting continuously increasing demands.

One approach to queuing analysis considered for this report was Newell's method because its use probably would have allowed a smooth transition from nonqueuing to queuing. However, the coefficient of variability, *I*, must be computed in Newell's method, and there is as yet no efficient algorithm for doing this (see *NCHRP Report 113* for details of Newell's method). Other disadvantages of Newell's method are that (1) the graphical solution requires more explanation to users than the graph that was derived from Webster's equation (Fig. 16); and (2) even after the user completes a graphical solution, other computations would be necessary to arrive at the final delay. If these problems were solved, however, approach probably would be superior to Webster's where demand levels are approaching saturation.

5. Corroboration and extension of tire wear costs, especially in view of radial ply and similar new tire types on the market with mileage guarantees, and especially for trucks and for higher speeds (more than 40 mph) using a variety of automobile aerodynamic shapes; and updating of fuel consumption data for new engine designs that meet air and noise pollution standards.

Comment: The problems with available data on tire wear costs are described in Appendix B. Especially for running cost factors in which tire wear is a significant proportion of total costs (curves, speed changes), improved tire wear cost factors would be valuable. Fuel consumption rates also are likely to vary from rates developed in the 1960's, over time.

6. Road tests to obtain field data (and related average travel times and running costs) for:

a. The speed profiles of trucks in comparison with passenger cars under the same or equivalent traffic conditions.
b. Speed profiles under a variety of highway conditions (especially queuing), vehicle classes, and times of day, week, and year.

Comment: (1) Field data on truck speed profiles are rarely if ever obtained in comparison with similar *and simultaneous* data for automobiles on the same facility. Such information would be valuable to compare with the assumptions made regarding truck speed changes for this report (see Appendix B). (2) The researchers' speed change costs, especially for queuing, were based on relatively sparse data, so more information on the subject would be helpful.

7. Updating of air and noise pollution estimating procedures as the results of current research become available, and addition of an air pollution diffusion model when feasible.

Comment: The accuracy and reliability of procedures for estimating air and noise pollution effects are limited

by present sparse research results in these areas. The estimating procedures in the report should be reviewed as results of current research become available, particularly development of an air pollution diffusion model and more widely acceptable ways of incorporating peak noise occurrences into noise criteria.

8. Improvements in accident cost and frequency information as functions of road geometrics, speeds, hourly and daily traffic density, etc.

Comment: A study to develop the following refinements in accident reporting practices would contribute to more reliable accident cost estimates:

a. More detailed descriptions and classification of the type of road by which accident statistics are gathered and summarized would be desirable. At present, gross classification categories tend to be used, such as "two-lane urban state highways," but such categories may differ widely over different sections with regard to curvature, traffic, roadside development, median barriers, frequency of intersections and stop signals, and other accident-affecting features.
b. The identification of unreported accidents by type of road in future accident studies would enable improvement of guidelines for estimating unreported accidents. Past state studies have adequately demonstrated the importance of unreported accidents in the total accident cost picture, but their failure to furnish data on unreported accidents by type of road handicaps the estimation of this number from statistics on reported accidents. It also would be desirable to be able to relate the degree of underreporting to state minimum reporting laws, police reporting policies, or other causative factors.

In addition, it would be desirable to develop sound economic criteria for inclusion of costs for the loss of future earnings for fatalities and total disability cases. Two possible approaches are to calculate either (1) average net loss of output of persons killed or permanently injured in highway accidents (output less consumption); or (2) average liability payments plus insurance overhead costs for accidental deaths on public transportation systems, or under workers' compensation insurance plans. Attempts to carry out the first of these approaches have been made, but as yet not very successfully.

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APPENDIX A

CHARTS OF AVERAGE SPEED AND RUNNING COST FACTORS

The charts in this appendix provide average speed as a function of the volume/capacity ratio and running costs versus the v/c ratio or speed for the three types of vehicles considered in this report: passenger cars of about 4 kips (4,000 lb), 12-kip single-unit trucks, and 50-kip 3-S2 combination tractor semi-trailer trucks. The following charts are included: *

A-1—A-3	Average Speed and Running Costs vs v/c Ratio for Freeways.
A-4—A-6	Average Speed and Running Cost vs v/c Ratio for Multilane Rural Highways.
A-7—A-9	Average Speed and Running Cost vs v/c Ratio for 70-mph, 2-Lane Rural Highways.
A-10—A-12	Average Speed and Running Cost vs v/c Ratio for 50-mph, 2-Lane Rural Highways.
A-13—A-15	Running Cost at Uniform Speed on Level Tangents vs Speed.
A-16—A-18	Excess Running Cost due to Grades vs Speed.
A-19—A-21	Excess Running Cost of Speed Changes Above Level Tangent Costs vs Initial Speed.
A-22—A-24	Excess Running Cost due to Curves vs Speed.

The use of these curves is discussed in connection with the worksheets for which they are employed to obtain speed or cost factors. For Figures A-1 through A-18, this is Worksheet 3, items 8.1 and 9. For the remaining figures, it is Worksheet 4, columns 7 and 8.

The components of running costs in these curves are fuel (less fuel taxes), tires, engine oil, maintenance, and that portion of depreciation that varies with mileage driven. Derivation of the curves was as follows:

1. The average speed curves (upper part of Figs. A-1 through A-12) are translations into average speed of the HCM curves of operating speed as a function of the v/c ratio, by highway type.

2. The running cost curves on level tangents (lower part of Figs. A-1 through A-12) are the sum of two different types of costs incurred in the normal operation of vehicles under different traffic conditions: one is the basic cost of running the vehicle on a level tangent road at a uniform speed, itself a function of traffic conditions measured by the v/c ratio. The second type of cost is due to the speed changes that occur in normal operations. The average frequency and magnitude of these changes were developed by the researchers from vehicle speed profiles as a function of traffic conditions as measured by the v/c ratio.

3. The other running cost curves (uniform speed on level tangents, the added cost of speed change cycles, and the added costs of grades and curves) are based on combinations of the components of running cost factors of Winfrey (*4*) and Miller (*11*), updated when necessary to 1970 price levels.

Appendix B provides further details on derivation and updating of these time and cost relationships.

The shape of the speed and cost curves is a consequence of the foregoing derivation procedures, and might not always be intuitively obvious. For example, the curves for passenger-car costs on level tangents as a function of the v/c ratio are notably flat, especially compared with the more familiar curve of such costs as a function of uniform speed (Fig. A-13). This is due both to the relatively narrow speed range represented up to a v/c ratio of 1.0 and to the compensating effect of increasing speed change costs as constant speed cost decreases with increasing traffic. The cost factors for trucks tend to go down with increasing v/c ratios, chiefly as a result of the assumed lower magnitude of speed change cycles for trucks with increases in traffic compared with their magnitude for passenger cars. Finally, the sharp changes in many curves as the v/c ratio approaches 1.0 result from corresponding changes in speed at those points in the HCM curves on which Figures A-1 through A-12 are based.

Most of the curves are plotted by a computer from data points or regression equations; hence, they lack the smoothness of hand-drawn curves. Minor irregularities in the curves should be ignored in reading off speed and cost values.

* In addition to these charts, average speed and running costs for level of service F are represented by Figures 9 and 13.

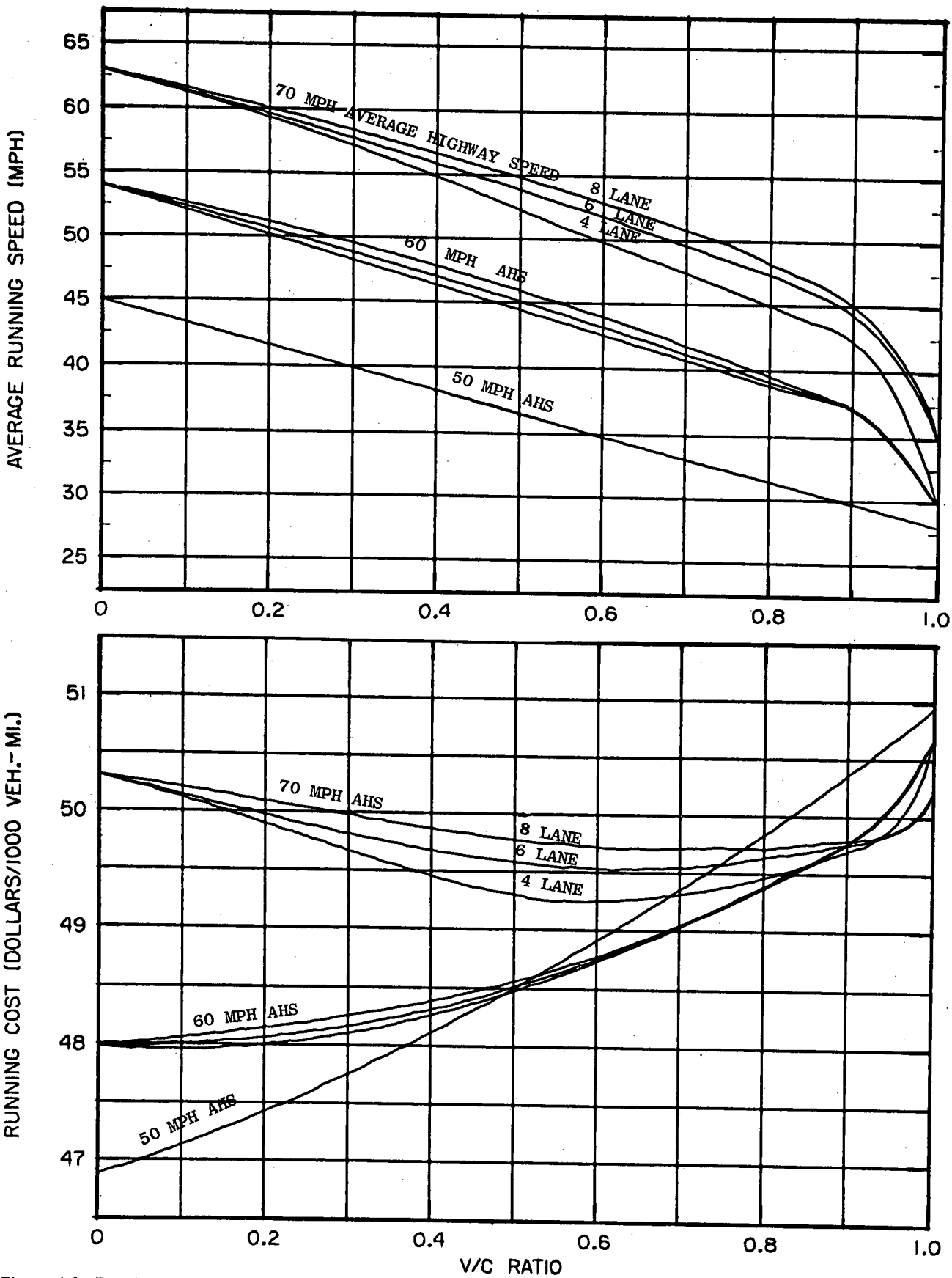


Figure A-1. Running speed and cost: Freeways—passenger cars.

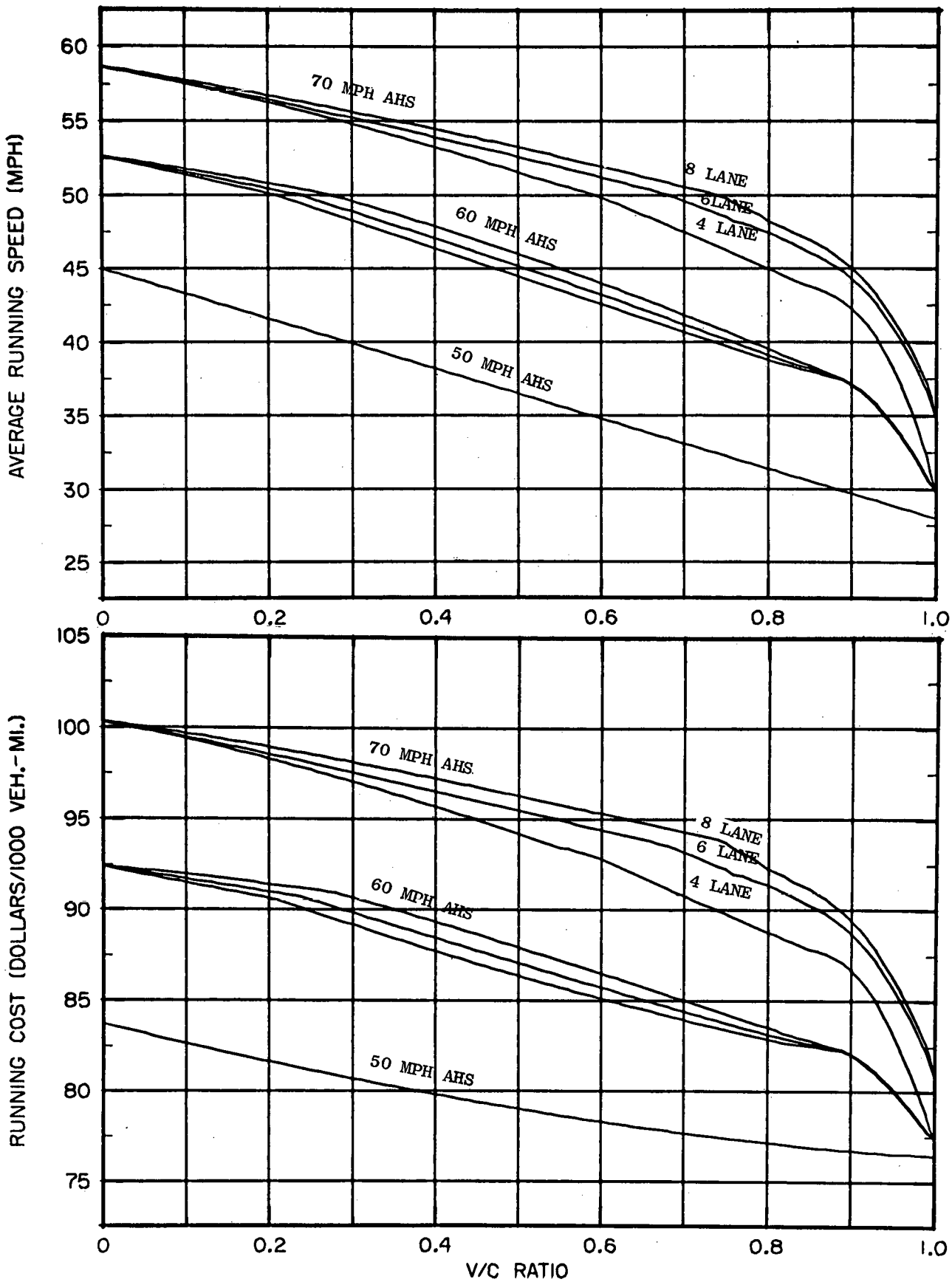


Figure A-2. Running speed and cost: Freeways—single-unit trucks.

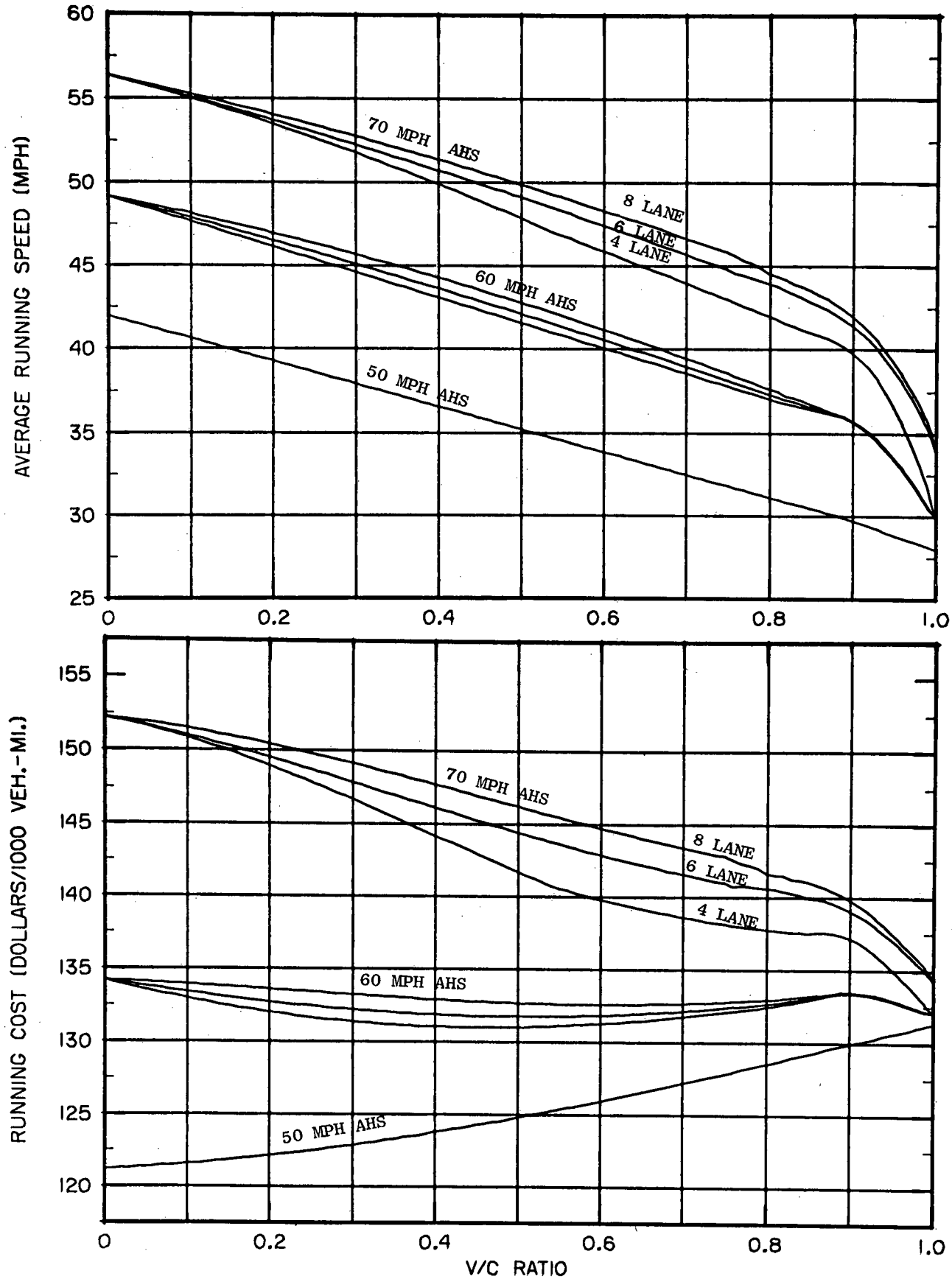


Figure A-3. Running speed and cost: Freeways—3-S2 trucks.

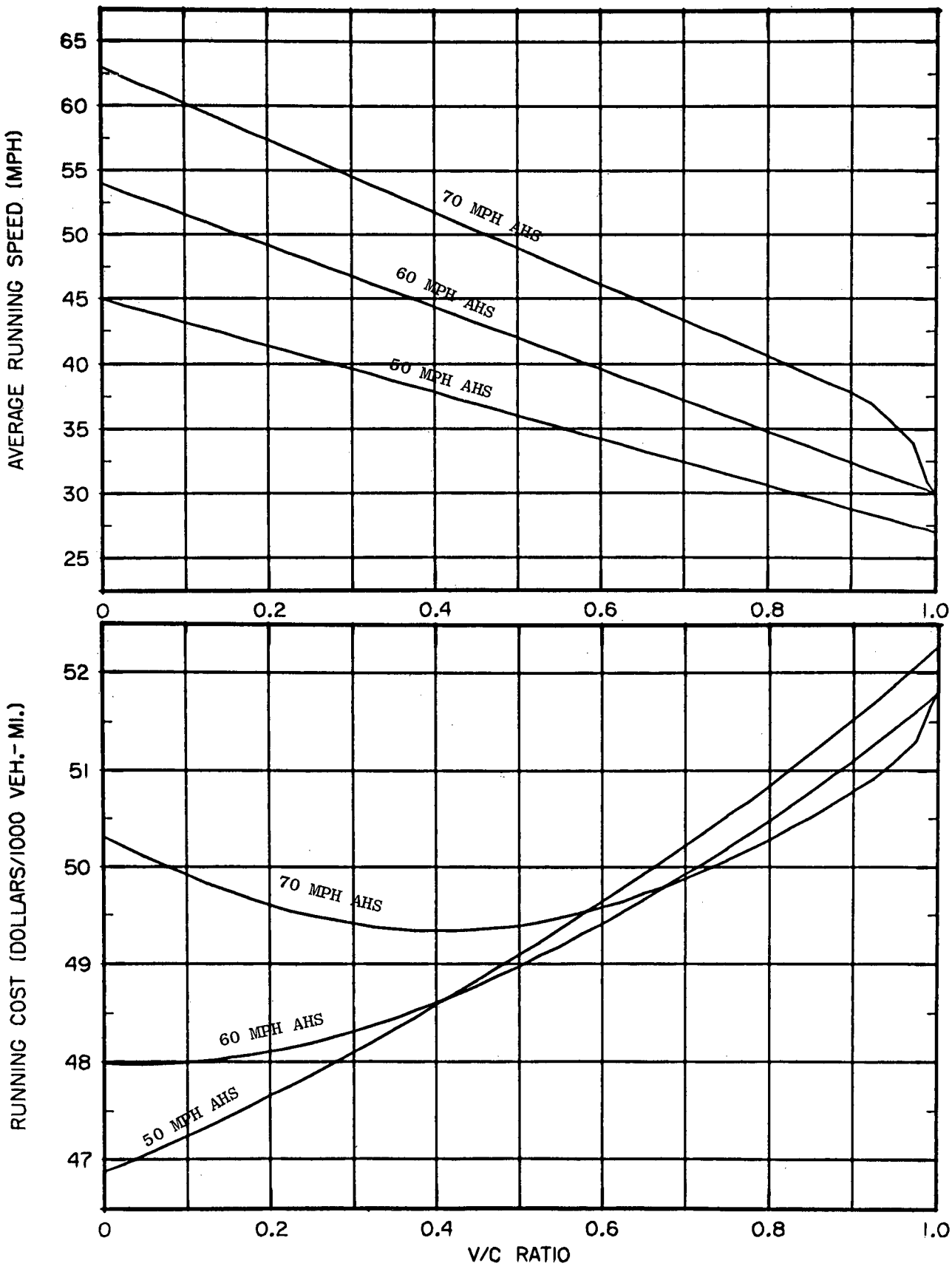


Figure A-4. Running speed and cost: Multilane rural highways—passenger cars.

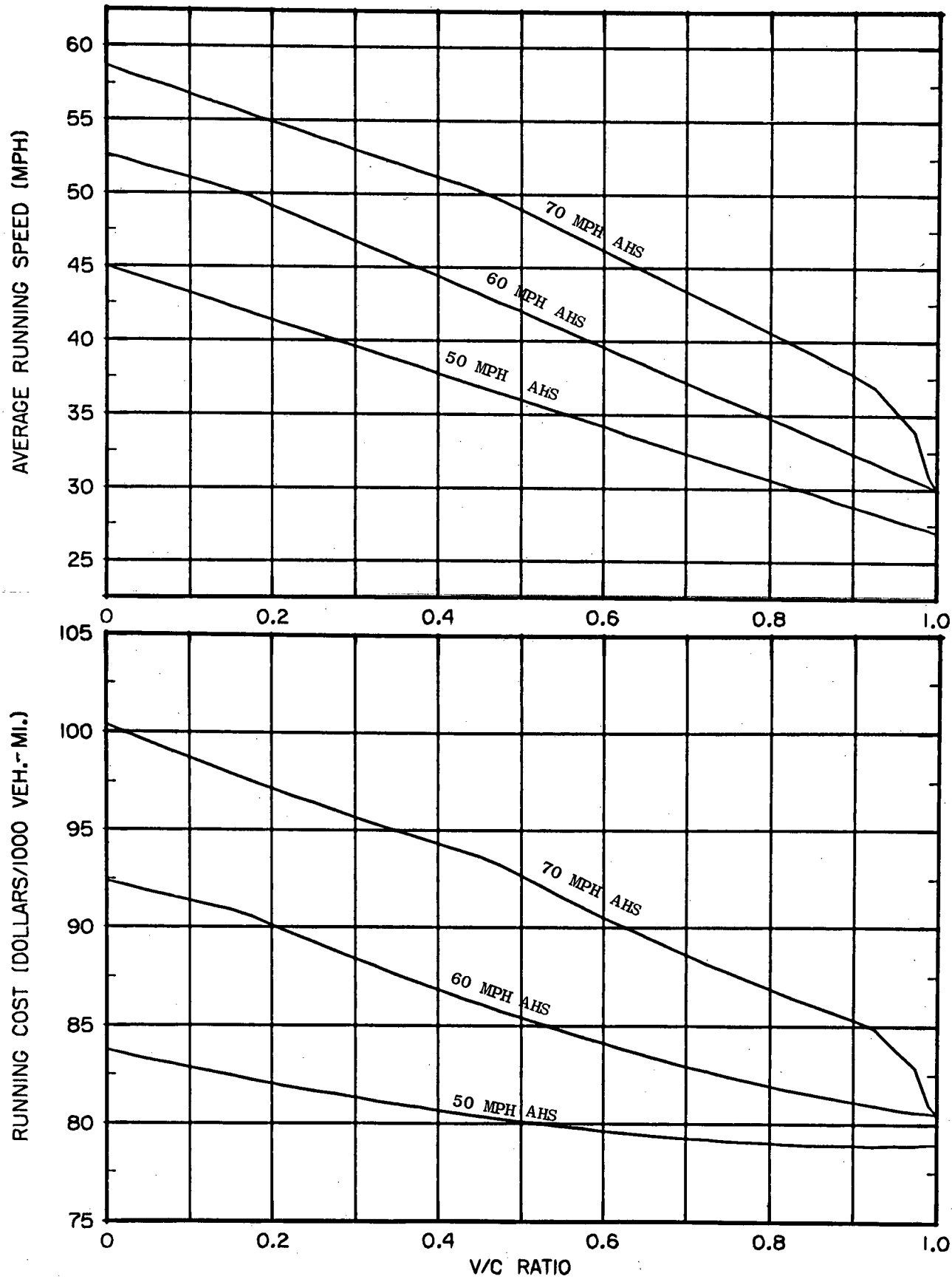


Figure A-5. Running speed and cost: Multilane rural highways—single-unit trucks.

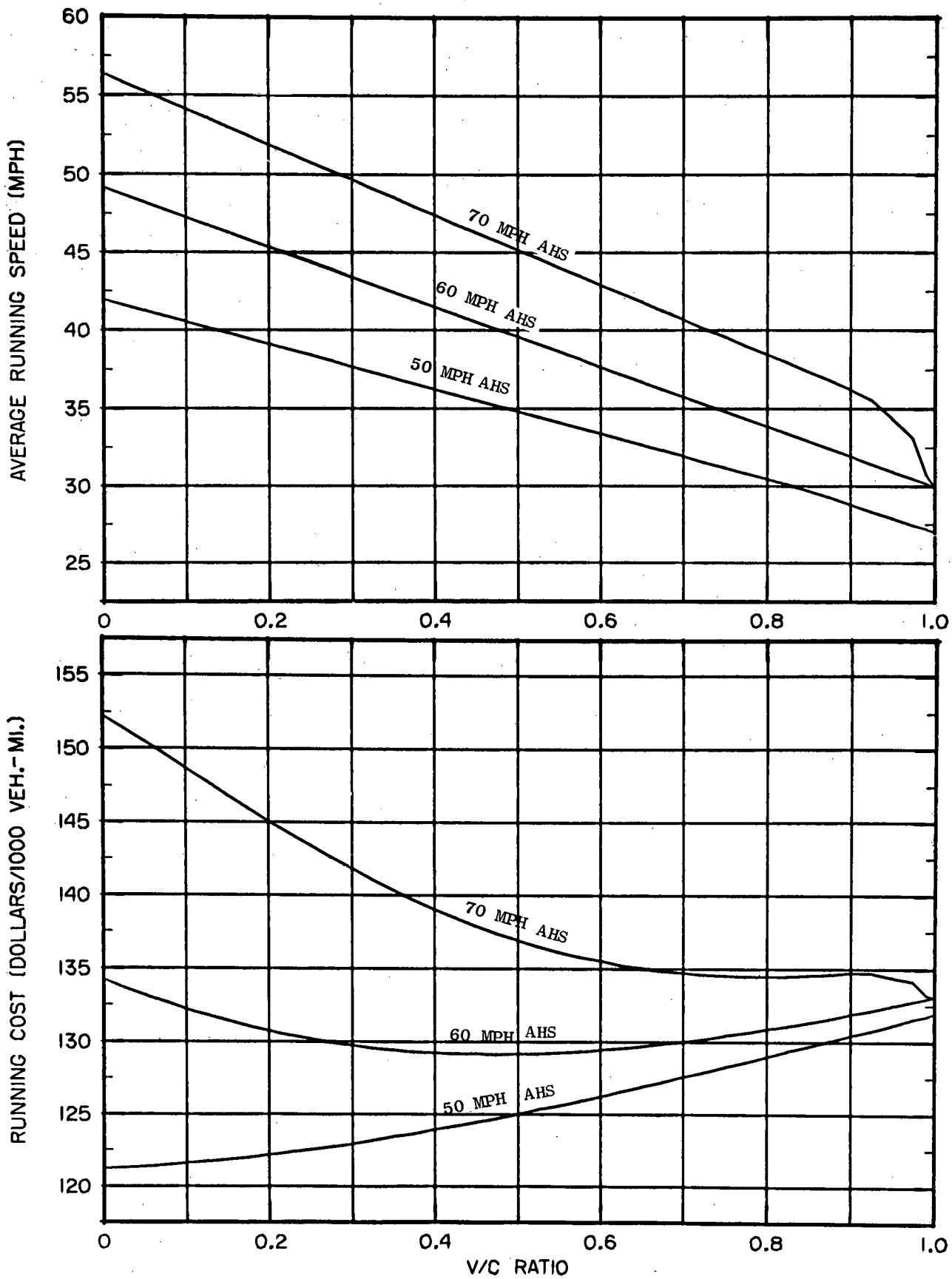


Figure A-6. Running speed and cost: Multilane rural highways—3-S2 trucks.

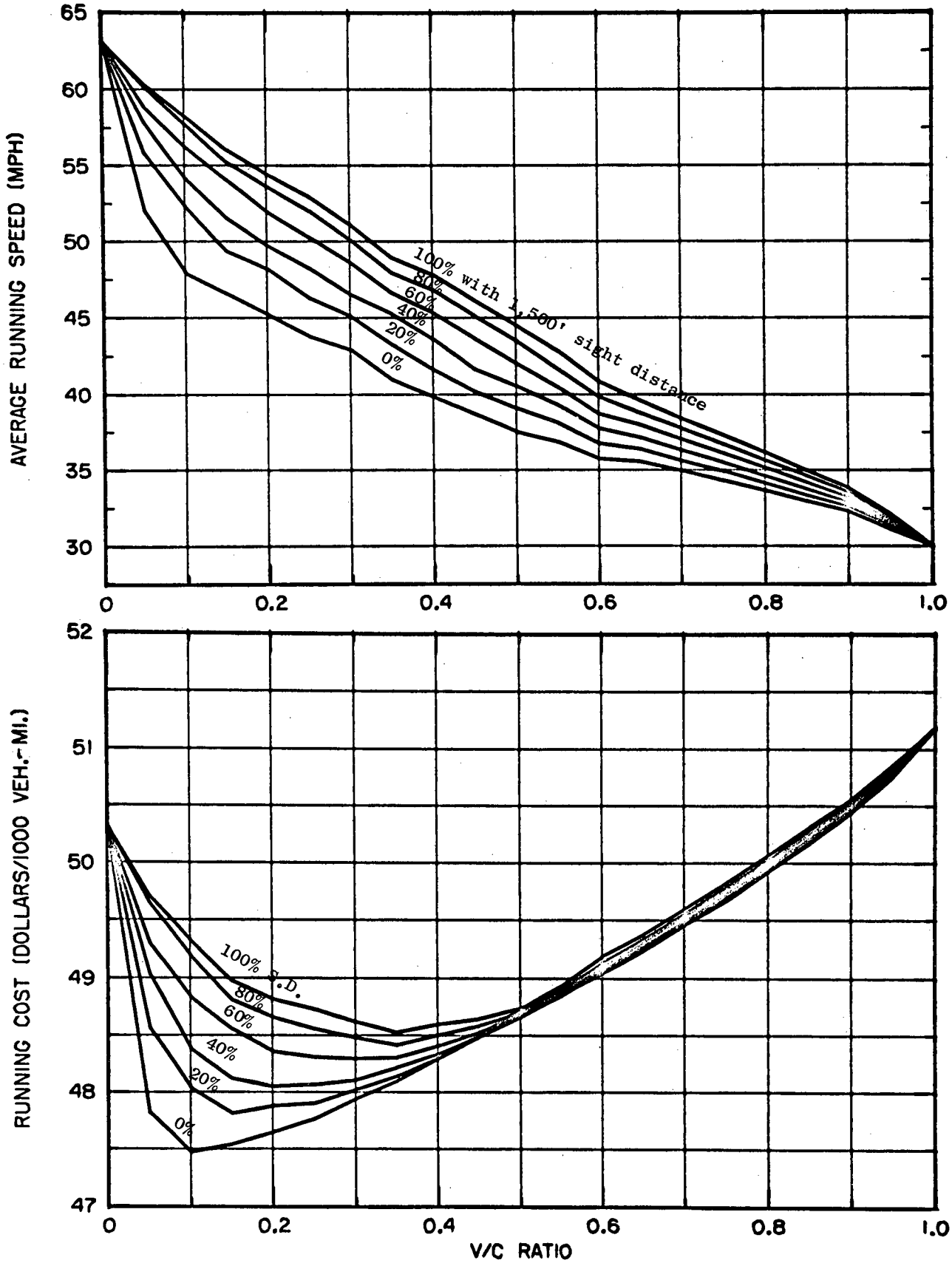


Figure A-7. Running speed and cost: 70-mph 2-lane rural highways—passenger cars.

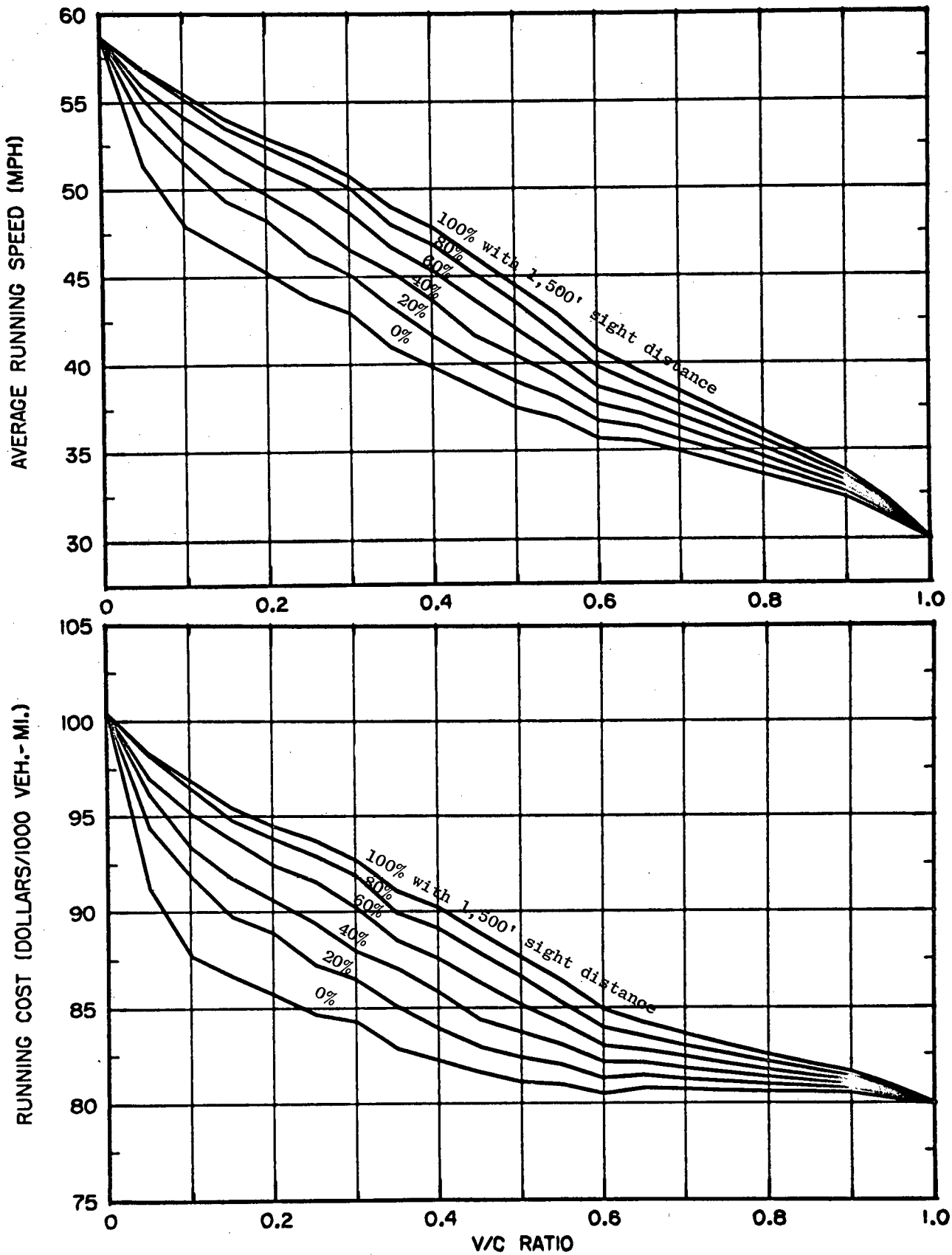


Figure A-8. Running speed and cost: 70-mph 2-lane rural highways—single-unit trucks.

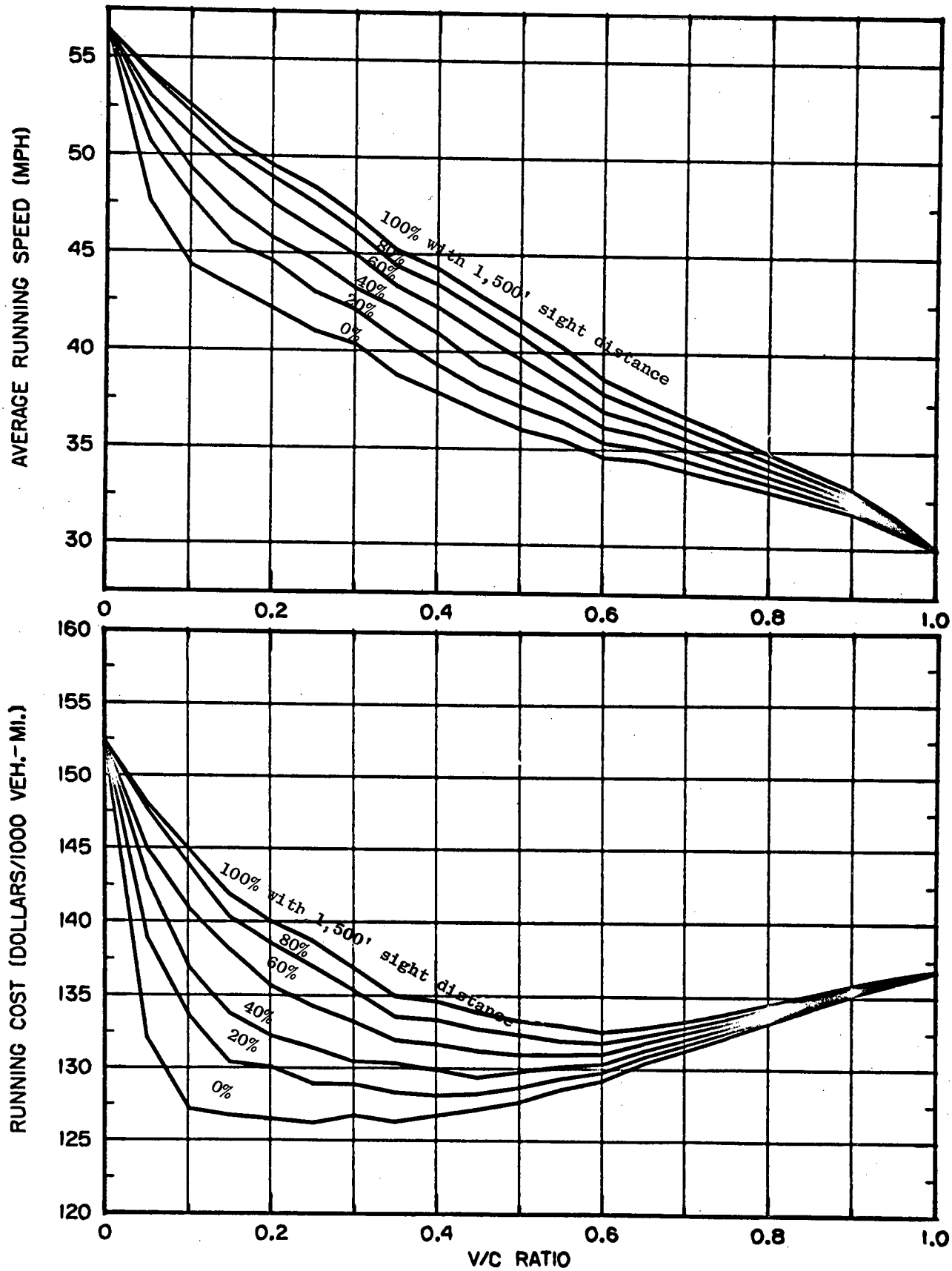


Figure A-9. Running speed and cost: 70-mph 2-lane rural highways—3-S2 trucks.

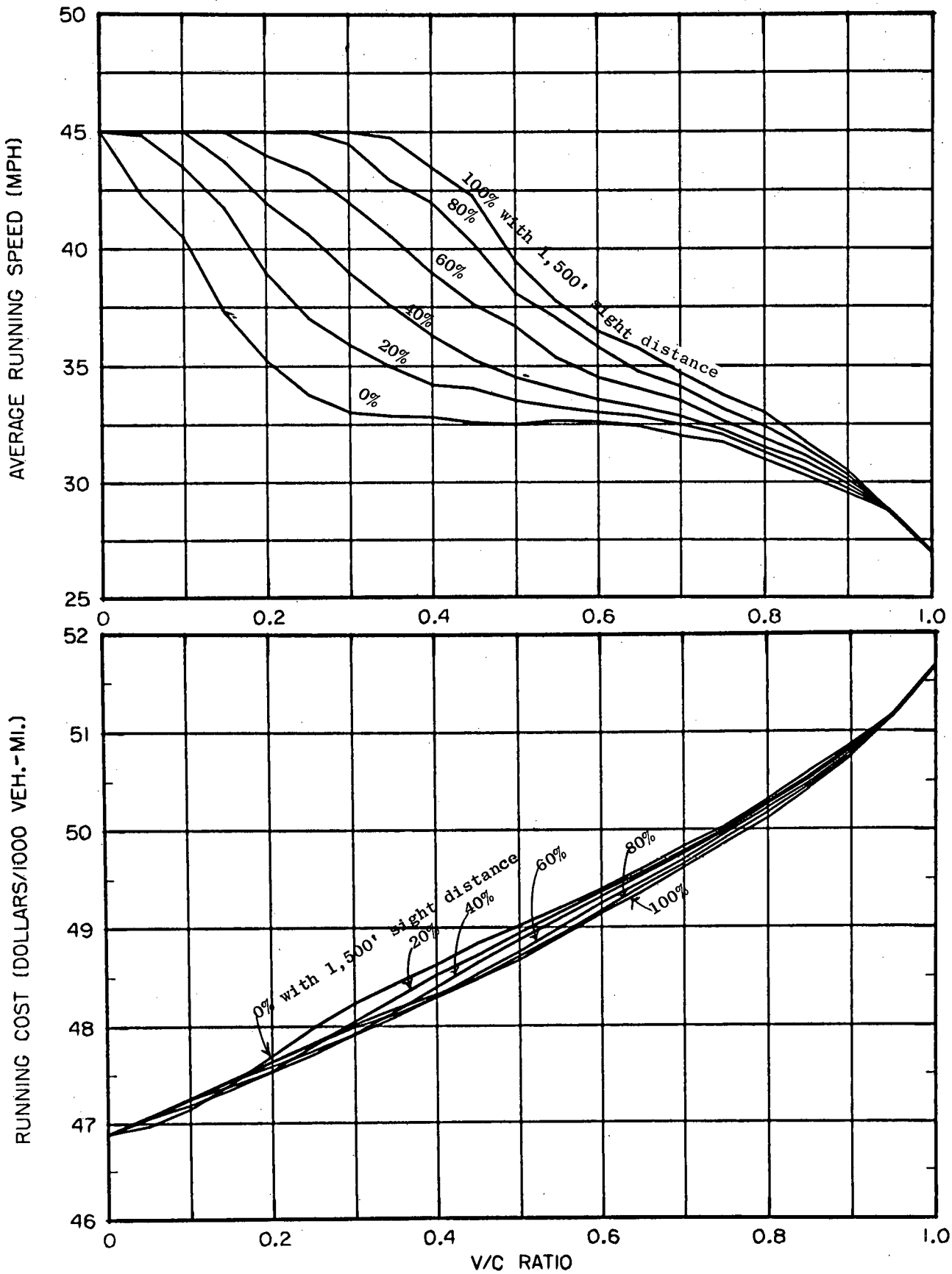


Figure A-10. Running speed and cost: 50-mph 2-lane rural highways—passenger cars.

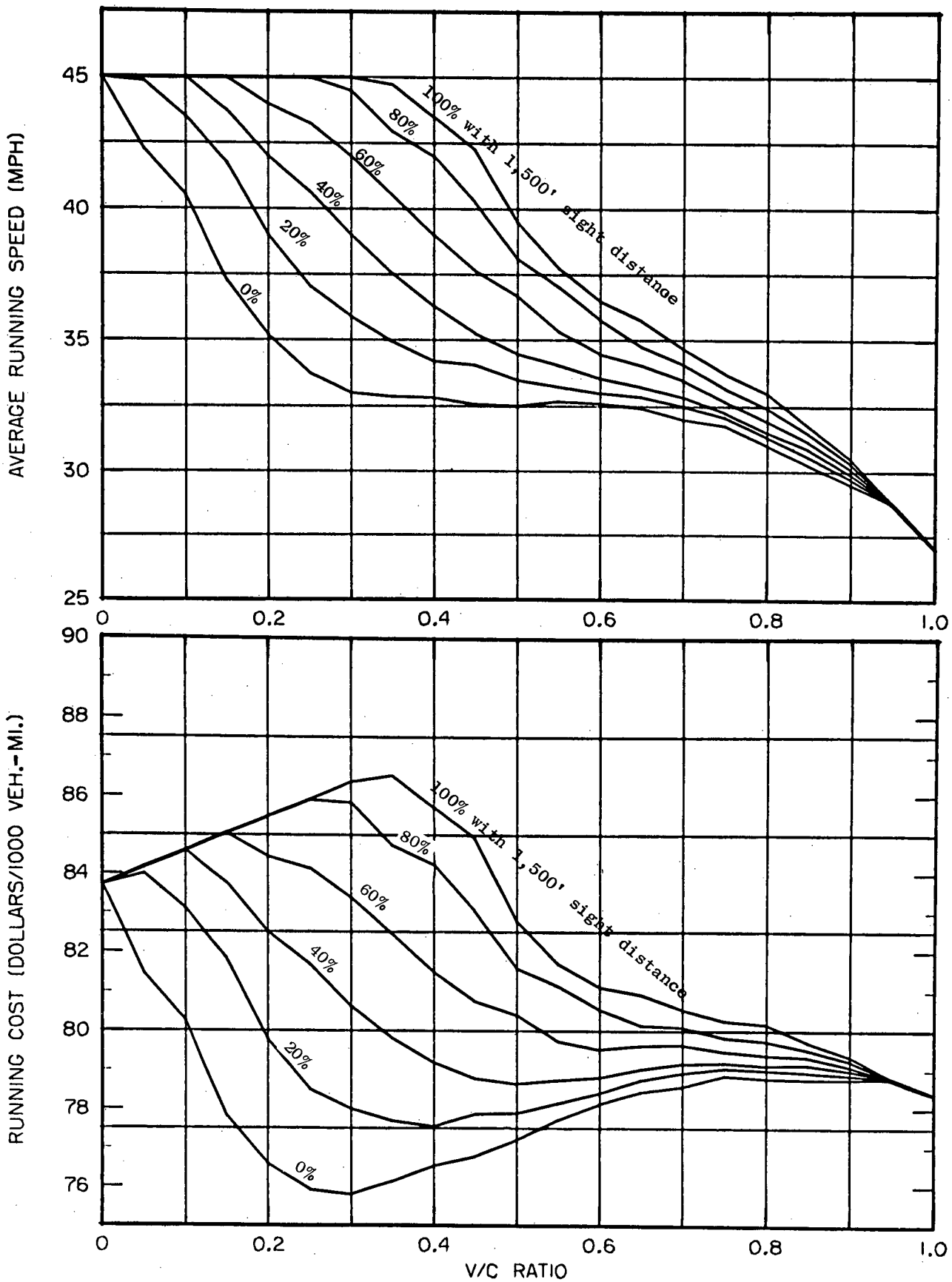


Figure A-11. Running speed and cost: 50-mph 2-lane rural highways—single-unit trucks.

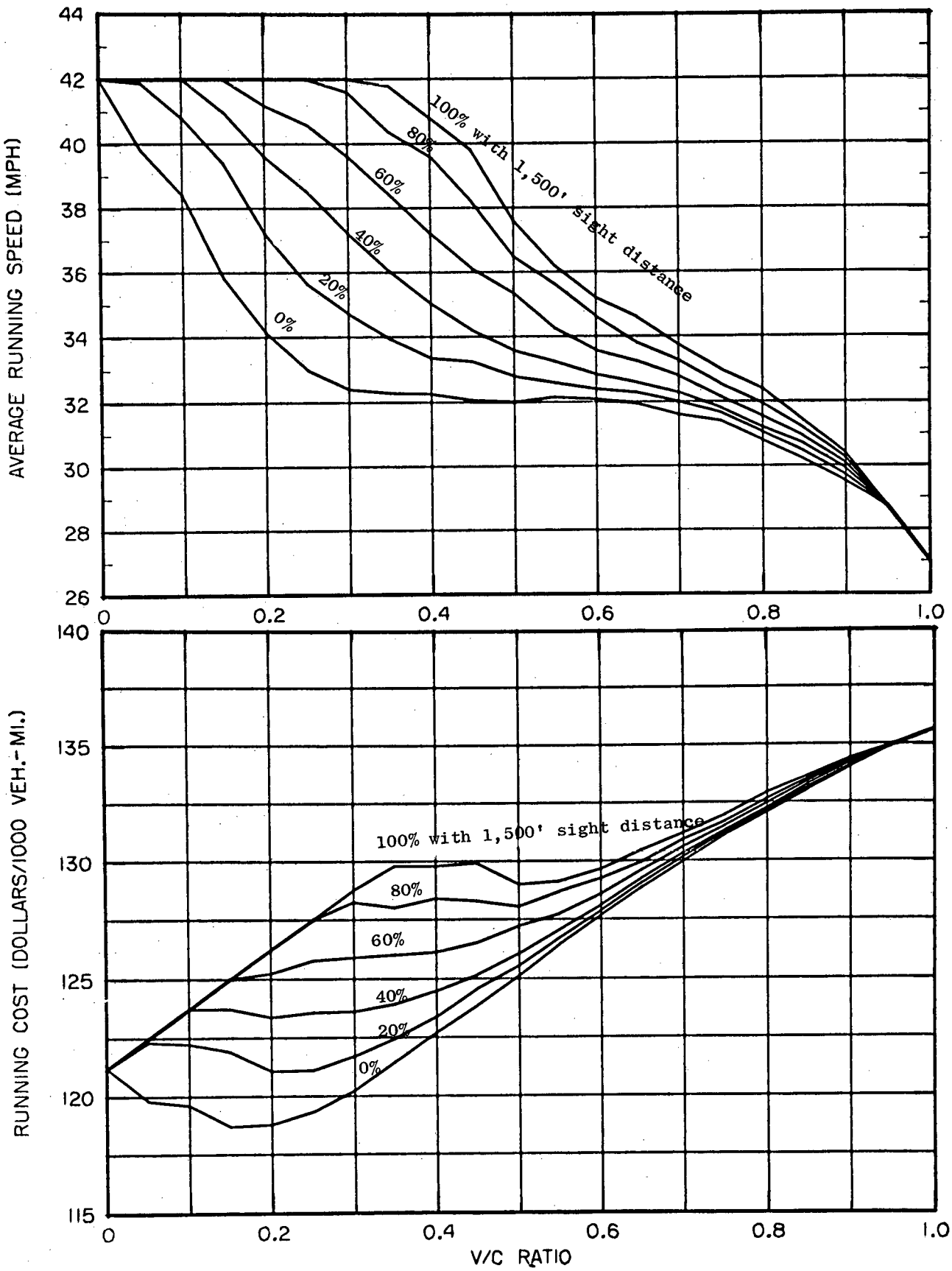


Figure A-12. Running speed and cost: 50-mph 2-lane rural highways—3-S2 trucks.

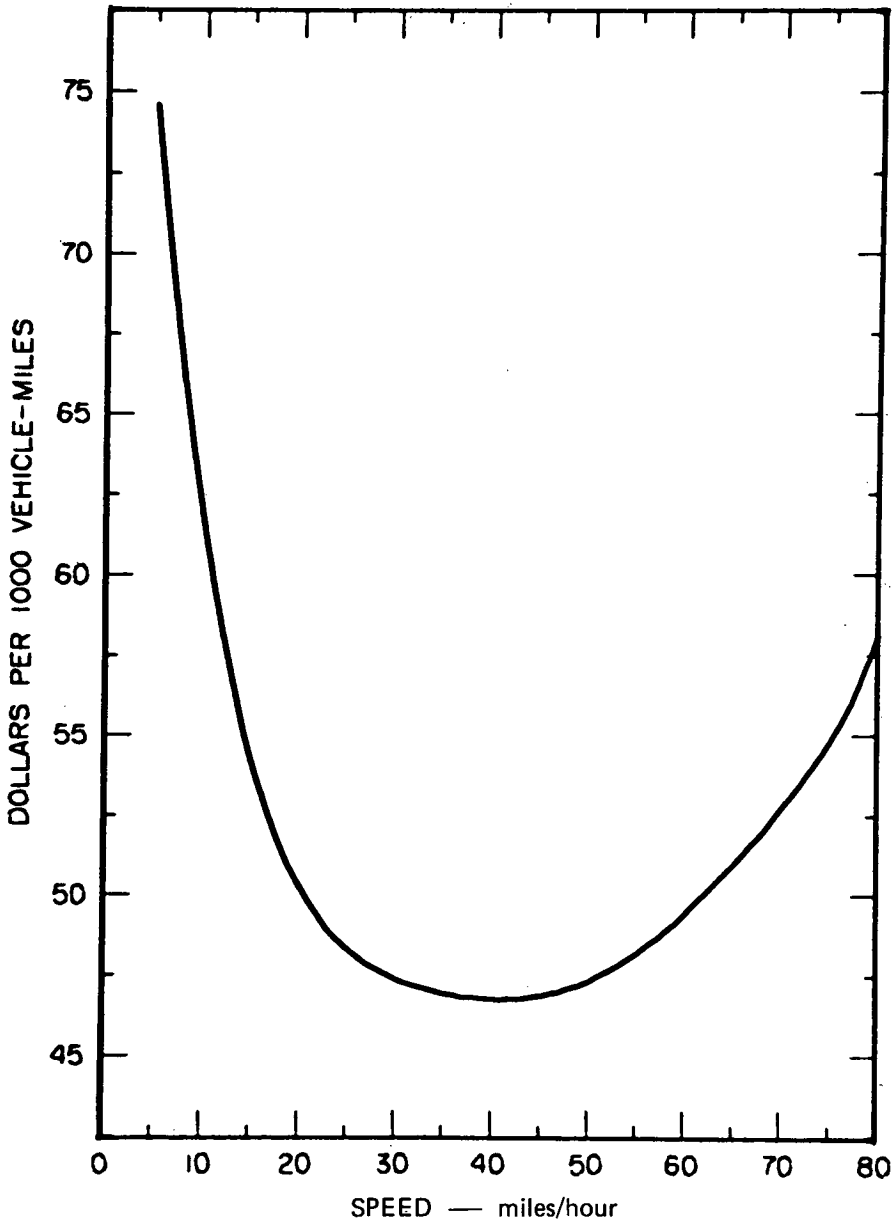


Figure A-13. Running cost at uniform speed on level tangent—passenger cars.

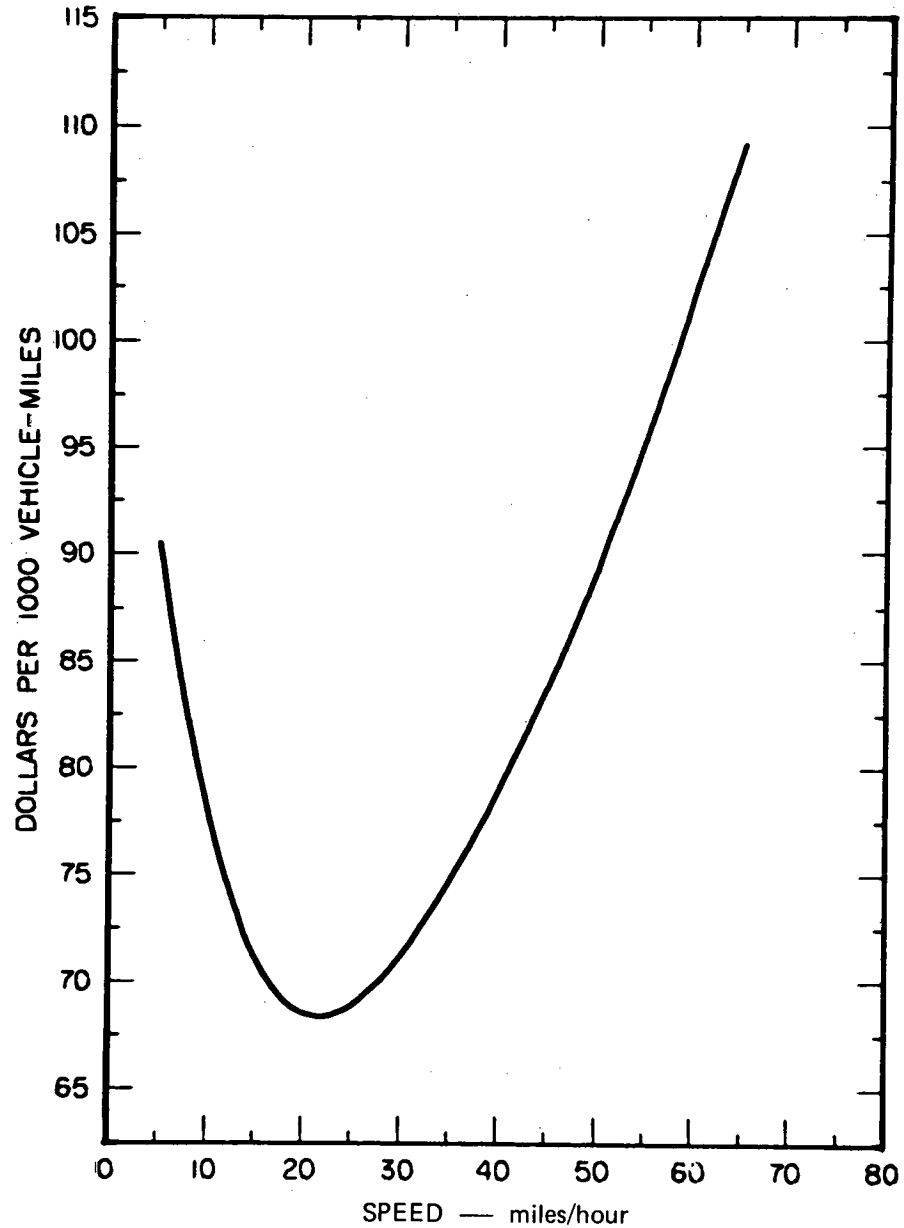


Figure A-14. Running cost at uniform speed on level tangent—single-unit trucks.

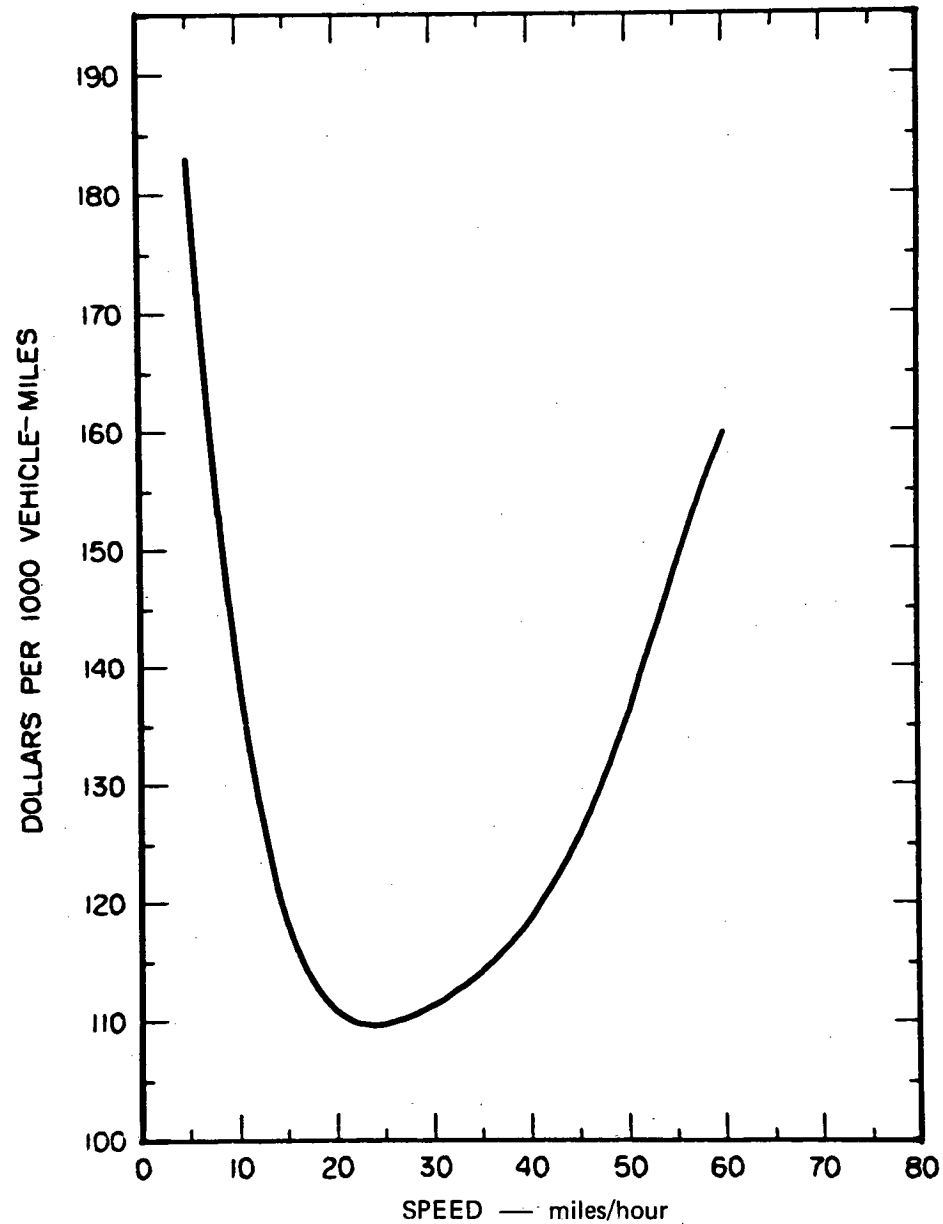
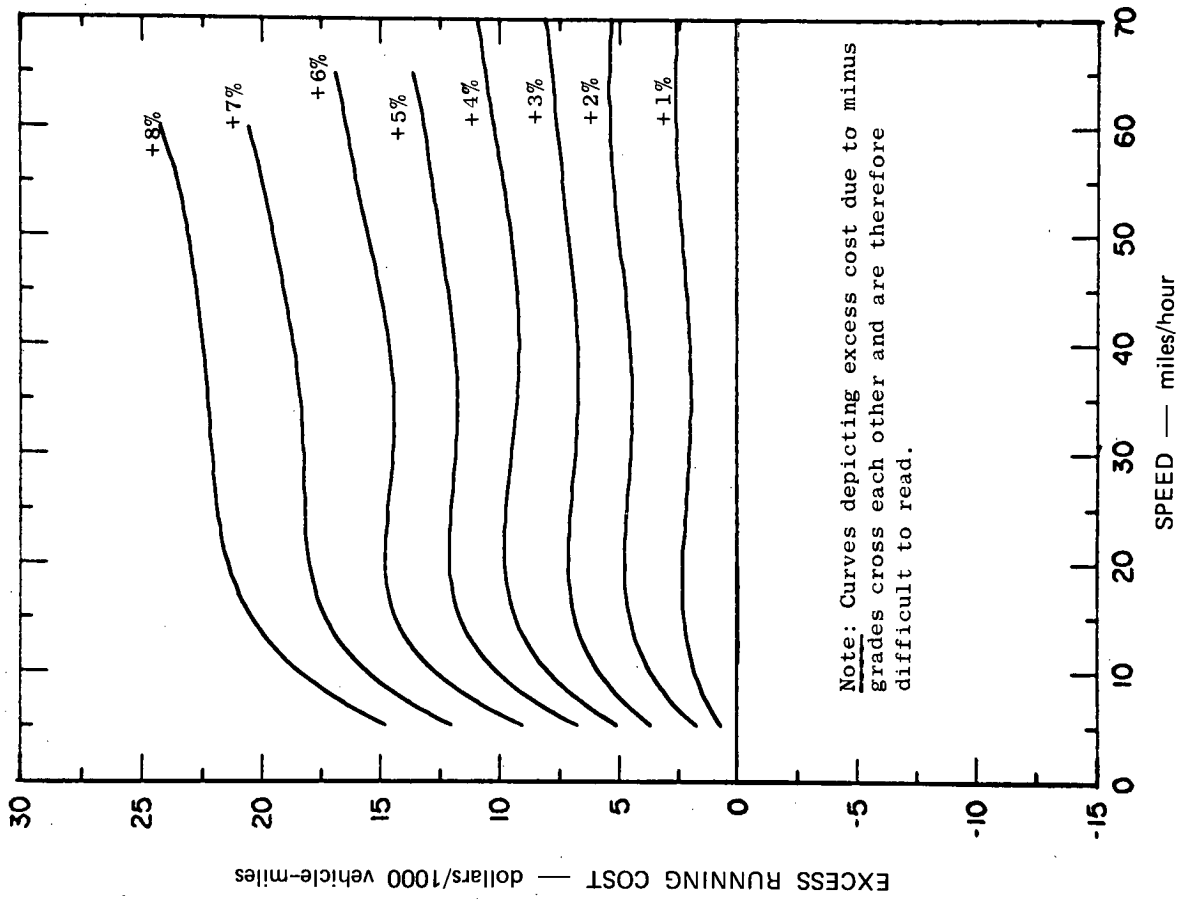


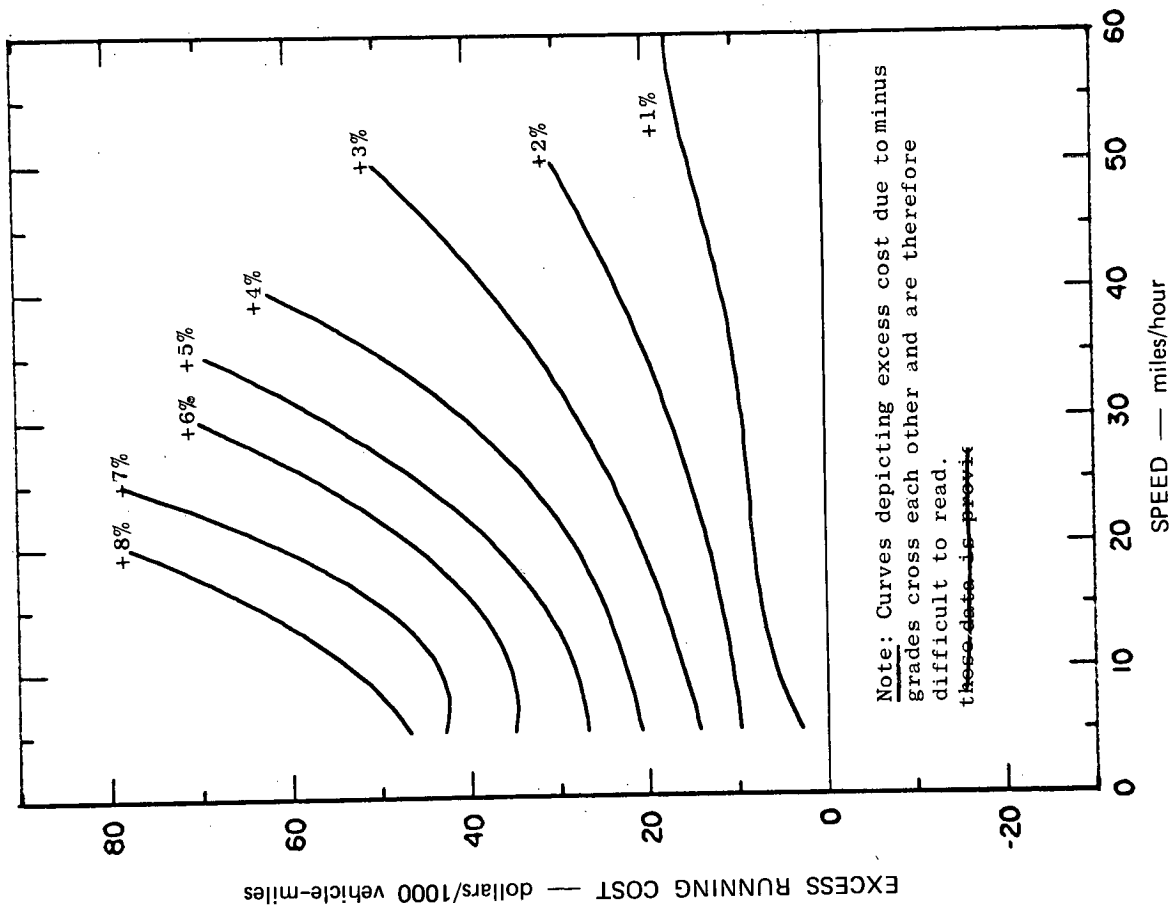
Figure A-15. Running cost at uniform speed on level tangent—3-S2 trucks.



PASSENGER CARS
EXCESS COST (OVER RUNNING ON LEVEL TANGENT) DUE TO GRADIENTS
M I N U S G R A D E S

SPEED	-1	-2	-3	-4	-5	-6	-7	-8
5.0	-4.24	-8.74	-9.07	-8.78	-8.38	-7.69	-6.76	-5.02
10.0	-2.97	-6.70	-7.79	-7.65	-7.32	-6.87	-5.98	-4.64
15.0	-2.55	-6.18	-7.48	-7.32	-7.02	-6.65	-5.90	-4.77
20.0	-2.62	-5.87	-7.14	-7.21	-6.96	-6.63	-5.98	-5.02
25.0	-2.90	-5.76	-7.16	-7.43	-7.28	-7.00	-6.42	-5.65
30.0	-2.91	-5.79	-7.28	-7.71	-7.75	-7.44	-6.94	-6.25
35.0	-2.91	-5.72	-7.53	-8.03	-8.29	-8.06	-7.57	-6.93
40.0	-2.99	-5.64	-7.41	-8.28	-8.61	-8.31	-7.81	-7.21
45.0	-3.00	-5.76	-7.43	-8.30	-8.60	-8.95	-9.62	-8.28
50.0	-3.17	-5.99	-7.44	-8.36	-8.83	-9.41	-9.04	-8.95
55.0	-2.89	-6.05	-7.11	-8.24	-8.70	-9.39	-9.13	-8.87
60.0	-2.87	-6.16	-6.95	-8.13	-8.64	-9.17	-9.91	-9.01
65.0	-2.69	-5.96	-6.95	-8.25	-9.02	-9.68		
70.0	-2.73	-5.61	-7.17	-8.41				
75.0	-2.55	-5.30	-7.29					
80.0	-2.51	-5.08						

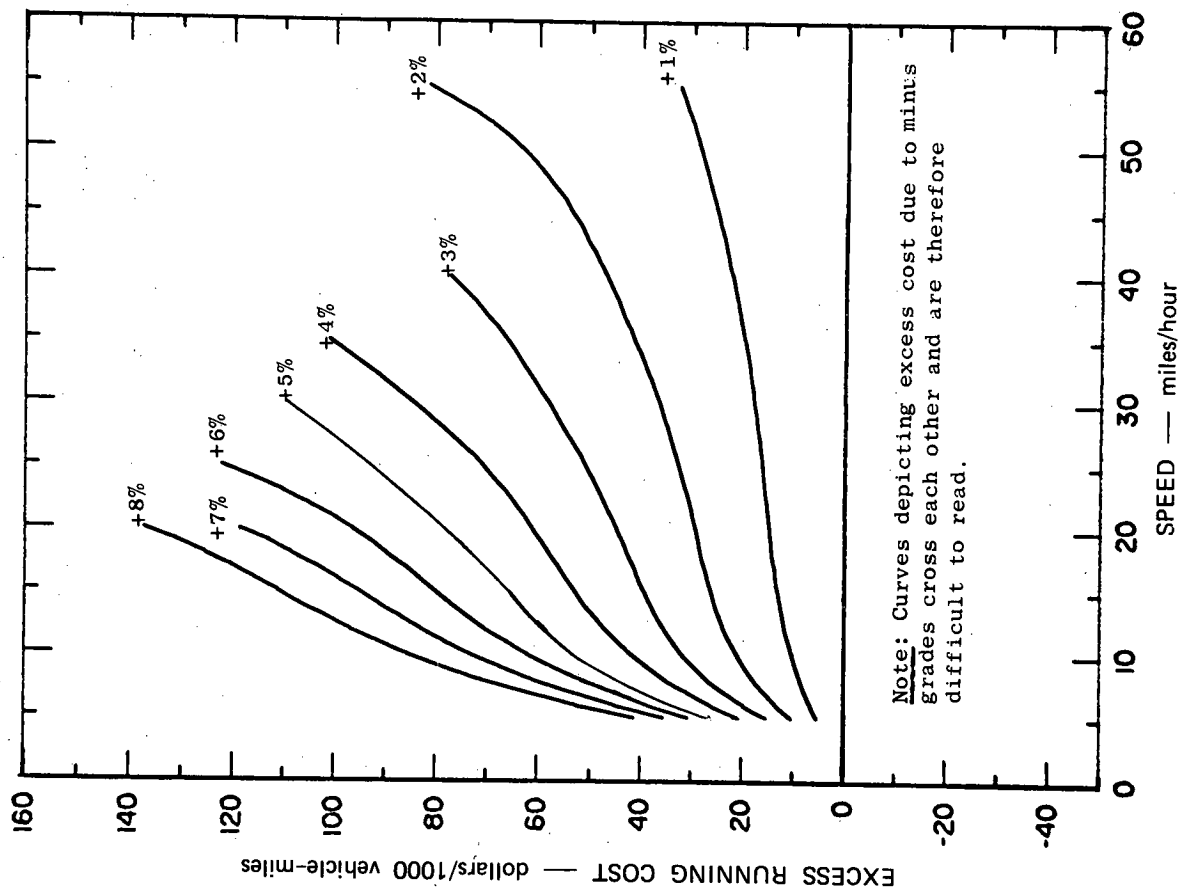
Figure A-16. Excess running cost due to grades—passenger cars (above cost on level tangent).



SINGLE UNIT TRUCKS
EXCESS COST (OVER RUNNING ON LEVEL TANGENT) DUE TO GRADIENTS
M I N U S G R A D E S

SPEED	-1	-2	-3	-4	-5	-6	-7	-8
10.0	-3.25	-5.55	-6.46	-6.62	-6.08	-5.61	-5.13	-4.72
15.0	-3.71	-6.30	-7.40	-7.98	-7.33	-6.67	-5.95	-5.29
20.0	-4.48	-7.60	-9.21	-9.55	-8.96	-8.11	-7.18	-6.30
25.0	-5.48	-9.25	-10.69	-11.49	-10.93	-10.06	-8.96	-7.81
30.0	-6.00	-10.50	-12.04	-13.15	-13.01	-12.26	-10.92	
35.0	-6.34	-11.81	-13.58	-15.06	-15.15	-14.78	-13.33	
40.0	-5.98	-12.34	-15.05	-16.49	-16.34	-15.27		
45.0	-6.56	-12.85	-16.44	-18.33				
50.0	-6.26	-11.53	-17.30					
55.0	-6.51	-12.97	-18.27					
60.0	-7.50	-13.29						

Figure A-17. Excess running cost due to grades—single-unit trucks (above cost on level tangent).



3-S2 COMBINATION TRUCKS
EXCESS COST (OVER RUNNING ON LEVEL TANGENT) DUE TO GRADIENTS
M I N U S G R A D E S

SPEED	-1	-2	-3	-4	-5	-6	-7	-8
5.0	-9.13	-13.27	-17.77	-17.02	-13.35	-10.96	-7.96	-4.93
7.5	-9.70	-13.93	-18.48	-17.52	-13.86	-11.17	-8.37	-6.51
10.0	-10.21	-14.73	-19.26	-18.06	-14.53	-11.59	-8.73	-6.43
12.5	-10.79	-15.57	-19.97	-17.58	-15.21	-12.12	-9.05	-8.08
15.0	-11.40	-16.40	-20.61	-19.02	-17.97	-12.51	-9.13	2.49
17.5	-11.89	-17.75	-20.97	-19.92	-17.14	-13.63	-9.74	
20.0	-12.42	-18.59	-22.01	-20.29	-17.75	-14.20	-10.29	
22.5	-12.86	-18.93	-22.10	-20.51	-17.57	-14.71		
25.0	-13.28	-20.09	-22.05	-20.70	-18.81	-15.18		
27.5	-13.70	-20.76	-22.15	-20.71	-19.31			
30.0	-13.95	-21.45	-22.33	-21.33	-19.76			
32.5	-14.10	-21.86	-22.74	-21.63				
35.0	-14.18	-22.25	-23.38					
37.5	-14.23	-22.52	-24.18					
40.0	-14.23	-22.71	-25.08					
42.5	-14.29	-22.87	-25.90					
45.0	-14.54	-22.98	-26.74					
47.5	-14.69	-23.14						
50.0	-15.11	-23.25						
52.5	-15.82	-23.55						
55.0	-16.99	-24.15						
57.5	-18.31							

Figure A-18. Excess running cost due to grades—3-S2 trucks (above cost on level tangent).

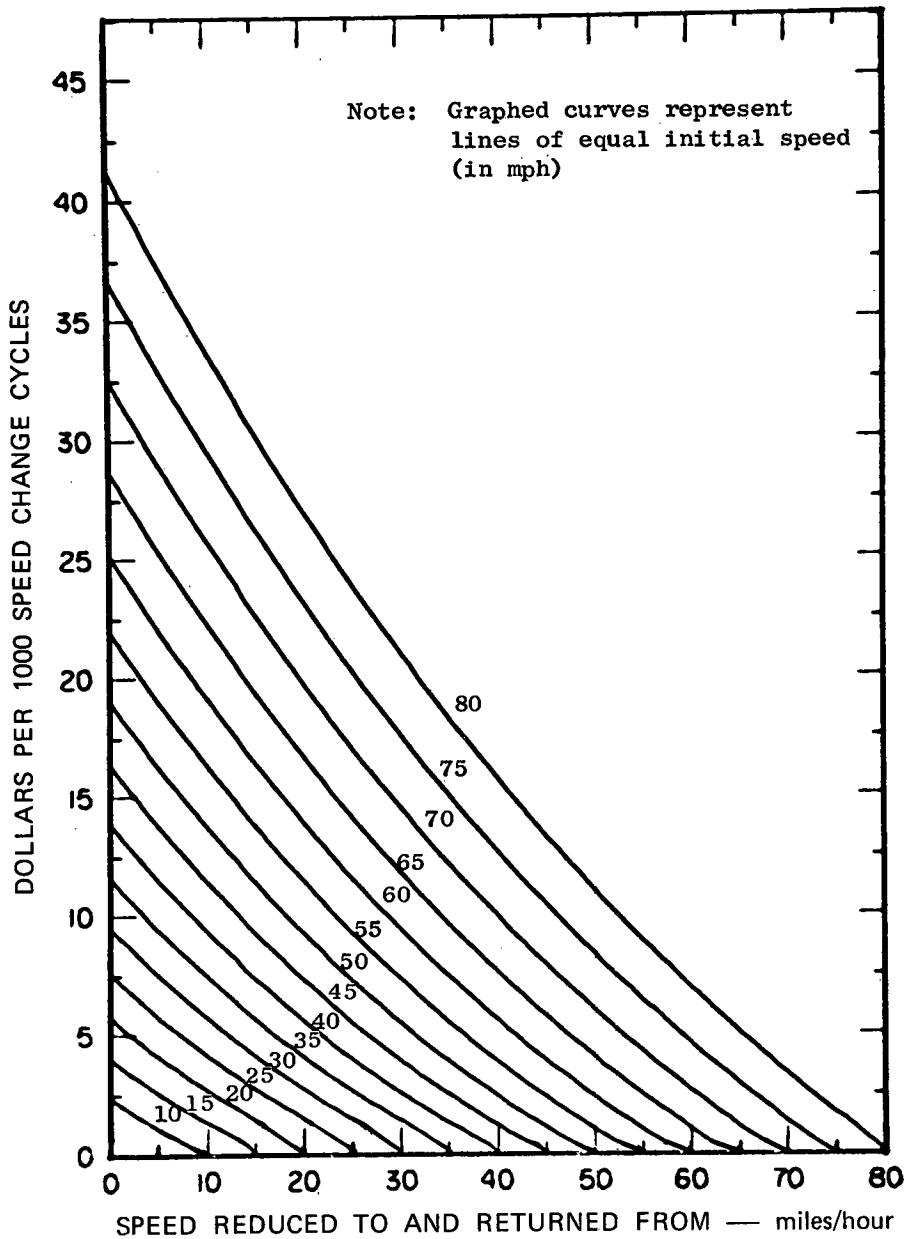


Figure A-19. Excess running cost of speed change cycles—passenger cars (above cost of continuing at initial speed).

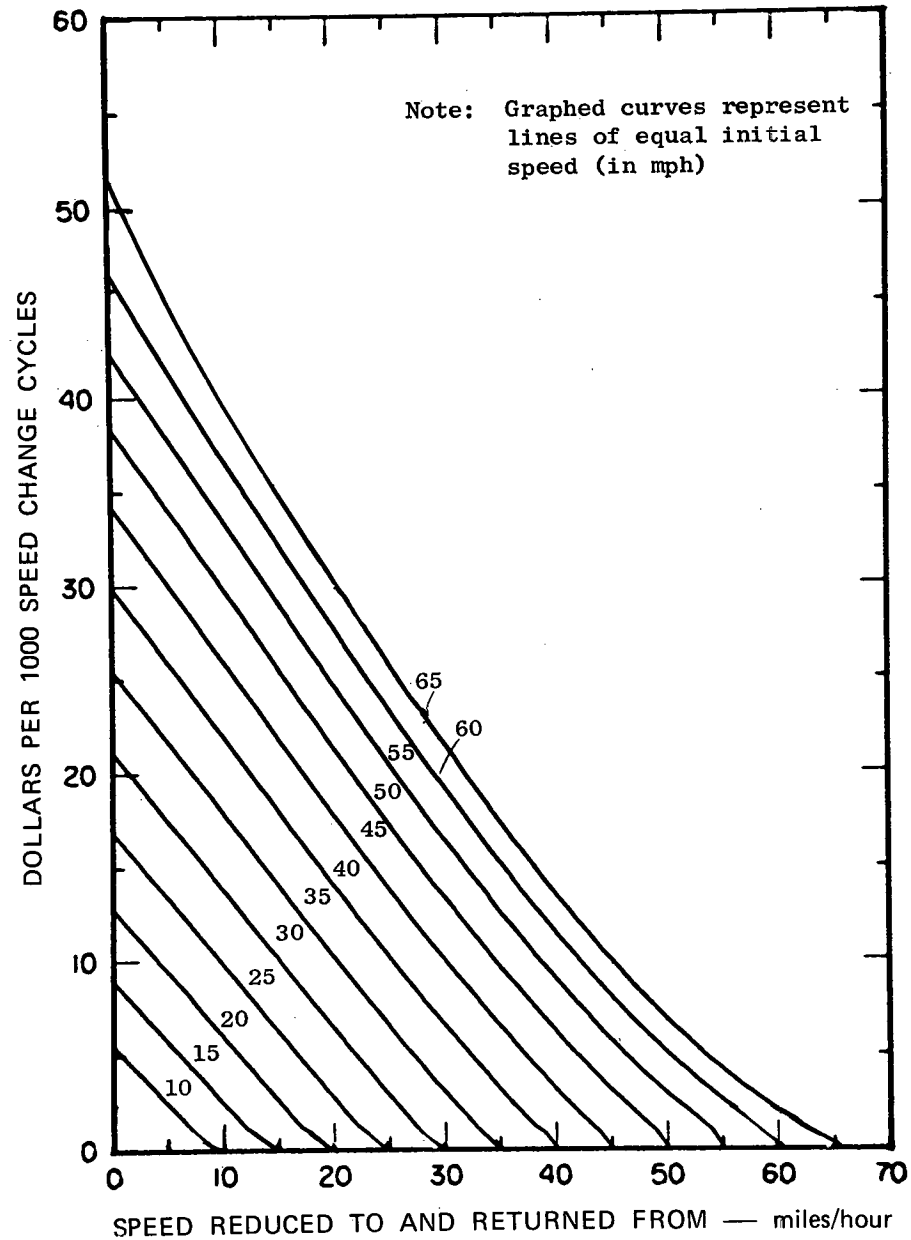


Figure A-20. Excess running cost of speed change cycles—single-unit trucks (above cost of continuing at initial speed).

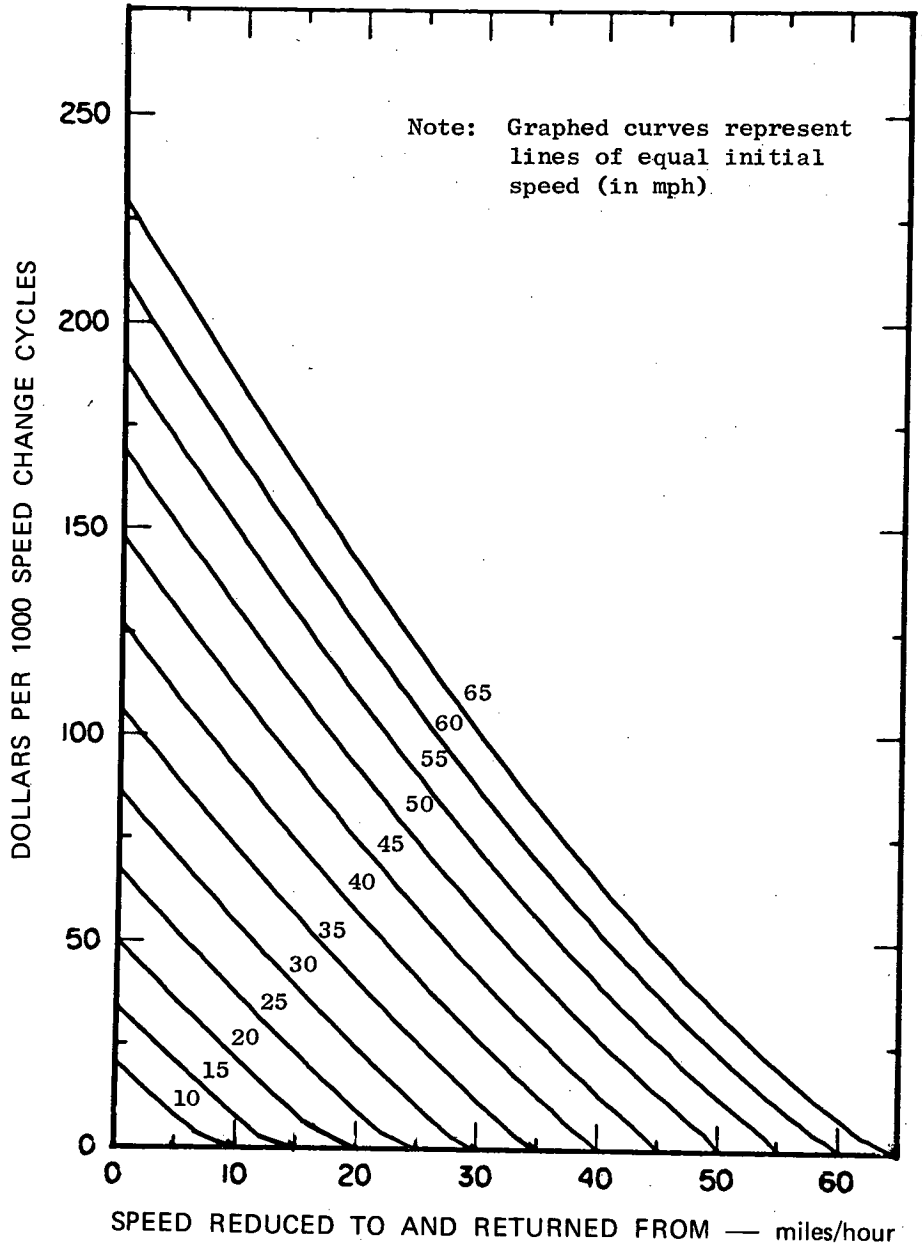


Figure A-21. Excess running cost of speed change cycles—3-S2 trucks (above cost of continuing at initial speed).

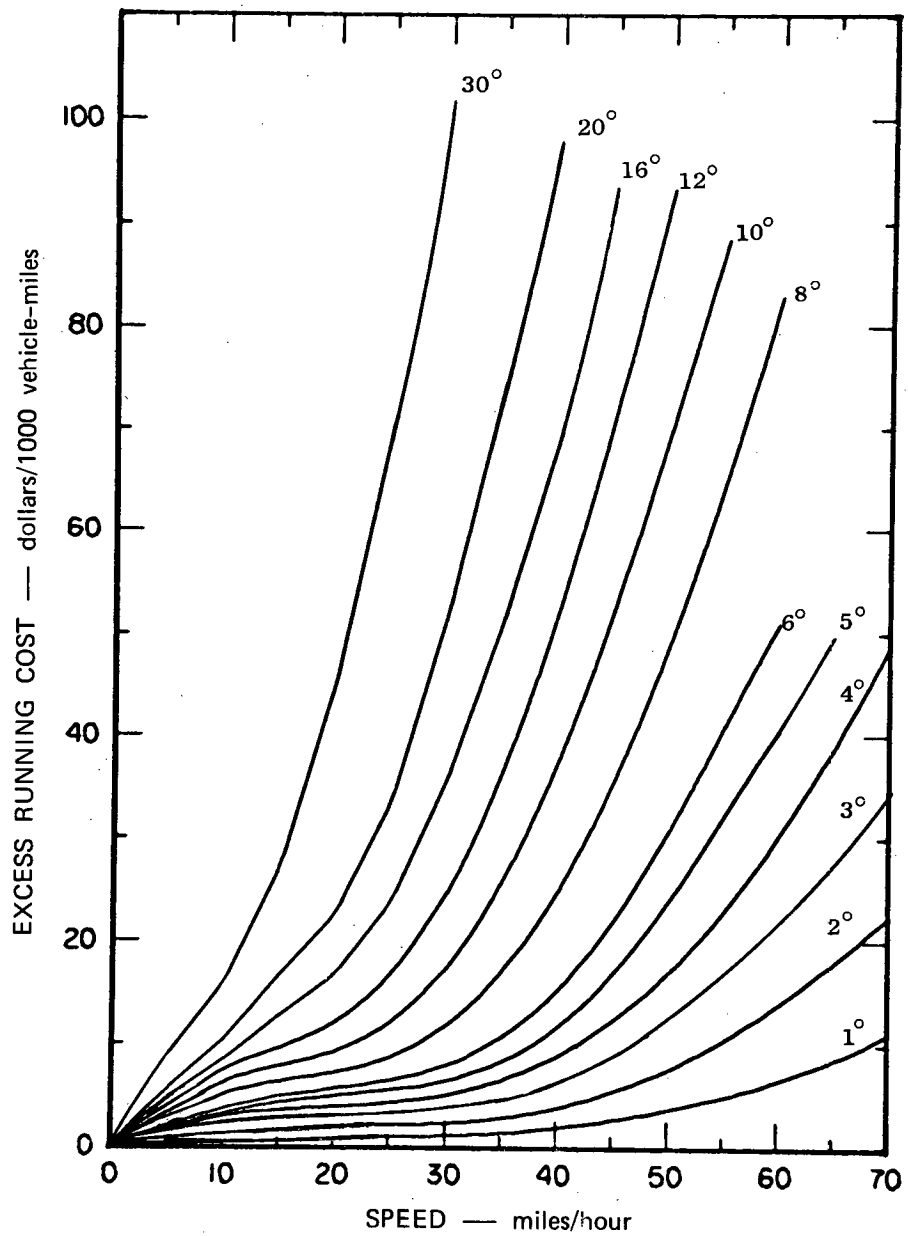


Figure A-22. Excess running cost due to horizontal curves—passenger cars (above cost on level tangent).

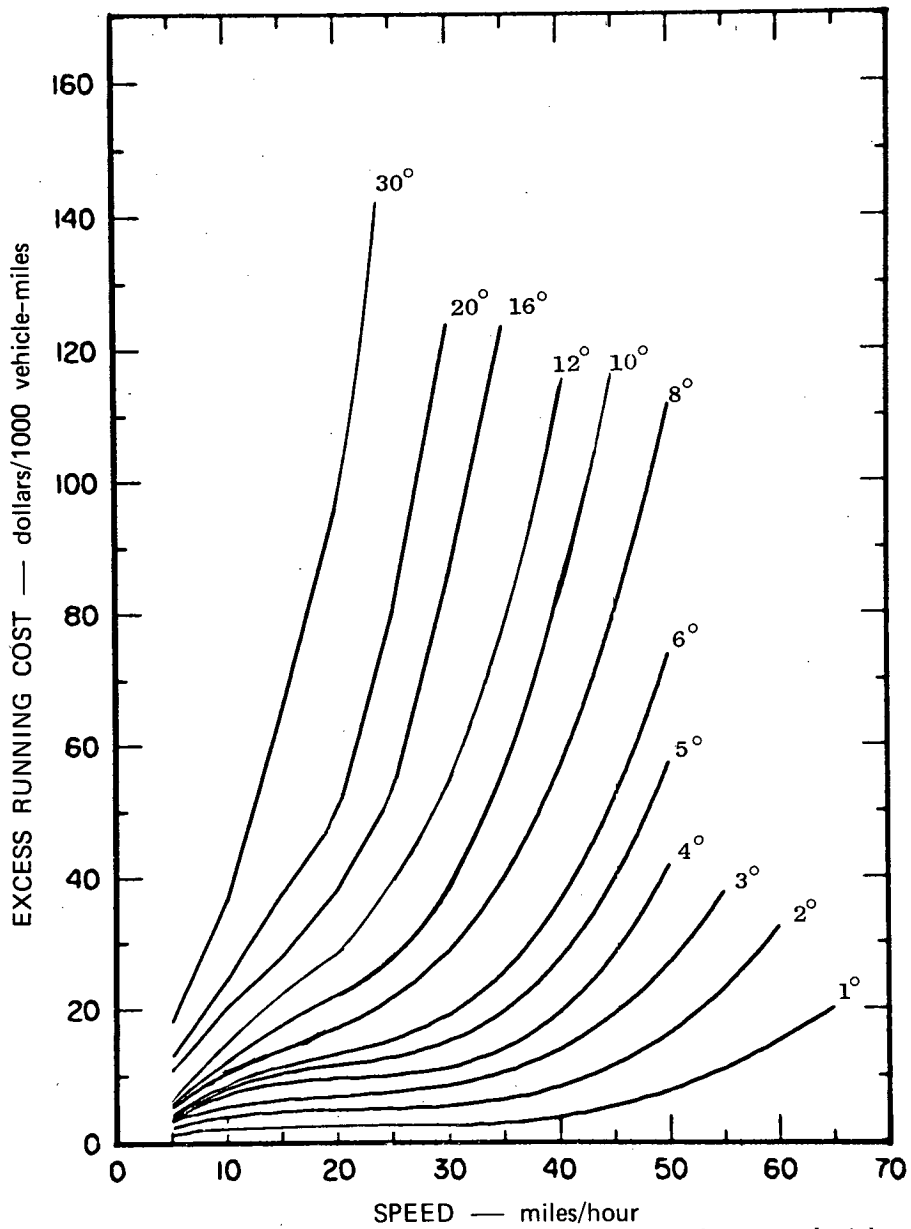


Figure A-23. Excess running cost due to horizontal curves—single-unit trucks (above cost on level tangent).

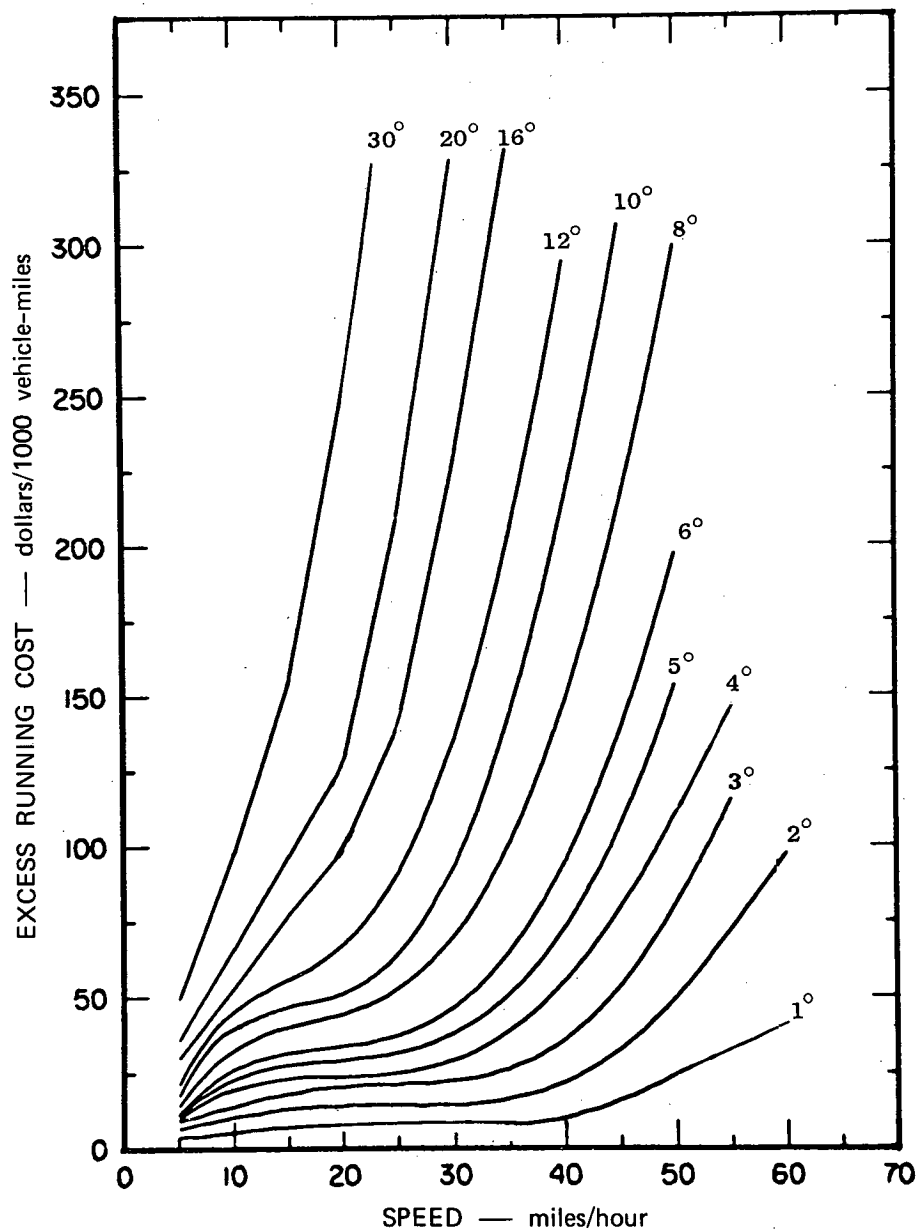


Figure A-24. Excess running cost due to horizontal curves—3-S2 trucks (above cost on level tangent).

APPENDIX B

RESEARCH APPROACH

The research approach for the study, summarized in Chapter One, is described in greater detail here. The emphasis is on technical details, data sources, and reasons for the choices of procedures and cost factors that are incorporated in the report.

DERIVATION OF UNIT RUNNING COST FACTORS

The research plan for this project included compilations and updating of motor vehicle running cost data for use in calculating relative road user costs at different levels of highway service and as affected by details of geometric design and traffic performance.

The components of motor vehicle cost that are affected by highway design and traffic conditions, and that were therefore included for purposes of this project, are fuel and engine oil consumption, tire wear, maintenance, and depreciation. Total running cost is the sum of these component costs, which were derived from Winfrey (*4*) and Miller (*11*) or from working papers obtained from them. All cost components were updated to reflect current price levels, and consumption data were converted to cost data using representative current prices. Tables were assembled of total running cost due to running on level tangents, on grades, and on curves and during idling and speed changes.

In terms of vehicle classes, running cost data were assembled for "average" passenger cars of about 4 kips (4,000 lb), for single-unit trucks of about 12 kips, and for diesel combination tractor semi-trailer (3-S2) trucks of about 50 kips. The rationale for this selection is presented later in this appendix.

Sources of Data

Other than isolated items in the literature, there are three major sources of running cost data:

1. *Economic Analysis for Highways,* by Robley Winfrey, International Textbook Co., 1969 (*4*).
2. "Running Costs of Motor Vehicles as Affected by Road Design and Traffic," by Paul Claffey, *NCHRP Report 111,* 1971 (*11*).
3. "Cost of Urban Car Travel," by E. Pelensky, Australian Road Research Board, Sydney, 1969 (*32*).

Because the third source deals with and was developed from vehicles found in Australia, and for Australian highway conditions, it was not directly applicable. Therefore, Claffey and Winfrey were the two major sources of motor vehicle running cost data for this project. These two sources together represent the most comprehensive and most recent data available. All components and factors of running cost as well as all vehicle classes considered in this project are covered to some extent in one or both of these sources. Other data sources, such as Pelensky (*32*) and Sawhill (*33, 34, 35, 36*), were used only in a supplementary way to confirm the validity of using either of the sources for given components of cost in the final tabulation.

A summary of the use of the two main sources for this project follows:

1. *Passenger Cars:* Claffey fuel data for all conditions (level, tangents, grades, curves, speed changes) and tire wear data for level tangents and speed changes; Winfrey data for all other cost components and conditions.
2. *Single-Unit Trucks:* Claffey fuel data for all conditions and tire wear data for speed changes; Winfrey data for all others.
3. *Tractor Semi-Trailers:* Winfrey data for all cost components and conditions.

In the following, the relevant contents of the two main data sources are summarized; the pertinent considerations for a choice of sources are given; and the analysis and conclusions are summarized, based on these considerations.

Winfrey (*4,* App. A) gives the following data for five types of vehicles: total running costs (including fuel, tires, oil, maintenance, and depreciation components) for a full range of highway speeds as affected by plus and minus grades, horizontal curvature, changes in speed, and idling engine, and a conversion table for gravel surfaces. In addition, data are supplied for fuel consumption in gallons and excess time consumption for speed changes.

The five types of vehicles are: 4-kip passenger car, 5-kip commercial delivery, 12-kip single-unit truck, 40-kip tractor semi-trailer (gasoline), and 50-kip tractor semi-trailer (diesel).

Claffey (*11*) gives partial running cost data for four classes of vehicles: a composite passenger car of about 4 kips, a composite pickup truck of about 5 kips, a composite of two six-tire trucks averaging 12 kips, and a tractor semi-trailer at 45 kips being a combination of a 40- and a 50-kip vehicle, both gasoline.

For these four classes of vehicles, Claffey gives fuel consumption for a full range of speeds as is desirable for economic analysis. Engine oil consumption is given only for the passenger car and the pickup, and then only for over-all use by a speed range. Oil consumption is not tied to highway design or to speed changes.

Tire wear cost is given for cars and pickups by speed on tangents, horizontal curves, speed changes, and for roadway surface. A few points for tire cost are given for the 12-kip truck. No tire data are given on the effects of vertical grades or for the 45-kip combination.

Partial vehicle maintenance costs are given for the car and the pickup in over-all general use, but are not related to highway design, speed, or traffic conditions. Vehicle

depreciation costs are not related to vehicle speed or highway design.

Claffey also provides some information on running costs for different hourly volumes of traffic, some information on the costs of speed changes on grades, and other types of information not given by Winfrey.

Considerations for Determining the Use of Data Sources

The first task in compiling unit cost factors from the existing data sources was to determine whether significant differences could or would occur if one of the major data sources was used rather than the other. If they would not, further consideration would not be necessary, and selection from among the sources could be arbitrary.

A sensitivity analysis was therefore performed in which a simple but comprehensive problem was evaluated using, in parallel, each of the two major data sources.* The general findings of the study indicated that significant differences could exist in the end results of the analysis (on the order of 30 percent), especially when the effects of curves and/or speed changes were included. A basic conclusion was that there can be situations where use of one data source might indicate acceptance of a given alternative while use of the other data source would indicate the opposite. Therefore, work on developing the most useful sources of data proceeded.

Data sources were then compared to determine which source would be most useful for each particular component of cost and how, if necessary, to combine the sources to develop, with a reasonable amount of effort, an improved set of running cost tables for this project. Criteria or areas for consideration were developed and used for judging the relative merits of using a particular data source for any of the different components or factors of running cost. Table B-1 gives a list of these areas for consideration. Topics 1 through 5 deal with relatively specific sets of data (e.g., fuel consumption on curves, or tire wear on gradients). Topics 6 and 7 are more general and deal with an over-all component of cost (e.g., fuel consumption or tire wear). Each topic can be analyzed in terms of a specific vehicle type.

As listed, these topics for consideration are overlapping and interrelated. Each consideration in and of itself is not meant to be an absolute, accept-reject criterion. Each individual situation will determine the relative importance of the considerations mentioned. When one is evaluating a given source for a particular component of cost, such considerations will have to be looked at in combination. The degree to which a given data source meets the conditions specified in the various consideration topics should be based as much as possible on quantitative information. However, owing to the nature of the problem, considerable qualitative or subjective judgment is also required.

In the following, the two basic data sources are analyzed with respect to the considerations given in Table B-1.

* For this purpose, the two distinct data sources were defined as (1) a "Winfrey" source, composed entirely of Winfrey-based data, and (2) a "Claffey" source, composed of Claffey data where possible and filled in with Winfrey or other data where gaps existed.

Analysis and Some Conclusions

Winfrey's data appear to be the most comprehensive available and are in a form most amenable to economic analysis. They also satisfy, at least to some degree, all of the considerations listed in Table B-1. For these reasons, Winfrey was used as a foundation on which to build the running cost data to be used for the project. His published tables chiefly provide data only in terms of total running cost. For use in this project, however, Winfrey provided the researchers with the original cost data by individual components for all situations. This facilitated the modification and updating of the data, component by component.

As the summary of the data sources indicates, Claffey provided few data with regard to the engine oil, maintenance, or depreciation components of running cost. Hence, for these components, the Winfrey data were used exclusively, after modification to reflect current price levels.

As a result of his extensive field research, Claffey presents the most recent empirical evidence available in the areas of fuel consumption and tire wear. These data were thus used where justified and appropriate, to modify or even replace Winfrey's data—some of which, in cases where empirical data were lacking, were based on theoretical considerations. The appropriateness of using the

TABLE B-1

CONSIDERATIONS IN CHOOSING A RUNNING COST DATA SOURCE

INDIVIDUAL FACTOR CONSIDERATIONS
1. Availability—The data should be available and in sufficient quantity so that interpolation and/or extrapolation can, if necessary, be accomplished so that data coverage does or would extend over a reasonable range of conditions.
2. Internal Consistency—For a given component or factor, do the related data points follow a consistent pattern; that is, when graphed, can the points be connected by a smooth curve or line? If they cannot, can these points be easily corrected so that such a logically consistent pattern results?
3. Acceptability—Can we accept for our purposes the data as given? Does the source of information appear reliable and representative?
4. Significant Differences—Are there significant differences between competing data sources? If there are not, the source most amenable to adaptation to the project's needs should be used.
5. Adaptability—If necessary, can the data, with a reasonable amount of effort, be modified so that they will suit the purposes of the project or meet its standards?
GENERAL COMPONENT CONSIDERATIONS
6. Coverage—For any given component of cost it would be preferable that costs for a given vehicle type attributable to various factors (e.g., gradients, curves, speed changes) be derivable from the same sources; this might be called consistency in terms of coverage.
7. Consistency Between Sources—If more than a single data source must be used, consistency should be maintained between these sources. As an extreme example, all data for different components from different sources should apply to the same vehicle.

various components of Claffey fuel and tire data was judged in terms of the considerations mentioned previously.

Table B-2 gives a summary of analysis of Claffey's fuel consumption and tire wear data with respect to the topics for consideration listed in Table B-1. The entries in Table B-2 indicate the degree to which these conditions are met by the various component or factor data. Much of the analysis summarized qualitatively in Table B-2 was performed by an extensive quantitative comparison of Claffey's fuel consumption and tire wear data with analogous data from Winfrey.

Claffey's fuel consumption data for both passenger cars and single-unit trucks appear for the most part to satisfy the conditions of data usability for this project. Fuel consumption data for grades and level tangents are similar to those of Winfrey, so that choice between either source is relatively unimportant. Claffey's data were chosen because they were derived from most recent empirical evidence. On the other hand, fuel consumption data related to curves and speed changes are markedly different from those of Winfrey. To preserve consistency, and because Claffey's empirical experience in the area of fuel consumption was relatively extensive, it was believed that the Claffey data in this area should also be used.

The evaluation of Claffey's tire wear data was more difficult than that of fuel consumption data. Although Claffey does provide considerable tire wear data for passenger cars, there are significant problems with the data. For example, the effect of gradients on tires is missing entirely, and a minimum of tire wear data for single-unit

TABLE B-2

QUALITATIVE EVALUATION OF THE DEGREE TO WHICH CLAFFEY DATA MEET CONDITIONS OF TABLE B-1 [a]

	CONSIDERATIONS						
	INDIVIDUAL					GENERAL	
ITEM	AVAILABILITY	INTERNAL CONSISTENCY	ACCEPTABILITY	SIGNIFICANT DIFFERENCES	ADAPTABILITY	COVERAGE	CONSISTENCY BETWEEN SOURCES
			(*a*) Passenger cars				
Fuel consumptioin:							
Over-all							
On tangents	Y	Y	Y	N	Y		
Due to gradients	Y	Y	Y	N	Y		
Due to curves	Y	M	M	Y	Y		
Due to speed changes	Y	M	M	Y	M		
Tire wear:							
Over-all						N	N [b]
Tangents	Y	M	?	Y	Y		
Gradients	N	n.a.	n.a.	n.a.	?		
Curves	Y	M	?	Y	M		
Speed changes	Y	M	?	Y	M		
			(*b*) Single-unit trucks				
Fuel consumption:							
Over-all						Y	Y
Tangents	Y	M	M	M	Y		
Gradients	Y	M	M	M	Y		
Curves	Y	M	M	Y	Y		
Speed changes	Y	M	M	Y	Y		
Tire wear:							
Over-all	A minimum of data is provided						
			(*c*) Tractor semi-trailers				
Over-all	Truck type is different from that of Winfrey						

Notation: Y = Yes. N = No. ? = Doubtful. M = Maybe; condition might be met with some extra effort. O = Condition which makes use of data questionable. n.a. = Data not available.

[a] Claffey data are from *NCHRP Report 111* (*11*).

[b] The trend of Claffey's tire wear data differs from that of Winfrey; e.g., at high speeds Claffey indicates tire costs go down, whereas Winfrey indicates that tire cost increases as speed increases.

trucks is provided. In addition, there is considerable divergence between Claffey and Winfrey, not only in terms of numbers but more importantly in the trend of the data. Claffey indicates that, after a certain speed, tire wear (cost) decreases as speed increases, whereas Winfrey indicates the opposite. Claffey points out that the effect of high speeds on tire wear is somewhat a function of vehicle body configuration and the aerodynamic forces on such shapes. His empirical findings imply that the effect of aerodynamic down pressure on the test vehicle (of a certain configuration) was greater than the effect on tire wear of increased traction forces needed to overcome increased air resistance at high speeds—hence, decreased tire wear (*11*, pp. 40-41). Winfrey's data imply different conclusions and possibly different assumptions regarding average auto body configurations. Further testing would be useful to resolve this issue.

In that they were empirically derived, it was decided to adopt Claffey's tire wear data for passenger-car travel on level tangents.* Because no comparable data are given for single-unit trucks, however, the Winfrey data were adopted. Winfrey data also provided the basis for tire wear costs due to gradients, for both passenger cars and single-unit trucks. Although Claffey provides data on tire wear costs due to curves (for passenger cars), the results were exceptionally high, and, in the researchers opinion, should be corraborated by further research before use. For this reason, Winfrey tire wear data were used for curves.

For a similar reason, the opposite result was applied in the case of tire wear costs due to speed changes. Winfrey's tire cost data related to speed changes appear to be unreasonably high.† In addition, these data had been theoretically rather than empirically derived—no such empirical data existed prior to Claffey's work. It was thus decided—lacking any other reasonable alternative—to adopt Claffey's tire wear cost data related to the effect of speed changes. Although Claffey provides speed change cost data for passenger cars only, it was believed that costs of the same relative magnitude could be attributed to single-unit trucks. In the absence of any better procedures, Claffey passenger-car costs were inflated in proportion to the difference between the original Winfrey speed change tire costs for passenger cars and single-unit trucks. Again, more empirical evidence would be useful.

Finally, as Table B-2 indicates, Claffey's data for tractor semi-trailers do not appear to be useful for the purposes of this project. Tire wear data for tractor semi-trailers are not given. The fuel consumption data for tractor semi-trailers is derived as a composite of a 40-kip and a 50-kip truck, and Winfrey provides separate data for each of these truck types. In addition, Claffey's trucks are gasoline powered, whereas Winfrey's 50-kip truck uses diesel fuel; diesel fuel is more common for large truck combinations. Although Claffey does give conversion factors between gasoline and diesel consumption, the component costs for the tractor semi-trailer vehicle are more consistent when Winfrey's data are used.

Choice of Vehicle Classes

Recommendations regarding the choice of vehicle classes for which running cost data are to be provided are summarized previously. The purpose of this section is to provide support for these recommendations.

The HCM refers to motor vehicles only as passenger cars, trucks, and buses. The problem in terms of this project then was to determine what modifications, if any, needed to be made to these vehicle classifications for purposes of economic analysis.

Passenger cars are the dominant vehicle in highway travel and therefore should be included as a separate vehicle class. On the other hand, buses are few in number compared, in general, to total traffic volume; hence, they may be classed, for purposes of economic analysis, in an appropriate weight group of truck or commercial vehicle. The problem thus is reduced to that of selecting an appropriate class or classes of truck vehicles.

For capacity calculations, the HCM treats pickups or commercial delivery trucks in the same manner as passenger cars. The first study undertaken in this area was thus to determine whether this assumption is also viable for economic analysis. Specifically, can the passenger-car cost data be used to represent both passenger cars and commercial vehicles, or are the separate cost relationships so different that separate tabulations (or resulting graphical displays) are necessary? In an attempt to resolve this question, running costs for passenger cars were computed ‡ for several different situations and for the same situations compared with an average or composite cost derived by combining appropriate passenger-car and commercial vehicle costs by what was believed to be an appropriate weighting scheme.§

The situations analyzed included running at uniform speeds of 5, 35, 60, and 70 mph on level tangents and on a + 3 percent grade; running on a level tangent with one 20-mph speed change cycle per mile at initial speeds of 35, 60, and 70 mph; and running at 70 mph on a 1 percent grade plus a unit stretch || at 30 mph on a 30° curve plus one stop per mile. This last situation was used to represent a situation where costs were of relatively large magnitude and where differences between passenger-car and commercial vehicle costs would be greatest.

For each situation, costs for passenger cars and for the composite situation were derived. The percentage difference in resulting cost between the composite situation and

* Claffey provides passenger-car tire wear data for both asphalt and concrete surfaces, whereas Winfrey provides data for "high type" surfaces only. Because asphalt appears to be the more predominant surface, it was decided to assume an asphalt surface wherever Claffey tire wear data were adopted.

† This judgment was brought to the researchers' attention in a private communication from Dr. G. N. T. Lack of Sydney, Australia, as a result of his experiences in applying such cost factors in actual problems.

‡ Costs were combined using the Winfrey's data prior to the researchers' development of updated cost tables. It is believed the conclusions would not be changed by using these updated cost tables.

§ On the average, passenger cars account for about 80 percent of all highway traffic and panel and pickup (i.e., 5-kip commercial) vehicles account for another 10 percent. Thus, if one considers only passenger cars and 5-kip commercial vehicles, an appropriate weighting for this group would be about 8/9th passenger cars and 1/9th commercial vehicles. This was the weighting used to derive the "composite" cost.

|| Costs are tabulated and calculations were made in terms of 1,000 vehicle-miles or 1,000 speed change cycles. Thus, a "unit stretch" was equal to 1,000 vehicle-miles.

the analogous passenger-car situation was computed. In no case was the percentage difference greater than 3.7 percent, and this occurred for the situation of a 70-mph uniform speed on a level tangent. The percentage difference for the last situation described in the previous paragraph was about 2.1 percent.

The general conclusion was thus that there is no significant difference between the results using only passenger-car cost data and those for the composite vehicle. Hence, for the purpose of economic analyses associated with highway capacity studies, it is recommended that passenger-car running cost data be used to represent, in general, both the passenger-car and commercial delivery truck vehicle classes.

A second study was carried out to determine what classes of large trucks (trucks other than pickups) would need to be represented for use in this project. Table B-3 gives the average distribution of vehicles on California highways in 1966. All single-unit trucks accounted for 36.2 percent of all large trucks, with two-axle six-tire single-unit trucks being by far the most prevalent type. Tractor semi-trailer combination trucks of the 3-S2 type accounted for 20.5 percent of large truck traffic, whereas 2-S2's accounted for only 4.5 percent.

The conclusion to be drawn from these limited data is that of the truck types for which running cost data are available, the two-axle six-tire single-unit truck and the 3-S2 tractor semi-trailer are most prevalent. The fact that there are a significant number of trucks on the road larger than the 3-S2, and the fact that most trucks larger than single-unit trucks today use diesel fuel, lend support to the idea—and thus the researchers' conclusion—that the 3-S2 type truck (and associated cost data) can be used alone (as opposed to using both 2-S2 and 3-S2 data) to represent all trucks larger than single-unit trucks.

Truck vehicles for consideration were thus reduced to the single-unit truck and the 3-S2 combination. The point of reducing the number of specific vehicle types to consider is to reduce the effort involved in data manipulation. If the effort could be warranted (it usually is not), then as many vehicle types as possible or desired for a given degree of accuracy should be used.

The question of whether and how these two truck types might be combined to produce an "average" truck also was considered, but, as is evident from Table 2, the relative distribution of trucks as represented by the single-unit and 3-S2 categories varies considerably according to highway type. Accordingly, provision was made in Worksheet 3 for users to form any desired weighted combination of the two truck types, rather than giving a single, inflexible composite average.

TABLE B-3

AVERAGE DISTRIBUTION OF VEHICLES, CALIFORNIA, 1966

VEHICLE	% OF TOTAL VEHICLES	% OF TOTAL TRUCKS (OTHER THAN PICKUPS)
Passenger cars	80.4	
Pickups	10.2	
Single-unit trucks:		
2-axle, 6-tire		26.1
Others		10.1
Subtotal	3.4	36.2
Tractor semi-trailer combination:		
2-S2		4.5
3-S2		20.5
Others		5.8
Subtotal	2.9	30.8
Tractive truck & trailer combination:		
5-axle		9.7
Others		0.9
Subtotal	1.0	10.6
Tractor & two trailer combination:		
5-axle		21.9
Others		0.5
Subtotal	2.1	22.4

Source: Winfrey (*4*, p. 429).

Method of Updating Unit Cost Factors

The choice of average prices for updating road user cost factors (as derived from the basic Winfrey and Claffey data described previously) is given in Table B-4. The prices are based on average 1970 levels, and were used either to update cost factors that had been based on earlier information or to place a price on reported rates of consumption. Price information was obtained by consulting national consumer and wholesale price indexes, trade journals, and local dealers in the San Francisco Bay area. Some observations concerning the prices in Table B-4 are as follows:

1. The average retail price of regular gasoline (used in both Winfrey's and Claffey's cars) has increased by 12

TABLE B-4

UNIT PRICES OR FACTORS USED IN CALCULATING UNIT RUNNING COSTS

	UNIT PRICE OR FACTOR, BY VEHICLE		
COST ITEM	PASS. CAR	SINGLE-UNIT TRUCK	3-S2 DIESEL TRUCK
Fuel ($/gal excluding tax)	0.24	0.20	0.16
Engine oil ($/qt)	0.72	0.40	0.20
Tires ($/tire)	30	105	250
Depreciation, vehicle base price ($)	3,400	4,100	22,600
Maintenance, increase over 1962 Winfrey costs (%)	35	35	35

percent since 1962 to a 1970 level of $0.24 per gallon, exclusive of state and Federal fuel taxes (*37*, p. 32). Claffey's passenger-car fuel consumption rates were multiplied by this value to obtain an updated fuel cost component. Average 1970 prices of $0.20 per gallon on gasoline and $0.16 per gallon of diesel fuel were determined for single-unit trucks and 3-S2 diesel trucks, respectively. Allowances for quantity discounts given to large trucking firms were made for truck fuel prices. Multiplication of consumption rates by updated unit prices yielded updated truck fuel costs. (Winfrey's 3-S2 fuel data were already in terms of costs at $0.16 per gallon. Hence, no data updating was needed in this case.)

2. The average 1970 retail price of engine oil is not as readily ascertainable as that of gasoline, but the retail price index for motor oil shows a gain of about 24 percent since 1962; applying this gain to Winfrey's 1962 price of $0.60 per quart gives a figure of $0.74 per quart. This price is somewhat higher than the average observed 1970 service station price of about $0.70 per quart in the San Francisco area for high-detergent, paraffin-base, variable-SAE motor oil. Thus, $0.72 per quart appears to be a reasonable compromise. Analogous findings and taking the effects of large quantity discounts into account resulted in the truck prices shown. Because Winfrey unit cost data were used as a base, the ratio of current prices to the prices Winfrey used in obtaining his costs was used to multiply the basic data for updating purposes.

3. Passenger-car tire prices have risen about 30 percent since 1962, increasing the Winfrey per-tire cost of $23 to about $30 in 1970 prices (a 7.50-by 14-in. 4-ply rated tire was used on both the Winfrey and Claffey test cars). The tires used by Claffey were reported to cost $29.75 each in 1969 prices, so $30 appears reasonable. Truck tire prices did not increase as much as passenger tire prices over the 1962-1970 period—about 14 percent. Inflating Winfrey's 1962 prices by this amount resulted in 1970 price estimates of $105/tire and $250/tire for trucks.

4. The new automobile price of $3,400 is the suggested retail price for the lowest-cost, standard-size, four-door Chevrolet sedan, the Biscayne model, including $120 transportation costs, $40 dealer preparation costs, and 4 percent local sales taxes, but no accessories. It was estimated that the average list price of about $510 * for accessories for such a car was approximately offset by discounts offered or available on request (or insistance) from dealers. This reasoning is also supported by the fact that $3,400 would cover the dealer's cost of about $2,950 for the indicated car, plus about a 10 percent profit margin (including commissions) on cost. Average 1970 prices for trucks were determined by inflating Winfrey's 1962 prices by the 13 percent rise in truck prices over the period 1962-1970. Winfrey's depreciation based costs were thus inflated by the increase in initial vehicle cost prices to obtain updated unit cost factors for this component.

5. Automobile repair and maintenance costs have risen about 35 percent in the period 1962-1970. Because similar information regarding trucks was lacking, the same figure was used in updating all of Winfrey's maintenance component cost figures.

The detailed updating approach described here was followed because it did not appear feasible to update unit costs factors as a whole by the use of some simple index or set of indexes. The problem is that the relative percentage of total running cost represented by each component cost varies widely depending on vehicle speed and types of cost (e.g., on level tangents, due to grades, curves, and speed changes). Complicating this is the fact that the prices of the different components of cost may change with changing technology. For example, tire technology may someday allow tires to be truly unconditionally guaranteed for a given mileage such that tire cost will be a constant per-mile cost. For these reasons, it is recommended that, if desired, future updating of unit cost factors be accomplished in the manner described previously, by obtaining current cost component price levels and applying these levels (or ratios of these levels to old levels) to each component cost factor separately over all speeds and situations.

Derivation of Appendix A Charts

The procedures by which the Appendix A speed and running cost charts were derived are noted at the beginning of that appendix. The process for deriving Figures A-1 through A-12 was particularly complex, and is therefore presented step by step here.

1. The HCM operating speed vs v/c ratio curves were first translated to average passenger car speed by

$$AS = OS - \frac{DS}{10}\left(1 - \frac{v}{c}\right) \qquad \text{(B-1)}$$

This formula is explained in Chapter One. Then, factors were added to reduce the speeds for single-unit trucks and truck combinations to levels ranging from 10 mph below passenger-car speed at a v/c ratio of 0 to the same as passenger-car speed at a v/c ratio of 1.0.

2. The average speed charts were then used, in combination with the unit running cost factors described earlier, to create interim cost tables of level tangent running costs without speed changes.

3. Finally, empirically developed functions relating v/c ratios on various highway facilities to average excess cost due to speed changes were added to the interim cost tables. Development of these speed change costs is discussed in the next section.

The final step implements the assumption in the methodology that the running cost at some average speed can be conceptually decomposed into a cost of running at a uniform speed (the average speed) plus the excess cost due to speed changes about this average speed.

Cost of Speed Changes

This section describes the method used for deriving speed change costs as a function of the v/c ratio for various highway facilities. The nature and importance of speed change cycles is discussed first.

* This figure was derived by assuming power steering, AM radio, V-8 engine, and automatic transmission in all cars, and air conditioning in 40 percent of the cars.

The running cost of a motor vehicle is less at a specific constant speed than at a variable speed which averages out to the same uniform speed. Richer fuel-air mixtures are made when the throttle is depressed, and the excess fuel used to accelerate above or back to uniform speed is more than the fuel savings in the corresponding decelerations. There are also other forces and wear at play during this process, in the transmission system, in the tires, and often in the braking system.

Speed changes occur mainly as a consequence of (1) features of the highway design, such as vertical curvature (gradients) or curves, (2) the driver's behavior or manner of driving, and (3) traffic conditions. In this analysis, the first type of speed change costs mentioned is included in the analysis of curves and gradients. The second and third types are not readily distinguishable, but both are important as an economic consequence of normal highway travel, and both were included in the speed change data.

The necessary data for analyzing speed changes were obtained by recording the speed of a vehicle moving in the traffic flow at the prevailing speeds. The speed was recorded at each second of time or at equal distances (i.e., 0.1 mile). The use of a clock recording instrument produced most of the speed profiles used in this study. Accurate hourly volume counts corresponding with each speed profile were available for the calculations of the corresponding v/c ratios. A computer program for freeway and highway capacity, developed by Collins and May (*38*), was used to obtain the v/c ratios for each section of highway under study.

The speed profile data base consisted of 210 observations made on freeways, multilane and two-lane highways, and urban streets for various traffic conditions, including those of queuing. The data were developed from road tests conducted by the researchers in the San Francisco Bay area and from speed profiles and hourly traffic volume counts for various roads and situations, developed in Georgia (*39*) and Detroit (*40*).

The speed profiles from the Detroit study were particularly useful in relation to freeway traffic. Accurate hourly counts are provided and different traffic situations are covered. Off-peak v/c ratios in the order of 0.30 and peak traffic values close to 1.00 were found. A separate speed change analysis for arterials was also based on data in this study.

Speed changes in level of service F (queuing) were analyzed mainly from the Georgia study data, complemented as in the case of freeways by road tests by the researchers. Speed changes for multilane roads and two-lane roads analysis were based on road tests conducted in the San Francisco Bay area.

The speed change patterns and costs found using the three different data sources are similar in the areas where v/c ratio values overlap, and final results for different types of roads are consistent.

A computer program, written in the FORTRAN language, was developed for use in obtaining speed change costs (excess above running at uniform speed) from the speed profile data. Input data included unit running cost functions for excess costs due to speed changes (developed as described previously) for passenger cars, single-unit trucks, and 3-S2 trucks, descriptions of all speed changes found in the 210 different speed profiles collected, and descriptions of road characteristics and hourly volume.

Because speed change *cycles* often could not be determined, the computer program assigned costs to speed *changes,* using the rule that an accelerative speed change was about 60 percent of a corresponding speed change cycle cost, whereas a change by deceleration was 40 percent of the cost of a complete cycle. This is because more fuel consumption and greater tire wear occur during the acceleration phase of a speed change cycle than during the deceleration phase. The costs due to the deceleration phase are related primarily to the absorption of kinetic energy by the vehicle. Speed changes of 2 mph or greater were considered in the analysis program in assigning speed change costs.

Separate speed profiles of trucks were not available to show actual truck behavior on the road. Hypotheses therefore were made to relate different speed changes in car traffic flow to that of trucks. A computer program was developed to construct tables based on assumptions of (1) proportionality to automobile speeds at speeds above 30 mph for 3-S2 trucks and above 50 mph for single-unit trucks, and (2) identity with automobile speeds below 30 or 50 mph, respectively. Also, considerations for slower truck acceleration and for maintenance of a more constant speed in cases of small truck speed changes were introduced. A flow chart giving all the assumptions introduced in the program is shown in Figure B-1. Final results appeared consistent with reality, but more research by systematic road tests is recommended.

The output of the speed change cost analysis program provided a data list of more than 600 relationships of v/c ratio versus excess speed change cost per 1,000 vehicle-miles. To develop reasonable representations of v/c ratio-speed change cost relationships, various forms of regression curves were attempted for fitting the data or segment of it. This work, carried out with the aid of a regression computer program, was performed for the various types of highways considered as well as for each vehicle class considered.

For other than level of service F (queuing) conditions, the fit that produced the most reasonable relationships was found to be a straight-line relationship between v/c ratio and excess speed change costs. To avoid the prediction of small negative costs near a v/c ratio of zero, a constraint was imposed in the regression program calling for the regression lines to pass through the origin. Although the data points were relatively scattered, the derived relationships possessed statistically reasonable multiple correlation coefficients of between 0.77 and 0.66.* Figure B-2 shows a plot of the final relationships.

* Actual values for the multiple correlation coefficient are as follows:

Freeways	Passenger cars	0.73
	Single-unit truck	0.69
	3-S2	0.66
Multilane roads	Passenger cars	0.76
	Single-unit truck	0.74
	3-S2	0.63
Two-lane roads	Passenger cars	0.77
	Single-unit truck	0.75
	3-S2	0.67

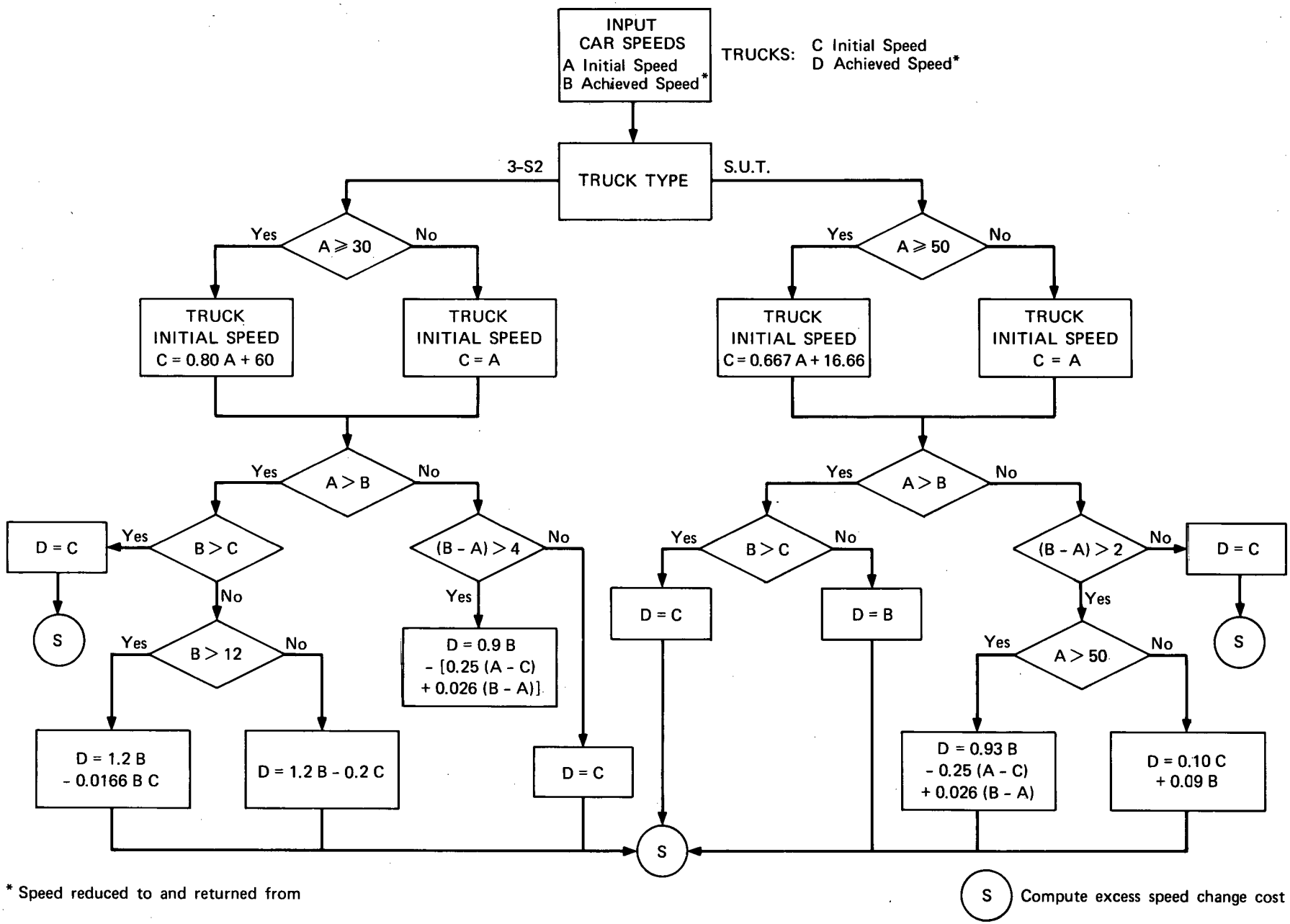


Figure B-1. Assumptions regarding truck speeds.

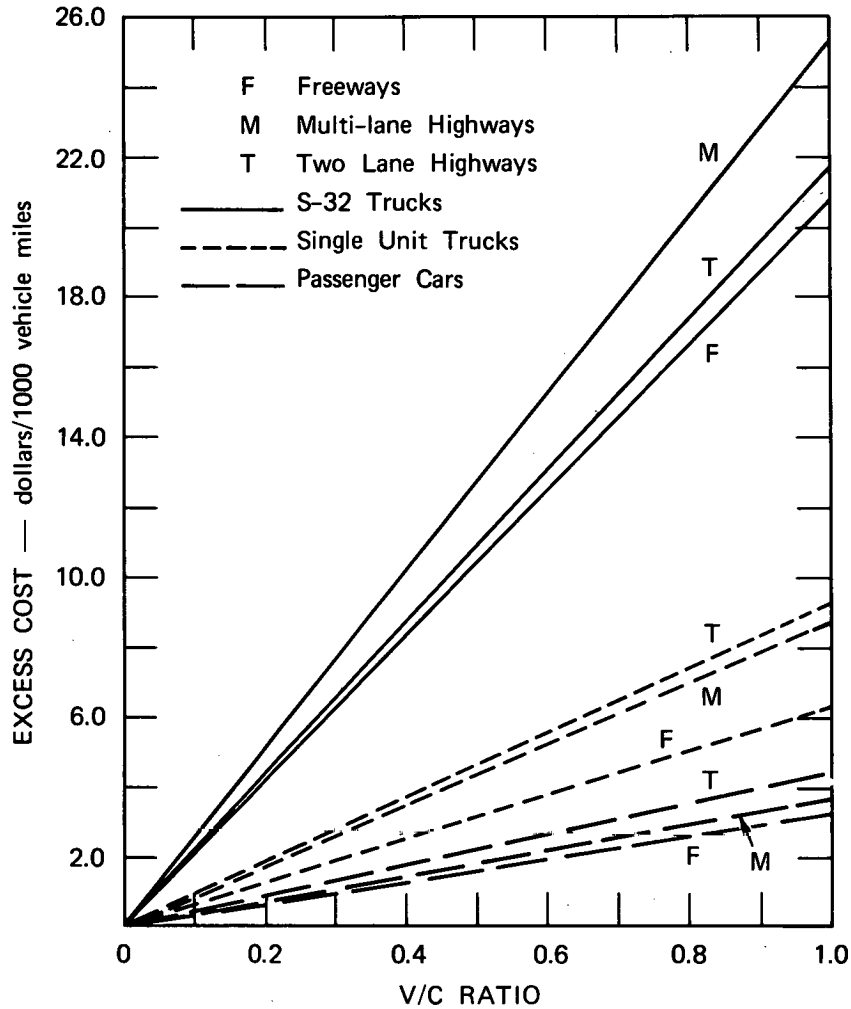


Figure B-2. Excess cost of speed changes as a function of v/c *ratio for selected highway facilities and vehicle types.*

Observations of level of service F conditions were considered independently, and a set of regression lines was calculated for passenger cars, single-unit trucks, and 3-S2 trucks in a manner similar to that explained previously. These relationships are discussed in the next section.

Running Costs vs v/c *Ratio for Level of Service F*

The curves shown in Figure 13 for running costs of level of service F were derived from 31 observations of level of service F conditions. These observations were taken in Georgia (*39*), in Detroit (*40*), and in the San Francisco Bay area (from speed runs by the researchers). The observations included accurate traffic counts in each direction of travel, speed profiles of a test car, length of the run, running time, and idling time. The data presented v/c ratios under queuing conditions ranging from as low as 0.25 to 0.977. These data served as the basis for estimates of average speed, the cost of speed changes, and total running costs.

The method for finding v/c ratios under queuing conditions was similar to that for levels of service A through E; that is, per-lane capacity was calculated in the usual way, and volume was determined by the number of cars per hour coming out the downstream end of the queuing section. The existence of queuing, especially the existence of one or more complete stops due to congestion (rather than to signalized intersections or accidents), was established by the erraticism of speed profiles for the traffic run.

The average speed as a function of v/c ratio was computed for each observation, and a regression curve was then fitted to the data. The results, shown in Figure 9, are similar to the corresponding HCM level of service F curves and produced a multiple correlation coefficient of 0.892. The results corresponded closely to the HCM speed curves for level of service F.

The level tangent running cost vs speed functions for different types of vehicles were combined with the average speed vs v/c ratio to obtain the running cost vs v/c ratio. The cost of speed changes was calculated using the same data and following the methods described in the previous section of this appendix. The output provided the relationship of v/c ratio vs cost per 100 vehicle-miles. With the aid of a regression computer program, several types of

curves were tried to fit the data; an exponential curve showed the best fit (multiple correlation coefficient of 0.50) and was adopted. The total cost (sum of the speed changes cost plus running cost) was then calculated for various v/c ratio levels, and was extended to trucks by methods previously described for speed change cycles. The results are plotted in Figure 13 for each type of vehicle.

It should be noted that running cost data corresponding to v/c ratios of less than 0.25 (corresponding to average speeds of less than 5 mph) were not available, so extrapolations of the regression line were used as one approximation. In addition, limits of probable running costs of less than 5 mph were established by modifying idling costs to include maintenance costs and hypothetical tire wear, and then dividing the results by speeds of less than 5 mph. The low v/c ratio results were sharply increasing costs at low speeds, as would be expected on a cost-per-mile basis as speed decreases. The results were reasonably comparable for these different methods, but the running costs shown for very low v/c ratios should be taken only as indicative and are shown as dashed lines below a v/c ratio of 0.1 to indicate increasing uncertainty. However, v/c ratios below 0.25 probably will rarely be found, because they imply average speeds of less than 5 mph.

Another approach used to check the validity of the results obtained was to consider the idling cost for the time at which the test vehicle was actually stopped and then add to it the running cost at the average moving speed. Two such tests were computed, for v/c ratios of 0.280 and 0.387; results for passenger cars confirmed the values of Figure 13. The same procedures for heavy trucks (3-S2) resulted in much lower costs; the reasons for this are presumed to be discontinuities in truck costs as a function of speed of less than 5 mph.

PEAK-HOUR VOLUME AS A FUNCTION OF AADT

The upper part of Figure 7, showing peak-hour volume per lane in relation to AADT per lane pair, was derived largely from data in the HCM (App. A). These data provide both AADT for given road sections and corresponding peak-hour volumes for selected highest hours: maximum 10th, 20th, 30th, 40th, 50th, 100th, and 200th.

The first step was to calculate the 30th highest hour and the 100th highest hour volume, using the percentage provided, for a single direction of travel (the volume for both directions of travel was not used because peaking seldom occurs in both directions at once). Also, to obtain more accurate results by making more observations available for statistical analysis, and also with the aim of simplifying the results, AADT data were converted to a per-lane-pair basis. This permitted all sizes of similar types of highway facilities to be analyzed together.

A total of 704 observations were organized according to type of facility and highest hour considered. Different types of curves and regression equations were then tested, and the final choice was regression equations of the quadratic type. The multiple correlation coefficients for freeways were 0.960 for 30th hour, and 0.961 for 100th hour; for multilane rural highways and two-lane highways they were 0.866 for 30th hour and 0.913 for 100th hour. The curves were smoothed slightly to converge more quickly with the 30th-hour freeway curves near the origin, to obtain a better fit through very low AADT data.

No significant difference was found for the regression lines of multilane and two-lane roads. For that reason, the regression lines included in Figure 7 combine data for multilane and two-lane roads. The "arterial" designation was added subsequently to the multilane and two-lane curves, because of the closer fit of arterial data to these curves than to the freeway curves.

The lower part of Figure 7 is a nomograph showing three curves for one to three one-way peak hours per day. The nomograph was constructed following arithmetic relationships in order to simplify the finding of approximate off-peak volumes for given ADT corresponding to the peak-hour volumes selected by the upper part of Figure 7. The curves were calculated in relation to a composite average peak-hour curve (not shown) that included both the 100th- and the 30th-hour curve for all types of roads.

QUEUING ANALYSIS

This section describes the two approaches used to analyze queuing in this report, based on the shock-wave method for uninterrupted flow and the deterministic method for interrupted flow. The key results needed in both cases, in order to assign costs to queuing, are average queue length, queue duration, and average vehicle delay due to the queue. (Further research on queuing methodology is suggested in the last section of this appendix.)

Shock-Wave Method

An example is given in Chapter Three for Worksheets 2, 2A, and 3 of queuing on a freeway (for uninterrupted flow). The v/c ratio in the congested subsection of the upstream section was computed as 0.67. The average number of miles of congestion (l_Q), defined as the distance from the bottleneck to the upstream location of the shock wave * caused by the queuing vehicles, is derived as follows.

The speed that the shock wave travels upstream during a time period, t, can be defined by

$$W_t = (-\Delta v_t / \Delta k_t) \qquad \text{(B-2)}$$

in which

$\Delta v_t =$ difference in vehicle flow during time period t between the congested section of roadway and the section of roadway upstream from the queue. This difference represents the rate of queuing. The minus sign in front of Δv_t ensures that the rate of queuing follows the convention that a positive value indicates a queue buildup, whereas a negative value indicates queue dissipation; and

$\Delta k_t =$ difference in density during time period t between the congested section of roadway and the section of roadway upstream from the queue. Density,

* Discontinuities in traffic flow are theorized to propagate in a manner similar to "shock waves" in compressible fluids. Ref. *19* describes this theory of traffic flow.

k_t, is defined as the average number of vehicles per unit length of roadway during the time period t:

$$k_t = (\Delta v_t / \text{Speed}_t) \qquad \text{(B-3)}$$

In the example of Chapter Three, $\Delta v_t = 3{,}880 - 4{,}200$ and $\Delta k_t = \frac{3{,}880 \text{ vph}}{13 \text{ mph}} - \frac{4{,}200 \text{ vph}}{49 \text{ mph}}$ during queue buildup. Note that $-\Delta v_t = 4{,}200 - 3{,}880$. Hence, as explained in the text (see Worksheet 2A), a positive rate of queuing represents an increasing queue (buildup), whereas a negative rate of queuing represents queue dissipation. The 4,200 value comes from line 1 on Worksheet 2A; the 3,880 corresponds to the value on line 4 of the worksheet for the peak period.

The average queue length that each vehicle will experience during a time period of T_t hours is:

$$l_Q = W_t \; T_t / 2 \qquad \text{(B-4)}$$

The congestion does not end until the queue has dissipated. The computation of the queue dissipation is based on the principle that vehicles must be conserved. This principle stated as an equation is:

$$\Delta v_{t+1} \; TDQ = \Delta v_t \; T_t \qquad \text{(B-5)}$$

in which T_t is the time duration of queue buildup and TDQ is the time duration of queue dissipation. Note that Δv_{t+1} is the difference between capacity flow of the bottleneck during the peak period and the flow through the bottleneck during the off-peak period. This difference represents the rate of queue dissipation and as such generally has a negative value.

The time TDQ to dissipate the queue is thus

$$TDQ = \Delta v_t \; T_t / \Delta v_{t+1} \qquad \text{(B-6)}$$

The time duration of the queue is $T_t + TDQ$, and capacity (of the bottleneck section) flow in sections qb and bc will occur during this time.

Completing the example of Worksheet 2A: The shockwave speed going upstream (rate of queue buildup) is

$$W_t = \frac{-(3{,}880 - 4{,}200)}{\left(\frac{3{,}880}{13} - \frac{4{,}200}{49}\right)} = 1.50 \text{ mph.}$$

Average queue length is $l_Q = 1.50 \times 1.69 \text{ hr}/2 = 1.27$ miles (line 10, Worksheet 2A).

Time to dissipate the queue is $TDQ = (3{,}880 - 4{,}200) \times 1.69/(1{,}956 - 3{,}880) = 0.28$ hr.

Total duration of the queue $= T + TDQ = 1.97$ hr.

Deterministic Queuing

Deterministic queuing is the method followed by Worksheet 5A (Fig. 17) to determine average delay in cases of queuing in interrupted flow. It is convenient to conceptualize delay during interrupted flow as the sum of two contributing delays:

1. The delay due to demand for some period of time, and disregarding interruptions to the flow of vehicles.

2. The delay caused by the signal cycling from red to green and back to red during the duration of the queuing.

Delay Due to Demand Exceeding Capacity

Figure B-3 shows the initial form of the model when $v(t)$ is the vehicle arrivals over time t. In this method, the discrete nature of the cars is disregarded, and traffic, thought of as a continuous fluid that arrives at a uniform rate (q_1), is released at a rate q_m and builds a queue while arrival rate exceeds departure rate ($q_1 > q_m$). At a later time, arrival rates become less than departure rates ($q_2 < q_m$) and the queue dissipates. The vehicle arrival rate is proportional to the density and speed of the arriving vehicles. The back of the queue is moving upstream while demand exceeds capacity; thus, the relative speed with which arriving vehicles approach the queue is greater than their speed over the ground, and therefore the vehicle arrival rate at the queue increases.

In the following equations a maximum density per lane (k_m) is assumed for all queued vehicles. A value for k_m based on vehicle spacing of 22 ft/veh is 240 veh/mi/lane. The rate of vehicles arriving in the queue is:

$$q_1' = q_1 \; [1 + (q_1 - q_m)/(NL \times SPEED_u \times k_m - q_1)] \qquad \text{(B-7)}$$

in which $SPEED_u$ is the average speed of vehicles approaching from upstream and NL is the number of lanes upstream. The number of vehicles N_1 in the queue at the end of T_1 is:

$$N_1 = T_1(q_1' - q_m) \qquad \text{(B-8)}$$

the time TDQ required to dissipate the queue N_1 is:

$$TDQ = T_1(q_1 - q_m)/(q_m - q_2) \qquad \text{(B-9)}$$

The average number of vehicles in the queue is $N_1/2$ and the average time spent in the queue is:

$$TSQ = (N_1/2)/q_m \qquad \text{(B-10)}$$

The average length of the queue, l_Q, is:

$$l_Q = N_1/(2 \; k_m \; NL) \qquad \text{(B-11)}$$

Delay Caused by Signal Cycling

Delay due to the red and green phases during a signal cycle of an intersection is based on Newell's approximate method for computing delays and queues (*23*). The method described in the previous section assumes that vehicles are released from the congested section of roadway at a constant rate, q_m. In this method it is assumed that vehicles are blocked for a time, R, then released for a time, G, at a rate, S, and that S and q_m are related by:

$$S = q_m(R + G)/G \qquad \text{(B-12)}$$

Figure B-4 shows the model of an approach to an intersection where q_1' and q_m are as defined previously and $N_1/2$ is the average number of vehicles in the queue that are at least one cycle length away from the signal. Then

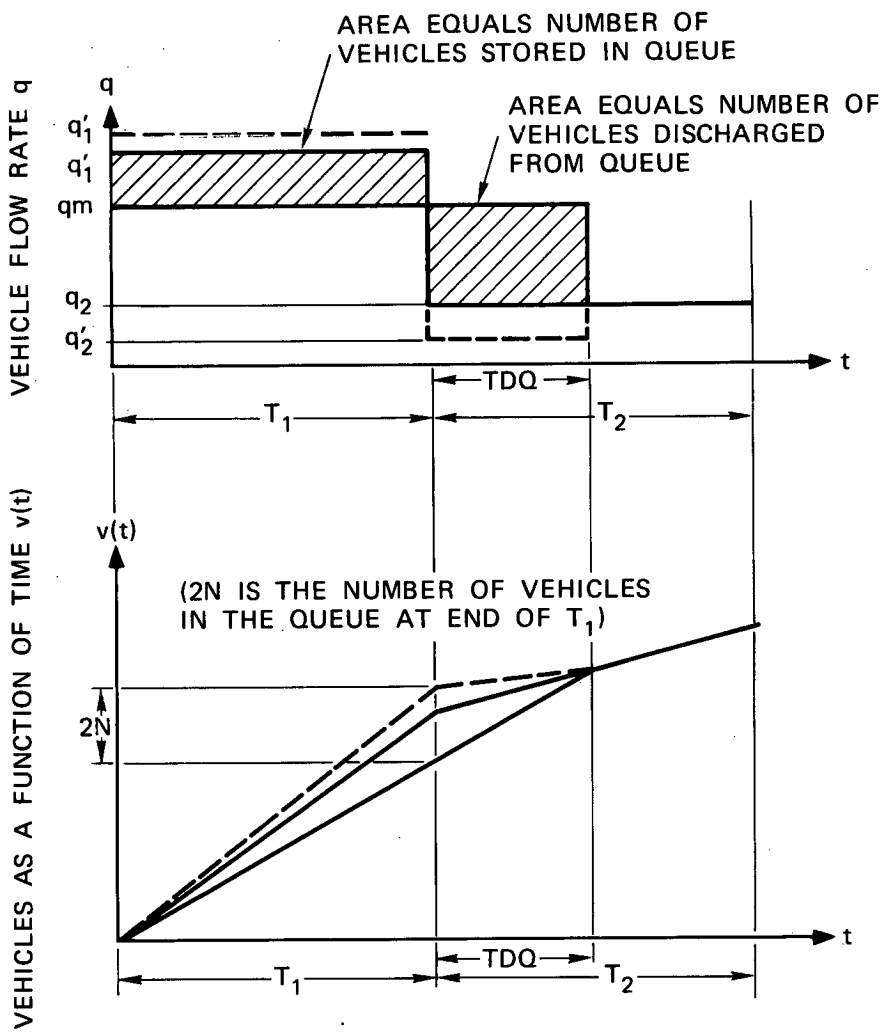


Figure B-3. Deterministic queuing model depicting delay when demand exceeds capacity.

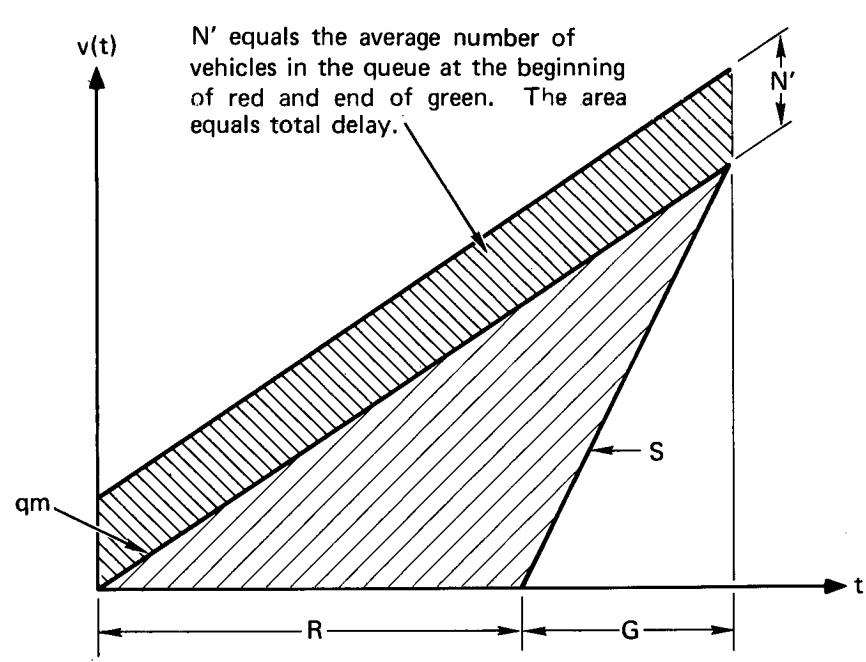


Figure B-4. Deterministic queuing model for one signal cycle.

the average number of vehicles in the queue during the time $R + G$ is:

$$N' = (N_1 + q_m R)/2 \qquad \text{(B-13)}$$

The average length of the queue is:

$$l_Q = N'/(NL \times k_m) \qquad \text{(B-14)}$$

and the average time delay per vehicle is:

$$AD = N'/q_m$$
$$= (T_1(q_1'/q_m - 1) + R)/2 \qquad \text{(B-15)}$$

In terms of the example worked out on Worksheet 5A (Fig. 17):

$q_1 = 1{,}900$ vph $\qquad q_2 = 1{,}000$ vph
$q_m = 1{,}830$ vph
$T_1 = 2$ hr
$R + G = 90$ sec
$R = 45$ sec
$NL = 3$
$SPEED_u = 30$ mph
$k_m = 240$ veh/mi/lane

The increased flow into the queue due to queue back-up is:

$q_1' = 1{,}900\,[1 + (1{,}900 - 1{,}830)/(3 \times 30 \times 240 - 1900)]$
$= 1{,}907$ veh/hr (line 9.2, Worksheet 5A)

The time to dissipate the queue is:

$TDQ = 2\ (1{,}900 - 1{,}830)/(1{,}830 - 1{,}000) = 0.169$ hr (line 12, Worksheet 5A)

The average delay per vehicle is:

$AD = [7{,}200\ (1{,}907/1{,}830 - 1) + 45]/2 = 173$ sec (line 11, Worksheet 5A)

AIR POLLUTION AND NOISE EFFECTS

Because another project funded by NCHRP was devoted exclusively to the problem of noise from highways, the work from that project was incorporated directly into the procedure for determining noise consequences presented in Chapter Four. Therefore, this section describes only the researchers' derivation of the methodology and assumptions that underlie the air pollution computations.

The most significant reduction in the amounts of air pollution resulting from motor vehicles ultimately will result from changes in the design of the motor vehicles, although some highway design features can result in reduced pollution. The process of improving the emission characteristics of motor vehicles is under way, given impetus by Federal legislation setting limits on the amounts of emission of specific substances. These limits are decreasing as time passes, so an approach to the forecast of emission levels resulting from alternative highway designs must recognize that the emission levels probably will decrease also. In keeping with this assumption, the suggested procedure for estimating air pollution emissions from operations over a given segment of roadway is as follows: (1) computation of the emission levels under the conditions that all the vehicles using the facility have emission levels equivalent to those required for a new 1969 automobile (this level of emissions is called reference emission); and (2) modification of this reference emission level to reflect the level of control requirements, the distribution of model year vehicles in operation in a given year, the increase in emission levels that probably will result as vehicles age, and the number of gasoline-powered trucks on the segment.

Reference Emissions

The emission data required for the computation of reference emissions over a uniform segment of highway have not been collected in a form and with sufficient number of samples to make a data bank readily available. The Office of Air Programs, U.S. Environmental Protection Agency, carried out a number of measurements in 1962 (*41*) that provide a statistical base for the determination of average emissions as a function of average route speed over a composite urban route. These data are not applicable here because the composite route is not representative of the route segments that are classified herein. Other statistical data have been taken as a result of testing by the Office of Air Programs and California's Air Resources Board. New vehicles are tested to determine whether they meet the requirements of the laws limiting emission. The test now consists of measuring the total amount of emissions over a programmed speed pattern, representing a trip over a composite urban route, using a dynamometer to simulate road loading. Conversations with personnel at the federal test facility at Ypsilanti, Mich., revealed that no data are taken to measure the emissions on individual portions of the route that would allow computation of emission for a large number of vehicles over that particular type of route segment.

California has published results (*42*) of emission tests performed on a large number of vehicles, but these tests again included a composite route and data were presented in terms of concentrations, rather than mass emissions per mile.

Because the researchers were unable to locate data in the proper form, the approach to the estimation of emissions is to compute emissions for a vehicle of a particular type and characterize the emissions from that vehicle as "typical" of the vehicles that will be built during the next few years. This is obviously made more difficult by the current controversy between automobile manufacturers and the Federal Government regarding the ability of the manufacturers to meet the requirements for 1975 model year vehicles in production vehicles. Such a controversy indicates that the technology may not exist for the level of control needed, and the technology that ultimately is used may result in emissions as a function of operating conditions that are significantly different from those developed here. The assumption is that the control technology will not change the patterns.

Controls of motor vehicles to reduce emission of air pollutants fall into three classes: (1) crankcase emission controls; (2) evaporative emission controls; and (3) ex-

haust emission controls. Crankcase emissions result from ventilation of the crankcase of a gasoline engine to the atmosphere. Since the 1963 model year, all U.S.-built vehicles have been required to have crankcase emission controls that ventilate the crankcase through the carburetor and into the engine, thus eliminating the crankcase emissions. Evaporation of fuel left in the carburetor results in a sizeable addition to the amount of hydrocarbons in urban air; controls of this evaporation will be required by 1975. However, the evaporation is not a function of the vehicle operation, and therefore emissions from this source are not affected by facility type, level of service, or other design factors. These emissions are not included in the analysis. Thus, exhaust emissions are the area of primary concern in this analysis.

Four approaches to the control of exhaust emissions are being used or are being discussed seriously as candidates for controls. These control types are: manifold air injection, engine modification, exhaust gas recirculation, and catalytic afterburning. Manifold air injection provides excess air in the exhaust manifold, where the heat and the air further oxidize unburned hydrocarbons and carbon monoxide. The engine modifications make the mixture leaner over some conditions, raise idling speed, retard the spark timing on deceleration and idle, and change design of the combustion chamber to reduce the emissions of hydrocarbons and carbon monoxide. These engine modifications are being used by most manufacturers in preference to the manifold air injection. Exhaust gas recirculation through the carburetor reduces combustion chamber peak temperatures and thus retards the formation of oxides of nitrogen. Finally, catalytic afterburners place a platinum catalyst in the path of the exhaust gases and induct air into the exhaust system in order to oxidize hydrocarbons and carbon monoxide. The platinum allows the oxidation to take place at relatively low temperatures for normal concentrations of these substances in the exhaust.

Other emission control approaches have been proposed, such as use of alternative powerplants: turbines, steam engines, Stirling Cycle engines, or electrical power. Modifications to the internal engine to incorporate stratified charge burning also have been proposed. Economics or performance factors place each of these proposals at a disadvantage to the gasoline engine, however, so it is difficult to forecast when, or if, such vehicles might make up a significant portion of the highway traffic.

Because such foreknowledge is lacking, the emission estimating procedure assumes that during the next few years vehicles will achieve carbon monoxide and hydrocarbon control through use of the engine modification package and a catalytic afterburner, and will achieve nitrogen oxides control through exhaust gas recirculation. Such changes might affect economy or performance by use of richer fuel mixtures in the engine, constriction of air flow or exhaust flow, or improved fuel oxidation. Nevertheless, in the absence of definite information it had to be assumed in this analysis of air pollution (as well as in the analysis of user costs) that these changes will not significantly affect the performance or economy of vehicles of this era.

Emission characteristics of a vehicle using the assumed type of engine modification package were obtained for a vehicle manufactured by the Chrysler Corporation (*43*). The emission data are presented in concentration form (i.e., parts per million), so airflow was estimated and modified by fuel-air ratio data (*43*) presented to convert the concentrations to mass flow. The vehicle was equipped with an engine modification package designed to meet the 1969 requirements. Because most manufacturers are using this type of control, it is assumed that the vehicle is typical in that regard.

Tests performed on a number of vehicles of various engine sizes and manufacture showed the average emissions per mile vs engine size (Fig. B-5). Note that the typical vehicle, with a displacement of 318 cu in., falls near the center of the range. Table B-5 gives the prevalence of various engine sizes in California for 1966, which confirms that this engine size is reasonably typical of existing vehicles.

The emissions for stopping and changing speeds were computed under the assumption that slowing down and speeding up would occur under constant acceleration conditions. Emissions were computed from computed airflow and fuel/air ratios that would exist under the various speed and acceleration conditions. The emissions added due to speed changes were computed by applying the emission characteristics of the test vehicle to the pattern of speed changes analyzed in the running cost section of the report and characterizing the results by the best straight line that could be drawn through the emission values plotted against v/c ratio for each type of facility.

Conversion of Reference Emissions to Emissions in Year X

The reference emissions provide an estimate of pounds of each pollutant generated by travel over the analysis segments if all of the traffic consisted of automobiles meeting the exhaust emission control requirements for the 1969 model year. The actual traffic will consist of a mixture of automobiles of various model years, having varying degrees of emission control and degradation of emission controls due to use, together with some trucks.

The requirements for exhaust emission control become increasingly strict over the 1970-1980 period. Table B-6 gives the emission limits in grams per mile and their rela-

TABLE B-5

DISTRIBUTION OF VEHICLES, BY ENGINE SIZE, CALIFORNIA, 1966

ENGINE SIZE CLASS	DISPLACEMENT RANGE (CU IN.)	VEHICLES IN OPERATION (%)
A	<140	8
B	140-200	9
C	200-250	30
D	250-300	25
E	300-375	17
F	>375	10

Source: Beckman et al. (*43*).

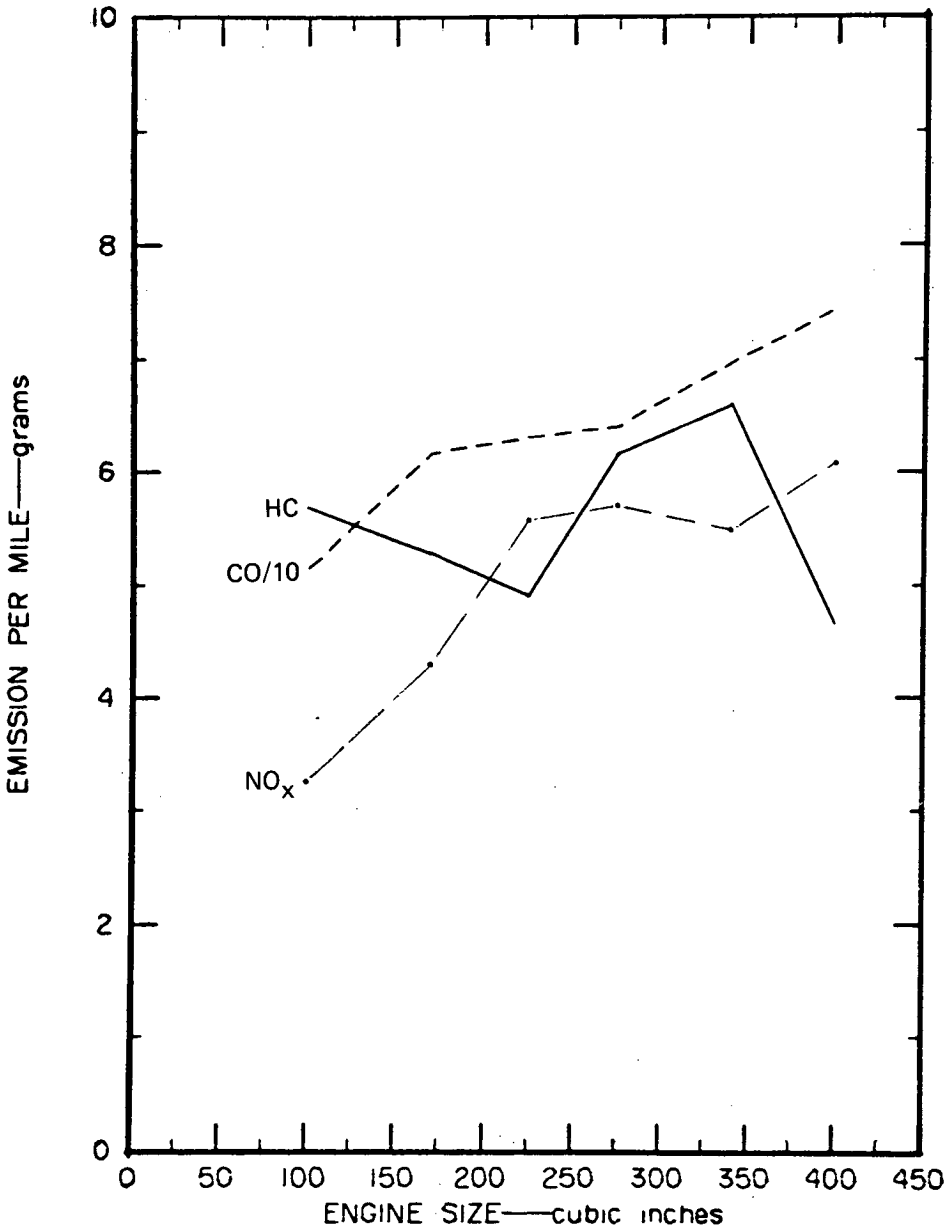


Figure B-5. Emission per mile as a function of engine size.

tive ratios for various model years as used in the conversion procedure (the ratios do not always track the emission limits exactly, owing to changes in test procedures). These relative values were combined with the fraction of the vehicle population represented by each model year as predicted by averaging the fractions for the years 1966-1970 (*44*).

The last step was to incorporate a degradation factor for the predicted increase in vehicle emissions due to wear, derived from data of Rose and Krostek (*45*). The degradation factor increases at a decreasing rate with average mileage driven, and hence with age:

VEHICLE AGE (YR)	DEGRADATION FACTOR
0	1.00
1	1.05
2	1.12
3	1.18
4	1.20
5	1.22
6	1.23
7	1.24
≥8	1.25

Use of the conversion factor assumes that the future controls will reduce emissions proportionally for all operating conditions. Use of a catalytic afterburner for control of carbon monoxide and hydrocarbons will result in emissions that approximate this assumption.

The emissions for single-unit gasoline trucks were estimated to be 2.5 times those of the reference automobile under all conditions. It was assumed that the pollution emission per horsepower-hour for the truck engine would be slightly lower than that of the passenger automobile because the truck engines run at lower engine speeds—hence, there is more time for combustion to take place. If the truck generated emissions in proportion to horsepower expended, the ratio would approximate 3.0, because the gasoline truck used in this analysis weighs three times as much as the passenger auto. The ratio was adjusted to 2.5 to account for the slightly lower emissions.

Diesel trucks have low emissions of hydrocarbons and carbon monoxide and nitrogen oxides; thus, they are not included in this analysis. The primary pollutant from diesel engines is smoke, which is coming under strict control.

ACCIDENT COSTS

The accident cost estimates in this report are derived from direct accident cost estimates from a 1958 Illinois study (*29*). In common with three other statewide accident cost studies that were sponsored by the Bureau of Public Roads, the Illinois study produced unit accident cost estimates that are not vulnerable to arguments about the validity of including indirect cost elements such as loss of future wages due to early death. Because it was the latest, most carefully planned, and most thorough of the four statewide studies, and because the results from the other studies vary considerably (*46*), this report relies exclusively on the Illinois study.

Some idea of the importance of the major accident cost components may be obtained from the following summary for the three degrees of traffic accident severity identified in the Illinois study.

	PROPORTION OF TOTAL COSTS BY DEGREE OF SEVERITY (%)		
ACCIDENT COSTS	FATAL	NONFATAL INJURY	PROPERTY DAMAGE
Property damage	16.9	31.3	95.9
Treatment of injuries	12.6	15.0	—
Value of wages lost	7.7	13.4	0.8
Loss of vehicle use	0.8	0.6	2.0
Legal and court costs	18.8	16.7	0.9
Damage awards in excess of known costs	42.3	23.0	0.4
Total	100.0	100.0	100.0
Proportion of total accident costs (%)	3.3	51.2	45.5

TABLE B-6

AUTOMOBILE EMISSION LIMITS FOR VARIOUS MODEL YEARS

	ACTUAL OR PERMISSIBLE EMISSION (GM/MI)			EMISSION FACTOR USED (1970=1.0)	
MODEL YEAR	HC	CO	NO_x	HC AND CO	NO_x
1967 and earlier	13.6	117	4.0	3.5	1.0
1968 and 1969	5.1	59	4.0	1.5	1.0
1970	3.4	39	4.0	1.0	1.0
1971-72	3.4	39	4.0	1.0	1.0
1973-74	3.4	39	3.0	1.0	0.75
1975	0.41	3.9	3.0	0.1	0.75
1976-79	0.41	3.9	0.4	0.1	0.1
1980 and later	0.20	2.0	0.2	0.05	0.05

Source of Data

The 1958 direct average accident costs were first obtained by dividing the costs for urban and rural roads (*29*, p. 91, Table C1-01.08-1) by the number of such accidents (*29*, p. 13, Table A1-01.08-1).

Although it would have been desirable to estimate average accident costs separately for freeways and surface roads (and also perhaps for other subcategories of roads and streets), the foregoing categories were not useful for such purposes. Furthermore, the number of observations in some categories (such as fatal accidents on divided highways) was too small to establish a statistically reliable average cost for that category. In any case, averages for the rural and urban categories were found to represent averages for the various subcategories reasonably well; and there were clearly larger costs (from 34 to 55 percent greater) in all three severity classes for rural accidents compared with urban accidents. The unadjusted average costs, which include costs for unreported accidents, were as follows:

	UNADJUSTED AVERAGE COSTS ($)		
AREA	FATAL	INJURY	PROPERTY DAMAGE
Urban	5,426	1,147	136
Rural	7,271	1,780	186

Updating to 1970 Price Levels

The two components of the CPI (consumer price index) believed to correlate most closely with accident costs are automobile insurance, for fatal accidents and nonfatal injury accidents, and automobile repairs and maintenance. These indexes stood at 181.9 and 143.5, respectively, in June 1970, compared with a 1957-59 base of 100. Because the base years of the CPI span the year 1958, in which the Illinois study was conducted, these indexes were used directly for updating purposes.

Insurance Overhead

The foregoing averages were first multiplied by 1.4 to include an approximation of insurance overhead costs. The 1.4 factor for inclusion of insurance was based on a comparison of total Illinois direct accident costs with 1958 auto insurance overhead for vehicles registered in Illinois.

The rationale for adding vehicle insurance overhead to accident costs, even though it was omitted from the direct cost estimates of the Illinois study, is that insurance overhead tends in the long run to be a relatively constant proportion of total premiums (although during a period of rapid increase in average claim costs, insurance overhead percentages and profit margins tend to decline because rate increases are slow to obtain). The long-run relative constancy of the overhead percentage is probably a result of insurance industry and state insurance commission policies of basing rates on a fairly constant ratio of anticipated losses to total premiums earned. This policy is feasible, in part because a large part of insurance overhead consists of (1) costs that are proportional to claims, such as the cost of claim settlement; or (2) costs that are proportional to total premiums, such as agents' commissions. Furthermore, general rises in prices tend to affect both accident costs and insurance company overhead expenses, such as salaries. It is therefore believed that, over a reasonably wide range of accident costs, vehicle insurance overhead will vary proportionately.

A further reason for including insurance overhead in motor vehicle accident costs is that comparisons with accident costs of public transportation systems are thereby facilitated. For public systems (bus, rail, airplane, etc.), the accident costs will be measured directly by the out-of-pocket insurance premium cost of the carrier, a figure that includes insurance overhead. Valid accident cost comparisons of public transportation systems with the highway system require, for consistency, the inclusion of insurance overhead in the highway system costs also. The widespread and increasing emphasis on comparisons of private motor vehicles with other forms of transportation makes this consistency highly desirable.

Procedure for Estimating Unreported Accidents

There is little doubt that some approximation of unreported accidents is necessary, because unreported accidents accounted for more than 40 percent of total costs in the Illinois accident cost study (*47*).

The Illinois study findings that 75 percent of all accidents were unreported would mean at least 80 percent of property-damage-only accidents were unreported, based on the fact that nearly all of the unreported accidents involved only property damage. If this finding applied generally to other areas of the United States, reported property damage accidents should be multiplied by 5 to estimate the number of property damage accidents. However, a more conservative expansion factor would be suggested by the fact that (1) the average cost of repairing motor vehicle damage tends to increase with time, whereas minimum reporting requirements usually have stayed fixed at levels of from $50 to $100, and (2) accident reporting procedures tend to improve over time.

A recent study by the California Division of Highways (*48*) indicates that even where state highway police make an effort to report all accidents that come to their attention, only about 40 percent of all property damage accidents on California state highways within state highway police jurisdiction are reported. And on state highway sections under city police jurisdiction, the reporting level for property damage accidents is less than 30 percent. The California study also indicates that only 11.6 percent of single-vehicle accidents are reported (compared to almost four times that level for multivehicle accidents), which helps to explain the low over-all reporting level for property damage accidents.

Minimum legal accident reporting requirements presumably would affect the level of accident reporting. However, although there is no legal minimum requirement in California for reporting property damage accidents to police authorities *—whereas Illinois law sets a $100 minimum and Chicago ordinances set a $50 minimum—the reporting level for California state highways appears to exceed that encountered in the Illinois study. From this fact, as well as from the lower reporting levels for many California cities, it may tentatively be concluded that police diligence or other factors are at least as important as legal minimums in stimulating a high level of accident reporting.

Admittedly, the unreported accident picture is somewhat speculative, but based on the foregoing discussion it would appear reasonable to use a multiplying factor of at least 2.5 for increasing the number of reported property damage accidents (this assumes a 40 percent reporting rate) to include the number of unreported accidents. Even higher factors may be justified in certain cases (such as accident rates on local rural roads and urban streets), but data for development of more refined factors are not available.

It is appropriate to ask how differences in the factor for numbers of unreported accident would affect over-all accident costs. The answer is that total accident costs (all classes of accidents) are not very sensitive to changes of 0.5 or so in this factor, but the sensitivity varies depending on the proportion of total accident costs accounted for by property damage accidents. Results of a sensitivity check indicate, for example, that the changes in total accident costs due to use of a 3.0 factor instead of a 2.5 factor would range from +10 percent for local city streets (where property damage accidents account for 48.5 percent of total accident costs) down to +3 percent for rural freeways (where property damage accidents account for only 17.3 percent of total accident costs).

Consideration also was given to showing accident rates and costs exclusive of unreported accidents in Tables 9 and 10, because state and local accident statistics normally will exclude unreported accidents and hence would be more comparable. However, the researchers thought it best to show the situation as it is, as closely as possible, and let users of the report adjust their own statistics for comparability.

* The financial responsibility laws of California do require a special report when there is damage to property of any one person in excess of $100, but these reports are not filed with state or local police and are not included in reported accident statistics.

The source of the advice given in this report that a total reported accident incidence of less than 50 cannot be considered statistically reliable is an article on the statistical significance of changes in accident rates by Michaels (*49*), who states:

> Although the selection of a sample size of 50 is arbitrary, it should generally minimize any bias that might be imposed by the rare accident in which a large number of fatalities or injuries occurred. The probability of more than five deaths or injuries occurring in a single accident is very low. Consequently, with a sample size of 50 or more, these single cases will inflate the accident frequency rarely more than 10 percent.

The National Highway Traffic Safety Administration (NHTSA) currently is conducting work on the costs of motor vehicle accidents. A preliminary report of this work, "Societal Costs of Motor Vehicle Accidents" (Apr. 1972), suggests much higher levels of accident costs than those identified in this report, primarily due to inclusion of the total discounted value of foregone earnings (without deduction of consumption expenditures). Time has not permitted an appraisal of the validity of this and other procedures followed in the NHTSA analysis.

APPENDIX C

PROJECT SELECTION CRITERIA

BUDGETARY POLICY OBSERVATIONS AND ASSUMPTIONS

The question of highway construction budget policies is relevant to project selection criteria. For this appendix, the following budget policy observations and assumptions were made:

1. Highway budgets are fairly well known for several years in advance because forecasts of gasoline taxes and other revenues can be made with reasonable accuracy. The total funds anticipated may be subdivided in various ways, such as along geographical, functional, or system lines; each such subdivision constitutes a separate constraint for advance planning purposes, and only construction projects within the same budgetary subdivision should be considered in direct competition with each other.

2. The major problems in highway priority determination lie beyond the current fiscal year and the subsequent fiscal year, because most construction projects for those two years will already have binding commitments. It is therefore the subsequent "advance planning program" on which priority determination should focus. For analysis purposes, it appears most convenient to take successive 3- to 5-year "budget planning periods" within, say, a 6- to 20-year advance planning program,* and consider all potential construction projects in competition for funds within that period by setting the zero year for the economy studies midway in the budget planning period. Projects with less desirable ratings then can be considered for funding in the succeeding budget period, and so on to the end of the advance planning program.

Very large projects, which have to be funded and constructed over a period of 2 or more fiscal years, may sometimes usefully be divided into sections for economic analyses. This may create problems with a project of moderate over-all priority that contains some very high- and very low-priority sections. Normally, the logical approach would be to schedule the high-priority sections early and either drop or defer the low-priority sections, unless the high-priority sections depend on the low-priority sections as traffic connections, or feeders, or in some similar way. Such problems could be avoided if sections are chosen for analysis purposes that are relatively independent of each other physically. Highly interdependent sections must be considered together, as one project, for economy study purposes.

The foregoing budget policy assumptions are believed to correspond reasonably well with the way that most state highway agencies proceed with the development of their advance planning programs (except that economic analysis is not commonly a part of the priority determination process at present). Naturally, the farther off a given year is in the advance planning period, the more tentative and subject to change is the selection of projects for that year. Changes in scheduling as a remote year draws closer to the budget year will arise for many reasons (e.g., political, financial, workload leveling) but none of these changes negates the basic value of the advance planning program as an aid to orderly consideration of potential highway construction projects.

* The suggested range for the budget planning period is based on judgment only. One consideration is that the longer the budget period used, the greater is the risk that low-priority projects that have rapid growth rates and that are scheduled late in the period might in fact have significantly higher benefit/cost ratios if their zero years were located toward the end rather than at the middle of the budget period. Another consideration is the masking effect of multiyear budget periods on evaluation of incremental investments, as explained later in this appendix under "Example of Postponement Analysis."

HIGHWAY ECONOMY STUDIES AND ADVANCE PROGRAMMING

Current state methods of establishing priorities for competing state highway construction projects rely almost exclusively on noneconomic methods. The most widely used method of state highway priority determination is some variation of "sufficiency ratings" involving the comparison of several important road or traffic variables with stated standards of adequacy on a numerical scale that is weighted subjectively according to the importance ascribed to each variable. Thus, one state defines standards for "dependability," "facility of movement," and "safety" on a ten-point scale, and rates the relative sufficiency of each road section according to estimates of remaining surface life (for dependability), operating speed during the design hour (for facility of movement), and number of accidents per mile (for safety). In this case, the interpretation of the resulting three-number rating places greatest emphasis on the facility of movement rating.

The following important parallels and contrasts exist between highway sufficiency rating procedures and the type of economic analysis of an entire highway construction program that is possible through use of this manual:

1. The results of either sufficiency ratings or economy studies may become part of the input to the extremely complex activity of "advance programming," as that activity is described by Granum and Burnes (*50*, p. 23).
2. Sufficiency ratings and economy studies both typically use basic highway data on speed, accident rates, and road resurfacing expectancy; but whereas sufficiency ratings convert the basic data into index numbers, economy studies convert the data into transportation costs or dollar equivalents.
3. Given two equally deficient sections of highway, and a budget limitation that prevents improvement of both sections at the present time, sufficiency ratings can make no contribution toward a decision between the two improvements, whereas the results of an economy study would help by giving the relationship of estimated dollar benefits to the estimated dollar costs of making each improvement.

The last point seldom appears to be considered in making up lists of highway "needs," because lists of needs are typically based on achievement of certain road standards without specific regard to whether the costs of a particular highway improvement bear a reasonable relationship to the road user benefits obtained by the outlay.* It has been argued in the past that economy studies lack the precision necessary to use them for comparison of the alternative routes of a given project. However, economy studies should be at least as reliable as sufficiency ratings for such comparisons, especially if the resulting economic indexes are not taken as absolute indicators apart from intangible considerations not readily translatable into dollars.

The following sections describe decision rules for use of the benefit/cost ratio in highway economy studies.

Current Practices for Interpretation of Benefit/Cost Ratio

The AASHO Red Book (*2*) gives three inconsistent criteria for interpreting the benefit/cost ratios that result from highway economy studies, as follows:

Criterion 1:

> To make analyses for two or more alternatives of highway improvement, a benefit ratio is calculated for each alternate compared to the basic condition. The indices for the several alternates indicate their relative merit as regards road user benefits (*2*, p. 14).

Criterion 2:

> A benefit ratio less than one indicates that in a road user benefit sense the basic condition is to be preferred over the alternate improvement (*2*, p. 28).

Criterion 3:

> There may be advantage [where several alternate locations or designs are being compared] in a second-step benefit ratio calculation to analyze the expenditure of additional increments of capital cost with resultant gain in benefits (*2*, p. 151).

Examples in the AASHO Red Book generally follow Criterion 1, ignoring the other two criteria. For instance, Example 6 (*2*, pp. 40-44) compares two levels of highway improvement, termed "Plan 1" and "Plan 2," for which the financial data given in Table C-1 are estimated.

The comparison of Plan 2 with Plan 1 in the final line of Table C-1 is not given by the Red Book, which concludes its example with the statement that "it appears that Plan 1 is more desirable than Plan 2 and should be selected for construction." By Criterion 3, however, as illustrated in the Red Book (*2*, App. B), Plan 2 should have been chosen, because the extra cost of Plan 2 over Plan 1 would be more than offset by the incremental benefits of Plan 2 compared with Plan 1.

It is generally acknowledged, even though it is far from universally practiced, that comparison of *incremental* benefits and costs as suggested by Criterion 3 is the only appropriate method for economy studies. However, even Criterion 3 is not a complete statement of the criterion problem. A full description of proper investment criteria has to allow for projects in which the least-cost alternative has a benefit/cost ratio of less than 1; in such cases, the incremental benefit/cost ratio of the higher cost alternative is not relevant. A full statement of criteria also must distinguish between (1) the simple case where no budget constraint is involved, and (2) the more complex but considerably more common case of a fixed investment budget. Proper investment decision rules for these two cases are discussed in turn in the following.

* For purposes of illustration, assume that road section M through a mountainous region is rated equally as deficient as a certain rural road section S, of equal length, through a rapidly growing suburban area. Section M could conceivably require several times the investment of section S to remove curves and grades, or to add lanes for expected increases in traffic, or otherwise to improve the roadway to a predetermined level of sufficiency. How can the priorities of work on the two sections be compared, taking expected traffic loads and road user benefits into account? It may also be relevant to ask how the merits of different levels of improvement on one or both sections compare. Sufficiency ratings by themselves cannot cope with such questions.

Decision Rules for First Case: Two Alternatives and No Budget Constraint

The case of two alternatives and no budget constraint is similar to Example 6 (*2*, pp. 40-44), with a least-cost alternative and a higher-cost alternative. The three possible decisions that may be taken in a given budget period and the correct investment rules for reaching each decision are summarized in Table C-2.

The left-hand column of Table C-2 lists the three benefit/cost ratios that must be obtained: the least-cost and the higher-cost alternatives versus doing nothing (ratios A and B), and the higher-cost versus the least-cost alternative (ratio C). The first two sets of benefit/cost ratios can be followed through the table: If ratio A $\geq$ 1, and ratio C $<$ 1, the right decision is to do the least-cost alternative; if ratio A $\geq$ 1, and ratio C $\geq$ 1, the right decision is to do the higher-cost alternative. In neither case is ratio B relevant to the question. As indicated, ratio B does become relevant when ratio A $<$ 1. In that event, the higher-cost project must stand on its own feet, so to speak. The deci-

TABLE C-1

SUMMARY OF FINANCIAL DATA FROM EXAMPLE 6

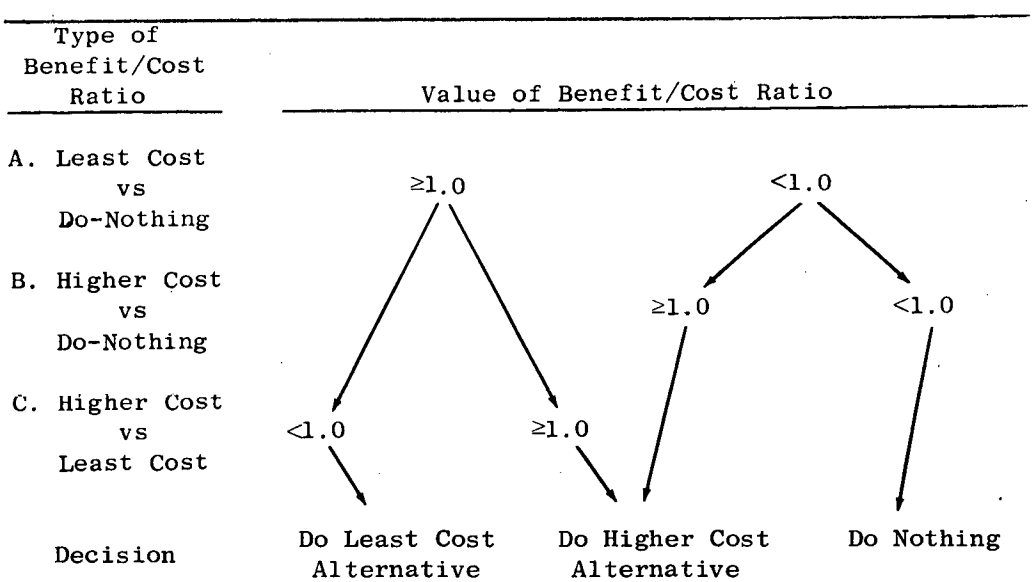
CONDITION	ANNUAL ROAD USER COSTS ($1,000)	ANNUAL HIGHWAY COST ($1,000)
Existing roads	$5,730	$ 46
Plan 1	4,760	234
Plan 2	4,697	264

CONDITION	ANNUAL INCREMENTAL BENEFITS ($1,000)	ANNUAL INCREMENTAL COSTS ($1,000)	B/C RATIO
Plan 1 vs existing roads	$ 970	$188	5.2
Plan 2 vs existing roads	1,033	218	4.7
Plan 2 vs Plan 1	63	30	2.1

Source: AASHO (*2*, pp. 40-44).

TABLE C-2

INVESTMENT DECISION RULES FOR THE CASE OF TWO ALTERNATIVES AND NO BUDGET CONSTRAINT



Procedure

Step 1: Obtain the three types of benefit/cost ratios identified in the first column.

Step 2: Compare each ratio successively with a benefit/cost ratio of 1, moving vertically down the arrows on the table to the correct decision at the bottom. (Note that the arrows bypass ratios that are not applicable.)

sion then will be to do the higher-cost alternative if ratio $B \geq 1$, and to do nothing at all if ratio $B < 1$.

One important observation is that although the decision rules in Table C-2 are specified only for the case of a single project with two alternatives, the rules may readily be generalized to permit a ranking of any number of projects by comparing each project successively with all other projects, one comparison at a time. However, such a procedure becomes extremely burdensome for the large number of projects that are usually encountered in state highway programs. For a large number of projects, the abbreviated procedures described in the next section are suggested.

Decision Rules for Second Case: Ranking a Series of Projects Within a Limited Budget

If all highway projects being considered for funding within a given budget period have only a single proposal for improvement, the economically optimum set of projects may be selected by simply ranking the projects in order of descending benefit/cost ratios until the budget is exhausted. The last project covered by the budget is defined as the marginal project, and its benefit/cost ratio is defined as the marginal benefit/cost ratio. Of particular interest is the fact that the marginal benefit/cost ratio becomes the cutoff or decision point, rather than a benefit/cost ratio of 1, as in the previous case where no budget limitation was involved. In a continuing highway program, of course, the decision not to do a particular project in one budget period does not prevent its reconsideration in a later budget period. In a later budget period, the project will be in competition with a different set of projects from those in the first period; in addition, traffic increases over time may give the project a higher benefit/cost ratio.

Although a simple ranking of benefit/cost ratios will work for projects involving only one proposal for improvement, it is far more usual for some potential highway projects to involve at least two alternative routes or levels of improvement (plus the "do-nothing" case). In that event, the first step in the selection procedure is to compute benefit/cost ratios for each alternative of a project compared with all other alternatives of the same project (including the "do-nothing" condition). The second step or series of steps is to follow the iterative procedure described in Chapter Four, which involves the selection of projects and of incremental investments with successively lower benefit/cost ratios until the budget is exhausted. In the course of this process, lower-cost project alternatives that were approved at a previous iteration may be displaced in a later iteration through approval of the incremental investment in a higher-cost alternative. This event may be illustrated by Example 6 (*2*, pp. 40-44), as summarized in Table C-1: if the marginal benefit/cost ratio (the benefit/cost ratio for the last project covered by a given budget) were less than the 2.1 incremental benefit/cost ratio of Plan 2, the $30,000 additional cost of Plan 2 would be approved. Plan 1, with a benefit/cost ratio of 5.2, would have been approved at a previous iteration, but would now be displaced by Plan 2.

The third and final step in the selection process when some projects involve two or more alternative improvement proposals is to consider the economic desirability of postponing any projects with incremental investments that have benefit/cost ratios greater than 1 but less than the marginal benefit/cost ratio for the period under construction. Again, in terms of the example in Table C-1, the question can be stated as follows: Assuming a marginal benefit/cost ratio higher than the 2.1 incremental ratio of Plan 2, would it be better from an engineering economy viewpoint to postpone the whole project in the expectation that the marginal benefit/cost ratio for some future budget period would be less than 2.1, thereby justifying Plan 2 at that future period?

To answer this question, it is first necessary to forecast the marginal benefit/cost ratios of succeeding budget periods. If a long-term supply of projects with benefit/cost ratios greater than 2.1 but less than 5.2 is anticipated, it obviously will not be profitable to postpone the entire project, because the lower-cost alternative (Plan 1) has a benefit/cost ratio of 5.2. On the other hand, if the marginal benefit/cost ratio is expected to decline to less than 2.1 in some future budget period, it may be profitable to postpone the project in order to justify carrying out the more expensive Plan 2. To determine the economic advantages of postponement requires analysis of total tradeoffs in costs and benefits that will be caused by postponing Plan 2. A simplified but reasonably realistic example of such a "postponement analysis" follows.

Example of Postponement Analysis

Although the benefit/cost ratios of Example 6 are retained to preserve some continuity of discussion, it is most convenient to change the benefit and costs from the Red Book example. Accordingly, the following assumptions are given:

1. In a certain highway agency, the six non-mutually-exclusive prospective construction projects, A, B, C, D, E, and F, are being considered for funding. For each project, the least-cost improvement is known as alternative 1 and the next higher cost improvement (if any) is known as alternative 2. Data on these projects are given in Table C-3. Project A has an alternative 1 with an initial cost of $10 million and a mutually exclusive alternative 2 with an added initial cost of $10 million (total of $20 million). The other five projects involve only the single alternative 1, with an initial cost of $10 million in each case.
2. Alternative 1 of Project A has a benefit/cost ratio of 5.2, and alternative 2 of Project A has an incremental ratio of 2.1—similar to the Red Book example in both instances. The other five projects have ratios of 4.0, 3.5, 2.0, 1.5, and 1.5. The highway agency anticipates a future supply of projects with benefit/cost ratios of at least 1.5 for many years to come.
3. The construction budget for the highway agency is $20 million for the next three budget periods, which means that two $10 million projects or one $20 million project can be funded in each budget period.

Given the foregoing facts, a highway economy analyst proceeds to rank the projects in order of priority, by budget

period. He notes that Projects B through F need simply to be scheduled in order of declining benefit/cost ratios, but that two choices exist with respect to Project A: it can be either undertaken at the $10 million level (alternative 1), in the first budget period, or postponed until the second budget period, at which time the 2.1 incremental benefit/cost ratio of its alternative 2 will exceed the 2.0 marginal benefit/cost ratio anticipated for the second budget period. The analyst makes a summary (Table C-4) of the effect of the foregoing choices on total benefits in each budget period (Options 1 and 2), including for good measure a summary of benefits from doing alternative 2 of Project A in the first budget period (Option 3).

It is evident from column 5 of Table C-4 that there are three significant economic effects of Option 2, postponing Project A until the second budget period and doing the more costly alternative, in comparison with Option 1:

1. A $17 million loss of benefits in Period I.
2. An $18 million gain of benefits in Period II.
3. A $5 million gain of benefits in Period III.

Assuming that each budget period is 1 year in length, conversion of the $23 million in gains of Option 2 in Periods II and III to equivalent present worths at 6 percent interest gives a total equivalent gain of $20.2 million in Period I. Taking the $20.2 million equivalent gain as "benefits" and the $17 million loss of benefits in Period I as "cost," a benefit/cost ratio of $20.2 ÷ $17.0 or approximately 1.2 is obtained. The ratio is close to but nevertheless greater than 1, and the analyst may therefore assert that there is a slight economic case in favor of Option 2—delaying Project A by a year in order to carry out its more expensive alternative. The comparison of Options 2 and 3 in column 6 shows that Option 2 is economically superior to 3; one would not spend $2 million in one year to get the same amount back next year.

It is of interest to observe that different results will be obtained in this example if the budget periods are assumed to be 5 years in length instead of 1 year. Discounting the gains in Periods II and III at 6 percent interest then results in an equivalent present worth in Period I of only $16.2 million, which is less than the $17 million loss of benefits in Period I (assuming end-of-period gains and costs in all cases).

Generalizing from this conclusion, it appears that the longer a project must be postponed for the incremental costs of its more expensive alternatives to be justified, the less likely is the postponement to be economically defensible.

A further observation on the postponement example is that column 6 of the summary of benefits shows there is no economic advantage in funding the higher cost alternative A2 in Period I, compared with funding it in Period II (because the gain of $2 million in Period II will be less than the loss of $2 million in Period I, when discounted to an equivalent present worth).

Combination of the three budget periods used in the example into a single period with a total fund availability of $60 million would have a masking effect on the data given in Table C-4, because the projects then would be evaluated on the basis of the same study period with the same zero year, and the three options given in Table C-4 would no longer exist. The masking effect of combining three 1-year budget periods into a single budget period is not significant, because the same conclusion would be reached in either case: do alternative A2 instead of A1. However, as noted in the previous paragraph, if each original budget period is 5 years long, the proper decision based on separate budget periods would be to do alternative A1; but the decision based on a combined 15-year budget period would be to do alternative A2. The conclusion suggested by this brief example is that the larger the number of years that is combined into a single budget period, the greater is the risk that masking will result in the approval of economically undesirable incremental investments. Pending further research on the optimum length of budget periods for investment programs of various types and sizes, the tentative position has been taken in this

TABLE C-3

BASIC DATA FOR ANALYZING POTENTIALLY POSTPONABLE HIGHWAY INVESTMENT

PROJECT	ALTERNATIVE	PRESENT WORTH OF BENEFITS ($MILLIONS)	INITIAL COST ($MILLIONS)	INCREMENTAL B/C RATIO
A	1	52	10	5.2
	2	73	20	2.1 [a]
B	1	40	10	4.0
C	1	35	10	3.5
D	1	20	10	2.0
E	1	15	10	1.5
F	1	15	10	1.5

BUDGET PERIOD	FUNDS AVAILABLE FOR NEW CONSTRUCTION ($ MILLIONS)
I	20
II	20
III	20

[a] This ratio is based on incremental benefits of $21 million divided by incremental benefits of $10 million.

TABLE C-4

BENEFITS FROM THREE OPTIONS

	BENEFITS ($ MILLIONS)				
				GAIN (+) OR LOSS (−)	
BUDGET PERIOD	OPTION 1: DO A1 IN PERIOD I	OPTION 2: DO A2 IN PERIOD II	OPTION 3: DO A2 IN PERIOD I	OPTION 2—OPTION 1	OPTION 3—OPTION 2
(1)	(2)	(3)	(4)	(5)	(6)
I	92	75	73	−17	−2
II	55	73	75	+18	+2
III	30	35	35	+5	—

study that periods of from 3 to 5 years in length are appropriate for the state highway construction programs under consideration.

Assessment

It may be objected that the foregoing example of a postponement analysis assumes more knowledge of the future on the part of the analyst than it is reasonable to expect. For instance, what if projects with benefit/cost ratios ≥ 2.5 appear in the second budget period, unforeseen until the decision to postpone alternative A2 has been made? One possible reply to this objection is that all economic analysis is based on assumptions about future events, and decisions must be reevaluated when the predictions on which they were based turn out to be significantly erroneous. The firmness of commitments to the planned time schedule and design for a project usually increases with the proximity of the project, but until the actual construction contracts are let, some degree of flexibility might still remain in the hands of administrators to adapt the highway design and schedule to important changes in the estimates on which the advance planning program was based.

Speaking more generally about the entire procedure of selecting projects by their benefit/cost ratios, it must be clearly understood that an economic ranking of projects is only as valid as the estimates underlying the economy studies, especially the estimates of value of time, average daily speeds, opportunity cost of capital, and initial highway cost. Due care needs to be exercised to use the most reliable sources for these important variables, but engineering precision cannot be expected. A caution should also apply to the interpretation of intangibles or nonmarket values and "spillover" costs connected with highway construction projects, because it is the job of the engineering economy analyst to identify any intangibles and spillover costs that influence the desirability of the project. A ranking by economic desirability has no validity until the differences between projects that cannot be converted into dollars are taken into account in some way in the decision-making process. Air and noise pollution are examples of such effects. (See Refs. *6* through *11* and *51* for suggestions on evaluating other community impacts.)

INCLUSION OF HIGHWAY MAINTENANCE COSTS IN NUMERATOR

Throughout this appendix it has been assumed that highway maintenance cost reductions are included in the numerator of the benefit/cost ratio, together with residual or terminal value. One common treatment of maintenance costs is to add them to construction costs in the denominator. The justification for this treatment of maintenance costs is usually that both construction and maintenance costs are constrained by the budget limitations of a highway agency, so that the proper ratio to maximize is total future benefits vs total highway agency costs (all reduced to equivalent present worths or annual costs, of course). However, this argument assumes a fixed future highway budget, whereas the researchers would maintain that when projects of high economic merit more than exhaust available highway funds, then (in the medium-to-long run) future fuel taxes could justifiably be increased.

Another reason for including Δm (present worth of annual highway maintenance cost reduction) in the numerator instead of the denominator of the benefit/cost ratio is to facilitate ranking projects in order of priority by descending benefit/cost ratios. The effect of the location of maintenance costs in the formula on its potential as a ranking criterion may be illustrated by the following simplified example: Assume that the three non-mutually-exclusive projects X, Y, and Z are being considered for inclusion in a given budget, and that it is desired to rank them in order of priority by means of their benefit/cost ratios. Lines 1, 2, and 4 of Table C-5 give the pertinent data for each of the projects, in comparison with the existing highway at each site.

Because each project requires the same investment of \$10, the relative savings in transportation costs created by each project will be an index of the project's comparative desirability according to the basic criterion of minimizing the present worth of highway transportation costs within a given highway construction budget. It is evident from the transportation cost savings in line 5 of Table C-5 that the three projects' relative order of economic merit is X, Y, Z, owing to their respective savings of \$90, \$80, and \$72. However, a comparison using benefit/cost ratios with maintenance costs added to initial highway costs, as in line 6, will produce exactly the opposite ranking: Z, Y, and X. A correct ranking solution using benefit/cost ratios is possible by first deducting maintenance costs from future benefits, as given in Table C-6.

It has been argued in the past that the benefit/cost ratio is not appropriate for ranking a series of investments, and should be used only for comparing two alternative investments. In Table C-5 the following results would be obtained from such a comparison:

1. Incremental benefits of X compared with Y = \$20.
2. Incremental costs of X compared with Y = \$10.
3. Benefit/cost ratio of X compared with Y = \$20/\$10 = 2.0.

Granted that the foregoing incremental benefit/cost ratio of 2.0 correctly selects project X over project Y, the question may still be raised: Is it not simpler and just as valid to rank projects X and Y by the method of Table C-6? Use of the benefit/cost ratio for ranking investments that include sets of mutually exclusive alternatives should, of course, be accompanied by an iterative procedure for handling the incremental investment in the mutually exclusive alternative sets, as described in Chapter Seven.

In addition to making project rankings feasible, having only the initial investment in the denominator seems intuitively more convenient. Each increase of 1 in the benefit/cost ratio can then readily be recognized as an additional return of 100 percent of the initial investment, and smaller changes in the ratio will have some consistent meaning relative to the original investment. Also, if one's objective is (as has been assumed) to maximize the excess of transportation benefits over transportation costs that will

TABLE C-5

COMPUTATION OF BENEFIT/COST RATIO BY CONVENTIONAL METHOD

ITEM	COSTS AND BENEFITS ($), BY PROJECT X	Y	Z
1. Initial cost (Δh)	10	10	10
2. Maintenance (present worth)	20	10	2
3. Total cost (line 1+line 2)	30	20	12
4. Benefits ($V\Delta t+\Delta u$)	120	100	84
5. Transportation cost savings (line 4—line 3)	90	80	72
Benefit/cost ratio, conventional menthod (line 4/line 3)	4.0	5.0	6.0

TABLE C-6

COMPUTATION OF BENEFIT/COST RATIO WITH MAINTENANCE COSTS IN NUMERATOR

ITEM	BENEFITS AND COSTS ($) BY PROJECT X	Y	Z
1. Benefits ($V\Delta t+\Delta u$)	120	100	84
2. Maintenance (present worth)	20	10	2
3. Net benefits (line 1—line 2)	100	90	82
4. Initial cost (Δh)	10	10	10
Benefit/cost ratio (line 3/line 4)	10.0	9.0	8.2

result from a present outlay, the present outlay alone must appear in the denominator when the benefit/cost ratio is used as an index for ranking competing proposals. As Table C-6 indicates, adding maintenance costs to construction costs will prejudice projects with a high ratio of maintenance to construction costs, even though their incremental future benefits are more than enough to offset their higher maintenance costs.

The preceding remarks apply equally to the treatment of residual value, which is equivalent to a positive cash flow so far in the future that it would be illogical to deduct it from initial construction costs anyway.

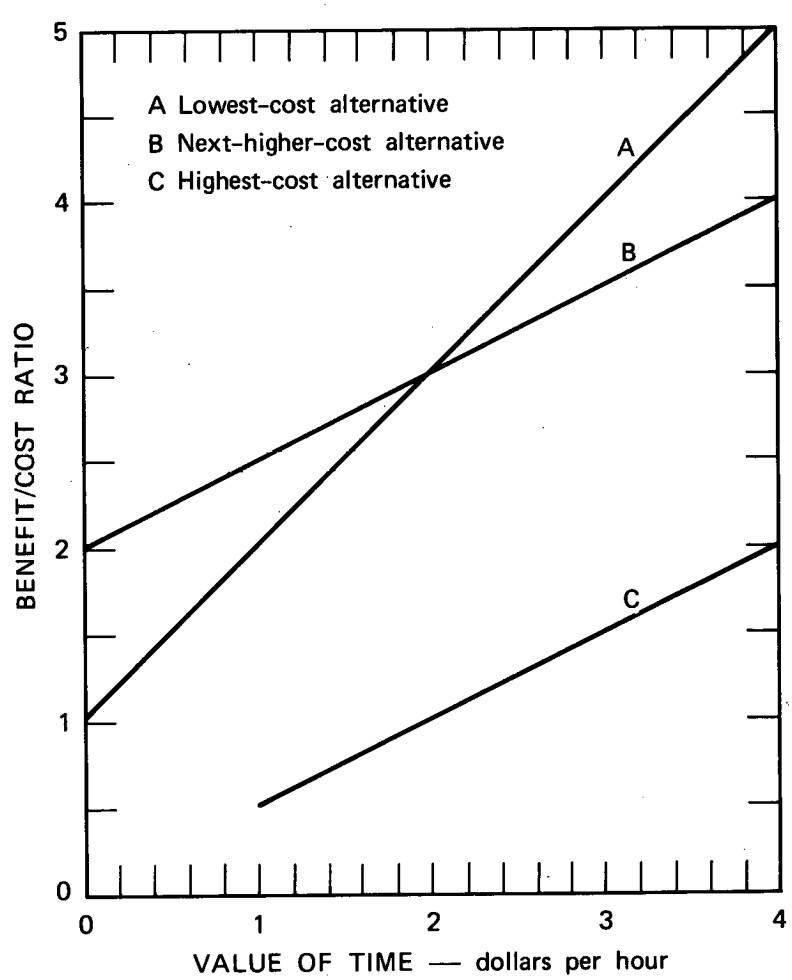


Figure C-1. Relation of value of time to benefit/cost ratio.

COST OF TIME

In addition to the benefit/cost ratio and rate of return on investment, Worksheet 10 provides for an index called the cost of time. The cost of time can be defined as the actual cost of providing time savings on a specific project, as contrasted with the value of time that is imputed to such savings based on travelers' willingness to pay. To find the cost of time, the value of time in the benefit/cost ratio is solved for, by dividing the *net increase in the present worth of highway and user costs* by the *hours of time savings for the project.* A negative cost of time indicates that total highway costs (construction costs plus the present worth of maintenance costs and residual value) are more than offset by the present worth of user costs; a positive value of time indicates the opposite.

In analyses of the economy of a proposed highway facility, the dollar value of passenger-car travel time reduction frequently constitutes 50 percent or more of total project benefits. Decisions among competing proposals may therefore be highly sensitive to the specific hourly value that is used.

Figure C-1 shows the relationship between the cost of time, the value of time, and the benefit/cost ratio. The dashed lines for projects A, B, and C show the benefit/cost ratios that would be obtained for each project as the assumed value of time is varied from $0 to $4. The cost of time for each project is shown where its line crosses the 1.0 benefit/cost ratio lines: $2 for project C, zero for project A, and some negative figure for project B. Note that for values of time between $0 and $2 per hour, project B would have a higher benefit/cost ratio than A, and C would be regarded as economically undesirable because its benefit/cost ratio is less than 1.0. For values of time greater than $2 per hour, A would have a higher benefit/cost ratio than B, and C would be regarded as economically desirable (though less desirable than either A or B).

The foregoing analysis of Figure C-1 would be exactly the same if it is assumed that lines A, B, and C, instead of representing three independent projects, represent the incremental investments in three mutually exclusive alternatives of a single project. For example:

1. Line A could represent the lowest-investment-cost alternative of a given project.
2. Line B could represent the incremental investment cost of the next-higher-investment-cost alternative of the project.
3. Line C could represent the incremental investment cost (above the investment cost of alternative B) of the highest-investment-cost alternative of the project.

APPENDIX D

BLANK WORKSHEETS

Worksheet 1

SUMMARY OF BASIC PROJECT INFORMATION

Project No. _____ Construction Period _____ Highway Type(s) _____

1. System, route, and location _____

2. Type of improvement _____

3. Description of alternative plans

4. Alternative					
5. Highway investment cost, thousands of dollars					
5.1 Land for right-of-way					
5.2 Earthwork, grading, and structures					
5.3 All other					
5.4 Total					
6. Annual maintenance cost, thousands of dollars					
7. Length, miles					
8.					
9.					
10.					
11.					
12.					

Worksheet 2

TRAFFIC DEMAND AND HIGHWAY CAPACITY

Project Number _____

• 1. Section	_____	_____	_____	_____	_____
• 2. Type of facility	_____	_____	_____	_____	_____
• 3. Design speed (mph)	_____	_____	_____	_____	_____
• 4. Length, miles	_____	_____	_____	_____	_____
• 5. Number of traffic lanes in one direction	_____	_____	_____	_____	_____
• 6. Lane width, feet	_____	_____	_____	_____	_____
• 7. Grade, percent	_____	_____	_____	_____	_____
• 8. Other factors affecting capacity					
8.1	_____	_____	_____	_____	_____
8.2	_____	_____	_____	_____	_____
8.3	_____	_____	_____	_____	_____
9. AADT, vehicles per day					
9.1 Total, Year 1	_____	_____	_____	_____	_____
9.2 Total, Year 20	_____	_____	_____	_____	_____
9.3 Per lane pair, Year 1 (9.1 ÷ 5.)	_____	_____	_____	_____	_____
9.4 Per lane pair, Year 20 (9.2 ÷ 5)	_____	_____	_____	_____	_____

10. Estimated one-way AHT (two-way AHT for 2-lane roads), vehicles per hour (9.3 and 9.4 to Figure 7 values x 5.)	Duration, Hrs/day* Year	Peak	Off-Peak	Peak	Off-Peak	Peak	Off-Peak	Peak	Off-Peak	Peak	Off-Peak
10.1	1	____	_____	____	____	____	____	____	____	____	____
10.2	20	____	_____	____	____	____	____	____	____	____	____
11. Percent Trucks											
11.1 Single unit				____	____	____	____	____	____	____	____
11.2 Truck combinations				____	____	____	____	____	____	____	____
11.3 Total, or average trucks				____	____	____	____	____	____	____	____
12. One-way section capacity (two-way for 2-lane roads), vehicles per hour				____	____	____	____	____	____	____	____
13. Demand/Capacity Ratio (10.1 and 10.2 ÷ 12)											
13.1 Year 1				____	____	____	____	____	____	____	____
13.2 Year 20				____	____	____	____	____	____	____	____

* Based on ____ hours per weekday and ____ per weekend (____ per year) for one-way peak period AHT, and ____ hours per weekday plus ____ per weekend (____ per year) for one-way off-peak AHT.

Worksheet 2A

QUEUEING IN UNINTERRUPTED FLOW

Project No. _____ Upstream section identification _____

Year _____ Bottleneck section identification _____

Time of Day _____

	Peak	Off Peak
1. Demand volume for bottleneck (W2, 10.1 or 10.2)	_____ veh/hr	_____ veh/mh
2. Time duration of volume (W2, 10.1 or 10.2)	_____ hrs	
3. Capacity of upstream section (W2, 12.)	_____ veh/hr	
4. Capacity of bottleneck (W2, 12.)	_____ veh/hr	
5. V/C ratios		
5.1 Noncongested subsection (W2, 13.1 or 13.2)*	_____	
5.2 Congested subsection (4. ÷ 3.)*	_____	
5.3 Bottleneck section (MIN (1.0, 1. ÷ 4.))*	_____	
6. Rate of queueing (1. - 4. of peak) (a positive value indicates an increasing queue)	_____ veh/hr	_____ veh/hr
7. Speed of vehicles through each section		
7.1 Upstream section (from Appendix A)*	_____ mi/hr	
7.2 Congested subsection (from Figure 9)*	_____ mi/hr	
7.3 Bottleneck section (from Appendix A)*	_____ mi/hr	
8. Density of vehicles using each section		
8.1 In upstream section (1. ÷ 7.1)	_____ veh/mi	
8.2 In congested subsection (4 ÷ 7.2)	_____ veh/mi	
8.3 In bottleneck section (1. ÷ 7.3)	_____ veh/mi	
9. Change in density in going from the upstream section to the congested section (8.2 - 8.1)	_____ veh/mi	
10. Average length of queue (congested subsection) (2. x 6. ÷ 9. ÷ 2)*	_____ mi	
11. Time required during off-peak to dissipate queue, in hours (6. of peak x 2./6. of off peak x -1)*		_____ hrs

* These results are utilized for Worksheet 3, lines 6, 8.1, 4, and 5.

Worksheet 3

TRAVEL TIME AND RUNNING COSTS

Project No. _____ Year ____

	All Vehicles		Passenger Cars		Trucks	
1. Section						
2. Facility Type						
3. Grade, percent						
4. Length, miles (W2, 4., or* W2A, 10.)						
	Peak	Off-Peak	Peak	Off-Peak	Peak	Off-Peak	Peak	Off-Peak
5. Duration of service volume period, hours per day (W2, 10., or* W2A, 11.)								
6. Volume/Capacity (V/C) ratio (W2, 13., or* W2A, 5.)								
7. One-way AHT (average hourly traffic) (W2, 10. or* 6. x W2, 12)								
8. Speed and time:								
8.1 Average tangent speed, mph (6. to Appendix A or Table 3 or* Figure 9)								
8.2 Hptm (Table 4)								
8.3 Hptv (4. x 8.2)								
9. Running cost factor, $ptvm (6. or 8.1 to Appendix A, or* 6. to Figure 13)								
10. Added delay from stops, etc.								
•10.1 Number of signals								
10.2 Average signal stops per vehicle (10.1 x W5, 16.)								
10.3 Added time for signals, hptv (10.1 x W5, 15. x .2778)								
•10.4 Number of stop signs								
10.5 Added hours per 1,000 stops (8.1 to Table 5)								
•10.6 Average hours stopped per 1,000 stop signs (__ sec./veh. x .2778)								
10.7 Total hours per 1,000 stop signs (10.5 + 10.6)								
10.8 Added time due to stop signs, hptv (10.4 x 10.7)								
10.9 Added time due to curves hptv (W4 Recap, 3. x 6.)								
10.10 Total added time, hptv (10.3 + 10.8 + 10.9)								
10.11 Idling time, hptv (10.3 - [10.2 x 10.5] + [10.4 x 10.6])								
11. Added costs from stops, etc.								
11.1 Stopping cost factor, $/1,000 stops (Table 5)								
11.2 Idling cost factor, $/hour (Table 5)								
11.3 Added cost due to stops, $ptv (10.2 + 10.4) x 11.1 + 10.11 x 11.2)								
11.4 Added cost due to curves, $ptv (W4 Recap, 7. + 8.)								
11.5 Grade cost factor, added $ptvm (3. and 9.1 to Appendix A)								
12. Time and cost ptv, speed, service level								
12.1 Total hptv (8.3 + 10.10)								
12.2 Total $ptv ([9.+11.5] x 4.+11.3+11.4)								
12.3 Average speed (12.1 ÷ 4., to Table 4)								
12.4 Level of service (6. or 12.3 to Table 1)								
13. Total traffic, millions (5. x ____ days/year x 7. ÷ 10^6)								
14. Total vehicle-miles, millions (4. x 13.)								
15. Travel Time, thousands of hours (12.1 x 13.)								
16. Running cost, thousands of dollars (12.2 x 13.)								

hptvm = hours per thousand vehicle-miles

* For queueing, bottleneck, or downstream sections only

Worksheet 4

ADDED TRAVEL TIME AND RUNNING COST FOR CURVES

Project Number _____ Year ____ ___ All Vehicles ___ Passenger Cars ___ Trucks

• Alternative, Section, & Curvature (degrees)	• Number of Curves	• Average Speed, mph		hptm Where Curve Speed Dominates	• Curve Mileage	Added Curve Cost Factors, Dollars ptvm	Added Cost per Thousand Speed Change Cycles, Dollars
		Tangent	Curve				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)

Recapitulation

1. Alternative and section					
2. Hours ptv where curve speed dominates (Σ col 5 x col 6)					
3. Total curve mileage where curve speed dominates*					
4. Average hptm on curves (2. ÷ 3.)					
5. Average hptm on tangents (W3, line 8.2)					
6. Added hptm for curves (4. - 5.)*					
7. Added cost for curves, dollars ptv (Σ col 6 x col 7)*					
8. Added cost for speed change cycles, dollars ptv (Σ col 2 x col 8)*					

* These results are utilized in Worksheet 3, lines 10.9 and 11.4.

Worksheet 5

INTERSECTION DELAY

Project No. _____ Intersection Identification _____

Year _____ Time _____

Intersection Approach Identification	(1) _____		(2) _____	
	Peak	Off-Peak	Peak	Off-Peak
1. Demand volume, veh/hr (W2, 10.)				
2. Demand volume duration, hrs (W2, 10.)				
• 3. Saturation flow, veh/hr (S)				
• 4. Effective green time of signal, sec (G)				
• 5. Cycle length of signal, sec (C)				
6. Green to cycle time ratio (λ) (4. ÷ 5.)				
7. Capacity of approach, veh/hr (3. x 6.)				
8. Degree of saturation (χ) (1. ÷ 7.) (if χ is greater than 1, do the queueing worksheet, W5A)				
9. Delay per vehicle, sec/veh (7. and 8. to Figure 16)				
10. Correction Factor, sec/veh (5. and 6. to Figure 16 insert)				
11. Average Delay per vehicle, sec/veh (9. + 10., or enter from W5A)				
12. Time to dissipate queue (if any) during Off-Peak period, hrs. (W5A, line 12.)				
13. Difference in delay between Peak and Off-Peak period, sec/veh (Peak 11. - Off-Peak 11.)				
14. Increase in average delay due to queueing that extends into Off Peak period, sec/veh (12. ÷ 2. x 13.)				
15. Average delay per vehicle, sec/veh (11. + 14.)*				
16. Proportion of vehicles that were stopped. MIN (1, (1 - 6.) ÷ (1 - 1. ÷ 5.)*				

* These results are utilized for Worksheet 3, lines 10.2 and 10.3

Worksheet 5A

QUEUEING IN INTERRUPTED FLOW

Project No. _____ Intersection Identification _____

Year _____ Time _____ Approach Identification _____

1. Demand Volume for peak hour (W5, 1.) _____veh/hr

2. Demand volume for off-peak hour (W5, 1.) _____veh/hr

3. Capacity of intersection during peak period (W5, 7.) _____veh/hr

4. Time duration of peak period (W5, 2.) _____hrs

5. Cycle length of signal during peak period (W5, 5.) _____sec

6. Effective green time during peak period (W5, 4.) _____sec

7. Speed of vehicles on the approach to the intersection during the peak period (W3, 8.1) _____mi/hr

8. Number of lanes of the approach (W2, 5.) _____lanes

9. Rate of arrival of vehicles into the intersection queue

   9.1 Density of vehicles per mile per lane when queued (240 veh/mi/lane assumes 22 ft/veh spacing in the queue) _____veh/mi/lane

   9.2 Adjusted arrival rate (1. (1 + (1. - 3.) ÷ (8. x 7. x 9.1. - 1.))) _____veh/hr

   9.3 Arrival rate to be used (1. or 9.2) _____veh/hr

10. Duration of interruption by signal (5. - 6.) _____sec

11. Average delay due to queue (MAX [((4. x 3600 x ((9.3 ÷ 3.) - 1) + 10.) ÷ 2), 5.])* _____sec/veh

12. Time to dissipate the queue built up during peak period (4. x (1. - 3.) ÷ (3. - 2.)* _____hrs

---

* These results are utilized for Worksheet 5, lines 11 and 12.

Worksheet 6

AIR POLLUTION

Project Number _____ Year 1

1. Section						
2. Facility type						
3. Length, miles (W3, 4.)						
	Peak	Off-Peak	Peak	Off-Peak	Peak	Off-Peak
4. V/C ratio (W3, 6.)						
5. Year 1 auto AHT, thousands (W3, 7.)						
6. Year 1 auto vehicle miles per hour, thousands (3. x 5.)						
7. Percent single unit trucks (W2, 10.1)						
8. Average tangent auto speed (W3, 8.1)						
9. Number of stops per vehicle (W3, 10.2 + 10.4)						
10. Idling time, hptv (W3, 10.11)						
11. Reference HC emissions for automobiles						
11.1 Steady speed factor (8. to Fig. 19)						
11.2 Speed changes factor (4. to Fig. 22)						
11.3 Running HC emissions, pounds per hr. (11.1 + 11.2 x 6.)						
11.4 HC per 1000 stops, pounds (8. to Fig. 20)						
11.5 HC emissions from stops, pounds per hr. (11.4 x 9. x 5.)						
11.6 HC emissions from idling, pounds per hr. (10. x 5. x 0.0087)						
11.7 Total reference HC emissions (11.3 + 11. 5 + 11.6)						
12. Reference CO emissions for automobiles						
12.1 Steady speed factor (8. to Figure 19)						
12.2 Speed changes factor (4. to Fig. 21)						
12.3 Running CO emissions, pounds per hr. (12.1 + 12.2 x 6.)						
12.4 CO per 1000 stops, pounds (8. to Fig. 20)						
12.5 CO emissions from stops, pounds per hr. (12.4 x 9. x 5.)						
12.6 CO emissions from idling, pounds per hr. (10. x 5. x 1.19)						
12.7 Total reference CO emissions (12.3 + 12.5 + 12.6)						
13. Reference auto NO_x, pounds (6. x 8.81)						
14. Reference single unit truck emissions, pounds						
14.1 HC (7. x 11.7 x 0.025)						
14.2 CO (7. x 12.7 x 0.025)						
14.3 NO_x (7. x 13. x 0.025)						
15. Total two-way emissions per hr. in Year 1, (19__) pounds						
15.1 Figure 23 HC & CO factor for Year 1 (____ x 2*)						
15.2 Figure 23 NO_x factor for Year 1 (____ x 2*)						
15.3 HC (11.7 + 14.1) x (15.1)						
15.4 CO (12.7 + 14.2) x (15.1)						
15.5 NO_x (13. + 14.3) x (15.2)						
	1 vs 0	2 vs 0	3 vs 0	4 vs 0		
16. Reduction in emissions from improvement alternatives vs. do-nothing case, pounds per hour						
16.1 HC (15.3, Σ Alternative 0 - Σ Alternative 1, etc.)						
16.2 CO (15.4, same as 16.1)						
16.3 NO_x (15.5, same as 16.1)						

* Omit this factor for any sections that are analyzed separately for each direction of travel.

Worksheet 7

NOISE IMPACTS

Project Number _____ Year 1

1. Section (or alternative)	_____					_____					_____				
	Fav-orable			Unfav-orable		Fav-orable			Unfav-orable		Fav-orable			Unfav-orable	
2. Degree of Impact	G	M	N	M	G	G	M	N	M	G	G	M	N	M	G
3. Residential Units (Number)															
3.1 Single-Family	___	___	___	___	___	___	___	___	___	___	___	___	___	___	___
3.2 Multi-Family	___	___	___	___	___	___	___	___	___	___	___	___	___	___	___
4. Schools (Number)	___	___	___	___	___	___	___	___	___	___	___	___	___	___	___
5. Churches (Number)	___	___	___	___	___	___	___	___	___	___	___	___	___	___	___
6. Hospitals (Number)	___	___	___	___	___	___	___	___	___	___	___	___	___	___	___
7. Office Space (Square Feet)	___	___	___	___	___	___	___	___	___	___	___	___	___	___	___
8. Other															
8.1 _____	___	___	___	___	___	___	___	___	___	___	___	___	___	___	___
8.2 _____	___	___	___	___	___	___	___	___	___	___	___	___	___	___	___
8.3 _____	___	___	___	___	___	___	___	___	___	___	___	___	___	___	___
8.4 _____	___	___	___	___	___	___	___	___	___	___	___	___	___	___	___

Note: G = great impact; M = moderate impact; N = no impact.

Worksheet 8

ACCIDENT COSTS

Project Number _____ Year _____ ___All Vehicles __ Passenger Cars __Trucks

1. Section or alternative	_____		_____		_____		_____	
	Peak	Off-Peak	Peak	Off-Peak	Peak	Off-Peak	Peak	Off-Peak
2. Accident rates per MVM								
2.1 Fatal	___	___	___	___	___	___	___	___
2.2 Nonfatal injury	___	___	___	___	___	___	___	___
2.3 Property damage*	___	___	___	___	___	___	___	___
3. Average cost per accident								
3.1 Fatal	___	___	___	___	___	___	___	___
3.2 Nonfatal injury	___	___	___	___	___	___	___	___
3.3 Property damage	___	___	___	___	___	___	___	___
4. Accident costs per MVM								
4.1 Fatal (2.1 x 3.1)	___	___	___	___	___	___	___	___
4.2 Nonfatal injury (2.2 x 3.2)	___	___	___	___	___	___	___	___
4.3 Property damage (2.3 x 3.3)	___	___	___	___	___	___	___	___
4.4 Total (4.1 + 4.2 + 4.3)	___	___	___	___	___	___	___	___
5. Billions of vehicle miles of traffic (W3, 14. x 10^{-3})	___	___	___	___	___	___	___	___
6. Total, thousands of dollars (4.4 x 5. x 2†)	___	___	___	___	___	___	___	___

* The indicated rates should include unreported accidents (multiply reported property-damage-only accident rate by 2.5 in the absence of a better estimate).

† Omit this factor for sections that are analyzed separately for each direction of traffic.

Worksheet 9

SUMMARY OF USER COST AND TIME REDUCTIONS

Project Number _____ Year _____ ___ All Vehicles ___Passenger Cars ____ Trucks

1. Alternative or section	_____	_____	_____	_____
2. Vehicle time, thousands of hours (Σ W3, 15., all sections with same traffic volume)	_____	_____	_____	_____
3. Running cost, thousands of dollars (Σ W3, 16., same sections)	_____	_____	_____	_____
4. Accident costs, thousands of dollars (Σ W8, 6., same sections)	_____	_____	_____	_____
5. Annual traffic volume, millions of vehicles (W3, 13., peak + off-peak)	_____	_____	_____	_____
6. Average costs and travel time:				
6.1 Vehicle time, hptv (2. ÷ 5.)	_____	_____	_____	_____
6.2 Running cost, $ptv (3. ÷ 5.)	_____	_____	_____	_____
6.3 Accident costs, $ptv (4. ÷ 5.)	_____	_____	_____	_____
7. Alternative pairs		_____		_____
8. Reduction in average costs and travel				
8.1 Vehicle time, hptv (6.1, Alternative 0-1, 0-2, etc.)		_____		_____
8.2 Running cost, $ptv (6.2, Alternative 0-1, 0-2, etc.)		_____		_____
8.3 Accident cost, $ptv (6.3, Alternative 0-1, etc.)		_____		_____
9. Average traffic volume, millions of vehicles (5. for alternative 0 + given alternative, ÷ 2)		_____		_____
10. Time and cost reductions (benefits)				
10.1 Vehicle time, thousands of hours (8.1 x 9. x 2*)		_____		_____
10.2 Running cost, thousands of dollars (8.2 x 9. x 2*)		_____		_____
10.3 Accident cost, thousands of dollars (8.3 x 9. x 2*)		_____		_____
10.4 Value of truck time if trucks are not analyzed separately, thousands of dollars (10.1 x W2, 11.3 x $ ____/hr)		_____		_____
10.5 Value of passenger car time, thousands of dollars ([10.1 x $____/hr] - 10.4)		_____		_____

* Omit this factor for any sections that are analyzed separately for each direction of travel.

Worksheet 10

ECONOMIC INDEXES

Project Number _____ ____ All Vehicles ____ Passenger Cars ____ Trucks

1. Alternative pairs	_____	_____	_____	_____
2. Year 1 cost and time value reductions, thousands of dollars				
2.1 User costs (W9, Σ 10.2 + 10.3 + 10.4 for all sections, both peak and off-peak)	_____	_____	_____	_____
2.2 Value of passenger car travel time (W9, Σ 10.5 for all sections, both peak and off-peak)	_____	_____	_____	_____
3. Year 20 cost and time value reductions, thousands of dollars				
3.1 User costs (same as 2.1)	_____	_____	_____	_____
3.2 Travel time value (same as 2.2)	_____	_____	_____	_____
4. Average annual increase in benefits				
4.1 User costs ([3.1 - 2.1] ÷ 20)	_____	_____	_____	_____
4.2 Value of travel time ([3.2 - 2.2] ÷ 20)	_____	_____	_____	_____
5. Present worth factors, for ____ years at __%				
5.1 Uniform series _____				
5.2 Gradient series _____				
5.3 Single payment _____				
6. Present worths, thousands of dollars				
6.1 User costs (2.1 x 5.1) + (4.1 x 5.2)	_____	_____	_____	_____
6.2 Highway maintenance cost reduction (5.1 x W1, 6.: Alternative 0-1, 0-2, etc.)	_____	_____	_____	_____
6.3 Residual value (5.1 x [W1, 5.1 + 0.5 x 5.2])	_____	_____	_____	_____
6.4 Subtotal (6.1 + 6.2 + 6.3)	_____	_____	_____	_____
6.5 Travel time value (2.2 x 5.1) + (4.2 x 5.2)	_____	_____	_____	_____
6.6 Total (6.4 + 6.5)	_____	_____	_____	_____
7. Highway investment cost (W1, 5.4, for given alternative)	_____	_____	_____	_____
8. Benefit/cost ratio				
8.1 Excluding travel time value (6.4 ÷ 7)	_____	_____	_____	_____
8.2 Including travel time value (6.6 ÷ 7)	_____	_____	_____	_____
Optional Indexes				
9. Rate of return				
9.1 Excluding travel time value (interest rate at which 6.4 = 7.)	_____	_____	_____	_____
9.2 Including travel time value (interest rate at which 6.6 = 7.)	_____	_____	_____	_____
10. Cost of time ([7 - 6.4] x $____ ÷ 6.5)	_____	_____	_____	_____

Worksheet 11

SUMMARY OF BENEFIT/COST COMPARISONS

Budget Period__________

1. Single-alternative projects (1-0)

Project No. __________ __________ __________ __________ __________
Benefit/Cost Ratio ______ ______ ______ ______ ______

Project No. __________ __________ __________ __________ __________
Benefit/Cost Ratio ______ ______ ______ ______ ______

Project No. __________ __________ __________ __________ __________
Benefit/Cost Ratio ______ ______ ______ ______ ______

Project No. __________ __________ __________ __________ __________
Benefit/Cost Ratio ______ ______ ______ ______ ______

Project No. __________ __________ __________ __________ __________
Benefit/Cost Ratio ______ ______ ______ ______ ______

Project No. __________ __________ __________ __________ __________
Benefit/Cost Ratio ______ ______ ______ ______ ______

2. Double-alternative projects

Project No. __________ __________ __________ __________ __________
Benefit/Cost Ratio: 1-0 ______ ______ ______ ______ ______
2-0 ______ ______ ______ ______ ______

2-1 ______ ______ ______ ______ ______

Project No. __________ __________ __________ __________ __________
Benefit/Cost Ratio: 1-0 ______ ______ ______ ______ ______
2-0 ______ ______ ______ ______ ______

2-1 ______ ______ ______ ______ ______

Project No. __________ __________ __________ __________ __________
Benefit/Cost Ratio: 1-0 ______ ______ ______ ______ ______
2-0 ______ ______ ______ ______ ______

2-1 ______ ______ ______ ______ ______

3. Triple-alternative projects

Project No. __________ __________ __________ __________ __________
Benefit/Cost Ratio: 1-0 ______ ______ ______ ______ ______
2-0 ______ ______ ______ ______ ______
3-0 ______ ______ ______ ______ ______

2-1 ______ ______ ______ ______ ______
3-1 ______ ______ ______ ______ ______

3-2 ______ ______ ______ ______ ______

Worksheet 11 (Concluded)

3. Triple-alternative projects (Continued)

Project No.					
Benefit/Cost Ratio: 1-0					
2-0					
3-0					
2-1					
3-1					
3-2					

4. Quadruple-alternative projects

Project No.					
Benefit/Cost Ratio: 1-0					
2-0					
3-0					
4-0					
2-1					
3-1					
4-1					
3-2					
4-2					
4-3					

5. Quintuple-alternative projects

Project No.					
Benefit/Cost Ratio: 1-0					
2-0					
3-0					
4-0					
5-0					
2-1					
3-1					
4-1					
5-1					
3-2					
4-2					
5-2					
4-3					
5-3					
5-4					

Worksheet 12

IDENTIFICATION OF OPTIMAL SET OF PROJECTS

Budget Period____________ **Total Investment Budget**____________

	Tentative Selection				
Iteration	Project No.	Alternative	Benefit /Cost Ratio	Incremental Investment Cost ($000)	Cumulative Total ($000)
(1)	(2)	(3)	(4)	(5)	(6)

---

* Indicates a project alternative displaced later in the list by one having a higher total investment cost.

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