NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM REPORT **255**

HIGHWAY TRAFFIC DATA FOR URBANIZED AREA PROJECT PLANNING AND DESIGN

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AREAS OF INTEREST:

PLANNING
FORECASTING
(HIGHWAY TRANSPORTATION)

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. DECEMBER 1982

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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FOREWORD

By Staff
Transportation
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Individual agencies have developed various approaches and techniques for applying system-level traffic data to specific highway design projects. For example, many state and urban area transportation agencies use traffic assignments developed in their long-range system planning activities to determine design-hour volumes at the project level. However, these techniques have not previously been documented or standardized for general use. This report provides a comprehensive compilation of the best techniques that are currently being used in urban areas to bridge the gap between system and project analyses. These techniques were identified through a survey of state and local agencies with follow-up field visits to obtain detailed information on procedural steps and typical applications. A user's manual with illustrative case studies is provided in the Appendix. This report should be of special interest to highway planners and design engineers who wish to modify their current procedures or to adopt new ones.

Estimating traffic volumes with the accuracy needed for use in highway design has always been a complex task. Typically, the analyst uses information obtained from land-use planning, traffic forecasting (e.g., trip generation, mode split, traffic assignment), volume counts, and other data to develop design volumes. Many agencies have established various procedures for this purpose, but in most cases these procedures have not been documented for wide dissemination.

JHK & Associates collected information from numerous state and local agencies regarding currently used procedures and developed complete documentation for others to use. The procedures are grouped into ten categories—refinement of computerized traffic volume forecasts; traffic data for alternative network assumptions; traffic data for detailed networks; traffic data for different forecast years; turning movement data; design hour volume and other time-of-day data; directional distribution data; vehicle classification data; speed, delay, and queue length data; and design of highway pavements. The selected procedures were found to be applicable in many situations and to provide a basis for standardization of traffic data analysis.

These same ten categories provide the framework for the user's manual that was developed as part of this research (see Appendix). The user's manual is applicable over a wide range of analyses including systems planning, corridor or subarea studies, evaluation of alternative plans, traffic operations studies, highway design, and environmental studies. Emphasis is placed on easily applied manual techniques, but computer applications are also addressed.

To demonstrate the use of the procedures, three case studies are included—the upgrading of a limited access highway; the evaluation of an arterial improvement; and the design of a highway volume intersection. Detailed information on procedureal steps is provided along with guidance regarding level of accuracy, time requirements, limitations, etc.

This report complements *NCHRP Report 187*, "Quick-Response Urban Travel Estimation Techniques and Transferable Parameters—User's Guide," which provides manual techniques for trip generation, mode split, and traffic assignment. Together, these two reports cover the full spectrum of techniques typically used in planning and design applications.

CONTENTS

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Sincerest thanks are extended to each of the agencies and individuals who contributed time and effort in responding to interviews, completing the questionnaire, and providing documentation of available procedures.

HIGHWAY TRAFFIC DATA FOR URBANIZED AREA PROJECT PLANNING AND DESIGN

SUMMARY

The development of highway traffic data for highway project planning and design requires close cooperation between the users and producers of such data. Unfortunately, until the present time, there have existed no standardized procedures to enable the results of highway system-level traffic assignments, historical data, land-use information, and other factors to be translated into traffic data for highway projects.

Accordingly, this research was conducted to meet the following objectives: (1) Identify, review, and evaluate typical procedures currently being used to develop highway traffic data for project planning and design in urbanized areas; and (2) Using existing techniques to the maximum extent possible, develop a user-oriented manual containing procedures for the full range of planning and design needs, together with illustrative case studies.

A research approach was developed which would enable various procedures to be efficiently identified and evaluated. The following tasks were included:

- Task 1: Investigate Current Needs and Existing Procedures
- Task 2: Evaluate and Recommend Appropriate Procedures
- Task 3: Prepare a User's Manual with Illustrative Case Studies

A literature search was performed to identify existing documentation of available procedures. Subsequent contacts were made with more than 45 state and local governmental agencies throughout the United States. A detailed questionnaire was distributed to both the users and producers of traffic data in these agencies. Based upon the responses to the questionnaire, a number of states were chosen for follow-up personal and telephone interviews. In these interviews more in-depth information was obtained regarding promising procedures and the use of traffic data in highway project planning and design.

These findings indicate that a large percentage of highway planning and design decisions are based on the results of traffic data forecasts. At the same time, it is apparent that the quality of the input data and the analysis procedures used for these forecasts are viewed as being deficient in many respects. Some of the primary issues revealed were the following:

1. The level of detail and precision of computer traffic forecasts varies tremendously from project to project.
2. The lack of quality land-use forecasts hampers the development of high quality traffic forecasts. There are wide variations in the format and quality of data produced by agencies.
3. There is no uniformity in the types of computerized traffic assignments performed (i.e., all-or-nothing; capacity restrained; stochastic).
4. Computer assignments are often not available for all highway alternatives or for all years under study.
5. The traffic data needs for evaluation, design, and environmental analyses are often very different.
6. The responsibility for producing traffic data is often fragmented among agencies.
7. Production of adequate traffic data requires considerable effort and time as well as judgment which comes with experience.
8. A large number of explicit and implicit assumptions are made every time traffic forecasts are performed for highway project planning and design studies.

While public agencies are in partial agreement with respect to techniques for using highway traffic data, they have virtually no uniform procedures to develop those data. Thus, results of analyses in one state cannot be readily compared with results obtained from adjacent states. This problem even manifests itself among urban areas within the same state.

The need existed to identify as many of the available procedures as possible, evaluate each of the procedures, draw upon the strongest points of the evaluated procedures, and develop a set of standard procedures that could receive national distribution. In response to this need, a user's manual was prepared.

The user's manual covers 10 categories of procedures related to traffic data development, as follows:

1. Refinement of computerized traffic volume forecasts.
2. Traffic data for alternative network assumptions.
3. Traffic data for more detailed networks.
4. Traffic data for different forecast years.
5. Turning movement procedures.
6. Design hour volume and other time-of-day procedures.
7. Directional distribution procedures.
8. Vehicle classification procedures.
9. Speed, delay, and queue length procedures.
10. Traffic data for design of highway pavements.

The procedures in these categories can be used singularly or in combination, depending on the analyses to be undertaken. In most cases, manually applied procedures have been described, although computer-aided techniques are presented where appropriate.

In the area of computerized traffic forecast refinements, two procedures are recommended at the corridor or subarea level. The first is a screenline adjustment process that compares base year and future year volumes and capacities across several facilities. The second procedure uses computer-generated select link or zonal tree data to aid the analyst in defining network travel patterns.

The procedures to derive traffic data for alternative network assumptions cover four basic situations: (1) change in roadway capacity; (2) change in roadway alignment; (3) construction of parallel roadways; and (4) addition or subtraction of links. Modifications of screenline adjustments and the use of select link or zonal tree data are used to perform the analyses.

To develop data for more detailed networks, two primary approaches are subarea focusing and subarea windowing. In focusing, a more detailed network is defined within the study area, leaving the remaining network intact. In windowing, a more detailed study area network is defined within a cordon. The remaining network is then replaced by a series of external stations. Both procedures are computer-aided. They are most applicable for conducting small scale corridor or subarea studies in which detailed link and turning volumes are desired on various highways which are not shown on a systems-level network.

In order to derive traffic volumes for different forecast years, various linear and nonlinear growth curves have been developed. These growth curves are based on projected land-use growth patterns or historical trends and can be used to interpolate or extrapolate traffic volumes to alternate years. For more detailed analyses in areas where wide variations in zonal growth are expected to occur, it is recommended that select link and zonal tree data be used to determine differential growth patterns on various facilities.

There are three sets of procedures presented for deriving turning movement data--factoring procedures, iterative procedures, and "T" intersection procedures. These procedures can be applied in situations where either "directional" or "nondirectional" volume data are available.

Procedures are documented to permit design hour volumes to be determined for typical urban facilities and for facilities characterized by sharp recreational or seasonal variations. Other time-of-day procedures are useful to convert daily volume estimates to hourly data for use in design or environmental studies. In both cases, emphasis is placed on the need to adjust base year time-of-day values to reflect changing land use, geometric, or traffic conditions in the future.

The directional distribution procedures try to establish statistical relationships between directional distribution and various factors, such as time-of-day, facility type, and orientation (i.e., radial, circumferential). In lieu of these data, other procedures adjust base year directional splits using professional judgment and knowledge of future land uses (e.g., commercial, residential, industrial).

The vehicle classification procedure provides basic background relevant to the estimation of various auto-truck percentages on urban facilities. It includes a review of expected future land-use changes that would be expected to affect the distribution of vehicles on a facility.

Procedures are presented for calculating speeds, delays, and queue lengths on grade-separated facilities and on surface arterials. The analyst is able to apply different methodologies for traffic flow conditions that are under- or over-capacity. The resulting data are directly applicable to small area design analyses and to environmental analyses.

The procedures presented for highway pavement design enable traffic volume and vehicle classification data to be converted into 18-kip equivalent single-axle loadings that are directly used in the calculation of flexible and rigid pavement design needs. These procedures are applicable using vehicle classification data specific to the subject facility or average values obtained on a regional or statewide basis.

The procedures in the user's manual are applicable over a wide range of analyses. The principal types of applications include systems planning, corridor or subarea studies, evaluation of alternative plans, traffic operations studies, highway design, and environmental studies. In order to demonstrate this applicability, the procedures were applied to three case studies--the upgrading of a limited access highway; the evaluation of an arterial improvement; and the design of a high volume intersection. These case studies describe the interactions of several procedures and indicate that reasonable results can be achieved in relatively short time frames.

The study suggests future areas of research relating to traffic data development. The key areas of emphasis should be the following:

1. The effects of over-capacity highway conditions on land-use development and on the temporal and geographic distribution of traffic.
2. The development of microcomputer or hand calculator applications of several procedures.
3. The quantification of additional factors contributing to or constraining traffic growth.
4. More systematic techniques for deriving turning volumes from intersection link volumes.
5. An improved statistical base for transferring time-of-day, directional distribution, and vehicle classification data to other settings. Particular effort should be given to quantifying truck time-of-day relationships.
6. The improved specificity and standardization of traffic data for use in environmental and evaluation models.

This research project represents the first major effort to document standardized procedures for producing traffic data for use in project planning and design. Therefore, it is recommended that a training course be developed to disseminate this information to both the producers and users of highway traffic data throughout the United States.

CHAPTER ONE

INTRODUCTION AND RESEARCH APPROACH

PURPOSE AND SCOPE OF PROJECT

Until the present time, there have been no nationally accepted or widely used procedures to translate the results of highway system-level traffic assignments, historical data, land-use information, and other factors into traffic data for individual highway projects. A need has been recognized not only to establish accepted procedures for translating various inputs into project traffic data, but also to specify the content, accuracy, and limitations of the data for the problem being addressed. This type of information is required to meet the diverse needs of highway designers, environmental planners, and decision-makers.

The specific objectives of this research were to: (1) identify, review, and evaluate typical procedures currently being used to develop highway traffic data for project planning and design in urbanized areas; and (2) using existing techniques to the maximum extent possible, develop a user-oriented manual containing procedures for the full range of planning and design needs together with illustrative case studies.

Background

During the past 30 years the science of forecasting travel demand in urban areas in the United States has undergone tremendous change. Until the mid-1950's the vast majority of traffic forecasting in the United States was done by projecting traffic trend lines into the future, occasionally taking into account exogenous factors. With the advent of the high speed electronic computer and the formulation of a series of mathematical travel demand models that related travel demand to land use, urban travel demand forecasting procedures changed dramatically. Instead of only being able to forecast traffic on a facility-by-facility basis, it now became possible to forecast changes in travel demand that could be expected to occur at both the systems and corridor level as a result of changing the transportation infrastructure. Emphasis shifted to developing long range system plans, and a great deal of credibility was placed in the computerized travel demand forecasts. Soon many transportation decisions were based on traffic projections produced "by the computer."

Research in travel demand continued to develop new mathematical models that could more accurately replicate human travel behavior. As more and more computerized travel demand forecasts were made, and as some of the transportation facilities opened for which these computerized travel demand forecasts had been made, it was soon apparent that a number of the forecasts had been far from correct. As a result, it was realized that multi-million dollar construction decisions had been based on projections that were not always reliable.

Much attention has been focussed on ways to make the mathematical models more sensitive to those variables that actually determine human travel behavior. However, in many cases the blame for errors in forecasting rests much more with the quality of the input data to the models than with the models themselves. For example, projecting future land use is a difficult and inexact science, even on a jurisdictional basis. To perform this task accurately at the level of travel analysis zones has proven to be almost impossible, yet future land use is probably the single most important input variable to the travel demand forecasting process.

Because of the amount of data that must be coded and the high cost of making travel demand forecasts, the transportation networks that have been used for travel demand projections are typically skeleton networks that simplify the actual highway system. In a computer simulation travelers are shown loading onto the network at only a limited number of entry points (zone connectors), when in reality they enter the network at many points. Traffic assignments have proven to be very sensitive to the coding of zone connectors in the network.

Because of the tremendous amount of network description data that must be developed for travel demand forecasts, general rules of thumb are often applied in order to obtain travel times and capacities for individual links. For example, both speeds and capacities are frequently defined by class of facility in the UTPS highway assignment model UROAD (115), yet both of these critical input parameters can vary widely among roadways within a particular class. Another problem which occurs because of the large data input requirements is that it becomes very easy to make subtle and largely undetectable network coding errors that affect the forecasting results.

Although problems with travel demand forecasts were recognized, the importance of the forecasts in the transportation planning and decision-making process continued to grow in the 1960's and 1970's. This was for several reasons. With the construction of urban freeways in most large metropolitan areas in the United States during the late 1950's and the 1960's, a better understanding was gained of the tremendous social, economic, and environmental impacts associated with these facilities. In recognition of the importance of these impacts, detailed socioeconomic and environmental analyses became a requirement in the evaluation of transportation alternatives. These analyses have been very dependent on a large number of detailed outputs from the travel demand forecasting process.

In addition, during the 1960's and 1970's, groups opposing highway construction projects became much better organized and required transportation planners and decision-makers to provide much more detailed justification for proposed projects. Since the primary justification for constructing most urban highway facilities has been to serve vehicular rather than person travel demand, traffic projections soon came under closer scrutiny and were often challenged by community and environmental groups.

The 1970's were a period during which highway construction costs escalated at a rapid rate, while government budgets in general and highway budgets in particular were restrained. As a result, potential highway construction projects were required to be evaluated not only on their own merits, but also in comparison with other highway alternatives. Insufficient funds were available to construct all of the facilities that were considered necessary. Expected travel demand became an important criterion in prioritizing projects.

The 1970's also witnessed a change in emphasis from the construction of new capital-intensive transportation facilities to improved management of existing facilities. In evaluating alternatives, it became necessary to analyze the expected travel demand impacts of a number of transportation system management (TSM) measures in addition to the traditional build and no-build alternatives. The standard travel demand forecasting models proved to be ineffective in estimating the impacts of many of these TSM alternatives; therefore, revised traffic forecasting procedures had to be adopted.

These changes in the transportation planning process dictated the need for improved travel demand forecasts. Subsequent research focussed on the development of better mathematical models that were sensitive to the critical variables that determined travel behavior. A second area given more attention was the quality of the land use and network description data used as input to the models. A third means to improve travel demand forecasts was to refine the assigned traffic volumes that result from the computerized travel demand process. This area, until recently, has not received a great deal of research focus, yet it is a task confronting almost all practicing travel demand forecasters. These refinements are essential if traffic forecasts are to pass reasonableness tests.

Although refinement of system-level traffic forecasts is widely practiced, until the present no standardized procedures existed which were documented nationwide use. One of the primary purposes of the research conducted in this study was to document procedures that could be used nationwide to develop and refine highway project planning and design traffic data.

Although the research investigated the role of computerized travel demand forecasts in the development of traffic data, its focus was not on means to improve the computer forecasts themselves, but instead it focussed on the use and refinement of the data produced by computer forecasts. The user's manual produced through the research should serve to provide a means to translate the results of system-level computerized forecasts into data required for highway project planning and design studies.

RESEARCH APPROACH

A research approach was developed that would enable various procedures used to develop highway traffic data to be efficiently identified and evaluated. The most promising procedures were later compiled into a user-oriented manual.

Three primary tasks were performed during the research, as follows:

Task 1: Investigate Current Needs and Existing Procedures. This task began with an extensive library literature search covering a wide range of related topic areas. Emphasis was placed on identifying documentation of procedures used to refine or supplement computer forecasts of travel demand, as opposed to documentation of travel demand models and their associated software packages.

Subsequent contacts were made with a number of state and local governmental agencies throughout the United States. A detailed questionnaire (Chapter Two) was distributed to both the users and producers of traffic data in these agencies. On the basis of the responses to the questionnaire, a number of states were chosen for follow-up personal and telephone interviews. In these interviews more in-depth information was obtained regarding promising procedures and the use of traffic data in highway project planning and design.

Task 2: Evaluate and Recommend Appropriate Procedures. The approach used in this task was to evaluate a large number of promising procedures for potential inclusion in the user's manual to be developed in Task 3. To accomplish this effort, series of evaluation criteria were established to serve as a basis for comparison. The available procedures within various categories were compared whenever possible using these criteria. The categories and criteria used in the study are documented in Chapter Two.

Using the findings obtained from the evaluation and knowledge of the current state of the art obtained from Task 1, a set of procedures was recommended for use by practitioners. These procedures were tested using data from actual traffic forecasting studies wherever possible. These results and subsequent modifications to the procedures became the basis for developing a user's manual.

Task 3: Prepare a User's Manual with Illustrative Case Studies. A primary thrust of the research effort was to develop a user-oriented manual of field-tested procedures. The recommended procedures from Task 2 were packaged along with three illustrative case studies as the basis for the manual (see Appendix to this report).

As a final step, the findings obtained from the development of the procedures and case studies were used to identify future research needs in this area. These needs are documented in Chapter Four of this report.

ORGANIZATION AND USE OF THE RESEARCH REPORT

This research report is structured to provide pertinent information to transportation managers and to traffic planners and designers regarding the findings of NCHRP

Research Project 8-26 which resulted in the documentation of a number of procedures for the development of highway traffic data for project planning and design in urban areas.

Chapters One through Four of the research report document the project findings, applications, and conclusions, which will be of primary benefit to administrators and project managers. This information will also provide traffic planners and designers with background relating to the technical procedures presented in the accompaning user's manual.

Exhibit 1 in Chapter Two provides a copy of the questionnaire sent to highway agencies around the country together with summary data of the responses to a number of the questions. The user's manual provided in the Appendix, represents a state-of-the-art presentation of procedures that can be used to refine, detail, and utilize traffic volume data obtained from computerized traffic forecasts. This user's manual is primarily for use by traffic analysts who must provide suitable traffic data to highway planners, designers, and environmental planners.

The user's manual provides an overview of the various uses of traffic data, followed by detailed descriptions of analysis procedures covering 10 related categories. Three case studies are included to illustrate the application of these procedures to typical highway planning and design situations. The manual is self-contained and requires no reference to other parts of this research report.

CHAPTER 2

FINDINGS

TRAFFIC REFINEMENT ISSUES

In order to obtain in-depth information about the development and use of traffic data for highway project planning and design, a three-stage analysis process was used:

1. A literature search was conducted.
2. A questionnaire covering various issues was sent to a number of agencies.
3. Personal interviews were conducted with selected users and producers of traffic data.

The literature search concentrated on identifying existing documentation of procedures available from research findings and agency reports. Much of the pertinent and usable information related to deriving time-of-day, directional distribution, or design hour volumes from average daily traffic (ADT) volumes or from traffic counts taken during specific periods of time at certain times of the year.

Two documents reviewed were user manuals on traffic refinement procedures for computer model output of travel demand. One article focused on generating turning movements from computer model output, while several documents pertained to specific uses of traffic data, such as for highway design or environmental (i.e., air, noise, energy) studies.

Many of the documents received from agencies were reports on studies that they had performed. Generally, the methodologies used in the studies were not discussed in enough detail to be useful as procedure documentation; however, the information was used to develop follow-up questions for the personal interviews.

In the second stage, a questionnaire was developed which was designed to cover various issues relating to traffic data development and use for highway project planning and design. A copy of the questionnaire is included in Exhibit 1.

Questions relating to departmental organization were asked to determine the relationships between traffic data providers and users and to obtain the names of persons to contact for additional information. Several questions related to the type and availability of traffic count data that are required for certain analysis procedures. A number of questions related to the type and use of system-level computerized travel demand forecasts, because these forecasts serve as the base for the development of most project-level traffic data. Next, respondents were asked to describe the procedures they used for refining computerized system-level travel demand forecasts for use at the project level. Information regarding traffic data used for evaluation of alternatives, environmental analyses, and highway design was also solicited. Finally, questions were asked about procedures for forecasting time-of-day characteristics of traffic, vehicle classification data, and speed, delay, and queue length data.

The questionnaire was sent to 45 governmental agencies responsible for developing project-level traffic data. Questionnaires were received from agencies in 38 of the 45 agencies contacted. Summary of questionnaire results from 38 agencies are displayed on the questionnaire. The number of respondents is shown in parentheses for each response. The total number of respondents answering any one question may vary. Some agencies answered more than one response to some questions and did not answer others.

Upon receipt of the completed questionnaires, personal interviews were conducted with developers and users of traffic data at both the state and local level in a total of 10 states. In developing a list of agencies to visit, two primary selection criteria were applied: (1) geographic distribution, and (2) availability of promising procedures. During these interviews in-depth questions were asked relating to the responses provided in the questionnaire, particularly regarding promising procedures and problems encountered in the use of traffic data. In addition, a number of follow-up telephone conversations were conducted with questionnaire respondents who were not able to be personally interviewed.

The following sections describe the major findings from the questionnaire responses and the personal interviews, segmented into various categories. In many cases, the personal interviews provided insight into specific techniques that had been summarized in the questionnaire responses. These findings have not been subjected to statistical analysis and are applicable only to the responding agencies. Therefore, the findings should only be used for informative purposes.

Exhibit 1 Questionnaire for NCHRP Project 8-26:
Development of highway data for project planning and design in urbanized areas.

DATA ABOUT PERSON BEING INTERVIEWED:
Name:
Title:
Address:

Telephone Number:
Brief description of interviewee's traffic forecasting responsibilities:

DEPARTMENTAL ORGANIZATION

1. Could we obtain an organizational chart which shows how the sections responsible for the collection, analysis, and forecasting of traffic data fit into the departmental structure?

2. Please identify the section within the department which is responsible for each of the following:

(a) Traffic counting
Name of section:
Name of responsible person:
Telephone number:

(b) Analysis of traffic count data
Name of section:
Name of responsible person:
Telephone number:

(c) Systems planning
Name of section:
Name of responsible person:
Telephone number:

(d) Traffic forecasting for systems planning
Name of section:
Name of responsible person:
Telephone number:

(e) Highway project planning and evaluation
Name of section:
Name of responsible person:
Telephone number:

(f) Traffic forecasting and traffic data analysis for project planning and evaluation
Name of section:
Name of responsible person:
Telephone number:

(g) Environmental analyses for project planning
Name of section:
Name of responsible person:
Telephone number:

(h) Preparation of traffic data for environmental analyses
Name of section:
Name of responsible person:
Telephone number:

(i) Highway design
Name of section:
Name of responsible person:
Telephone number:

Exhibit 1 Continued

(j) Traffic forecasting for highway design
Name of section:
Name of responsible person:
Telephone number:

(k) Traffic operations analysis for highway design
Name of section:
Name of responsible person:
Telephone number:

3. In addition to the groups identified above, which other sections within the department use traffic data?

Typical Responses:

- District engineers
- Safety
- Structures (bridge)
- Financial analysis
- Right-of-Way
- Research
- Materials (Geotechnic)
- Maintenance
- Developers
- Citizen Groups

4. For those sections which are responsible for forecasting and analyzing traffic data for highway project planning and design, could we obtain a job description for section staff members, including educational requirements?

Several responded.

5. What is the role of MPO's in providing traffic data for use in highway project planning and design studies?

- Land use/socioeconomic projections (13)
- Traffic forecasts (6)
- Perform traffic counts (6)
- Policy guidance (2)
- No role (14)
- Other (3)

EXISTING TRAFFIC DATA

1. Which of the following traffic counts are made as part of highway project planning and design studies?

(a) Road tube counts Yes (38) No (0)
How long are counts made at each location?
24hr (15); 48hr (14); 3 to 7 days (4); 2 weeks (2)
What time increment is reported?
15 min (4) 30 min (1) 1 hr (32) 24 hr (11) Other (1)

What type of correction factors are applied to the count data?

- Axles (15)
- Seasonal variation (22)
- Daily variation (11)
- Monthly variation (15)
- ADT (2)
- None (3)

(b) Turning movement counts Yes (37) No (1)
How long are counts made at each location?
4-6hr (8); 8-12hr (17); 14-16hr (7); 24hr (2)
What time increment is reported?
15 min (20) 30 min (3) 1 hr (13)

What type of correction and expansion factors are applied to the count data?

- ADT (20)
- Daily (4)
- Seasonal (12)
- Diurnal (2)
- Other (5)
- None (7)

Exhibit 1 Continued

(c) Vehicle classification counts Yes (37); No (1)
How long are counts made at each location?
3-4hr (3); 6-12hr (19); 14-24hr (13)
What time increment is reported?
15 min. (2); 60 min. (33)

2. Are any other traffic count data normally requested as part of a highway project planning or design study?

- . Pedestrian (4)
- . Directional split (5)
- . Design hour volume (7)
- . High occupancy vehicles (2)
- . Other (2)

TRAFFIC FORECASTING

1. Is a statewide travel demand forecast performed by your department? If so, is it computerized and for what years is traffic forecast?

Yes (13); No (23)

If yes, computerized? Yes (8); No (4)
If yes, time increments used? 5 yr (2); 10 yr (1); 20 yr (5); over 20 yr (2)

2. What urban areas within your state have ongoing computerized travel demand forecasting processes? Are highway project planning and design traffic data based upon these computerized forecasts?

Virtually all urban areas reported have computerized processes.
Highway plans based on forecasts? Yes (26); No (2)

3. What type of regular traffic counting program does the department have?

- . Permanent count stations (24)
- . Seasonal stations (5)
- . Cordon counts (3)
- . Periodic counts
  - .. 1 yr (6)
  - .. 2 yr (4)
  - .. 4 yr (2)
  - .. Over 4 yr (1)

4. Are annual reports summarizing basic traffic data issued? Could we obtain copies of any which are used to develop correction factors or growth factors which are used in traffic forecasting?

Frequency of issue:	1 yr	(24)
	2 yr	(3)
	Over 2 yr	(1)
	Annual map only	(1)

5. Do you have standard request forms for traffic counts? Yes(11); No(24)
If so, could we obtain a copy of each? If not, how are traffic counts requested?

- . Several obtained
- . Other requests via
  - . memo (19)
  - . phone (8)

6. What is the average turnaround time from date of request to actual receipt of traffic count data?

- . Less than 2 wk (13)
- . 2-4 wk (17)
- . Over 1 month (4)

7. Could you provide us with copies of traffic count data collection and data summary forms?

Several provided.

Exhibit I Continued

3. Who has responsibility for producing the computerized travel demand forecasts in each urban area?

- . State DOT (34)
- . MPO (15)
- . Local Agencies (4)
- . Consultants (1)

4. Are separate computerized forecasts typically made for each alternative being studied in a project planning study? What years are forecasts normally made for?

. Base year	(14)	. 15 yr	(3)
. Construction year	(5)	. 20+ yr	(33)
. 5 yr	(4)		
. 10 yr	(9)		

5. Are standarized FHWA or UMTA procedures used in performing the travel demand forecasts? Is so, are they flowcharted? Is the process documented? Could we obtain copies of documentation, including flow charts, if available?

- . FHWA only (6)
- . UTPS only (5)
- . Both FHWA and UTPS (22)
- . Other (3)

Very few are flow charted.

6. When were the models last calibrated? Has any work been done to validate or update the models since that time?

Last calibrated				Validated since then
. Before 1970	(5)	. 1977-1979	(10)	. Yes (18)
. 1970-1972	(4)	. 1980 or later	(4)	. No (13)
. 1973-1976	(12)			

7. Do you perform base (present) year validation runs as part of the computerized travel demand process for highway project planning or design studies?

- . Yes (20)
- . No (14)

8. Do you use a more detailed zone system and code a more detailed highway network within the corridor being studied?

- . Yes (13)
- . No (21)

9. Who provides the land use (socio-economic) data that is used in the forecasting process? What land use (socio-economic) variables are used? For what years are these forecasts available?

Providers:		Number of data variables used:		Years available:	
. MPO	(15)	. less than 5	(15)	. 2000	(17)
. Local Agencies	(6)	. 6-10	(9)	. 2005	(3)
. State DOT	(4)	. over 10	(1)		

10. What type of modal choice process is used in the travel demand forecasts?

Computer models used?
- . Yes (17)
- . No (6)

Various manual and computer models used.

11. Is your computer assignment process all-or-nothing, capacity-restrained, or stochastic? Which model do you use for computerized forecasts? Do you code global speed/capacity tables or separate speeds and capacities for each link in the network?

Type of Assignment:		Coding used for speeds and capacities:	
. All-or-nothing	(16)	. Global values	(5)
. Capacity restrained	(14)	. Link specific values	(17)
. Stochastic	(3)		

Exhibit 1 Continued

12. Are your assignments ADT, peak period, or peak hour? If they are peak period or peak hour, what factors do you apply to 24 hour trip tables to obtain peak period or peak hour trip tables for assignment?

ADT	(31)	
Peak Period	(3)	- Home interview survey results, diurnal
Peak Hour	(0)	count data.
AWDT	(2)	

13. Do your assignments produce turning movements at major intersections?

. Yes (30)

. No (6)

14. Do you plot computerized assignments manually or use computer plots of traffic assignments?

. Manually (14)
. Computer (16)
. Both (6)

15. In areas where computerized traffic assignments are not available, how do you perform traffic forecasts? Is this process documented? Is so could we receive a copy of the documentation?

. Historical trends (29)
. Regression equations (2)

Processes were rarely documented.

16. Has your department analyzed high occupancy vehicle priority treatment alternatives? If so, what travel demand forecasting procedures were used? Could we receive documentation of these procedures?

. Yes (17)
. No (17)

Procedures:
. Manual pivot point (1)
. NCHRP 187 Quick Response (1)
. Manual diversion curves (1)
. FREQ models (2)
. Other (5)

Documentation provided for most procedures.

TRAFFIC REFINEMENT PROCEDURES

1. Has your agency adopted standardized procedures for refining computerized system level travel demand forecasts for use at the project level? Is so, are these procedures documented? Can JHK receive a copy of the documentation? If standardized procedures have not been adopted for refining system level forecasts, describe how refinements are normally made?

. Yes (10)
. No (13)

Received documentation for available procedures.

2. For any refinement procedures used by your agency in developing project level traffic forecasts, please provide the following information: Typical responses follow:

(a) Give a basic description of methodology.

. Use historical trends (9)
. Check land use (3)
. Professional judgment (10)

Exhibit I Continued

(b) What are the required data inputs?

. Historical traffic counts (10)
. Turning movements (3)
. Land use (base and future "years") (8)
. Traffic assignments (4)

(c) What are the manpower, training, and cost requirements?

. Time consuming (11)
. Other variable answers.

(d) What level of accuracy is required of the computer forecasts?

± 10% (5)

± 5% (1)

± 15% (1)

± 20% (1)

(e) Are there built-in biases in the procedure?

. Requires knowledge of study area (11)
. Doesn't account for induced land use changes (11)
. Uses straight line extrapolation (1)
. All local roads must be manually assigned (1)
. Based on unreasonable land use forecasts (1)

(f) Are reasonableness checks used to check outputs of the procedure?

. Yes (16)

(g) In what types of applications has the procedure been used?

. System planning (6)
. Corridor studies (2)
. Highway design (8)
. Evaluation of alternatives (1)

(h) Have there been problems in applying the procedure?

. Computer turnaround time (1)
. Unavailability of data (1)
. Inconsistencies (1)
. Unreasonable growth rate (1)
. Difficult to comprehend future conditions (1)

(i) What suggested improvements to the procedure do you have?

. More current traffic counts (1)
. More current land use data and forecasts (1)
. More detailed networks and zones (1)
. Bring policy forecasts to reality (2)

3. How do you adjust system level forecast data in cases in which the forecast year for the highway project is different than the forecast year for the computerized systems level forecast?

. Extrapolate or Interpolate (11)
. Use historical growth rates (14)
. Factor trip table (2)

4. Do you have procedures for deriving turning movement data from link volume data? If so, are they documented, and could JHK receive a copy of the documentation? If documentation is not available, please describe the procedure.

. Yes (1) - Documentation sent
. No (4)

Most agencies use professional judgment.

Exhibit I Continued

5. Do you have procedures for developing traffic volume data for a more detailed network than that in the systems forecast? If so, are they documented, and could JHK receive a copy of the documentation? If documentation is not available, please describe the procedure.

- Windowing technique (3)
- NCHRP Report 187 Quick Response (2)
- Professional judgment (3)

6. How do you derive traffic volume data for alternative network assumptions for which separate travel demand forecasts have not been prepared? If such procedures are documented, could JHK receive a copy of the documentation?

- Select link analysis (3)
- Professional judgment (11)

TRAFFIC DATA FOR EVALUATION OF ALTERNATIVES

1. What traffic data are usually produced for the evaluation of highway project alternatives?

- ADT (8)
- Diurnal percentage (5)
- Directional distribution (2)
- Truck percentage (4)
- VMT (3)
- VHT (2)
- Speeds (1)
- V/C ratios (1)
- Turning movements (2)
- 18-kip equivalents (1)

2. Is a standardized format used for presenting traffic evaluation data? If so could we receive a copy of the specifications for presenting the data or a copy of a sample report which shows how traffic evaluation data are presented?

- Yes (8)
- No (23)

Some documentation received.

TRAFFIC DATA FOR ENVIRONMENTAL ANALYSES

1. What traffic data are normally produced for input to environmental analyses?

- ADT (6)
- Speed (1)
- Vehicle classification (7)
- Design hour volume (4)
- VMT (3)
- Diurnal percentages (1)
- Directional distribution (1)

2. If the data which are input to environmental analyses are prepared in a standard format, would you provide us with a copy of the forms which are used for preparing the data?

- Yes (9)
- No (18)

Some forms provided for specific models.

3. What environmental models which your agency uses require traffic data, and what traffic data are required as input to each model?

Air Quality:
- MOBILE 1 (15)
- CALINE (14)
- APRAC (2)
- HIWAY 2 (1)
- Kansas Air Pollution Package (2)
- Other (2)

Noise:
- FHWA procedure (3)
- STAMINA 1.0 (8)
- SNAP (6)
- HUSH (1)

Energy:
- NCHRP 20-7 (1)
- ENERGY (4)

Exhibit I Continued

TRAFFIC DATA FOR HIGHWAY DESIGN

1. What traffic data are normally produced for input to highway design studies? How are these data used in highway design?

. ADT	(34)	. Turning movements	(13)
. Diurnal percentages	(30)	. Geometrics	(2)
. Directional distribution	(11)	. Speed	(2)
. Truck percentages	(27)	. Accidents	(1)

2. Are standardized formats used for highway design traffic data? Is so, would you provide us with a copy of the format for presenting these data?

. Yes (13);
. No (21)

Several forms provided.

3. What capacity analysis procedures is your agency presently using, both an arterial streets and freeways?

. 1965 Highway Capacity Manual	(22)
. TRB Circular 212	(10)
. Leisch Charts	(2)
. Critical Lane Volumes	(5)
. AASHTO	(3)
. NCHRP 187 - Quick Response	(1)
. V/C ratios	(2)

SPECIFIC PROCEDURES FOR PRODUCING TRAFFIC DATA

1. Do you have procedures for forecasting changes in the percentage of traffic which travels during the AM and PM peak hours, changes in directional distribution by time of day, and changes in diurnal curve characteristics? If these procedures are documented, would you send JHK a copy of the documentation? If not, please describe the procedures.

. Yes (11) - Historical count trends (8); land use changes (2)
. No (21)

2. How do you forecast changes in vehicle mix? If you have documented these procedures would you send the documentation to JHK?

. Historical classification count trends (12)
. No procedure used (12)

3. How do you forecast operating speed data? Do you have special procedures for calculating average operating speeds in the vicinity of intersections or bottlenecks where traffic is stopped at certain times? If your procedures are documented would you send the documentation to JHK?

. V/C ratio and speed relationships (7)
. Speed and delay studies (5)
. 1965 Highway Capacity Manual Curves (3)
. Engineering judgment (4)
. No procedure used (5)

4. Are you required to perform queuing analysis for intersections or at bottlenecks? Is so, what procedures do you use? Do you use special procedures for calculating queues where demand exceeds capacities? Would you provide JHK with a documentation of your queuing analysis procedures?

. No (18)
. Yes (10) - Poisson distributions (3); Alternate arrival method (1); engineering handbook (1); 1965 Highway Capacity Manual (3); AASHTO (2)

No special procedures cited for over-capacity conditions.

Role of the Metropolitan Planning Organization in Producing Traffic Data

Metropolitan Planning Organizations (MPO's) play a variety of roles in providing traffic data for use in highway project planning and design studies. In 14 states, the MPO plays no role in providing traffic or socioeconomic data for use in the traffic forecasting process. Few MPO's perform the traffic forecasts or provide traffic count data; approximately half of the MPO's mainly provide land-use/socioeconomic data and policy guidance. In the vast majority of states the state DOT is the agency primarily responsible for developing facility level traffic data for major highway improvements in urban areas, while the MPO's role is to provide selected input data and policy guidance.

Traffic Data Collection

All agencies conduct some type of regular counting program. Two-thirds of the agencies have permanent counting stations; one-third count major state highways at least once every 2 years, and less than 10 percent report seasonal counts. A majority of states publish an annual report of traffic volumes, although some now only publish an annual traffic flow map. The permanent count stations are important because they provide good historical trend data, diurnal curve information, and indications of seasonal, monthly, and daily traffic variations.

All of the responding agencies take road tube counts, usually for periods of 24 or 48 hours at each location. Most agencies report the counts in 60 minute intervals. Two-thirds of the responding agencies apply seasonal correction factors, one-third apply axle and daily variation factors, and three states do not adjust their counts. It would appear that in a number of cases additional refinements to road tube count data are called for if these data are to be useful in the development of project level traffic forecast data.

All agencies, but one, take turning movement counts. Most counts are for 8 to 12 hours, with the remainder evenly split for shorter and longer durations. Two-thirds of the agencies report in 15-minute time increments and the remainder report in 60-minute increments. The majority of the agencies use an expansion factor to a 24-hour count, and several apply a seasonal factor. In those states where only 60-minute time intervals are reported, peak hour turning movement volumes may be underreported if the peak hour does not correspond to the reporting period. In most states, however, data are collected during peak hours; therefore, a key data input to the development of future year turning movement data is almost universally available.

All agencies, but one, have vehicle classification counts available. Most counts range from 6 to 12 hours with one-third ranging from 14 to 24 hours. Almost all of the states report these counts in 60-minute increments, which are then factored up to 24-hour values based on road tube counts. Because truck percentages are quite different during the hours that are not normally counted, truck ADT's are often misreported. In several states this has resulted in inadequate pavement thickness design.

Two-thirds of the respondents do not use a standard request form for traffic counts, relying on a memo letter or a phone call for traffic count requests. The average turnaround time for traffic count data is 2 to 8 weeks, depending on the type of data requested and the staff work load at the time of the request. The length of time required to obtain traffic count data must be incorporated into the development of schedules for producing highway traffic data, particularly in those states where slow turnaround times are common.

Traffic Forecasting

Statewide traffic forecasting is performed in a minority of the states surveyed with two-thirds of the statewide forecasts computerized. All but two of the agencies responded that the urban areas within their state have ongoing computerized travel demand forecasting processes, and that highway project planning and design traffic data are based on these forecasts. The state DOT's are mainly responsible for producing the computerized forecasts within the urban areas, while the MPO is in charge of the forecasts in a number of the larger urban areas. A few agencies employ consultants to perform their traffic forecasts.

Most of the responding agencies run separate computerized traffic forecasts for each highway alternative being evaluated in a project planning study. All but four of these agencies use standardized FHWA (104, 111) or UMTA (115) models for developing travel demand forecasts, many using a combination of the two modeling chains. Typically there is an FHWA modeling base combined with a few UTPS programs. A few agencies have developed supplemental programs to work with the FHWA/UTPS packages.

Three-quarters of the traffic forecasting models have been calibrated since 1973, with half of these calibrated since 1977. Ten percent have not been calibrated since they were developed in the late 1960's. Several states mentioned they were waiting for the 1980 census population and land-use data to recalibrate their models. Approximately 60 percent of the models have been validated since they were calibrated, and the same number of agencies perform base year validations as part of their computerized travel demand forecasts.

In the majority of the states, the MPO's and local governments provide the land-use/socioeconomic data for input to the forecasting process. Approximately two-thirds of the states use five or fewer variables in the model, and most states have available land-use forecasts for every 10 years up to the year 2000. A few states are currently developing forecasts for the year 2005, but generally system-level traffic forecasts are not available for the years for which facilities are presently being designed (i.e., construction year plus 20 years).

The agencies are equally split on using all-or-nothing or capacity restraint assignment processes. Only three agencies responded that they use a stochastic assignment process. Several agencies indicated they had capacity restraint capability but did not always exercise it either because of the costs involved or because they would rather manually restrain the roadways. Most of the agencies code specific speed and capacities for each link in the network. However, these speeds/capacities may be based on facility type and the number of lanes instead of on actual conditions.

The majority of the states produce 24-hour traffic assignments in those urban areas where computerized forecasts are performed. A constant peak hour factor of 8 to 10 percent is used, depending on the type of facility, historical count trends, and knowledge of future land use. Most of the agencies have the FHWA PLANPAC (104, 111) capability of producing turning movements but they do not exercise it on all runs. The agencies are split equally on manual versus computer plots, and several agencies use both, depending on the extensiveness of the project.

Traffic Refinement Procedures

Few of the agencies reported that they had standardized procedures to refine computerized system-level traffic forecasts for use at the project level. Almost all of the documented refinement procedures involved some type of comparison between base year simulated and actual traffic volumes. If such refinement procedures are to be used in agencies which do not presently use them, base year validation runs would have to be made in the states that do not presently do so. Undocumented procedures obtained with the responses typically combine an extensive amount of engineering judgment with local knowledge of historical traffic volume and land-use changes. Few agencies have specified a level of accuracy required of the computer forecasts in matching base year traffic volumes on the facilities being studied.

Several problems associated with the refinement procedures presently in use were cited. By factoring forecast volumes up or down by as much as base year simulated volumes are over or under base year traffic count data, the refined future year traffic volumes tend to be biased toward existing land-use patterns. Therefore, changes in traffic volume due to large new developments may be inadvertently lowered or raised more than is appropriate. In addition, almost all the documented refinement procedures are time consuming and require that considerable professional judgment be applied. Additional expense is often involved in obtaining base year traffic count data throughout the entire study area affected by a proposed roadway improvement.

The majority of agencies use growth factors derived from historical trends or from interpolation/extrapolation curves to adjust system-level traffic forecasts in cases where the forecast year for the highway project is different from the forecast year for the computer forecast. The exact year in which planned land-use developments will occur is often not taken into account.

Most agencies have turning movement capability within their computerized traffic assignment processes. However, several agencies responded that the turning movement data from the computerized process are not usable without substantial refinement. Of those states that do not forecast turning movement data with the computer, engineering judgment based on historical counts is the most common methodology employed.

Most states do not have procedures for developing traffic volume data for a more detailed network than that used in a system-level traffic forecast. Several states indicated that their highway networks were already detailed enough, thus obviating this need. Other states use various manual assignment procedures, windowing techniques, and/or engineering judgment.

Approximately one-half of the respondents indicated that separate travel demand forecasts are made for all alternative network assumptions. The remainder use engineering judgment or supplemental computer data to redistribute trips.

In areas where computer traffic assignments are not available, the use of historical traffic trends to forecast traffic is widespread. In these cases, at least cursory consideration is given to planned land-use changes in the study area surrounding a proposed highway improvement.

Traffic Data for the Evaluation of Alternatives

Very few of the respondents reported that they have a list of standardized traffic data that are produced for use in the evaluation of alternatives. Data requirements vary from project to project depending on the critical issues associated with each project. Most agencies do perform some type of benefit-cost analysis during project planning studies. In addition, traffic data are normally included in some type of evaluation report or matrix used by decision-makers to choose among alternatives.

Traffic Data for Environmental Analyses

Traffic data are required for three major categories of environmental analysis: air quality, noise, and energy consumption. These types of analyses are performed in almost all states as part of the environmental impact statement process, although simplified procedures are usually used where impacts are not expected to be significant.

Most agencies responded that they used some version of the air quality computer models MOBILE (33), CALINE (12, 100), and HIWAY (78) for emissions and dispersion analyses. These models require hourly traffic data stratified by vehicle class and by operating speed categories. Although intersection-level air quality analyses are not performed in most states, these analyses have recently been performed with a greater degree of regularity.

Virtually all agencies that perform energy analyses base these analyses on procedures developed by the California Department of Transportation and contained in the U.S. Department of Transportation Manual Energy Requirements for Transportation Systems (102). These procedures require average daily traffic data throughout the design life of the facility. These data must be stratified by vehicle class, congestion level, and operating speed.

The most commonly used noise models are those based on the FHWA Highway Traffic Noise Prediction Method (10, 105, 110) and its computerized versions SNAP (2, 14) or STAMINA (83). These models require as input data level-of-service "C" auto volumes, operating speeds, and design hour truck volumes.

Traffic Data for Highway Design

The major uses of traffic data in highway design are for capacity analyses and pavement design. Two-thirds of the agencies report that they exclusively use the 1965 Highway Capacity Manual (38) for capacity calculations on arterial streets and freeways. One-third are using the interim capacity materials in TRB Circular 212 (45), and scattered agencies are using the Leisch tables (35), other critical movement analyses, and/or AASHTO (6) procedures.

Most agencies report that they use procedures outlined in AASHTO's Interim Guide for Design of Flexible Pavement Structures (5) for pavement design. These procedures require that annual vehicle classification data be converted into equivalent 18,000-pound single-axle loads (18-kip equivalents) for all years during the design life of the pavement structure.

Other Data Requirements

Almost all agencies report that system-level traffic forecasts are performed using 24-hour data. Design-hour or peak-hour volumes are then derived by multiplying daily volumes by a peak-hour percentage. In almost all states the peak-hour percentage used is either a standard percentage determined by roadway type or a percentage derived from historical traffic count data on the facility being studied. In the few cases where peak-hour percentages are changed from base year conditions, these changes are based primarily on professional judgment or diurnal data from other facilities with traffic characteristics similar to those forecasted for the facility under study.

Most agencies report that base year directional distribution and vehicle classification percentages are assumed to hold for future years. For new facilities percentages are typically derived from similar facilities elsewhere in the same urban area. In some cases these percentages are modified using professional judgment to account for new land use developments that are forecasted to occur in the area of the facility being studied.

Most of the responding agencies use volume-to-capacity (V/C) ratios, or speed and delay studies to forecast highway speed data. Some agencies assume no difference between base year and future year speeds. No responding agency had special procedures for calculating speeds in the vicinity of intersections or bottlenecks where traffic is stopped at certain times. Similarly, few of the responding agencies are required to perform queuing analyses for intersections or bottlenecks, or to use special procedures for calculating queues where demand exceeds capacity. The agencies that perform these analyses use either the 1965 Highway Capacity Manual (38), Poisson distributions, or a measurement of delay procedure.

PROCEDURES TO PRODUCE TRAFFIC DATA

On the basis of the procedures identified during the literature search, interview processes, and subsequent development and refinement of additional methods, a set of procedures has been prepared that can be used to develop traffic data for highway project planning and design. These procedures, presented in detail in the Appendix (User's Manual), represent a combination of existing techniques and new or modified procedures.

A total of 10 categories of procedures were identified for consideration. These categories are given in Table 1. The categories include procedures to refine and detail system-level link-volume forecasts (categories 1 through 4) and procedures to derive specific traffic data needs, such as turning movements, hourly volumes, directional traffic distributions, and vehicle classifications (categories 5 through 8). Category 9 concentrates on procedures which use these traffic data to produce speed, delay, and queuing information for input to environmental models and capacity analyses. The final set of procedures (category 10) produce appropriate data for highway pavement design.

Table 1. Categorization of procedures.

Category	Procedure
1	Procedures to refine computerized traffic volume forecasts
2	Procedures to derive traffic data for alternative network assumptions
3	Procedures to derive traffic data for more detailed networks
4	Procedures to derive traffic data for different forecast years
5	Procedures to derive turning movement data
6	Procedures to determine design hour volume and other time-of-day data
7	Procedures to derive directional distribution data
8	Procedures to determine vehicle classification data
9	Procedures to calculate speed, delay, and queuing data
10	Procedures to produce traffic data for highway pavement design

Once an inventory of existing procedures had been prepared, as discussed above, an evaluation process was used to select the most appropriate procedures. In order to accomplish this task, a basic list of evaluation criteria was developed.

These criteria were based on the following considerations:

- In what circumstances can the procedure be effectively used?
- Is the procedure logical and sensible?
- Are the procedure's underlying assumptions and mechanics intuitively correct?
- Is the procedure sensitive to the critical variables which determine the values of the traffic data?
- What is the relative accuracy of the procedure?
- What is the general availability of required data inputs?
- What are the time and cost requirements for obtaining the required input data and for applying the procedure?
- Is the procedure easy to use? Is it understandable, or is it effectively a black box to its user?
- How easy is it to make errors with the procedure?
- Can the results be easily checked for reasonableness?
- Has the procedure been adequately documented and field tested? If not, what is required in terms of documentation and field testing?
- Have special problems been identified with the procedure?

It was necessary that existing alternative procedures be evaluated within the context of the data requirements for each of a number of specific types of projects and for varying conditions of data and systems-level forecast availability. For example, a procedure that would be evaluated as "poor" under the condition in which detailed computer travel demand analysis information is available for each alternative might be the best procedure available to develop certain traffic data when only a single systems-level computer forecast is available. Therefore, each alternative procedure was assessed within the context of different scenarios.

These scenarios were defined along several dimensions, as follows:

1. Type of project:
   - Construction of a new freeway or major arterial through a corridor.
   - Upgrading an existing highway facility.
   - Localized roadway improvements.
   - Transportation system management alternatives.

2. Amount of computerized forecast data available:
   - Detailed, high quality forecasts are available for each alternative being studied.
   - Sketch planning level forecasts are available for each alternative being studied.
   - A single systems-level forecast is available.
   - The computerized forecasts either lack enough detail or are nonexistent in the corridor under study.

3. Time and budget available:
   - Adequate budget, relatively long time available for analysis.
   - Small budget and/or short time available for analysis.

During the application of this evaluation process, it was soon found that many of the categories in Table 1 had few or no alternative procedures identified. As a result, procedures for these categories had to be developed or synthesized. Some categories had only one procedure identified in the inventory, and as a result no comparative evaluation of alternatives was necessary prior to recommendations.

In virtually all cases, a detailed comparative evaluation of alternatives was not found to be necessary although the available procedures were still rated for each of the criteria. This rating highlighted the strengths, weaknesses, and key aspects of a recommended procedure, thereby providing significant information for the user's manual. In some instances, identified alternatives were clearly inferior and were eliminated. In other instances, more than one alternative was recommended, as each was more applicable than the other under the different scenarios of data availability, analysis time and cost limitations, and specific characteristics of the highway project under analysis. And finally, in some instances it was concluded that the best elements of each of the identified alternatives should be combined into a new procedure. The primary findings relating to the alternative and recommended procedures in each of the 10 categories are discussed in the remainder of this chapter. The details of each procedure are presented in the user's manual.

Category 1 -- Procedures to Refine Computerized Traffic Volume Forecasts

The procedures in this category are aimed at refining link volumes. The available techniques ranged from simplified single-page guidelines to complex screenline adjustments (46, 88). Virtually all of them involved considerable professional judgment in determining how traffic should be adjusted between facilities. Most procedures look at a network of street assignments, while some are applied only on a link-by-link basis. One commonality of the reviewed procedures seemed to be the explicit or implicit consideration of base year traffic counts, land-use patterns, and traffic growth patterns in the refinement procedures. The level of documentation in this category was fair to good.

One link refinement procedure recommended is an adaption of a methodology developed by JHK & Associates for the Maryland Department of Transportation (46). This procedure includes an overall check of the computer assignments followed by traffic refinement at the link level. A comparison is made between the base year and future year volumes and capacities across a screenline to arrive at a refined assignment. The procedure is modified by embedding within it a methodology developed by the New York State Department of Transportation (77). This technique adjusts for discrepancies between the base year traffic forecasts and actual base year traffic counts. The procedure is most applicable for performing corridor-level analyses.

A second procedure, select link analysis (104, 115), is recommended for refining link volumes within a small study area or for defining travel patterns for reassignment of traffic in over-capacity conditions. A companion method using zonal tree analysis (104, 115) is also recommended for these applications.

Category 2 -- Procedures to Derive Traffic Data for Alternative Network Assumptions

There are four basic situations to consider in this category: (1) change in roadway capacity, (2) change in roadway alignment, (3) construction of parallel roadways, and (4) addition or subtraction of links.

Since most agencies surveyed rerun a computer model for each network change, there were very few documented procedures. The most sophisticated techniques used are the "windowing" or "focussing" procedures described in Category 3. These procedures enable several alternative networks to be quickly analyzed using a computer.

In other cases, the general trend has been to judgmentally redistribute volumes from parallel links onto a new or modified facility. In the first situation, a modified screenline refinement procedure from Category 1 is suggested for use. This procedure can account for relative changes in roadway capacity, as long as total screenline trips remain constant. Guidelines developed by the State of Washington (119) provided a good basis for the development of a manual procedure, although more specific explanations were required.

The latter three situations require a more rigorous analysis of travel patterns using select link or similar data. Once this is done the screenline procedure from Category 1 can be used to further smooth the volumes across a screenline. Another procedure that is suggested is essentially a manual reassignment process using modified travel times (user-supplied) and the NCHRP Report 187 assignment method (88). Therefore, the procedures developed in this category utilized a combination of computer and manual techniques to produce alternative network assignments in an efficient manner.

Category 3 — Procedures to Develop Traffic Data for More Detailed Networks

The two primary approaches documented in this category involved either subarea focussing or subarea windowing. In focussing, a more detailed network is defined within the study area, leaving the remaining network intact. In windowing, a more detailed study area network is defined within a cordon. The remaining network is then replaced by a series of external stations.

Each of the procedures involves the use of computer models. Subarea focussing is presented as a computer-aided method based on the documentation of the North Central Texas Council of Governments and Maricopa Association of Governments, among others (75, 61). Subarea windowing is documented in greater detail, emphasizing the process used to conduct either a manual or computer-aided procedure (76). The UMTA and FHWA programs NAG and DONUT provide the base for several expanded windowing procedures used in certain urban areas (115, 104). The experiences of the Minnesota and Ohio Departments of Transportation were used as prototypes (76). There was a divergence of opinion among agencies between focussing versus windowing procedures, but generally the approach has been to detail the study area network prior to running any model. No fully manual procedures were documented.

Less rigorous and computationally and data intensive techniques were also developed. These techniques concentrate on modifying the screenline and select link procedures from Category 1 to reallocate trips based on relative base year and future year volumes, capacities, and/or travel patterns. All of the above procedures can be considered in conjunction with one another.

Category 4 — Procedures to Derive Traffic Volumes for Different Forecast Years

The appropriate procedures in this category depend on the availability of historical traffic count data and adequate land-use or demographic data for the target year for which a traffic forecast is desired. Where these data are available, the suggested procedure is to interpolate or extrapolate the target year trips using a linear or nonlinear method. This decision would depend on the uniformity of expected growth inside and/or outside of the study area. Where full build-out growth data are also available, the rate of growth for an extrapolated year can be modified based on how close the study area is to its development capacity, as discussed by Memmott and Buffington (66).

Where land-use data are unavailable or inadequate, the suggested procedure is to extrapolate (linear or nonlinear) to a target year based on historical traffic and/or demographic trends. This procedure is usually only valid for short time frames.

For more detailed analyses in areas where wide variation in zonal growth are expected to occur, it is recommended that select link and zonal tree data (104, 115) be reviewed for changes in travel patterns and growth on specific facilities. This incorporates a procedure used by the Maricopa Association of Governments (60, 61). The target year assignments are then made on a facility-by-facility basis by interapolating or extrapolating these trends.

Category 5 — Procedures to Derive Turning Movement Data

There are three sets of procedures recommended for this category: factoring procedures, iterative procedures, and "T" intersection procedures. None were documented in the literature or in the field. The factoring procedure is a simple factoring of future year turning movements based on the degree of discrepancy between the base year counts and forecasts. Both a "ratio" and a "difference" factor are presented.

Iterative procedures have been developed for situations in which either "directional" or "nondirectional" volume data are available. The directional method is based on a row and column matrix balancing procedure developed by Mekky (64). This method can be applied to most intersection situations; however, it requires a realistic initial estimate of turning percentages in order to produce a final set of turns within a reasonable number of iterations. A related noniterative mathematical model developed by Norman and Harding (73) was found to provide some realistic solutions; however, its applicability was limited to selected intersection conditions and its calculations, while noniterative, were mathematically complex. Therefore, it was not included in the user's manual.

The nondirectional iterative method is a modification of a procedure prepared by the Middle Rio Grande Council of Governments (63). This procedure assumes that intersection link volumes are surrogates for downstream land-use productions and attractions. Its major limitations are a heavy reliance on professional judgment and a lack of a theoretical base. It is therefore most useful for sketch-planning purposes.

Finally, a special procedure for "T" or 3-legged intersections is presented. Because of the simplicity of turning movements in this situation, nondirectional turns can be directly calculated using an equation; directional turns can also be estimated by comparing relationships among various approach link volumes.

Category 6 — Procedures to Determine Design Hour Volume and Other Time-of-Day Data

Most of the available procedures in this category involved an analysis of local or statewide data for different time periods. Tables classifying the diurnal or time-of-day data by trip purpose, mode, or other categories were then constructed using these data. Several sources attempted to establish statistical correlations within the classification tables, so that the time-of-day curves could be readily transferred to other locations. One procedure included regression equations that related time-of-day information in Milwaukee to several trip-making characteristics (3). However, these equations were not statistically significant for transfer to other urban areas.

Procedures for forecasting design hour volume, hourly volumes over an average weekday, and peak hour factors have been recommended. With respect to design hour volume, different procedures were developed for those typical urban facilities with peaks defined by work travel and for those atypical urban facilities with peaks defined by recreational travel (6). For typical urban highway facilities whose peaks are determined by work travel, transfer of known design hour volume/average daily traffic ratios were recommended based on comparable highway type, location, orientation, adjacent land use, and level of service (6, 70). For urban facilities whose peaks are determined by recreational travel, the procedures recommended involved the transfer of base year known design hour volume/average daily traffic ratios from facilities that operate in a manner similar to how the facility under analysis is expected to operate in the future.

Procedures for weekday hourly volume forecasting were similarly based on transferring known hourly volume proportions based on several facility characteristics (88). Peak hour factor forecasting procedures were dependent on the availability of base year data and ranged from use of base year factors on similar facilities to the use of areawide peak hour factors.

No transferable documentation was found describing procedures for adjusting time-of-day curves based on the level of congestion on a facility or in a subarea or corridor. One study developed relationships between traffic level of service and the percentage of daily traffic in order to produce an estimate of the duration which congested conditions occurred within a study area. Unfortunately, the relationships were specific to local areas and required data on daily travel stratified by level of service ranges. Therefore, its applicability became severely limited. This is an area for further research.

Category 7 — Procedures to Derive Directional Distribution Data

The procedures in this category try to establish relationships between directional distribution and various factors, such as time-of-day, facility type, and orientation (i.e., radial, circumferential). The efforts to establish the statistical significance of these relationships have not been very successful. In lieu of these data, other procedures basically begin with a base year directional split (e.g., 60-40) and then make manual adjustments for future years using professional judgment and knowledge of abutting land uses (e.g., commercial, residential, industrial).

Two procedures to forecast peak hour traffic directional distribution were recommended. The first procedure, developed for the Maryland State Highway Administration by JHK & Associates, consists of the modification of base year directional distributions of peak hour traffic. The modification is based on a comparison of base year and future year work purpose traffic directional distribution in the facility corridor. One way to conduct this comparison is to perform traffic assignments of work purpose traffic in a production-attraction format for both the base year and future year. A less data-intensive, but more judgmental way to conduct this analysis is through a comparison of total base year and future year work trip (or residential trip) productions and attractions in the corridor.

The second procedure involves the transfer of peak hour directional distribution factors from facilities which today have characteristics like those envisioned in the future for the facility under analysis (88). The key characteristics that should be considered in such a transfer are highway type, location, orientation, and land use.

Category 8 — Procedures to Determine Vehicle Classification Data

Vehicle classification data usually consist of the percentage of total traffic that is comprised of light, medium, and heavy vehicles. Of these, the heavier truck classifications are the key variables to consider for highway design and environmental studies. The typical procedure used to determine vehicle classification data has been to assume that the base year vehicle classification of traffic on a facility will not change in the future. Similarly, the existing procedures to forecast vehicle classification characteristics are very judgmental and rely on data collected in a specific local area.

The recommended procedure includes an additional step. In this step the land-use changes in the traffic-shed of the facility under analysis are reviewed for the base year and future year. An estimate is then made of the degree of change in the proportion of those land uses in the traffic-shed that are known to generate truck traffic. This information is then used to modify the base year vehicle classification data. Similar relationships could not be established between vehicle classifications and such factors as time-of-day, facility type, and orientation of the facility. This is an area for further research.

Category 9 — Procedures to Calculate Speed, Delay, and Queuing Data

Various procedures were investigated to calculate speed, delay, and queuing data on grade-separated facilities and surface arterials. It was found that separate procedures were applicable for under-capacity and over-capacity conditions, a key distinction to be made in several environmental models. The characteristics of grade-separated facilities and surface arterials differ considerably because of the impacts of traffic signals and other controls for at-grade intersections.

The existing speed calculation procedures all involve a relationship between operating or average speeds, and the level of service or volume-to-capacity ratios on a facility. A series of curves have been developed in several studies (38, 45, 90) and in some computer software documentation (104, 115).

None of the available procedures adequately address the sensitivity of traffic speeds close to bottlenecks or to intersections. This sensitivity can be especially important in air quality and energy modeling. The procedures also differ in the calculation of speeds in over-capacity conditions.

The primary interest of delay and queueing procedures is at intersections where queuing can affect design needs (e.g., length of turn lanes) and localized environmental conditions (e.g., carbon monoxide hotspots). Several theoretical equations are available in the literature for modeling under-capacity conditions (120, 124). A deterministic procedure using various worksheets was provided in NCHRP Report 133 (91). In oversaturated conditions, fewer documented procedures were available. Linear models were reviewed from various sources (91, 104).

The recommended procedures combine the most relevant and straightforward techniques to calculate speeds, delay, and queuing. For under-capacity conditions on grade-separated facilities, the speed procedure uses a curve developed as part of the interim capacity materials of TRB Circular 212 (46). It is recommended that arterial speeds be determined through procedures documented in A Manual on User Benefit Analysis of Highway and Bus Transit Improvements published by AASHTO (90) and procedures documented in Signal Operations Analysis Package (SOAP) published by USDOT/FHWA (112). The arterial speed forecasting procedure combines relationships between mid-block average running speed and volume-to-capacity ratios with forecasts of intersection delays.

Procedures to calculate delay and queue lengths for under-capacity conditions are only applicable to surface arterials. The recommended procedure is based on Webster's equations (120) and is similar to the technique contained in the above referenced AASHTO Manual (90).

The procedures proposed for speed, delay, and queuing calculations for over-capacity conditions are those contained in NCHRP Report No. 133 (91). The procedure for grade-separated facility speed and queue length forecasts is based on a shock-wave method of queuing analysis. The procedure for surface arterial speed, delay, and queue length forecasts is based on a deterministic method of queuing analysis.

Category 10 — Procedures to Produce Traffic Data for Highway Pavement Design

The procedures most commonly used and recommended are those in the AASHTO Interim Guide for Design of Pavement Structures (5). The procedure involves the conversion of traffic data to 18-kip (18,000-lb) equivalents based on the forecast vehicle classification on the facility and statewide or station-specific rates of 18-kip equivalent single-axle loadings per 1,000 trucks. The 18-kip equivalent truck factors are then applied to each classification of vehicle in order to obtain a composite value for design purposes. Therefore, the time-of-day, directional distribution, and vehicle classification data obtained from procedures in Categories 6, 7, and 8 are directly used in this methodology. Some state agencies have computerized a similar version of the AASHTO 18-kip procedure, although most surveyed locations still use manual computations.

SUMMARY

This chapter has presented the major findings of the research study. A literature search, followed by the distribution of a questionnaire to several public agencies provided background on existing practies in producing highway traffic data in the United States. It was found that most agencies conduct regular base year traffic counting programs but do not have standardized procedures for forecasting traffic data for future year conditions.

The questionnaire results and subsequent personal and telephone interviews confirmed many of the insufficiencies in the traffic forecasting process. These include a lack of documentation of transferable procedures that can be applied in various situations, a lack of standardized formats for requesting and displaying traffic data for different applications, and the inability of current forecasting efforts to consistently produce realistic traffic data for various highway alternatives.

It was found that many traffic forecasting activities are performed using a vast amount of professional judgment with minimal reliance on any standardized procedures. As a result, documentation of procedures was incomplete or totally lacking in several of the traffic forecasting categories investigated in this study. Heavy emphasis, therefore, was placed on synthesizing portions of existing procedures and developing new procedures in response to the needs identified by practicing traffic analysts.

The procedures summarized in this chapter and fully described in the user's manual included in the Appendix to this report cover a total of 10 categories. These include traffic refinement and detailing procedures, procedures to produce specific traffic data needs (e.g., turning movements, hourly volumes, directional traffic distributions, and vehicle classifications), and procedures that use these data for environmental and highway design purposes. Various situations commonly encountered by the traffic analyst are addressed using examples and case studies wherever possible. The product of this effort is a manual of procedures that can be used to supplement, but certainly not replace, many traffic forecasts currently conducted using judgment alone.

CHAPTER 3

INTERPRETATION,APPRAISAL, AND APPLICATION

This chapter presents an interpretation and appraisal of the key issues and technical procedures involved in forecasting traffic data for highway project planning and design. Following this discussion suggestions are made for the application of these findings to current and future traffic forecasting efforts.

INTERPRETATION AND APPRAISAL

The major focus of this project was the examination of procedures for producing traffic data for use in highway planning and design activities. In the preceding two chapters, several findings were examined with regard to traffic forecasting issues revealed through the results of a questionnaire, telephone contacts, and personal interviews held with public agency staffs throughout the country. In addition, various procedures were identified and evaluated. These aspects of the study are explored more fully in terms of their meaning to practicing traffic analysts and their implications for needed improvements.

Development of Traffic Data

The study findings clearly indicate that a large percentage of highway planning and design decisions are based on the results of traffic data forecasts. At the same time, it is apparent that the quality of the input data and the analysis procedures used for these forecasts are viewed as being deficient in many respects. The following discussion focusses on the critical problems faced by the analyst who must develop the traffic data that are used for project planning and design. An understanding of these problems is necessary before the findings of this study can be fully interpreted and appraised.

The level of detail and accuracy of computer traffic forecasts vary tremendously from project to project. In one scenario a computerized travel demand forecast will have been made for each alternative under study using a stochastic capacity-restrained assignment procedure, with a great deal of effort having been expended on fine-tuning the land-use data inputs and defining a detailed highway network. Forecasts will have been made for each future year under study, and turning movements will have been produced for each critical intersection in the study area. In some cases a design hour computer assignment may even be available. In the more common scenario, however, computer forecasts are not available at this level of detail or accuracy or with the amount of fine-tuning of land use and network data that is desirable.

The lack of quality land-use forecasts was cited as a major problem facing the traffic analysts. Frequently, the analyst is required to manually adjust a traffic assignment to compensate for inaccuracies in land-use assumptions both in the base year and the future years. This problem occurs often when forecasts are requested for target years that do not match years for which land-use forecasts have previously been made. In such cases, the available land-use data must be manually interpolated or extrapolated to correspond to the target years. These extra computations and required assumptions can create land-use data errors or inconsistencies. Similarly, when the traffic analyst is performing small area studies, the available land-use data at the district or even zonal level is not accurate enough to produce reliable traffic forecasts on the specific facilities being examined.

The questionnaire results show that many traffic forecasts are still performed with all-or-nothing assignment procedures that assign all trips for a zonal interchange onto the same travel path, even though in reality travelers between the zones will choose a number of different travel paths of approximately equal travel time. The net result is imbalanced loadings on parallel routes. This situation still occurs to a lesser degree in the case of capacity restrained assignments that assign trips on the basis of available roadway capacity. In the majority of cases computer assignments are made using a 24-hour trip table and a systems-level highway network which does not provide the level of detail required for most project planning and design studies. Although capacity restraint procedures will lower the speeds on overloaded links during assignments, minimum travel paths may continue to be built through the overloaded links, thereby resulting in unrealistic assignments with link far exceeding capacities.

Many analysts showed a preference for all-or-nothing assignments, because travel patterns could be more readily traced and adjusted manually. Indeed, some of the procedures described in the user's manual, such as select link and zonal tree analyses, are more straightforward using all-or-nothing assignments. However, with continuing advances in assignment processes and more emphasis being placed on providing better coded highway networks and input data, capacity restrained methods can be expected to provide traffic assignments that will require fewer time-consuming manual refinements.

Due to limits on budgets, time permitted to perform analyses, and staff capabilities, computer assignments are often not available for all alternatives being considered. Many agencies are set up to forecast volumes for only a single year in the future, a year that is often somewhere in between the build year and the design year. At the present time the design year for most projects in project planning is somewhere between 2005 and 2010, but most agencies are performing systems-level forecasts for only 1995 or 2000.

Because of the cost of running large-scale computerized travel demand forecasts, the analyst on a project planning study must often be content with having a single systems-level traffic assignment with which to work. Network assumptions in the vicinity of the project under study may be different or much less detailed than the network assumptions the analyst has been told to use. Most analyses of alternative network assumptions must be done manually, traditionally through the use of judgmental procedures.

Similar is the case where either no computer forecast is available for use in the analysis or where the network used in the systems-level forecasts simply does not provide enough detail in the vicinity of the project under study. In most urban areas the majority of highway project planning and design studies are in rapidly growing fringe areas where the computer zone system and coded network are very coarse and in many cases even nonexistent. In these cases manual procedures must be relied upon to produce traffic forecasts for use in design and project planning.

Even under a scenario in which detailed computer assignments are produced for each alternative under study for both the build and design year, there is a large amount of additional data which must be developed for input to evaluation, environmental analysis, and design processes. The following is a list of traffic data which are often required in project planning and design studies:

- Average daily traffic volumes by link.
- Design hour traffic volumes by link.
- Turning movements for each intersection approach.
- Levels-of-service (mid-link, intersection, and interchange).
- Capacities (design and maximum).
- Level-of-service C volumes (for input to noise models).
- Diurnal curve (time-of-day) data.
- Vehicle classification data.
- Speed and delay data.
- Queuing data.

In some cases these data are required on all the links of a detailed network in order that the impacts of alternatives on total air pollutant emissions and energy consumption can be determined. Similarly, the impacts of the project on certain parameters such as time-of-day distribution, directional distribution, and vehicle classification characteristics are difficult to predict. As a result, existing patterns are often assumed to remain the same in the future, when in fact the effects of increased congestion levels and development patterns will cause these parameters to change. Guidance is needed on ways to predict changes in these variables. It seems paradoxical that extremely detailed traffic data must be developed for input to project planning and design when the systems-level computer forecasts that are used as a basis for producing these data are often very coarse and prone to error.

The standard computer traffic forecasting process consists of a chain of four separate models (trip generation, trip distribution, modal split, and trip assignment), each of which has inherent errors and biases. In some cases, these errors and biases are offsetting, and reasonable forecasts are generated for the facility being studied. However, in many cases the resulting traffic assignments require substantial refinement. Even validation assignments of base year traffic can be quite inaccurate, although validations are certainly of more benefit to the assignment process than the prevalent situations in which base year validations are not performed. A general rule of thumb for base year assignments for a particular roadway states that a good assignment has been performed if the assigned volumes are within 20 percent of actual observed volumes. Yet a 20 percent difference in traffic volumes can frequently mean the difference between providing a design level of service and exceeding the maximum capacity of a facility.

The survey results show that the responsibilities for various traffic analyses are fragmented among agencies. The state Departments of Transportation (DOT) provide the majority of these analyses, and in several states these functions are quite centralized. However, some states revealed that many traffic forecasting duties were allocated to the metropolitan planning organizations (MPO's), to district offices of the state DOT and in some cases to local agencies.

The variable role of the MPO's across the country with respect to traffic data development points to a need to better define their responsibilities. Whereas the land use and socioeconomic projections have traditionally been the responsibility of MPO's, the survey results indicate that other MPO roles vary from doing nothing to physically performing traffic counts.

Related to this issue is the observation that a majority of the surveyed agencies did not have any standardized format for requesting traffic data for various uses. The agencies that did use forms stated that this activity definitely reduced misunderstandings between the producers and users of the data. In most cases, the forms were simple one- or two-page requests for specific data to be used for planning, design, or environmental studies.

Analysis Procedures

Most transportation analysts have recognized the need to refine computer traffic assignments before submitting traffic projections for use in highway project planning and design. Various procedures are used throughout the nation, with varying levels of sophistication, standardization, and documentation.

The questionnaire results showed that over 50 percent of the responding agencies do not use standardized procedures for producing traffic data. However, there appears to be considerable standardization of procedures for using the resulting data. These procedures include the AASHTO user benefit analysis (90) and highway design methodologies (5, 6), the Highway Capacity Manual (38), and a number of environmental models. The primary implication of this disparity is that while public agencies are in partial agreement with respect to techniques for using highway traffic data, they have virtually no uniform procedures to initially develop those data. Thus, results of pavement design or air quality computations in one state cannot be readily compared with results obtained from adjacent states. This problem even manifests itself among urban areas within the same state.

The few standardized procedures currently being used to produce traffic data are typically poorly documented, poorly disseminated, and often only applicable to specific conditions. The documentation problem occurs because traffic analysts are typically not requested to fully document the procedures which they use to develop the traffic data. Documentation is also often performed as an afterthought some time after the analysis is completed, causing the analyst to overlook key details or helpful suggestions. Finally, the person who writes the documentation may not be the same person who performed the analysis. Thus, a very general report may result.

Some of the better documented procedures obtained in this study were retrieved from old project files or from a person's bookshelf. The procedure had often never been distributed outside of the department, much less the agency. This dissemination problem was not intentional in most cases, yet the information has failed to reach many of the analysts who could most benefit from it.

Many procedures were developed in response to the needs of specific project conditions, and therefore were limited in scope. For instance, regression equations used to forecast time-of-day distributions were typically based on a small set of localized data, and thus were not transferable to other urban areas or conditions. Other procedures were only partially developed to the extent required for use in specific traffic studies; the extra steps required to complete the procedures so that they would become more widely applicable were not undertaken.

The need existed to identify as many of the available procedures as possible, evaluate each of the procedures identified, draw upon the strongest points of the procedures evaluated, and develop a set of standard procedures that could receive national distribution. However, because of the great variance in the type and quality of computer forecasts that are used, and because of differing data requirements for different types of highway projects, it was necessary that a series of procedures be developed from which the analyst could select the most appropriate procedure for the particular study being performed.

The interpretations and appraisals of specific recommended procedures are thoroughly discussed in the user's manual. In terms of categories of procedures (see Table 1), it was apparent that the link-level traffic data refinement and detailing procedures (categories 1 through 4) were the least well documented and offered the greatest opportunity for variations among analysts. Typically the analyst is confronted with the need to convert a systems-level traffic forecast to some more detailed forecast within the immediate area of a proposed highway improvement. Several assumptions are required to perform such conversions. Therefore, the analyst must use a considerable amount of professional knowledge and judgment to apply even the most "standardized" procedures.

Because so many situations can occur which render any "cookbook" procedure useless, many analysts have resorted to using pure judgment for making such refinements. As a result, few documented procedures exist. The attempt in this research study was to combine the few available procedures with comments offered verbally by practicing analysts.

The second grouping of categories (categories 5 through 8 in Table 1) relates to procedures used to produce specific traffic data items, such as turning movements, hourly and directional distributions, and vehicle classifications. These procedures were somewhat better documented, possibly because they focussed more on data that could be obtained using mathematical computations rather than using pure judgment. Even so, several basic assumptions are required on the part of the analyst, such as whether or not traffic conditions in a future year would be expected to change significantly from those in the base year. Procedures to adjust for changing conditions were not readily available.

The final grouping of categories (categories 9 and 10 in Table 1) included procedures for translating the basic traffic data into inputs for evaluation, environmental, and design analyses. Procedures for computing speeds, delays, and queuing were readily available in the literature; however, the effects of over-capacity highway conditions on these variables were rarely examined. It was apparent that most analyses do not adequately represent traffic flow on congested facilities, a situation that is becoming increasingly familiar in urban areas. The highway design procedures were straightforward and related well to other procedures used to generate the input data. One realization was that the vehicle classification data required for the AASHTO pavement design procedure (5) are considerably more detailed and in a different format than data typically prepared for highway planning and environmental studies. Therefore, special care was taken to explain these characteristics in the procedure.

APPLICATION OF FINDINGS

The findings presented in this report and in the user's manual are of use to persons engaged in producing and using highway traffic data, such as transportation planners, traffic engineers, environmental analysts, and highway designers in federal, state, regional, and local agencies. Others who will derive benefits from these findings include persons engaged in safety studies, structural design, right-of-way acquisition, geotechnical and materials analysis, maintenance, and financial analysis. Outside of public agencies, land developers, consultants, and citizen groups will also find portions of these findings to be of use.

The findings of the study questionnaire and agency interviews provide an understanding of the strengths and weaknesses of current practices used in the traffic analysis field. Agencies can benefit from the organization and processes established by others to more efficiently perform these studies.

The procedures presented in the user's manual are applicable over a wide range of analyses. The principal types of applications include systems planning, corridor or subarea studies, evaluation of alternative plans, traffic operations studies, highway design, and environmental studies. In order to demonstrate some of this applicability, the procedures were applied to three case examples based on actual studies--a project planning study involving the upgrading of a freeway; a detailed subarea study involving the upgrading of an arterial facility; and a highway design study for constructing an interchange where two major arterial streets intersect. These illustrative examples show the types of procedures that can be applied as well as the level of judgment that is typically required. In all cases, emphasis has been placed on developing manual procedures, although the applicability of several techniques is enhanced with the aid of computer methods.

The procedures for refining systems-level traffic assignments (category 1 of Table 1) are applicable in corridor or subarea settings whether or not base year data are available. An adaptation to handle over-capacity conditions is also provided.

The material relating to alternative network assumptions (category 2) can be used to analyze changes in roadway capacity, changes in roadway alignment, construction of parallel roadways, or the addition or subtraction of network links. These situations occur in various combinations in most traffic analysis studies.

The windowing and focussing procedures for analyzing detailed highway networks (category 3) represent computer-aided approaches. Both are most applicable for small scale corridor or subarea studies in which detailed link and turning traffic volumes are desired on various highways that are not shown on a systems-level network. Simplified approaches are also described that are more applicable for quick-response studies.

Often the analyst is faced with the need to provide traffic data for study years for which no computer forecasts are available. Materials are presented (category 4) which permit available forecasts to be modified based on expected changes in land-use patterns. The procedures are flexible to permit an analyst to select between linear and nonlinear growth rates to be applied on a zonal or subarea corridor level. Treatment is also given to situations where development is approaching the full-buildout level.

The turning movement procedures (category 5) can be used to develop directional or nondirectional (i.e., two-way) turning volumes given various types of link volume data. Therefore, the analyst can use a systematic approach to estimate intersection turns for use in planning or design studies.

Design hour volumes (category 6) are the key data to produce for many traffic studies. Procedures are documented to permit design hour volumes to be determined for typical urban facilities and for facilities characterized by sharp recreational or seasonal variations. Other time-of-day procedures are useful to convert daily volume estimates to hourly data for use in design or environmental studies.

The procedures for determining directional distributions (category 7) are most applicable in design studies requiring estimates of peak direction traffic flows. They can also be of use in analyzing other transportation systems management actions, such as reserved bus and carpool lanes or reversible flow lanes.

The vehicle classification procedure (category 8) provides basic background relevant to the estimation of various auto-truck percentages on urban facilities. These data, in various formats, are key inputs to the calculation of highway design needs and to the determination of environmental impacts, including air quality, noise and energy consumption.

Procedures are presented for calculating speeds, delays, and queue lengths (category 9) on grade-separated facilities and on surface arterials. The analyst is able to apply different methodologies for traffic flow conditions that are under- or over-capacity. The resulting data are directly applicable to small area design analyses, such as the determination of turning lane length requirements, and to environmental analyses.

Highway pavement design (category 10) is a critical area for which specific traffic data are required. The procedures presented enable traffic volume and vehicle classification data to be converted into 18-kip equivalent single-axle loadings, which are directly used in the calculation of flexible and rigid pavement design needs. These procedures are applicable using vehicle classification data specific to the subject facility or average values obtained on a regional or statewide basis.

Generally, the procedures contained in the user's manual can be applied whether the system level traffic assignments have been produced through a computerized or manual process. Although the majority of applications would likely be in conjunction with a conventional UTPS traffic assignment, the procedure could also be used with assignments produced through manual or quick-response procedures, such as those contained in NCHRP Report 187 (88).

In summary, the findings provided in this report and in the user's manual have been shown to be appropriate for several types of applications. It is anticipated that some or all of the recommended procedures would be adopted by various agencies and personnel. The procedures presented are state of the art and are suggested to provide the traffic analyst the best analytical base for traffic estimates. It is expected that as the procedures receive widespread use, additional applications and suggested revisions or improvements will become apparent.

CHAPTER 4

CONCLUSIONS, SUGGESTED RESEARCH, AND RECOMMENDATIONS

CONCLUSIONS

The following general conclusions are presented based on the findings of the research:

1. Traffic data are used for three primary purposes in highway project planning and design in the United States: (a) for evaluation of alternative highway improvement projects; (b) for input to air quality, noise, and energy analyses of highway improvement projects; (c) for input to capacity and pavement design analyses.

2. The traffic data that are produced by systems-level computerized traffic assignment procedures must, in virtually all cases, be refined and subjected to further analysis in order that traffic data can be produced which can be used for highway project planning and design.

3. To date there has been virtually no national standardization of procedures for the development of traffic data that are used as input to evaluation, environmental, and design analyses. As a result, there are wide variations in the format and quality of traffic data produced by agencies.

4. Travel behavior is determined by a complex combination of a large number of factors. In response, the mathematical models used to forecast travel demand must make a number of simplifying assumptions and cannot take into account factors that are sometimes very important in determining travel behavior. As a result, traffic forecasts, particularly for individual facilities within a systems-level forecast, can vary significantly from actual observations.

Procedures to refine systems-level forecasts for use at the facility level are documented in the user's manual. It is critical that the user of these procedures realize that they are merely mechanisms to overcome some of the inability of the computer models to exactly replicate travel behavior. These procedures must be applied with considerable judgment and should only be applied after the analyst understands how the procedures work.

5. The procedures documented in the user's manual are designed to be used to produce facility-oriented data. Their applicability to larger sub-area studies is limited by difficulties in getting all routes in all directions to balance. Other new and emerging techniques, such as MICRO and TRAFFLO, should be considered when performing sub-area, rather than facility-oriented studies.

6. The procedures contained in the user's manual should be applied only after computer forecasts have been produced which pass a number of reasonableness tests. The types of checks that should be made and degree of accuracy required of the computer forecasts are documented in Chapter Three of the user's manual.

Special emphasis needs to be placed on ensuring the accuracy of land-use (socioeconomic) input data and coded network data. The majority of problems with systems-level forecast data used for highway project planning and design studies can be traced to problems with these data.

7. Production of adequate traffic data requires considerable effort and time as well as judgment that comes with experience. It is critical that agencies devote the time and effort necessary to produce a high quality forecast, because planning and design decisions that can raise or lower the cost of a highway project by millions of

dollars are often based on traffic data.

8. A large number of explicit and implicit assumptions are made every time traffic forecasts are performed for highway project planning and design studies. For instance, too often future traffic volumes have been forecasted using the assumption that existing or base year conditions will not change. Preliminary research in this study indicates that this assumption is not valid in many situations, especially in fast-growing suburban and rural areas. Therefore, it is important that both the producers and users of traffic data fully understand the sensitivity of the analyses to these assumptions and the implications of making alternative assumptions.

9. It is important that the producers of traffic data have a general understanding of how the traffic data are to be used to ensure that the proper data are prepared. Serious errors have often been caused in subsequent environmental or design analyses because of definitional misunderstandings about what data were required.

10. The users of the traffic data must understand the limitations and degree of uncertainty associated with traffic forecast data. Evaluation, environmental, and design analyses all require extremely detailed traffic data as input. These data often influence important decisions; therefore, it is important that the use of traffic data as input to these decisions be tempered by the degree of uncertainty associated with the forecasting process.

SUGGESTED RESEARCH

The following areas of research are suggested based on the results of this project:

1. The effects of over-capacity conditions on highways should be examined with respect to future land-use development as well as to the temporal and geographic distribution of traffic. It is apparent from this research that insufficient data currently exist to determine what dampening effects recurring congestion will have on future land-use growth in a corridor or subarea. These effects will influence the magnitude and shape of growth curves used to interpolate or extrapolate traffic volumes to alternate study years. Similarly, the extent to which congestion causes motorists to divert to alternative routes or to change the time at which the trip is made (e.g., "spreading of the peaks") is an important factor to examine further.

Future research could also include assessments of trip generation and trip distribution changes that occur as the result of various network modifications. For instance, the addition of a parallel facility in a corridor would likely influence the interzonal distribution of work and nonwork trips. A temporal shift in trip generation could also occur. The magnitude of these effects should be carefully determined.

2. Many of the manual computational procedures presented in the user's manual could be adapted to hand-held calculator or especially to microcomputer applications. For example, the repetitive screenline refinement calculations in Chapter 4 of the user's manual could be readily performed in much less time and with greater accuracy using a microcomputer. Additional screenlines could also be examined in an efficient manner. Other calculations such as those in the iterative turning movement procedure, in the speed, delay, and queue length procedures, and the manual assignment procedure are also candidates for microcomputer or calculator applications.

3. The windowing and focussing procedures presented in the user's manual would be enhanced by providing additional examples of their applications to various subarea network situations. The directional subzoning technique presented as a windowing option should also be applied to several network configurations in order to determine its maximum usefulness to traffic analysts.

4. The traffic growth curves developed for adjusting forecasted volumes to alternate study years are influenced by various factors, including land-use development trends, timing of development and highway improvements, and level of congestion (discussed previously). There is a need to better quantify these factors such that transferable parameters that influence traffic growth can be developed. The need is particularly acute to develop means to adjust traffic volumes in the vicinity of zones that are expected to have wide variations in expected growth. Such research should focus on specific effects on externally and internally generated traffic. If reasonable transferable parameters can be developed, the need to produce additional computer forecasts will be reduced.

5. The turning movement procedures require additional research to derive nondirectional and directional turns from nondirectional link volumes. This research would require more explicit accounting of land-use changes, roadway geometric modifications, and the development of transferable data for various facility types (e.g., freeways, arterials), locations (e.g., CBD, fringe suburban), and geographic orientation (e.g., radial, circumferential). These data would better systematize much of the judgment currently utilized in the procedures.

A noniterative procedure to derive directional turning volumes (73) should be further researched to increase its applicability and to simplify its calculations. Such a procedure, properly mechanized in a microcomputer or calculator, could enable reasonable turning movements to be derived in a more efficient manner than the iterative procedures.

6. Improved data and statistics are needed to transfer time-of-day, design hour volume, directional distribution, and vehicle classification data to other roadway types, to other geographic settings, and to future year scenarios. In the future year situation, techniques should be researched to adjust these relationships based on changes in land use, demographic data (e.g., employment, population, households), or expected roadway congestion. These data will increase the accuracy of future year traffic volumes used as key inputs for evaluation, design, and environmental studies. This research could build on data contained in the report An Analysis of Urban Travel By Time of Day (93).

7. Improved time-of-day data are required to relate design hour volumes to the average weekday peak hour and to establish truck hourly percentages throughout the day. The design hour volume (DHV) has often been substituted by the average weekday peak hour (AWPH) in performing traffic and design analyses. Although generally accepted for use by traffic and design analysts, the AWPH in several cases is not equivalent to the 30th highest annual hour. The magnitude of these differences and their implications on highway evaluation and design should be closely examined.

Similarly, improved time-of-day truck distributions are necessary. Current data do not accurately reflect the variations of truck volumes that occur during off-peak hours. Because several air quality and noise analyses often require detailed off-peak hour data for all highway modes, the inaccuracies in hourly truck volume estimation bias these results. Truck volume data in various categories (e.g., light, medium, heavy-gas, heavy-diesel) should be assembled over several time periods on facilities of different type, location, and orientation to major activity centers.

8. Improved relationships should be developed between various highway speed groups, such as design speed, operating speed, average speed, and average running speed. These relationships are important since current evaluation, design, and environmental models each require different speed data. One additional step may be to incorporate other factors besides the volume-to-capacity ratio into speed curves and equations. Such factors as land-use development, specific roadway

geometrics (e.g., lane widths, sight distance), and traffic signal characteristics (e.g., cycle length, phasing, progression) should be more explicitly considered in estimating speeds on different facility types.

9. Research should be conducted to better relate typical vehicle classification counts performed by agencies to truck loadometer station data required for highway pavement design. The research would establish statistical distributions of truck axle loadings for various truck types, highway types, geographic locations, and orientations to major activity centers. These transferable data would reduce the need to perform classification counts and investigate specific loadometer station data for each facility being analyzed.

Similarly, better means should be established to estimate truck classifications for each year of the highway design life, rather than assume that the annual truck rate will remain constant over that period. Providing this extra level of detail may improve the accuracy of the design calculations and increase the probability that the pavement will be properly designed.

10. All of the environmental models examined would benefit by better specificity and often simplicity of traffic data needs. The documentation should clearly distinguish between the types of traffic volumes (e.g., peak hour, 24 hour), speeds (e.g., average running speed, operating speed), and vehicle classifications (e.g., light, medium, and heavy trucks; motorcycles, etc.) which are required for application. Additional efforts should focus on standardizing and, if possible, reducing the traffic data needs for various air quality, energy, and noise models, as well as for currently used evaluation models. A common traffic data base for most models would improve the ability of the traffic analyst to produce quality data in a timely manner, and would improve the comparability of results.

RECOMMENDATIONS

This research project represents the first major effort to document standardized procedures for producing traffic data for use in project planning and design. It is critical that an effort is made to disseminate this documentation to both the producers and users of highway traffic data throughout the United States.

It is recommended that a training course be developed to facilitate the transfer of information contained in this report. At the same time, the U. S. Department of Transportation should make efforts to ensure that standardized procedures for developing traffic data are used on highway projects involving federal funding.

REFERENCES AND BIBLIOGRAPHY

1. Abrams, C. M. and DiRenzo, J. F., "Development and Evaluation of TSM Strategies." Measures of Effectiveness for Multimodal Urban Traffic Management. JHK & Assoc. and Peat, Marwick, Mitchell & Co., prepared for Dept. of Transp., Federal Highway Admin., Washington, D. C. (Dec. 1979).
2. Agent, K. R., Evaluation of the FHWA Highway Traffic Noise Prediction Procedure SNAP 1. Kentucky Bureau of Highways, prepared for U. S. Dept. of Transp., Federal Highway Admin., Washington, D. C. (Jan. 1980).
3. Ahmad, A. K., "Peak Period Urban Traffic Estimation." Masters Thesis, Center for Urban Transportation Studies, The University of Wisconsin-Milwaukee (Aug. 1981).
4. Alfa, A. S., "Time-Dependent Route Assignment of Peak Traffic." Transportation Engineering Journal, Vol. 107. American Society of Civil Engineers (Mar. 1981) pp. 153-163.
5. American Assn. of State Highway and Transp. Officials, AASHTO Interim Guide for Design of Pavement Structures 1972 (Revised). Washington, D. C. (1981).
6. American Assn. of State Highway and Transp. Officials, "A Policy on Geometric Design of Highways and Streets" (Review Draft No. 2). NCHRP Project 20-7 (1979).
7. Bacharach, M., "Biproportional Matrices and Input-Output Change." Cambridge University Press (1970).
8. Baerwald, J. E. (ed.), ITE Transportation and Traffic Engineering Handbook. Prentice-Hall, Inc. (1976).
9. Bailey, M. J., "Estimation of Design Hour Volume." ITE Journal. Washington, D.C. (Aug. 1981) pp. 50-55.
10. Barry, T. M. and Reagan, J. A., FHWA Highway Traffic Noise Prediction Method. U. S. Dept. of Transp., Federal Highway Admin., Washington, D. C. (Dec. 1978).
11. Barton-Aschman Assoc., Inc. and R. H. Pratt & Co. Division, Traveler Response to Transportation System Changes. Prepared for U. S. Dept. of Transp., Washington, D. C., 2nd edition (July 1981) & 1st edition (Feb. 1977).
12. Benson, P. E., Caline 3 - A Versatile Dispersion Model for Predicting Air Pollutant Levels Near Highways and Arterial Streets. California Dept. of Transp. (Nov. 1979).
13. Berman, N. et al., Trips in Motion: Methodology and Factors for Estimating Hourly Traffic Volumes from Average Daily Traffic. CALTRANS/LARTS (Sept. 1975).
14. Bowlby, W., SNAP 1.1: A Revised Program and User's Manual for FHWA Level Noise Prediction Computer Program. U.S. Dept. of Transp., Federal Highway Admin., Washington, D.C. (Dec. 1980).
15. California Dept. of Transp., Noise Manual. CALTRANS (n.d.)
16. Christiansen, D. L., Urban Transportation Planning for Goods and Services - A Reference Guide. Texas A & M University, prepared for Dept. of Transp., Federal Highway Admin., Washington, D.C. (June 1979).
17. Cohen, G. S. and Kocis, M. A., " Components of Change in Urban Travel." Research for Transportation Planning, Preliminary Research Report No. 159. New York State Dept. of Transp., Albany, N.Y. (July 1979).
18. Comsis Corp., Traffic Assignment. U. S. Dept. of Transp., Federal Highway Admin., Washington, D.C. (Aug. 1973).
19. Connecticut Dept. of Transp., Traffic Assignment Analysis Techniques. (n.d.)
20. Crowley, K. W. and Habib, P. A., Mobility of People and Goods in the Urban Environment: Facilitation of Urban Goods Movement. Prepared for U. S. Dept. of Transp., Washington, D. C. (Dec. 1975).
21. Curry, D. A. and Anderson, D. G., "Procedures for Estimating Highway User Costs, Air Pollution, and Noise Effects." NCHRP Report 133 (1972) 127 pp.

22. Darling, E. M., Jr. and Garlitz, J. D., Computer Modeling of Transportation-Generated Air Pollution, State-of-the-Art Survey II. U.S. Dept. of Transp., Washington, D. C. (Mar. 1978).
23. Davis, M. B., "Background and Description." The 1976 Urban and Rural Travel Survey. California Dept. of Transp. (1976) Vol. I.
24. DeLeuw, Cather & Co., Characteristics of Urban Transportation Systems. Prepared for U. S. Dept. of Transp., Washington, D. C. (May 1974).
25. Dunn, S. E., Highway Noise Prediction. Florida Atlantic University, Boca Raton, Florida (n.d.).
26. Erlbaum, N. S., "Procedures for Estimating Energy Consumption in Transportation Projects." Preliminary Research Report No. 174. Research for Transp. Planning, N. Y. State Dept. of Transp. (June 1980).
27. Ferlis, R. A., Guide for Estimating Urban Vehicle Classification and Occupancy. U. S. Dept. of Transp., Federal Highway Admin., Washington, D. C. (Sept 1980).
28. Fisher, R. F., Williams, G. J., and Boyd, P., Atlanta Vehicle Occupancy Monitoring. Georgia Dept. of Transp., Planning Data Services Section and Atlanta Regional Commission (Sept. 1979).
29. Florida Dept. of Transp., Flexible Pavement Design Manual. Div. of Road Operations, Florida Dept. of Transp. (Nov. 1980).
30. Furness, K. P., "Time Function Iteration." Traffic Engineering and Control, Vol. 7:7 (Nov. 1965) pp. 458-460.
31. Gerlough, D. L. and Huber, M. J., "Traffic Flow Theory, A Monograph." TRR Special Report 165, Transp. Research Board (1975) 222 pp.
32. Gordon, G. G., Galloway, W. J., and Kugler, B. A., "Highway Noise: A Design for Highway Engineers." NCHRP Report 117 (1971) 79 pp.
33. Guthman, L. E., User's Guide to MOBILE 1: Mobile Source Emissions Model. U. S. Environmental Protection Agency, Washington, D. C. (Aug. 1978).
34. Habib, P. A. and Crowley, K. W., "An Economic Approach to Allocating Curb Space for Urban Goods Movement." Paper for the Transp. Research Board Annual Meeting, TRB, Washington, D.C. (Jan. 1976).
35. Harmelink, D., Estimation of Annual Average Daily Traffic (AADT) and Thirtieth Highest Hour Volume. Ontario Department of Highways (1968).
36. Harrison, D. T. and Ward, T., "Carbon Monoxide "Hot Spot" Analysis: Providence, Rhode Island." Technical Paper No. 96. Office of State Planning, Providence, R. I. (May 1981).
37. Hartgen, D. T., "What Will Happen to Travel in the Next 20 Years?" Research for Transportation Planning, Preliminary Research Report No. 185. New York State Dept. of Transp., Albany, N.Y. (Aug. 1980).
38. Highway Research Board, "Highway Capacity Manual." HRB Special Report 87 (1965).
39. Huber, M. J., Boutwell, H. B., and Witheford, D. K., "Comparative Analysis of Traffic Assignment Techniques With Actual Highway Use." NCHRP Report 58 (1968) 85 pp.
40. Hupp, C. and Palombo, C., "Evaluation of the FHWA Vehicle Classification and Auto Occupancy Sampling Manual." Paper for the Transportation Research Board Annual Meeting, TRB, Washington, D. C. (Jan. 1980).
41. Illuminating Engineering Society of North America, American National Standard Practice for Roadway Lighting. IES, New York, N.Y. (July 1977).
42. Institute of Traffic Engineers, "Goods Transportation in Urban Areas." ITE Informational Report. Washington, D. C. (n.d.).
43. Jeffreys, M. and Norman, M., "On Finding Realistic Turning Flows at Road Junctions." Traffic Engineering and Control (Jan. 1977) pp. 19-21, 25 (Addendum Apr. 1977, p. 207).
44. JHK & Associates, Identification of Procedures Used to Develop Highway Traffic Data, (Technical Memorandum No. 1). Prepared for NCHRP Project 8-26 (Jan. 1982).
45. JHK & Associates, "Interim Materials on Highway Capacity." Transportation Research Circular No. 212, Transp. Research Board (Jan. 1980) 276 pp.
46. JHK & Associates, Refining Traffic Assignments: User's Manual. Prepared for Maryland Dept. of Transp. (1978).
47. JHK & Associates, Traffic Forecast Refinement. Prepared for Washington Council of Govt. and Maryland Dept. of Transp. (Apr. 1978).
48. John Hamburg & Assoc., Inc., "Improved Methods for Vehicle Counting and Determining Vehicle Miles of Travel. Evaluation of the Current State of the Art." NCHRP Project 8-20 (Nov. 1978) 2 vols.
49. Klais, A. E., Simplified Aids for Transportation Analysis, Annotated Bibliography. Wisconsin Dept. of Transp., Madison, Wisc. (Dec. 1975).
50. Klein, R. J., Review of the Baltimore Regional Planning Council Truck Modeling Procedures. Baltimore Region 3-C Process Regional Planning Council, Maryland (Dec. 1980).
51. Kopitzke, K., "Peak Period and Seasonal Adjustments of Traffic for Air Pollution Computations." Transportation Analysis. Minnesota Dept. of Transp. (Dec. 1977).
52. Kugler, B. A., Commins, D. E., and Galloway, W. J., "Highway Noise: A Design for Prediction and Control." NCHRP Report 174 (1976) 194 pp.
53. Kunselman, P. et al., Automobile Exhaust Modal Analysis Model. U. S. Environmental Protection Agency (Jan. 1974).
54. Lefkowitz, B., D'Esopo, D. A. and Korpi, M., Manual for the Gravity Model Computer Programs (Developed for the San Mateo County Traffic Ways Study). Stanford Research Institute, prepared for George S. Nolte Consulting Civil Engineers, Inc., Palo Alto, California (July 1962).
55. Leisch, J. E., "Capacity Analysis Techniques for Design of Signalized Intersections." Public Roads (Aug. 1967; Oct. 1967).
56. Lepore, T. F., An Interdisciplinary Approach to Queue Modeling for Use in Preparing Indirect Source Construction Permit Applications. Connecticut Dept. of Transp. (Feb. 1976).
57. Levenson, M., "Twin City Metropolitan Area Hourly Traffic Volume Study." Transportation Analysis. Minnesota Dept. of Transp., Planning Division (Oct. 1981).
58. Lobb, O., "Summary of Findings: Travel Data." The 1976 Urban and Rural Travel Survey. California Dept. of Transp. (1976).
59. Ludwig, F. et al., User's Manual for the APRAC-2 Emissions and Diffusion Model. Stanford Research Institute, Menlo Park, California, prepared for U. S. Environmental Protection Agency (1977).
60. Maricopa Assn. of Governments, Computer Analysis of West Phoenix Village Core: Documentation for PTI Project (Draft). MAGTPO (Sept. 1980).
61. Maricopa Assn. of Governments, MAGTPO Traffic Projection Procedures and Output (n.d.).
62. Maricopa Assn. of Governments, Peak-Hour Volume Characteristics - 1990 Forecast. Transp. Planning Program (n.d.).
63. Marshment, R., "Turning Movement Estimation Procedure." Middle Rio Grande Council of Governments of New Mexico. Unpublished procedure sent by letter to JHK & Assoc. (Jan. 8, 1982).
64. Mekky, A., "On Estimating Turning Flows at Road Junctions." Traffic Engineering and Control, Vol. 20:10 (Oct. 1979) pp. 486-487.
65. Mellem, N. and Halvorson, J., Study on the Attainment of Carbon Monoxide Standards.

Minnestoa Dept. of Transp. (Aug. 1980).

66. Memmott, J. L. and Buffington, J. L. "Predicting Traffic Volume Growth Rates Resulting From Changes in Highway Capacity and Land Development." Research Report 225-23. Texas Transp. Institute (Jan. 1981) 115 p.
67. Metropolitan Washington Council of Governments (MWCog), "Calibration of a Truck Trip Model for Use in the Transportation Integrated Model System (TRIMS)." Paper. MWCog (Feb. 23, 1978).
68. Metropolitan Washington Council of Governments (MWCog), "TRIMS MODEL (TRansportation Integrated Modeling System) - for Subarea and Quick Response Transportation Planning - User's Manual." Technical Report No. 9. MWCog (July 1976).
69. Metropolitan Washington Council of Governments (MWCog), "TRIMZONE (TRansportation Integrated Model at Zone - level) - for Highway Network Traffic Assignments." Technical Report No. 13. MWCog (April 1978).
70. Middle Rio Grande Council of Governments, "Arterial Utilization in the Albuquerque Urban Area." Technical Report No. 81 (1981).
71. Minnesota Dept. of Transp., "Methods of Traffic Analysis." System Planning and Analysis Report No. M-137 (n.d.).
72. Neveu, A. J., Quick Procedure to Forecast Rural Traffic. Transp. Statistics and Analysis Section Planning Division, New York State Dept. of Transp. (Oct. 1981).
73. Norman, M., Hoffman, N. and Harding, F., "Non-Iterative Methods for Generating a Realistic Turning Flow Matrix for a Junction." Traffic Engineering and Control (Dec. 1979) pp. 587-589.
74. Norris, G. A. and Nihan, N. L., "Subarea Transportation Planning: A Case Study." Traffic Quarterly, Vol. 33:4 (Oct. 1979).
75. North Central Texas Council of Governments, "Travel Model Development Report - Volume I." Dallas North Central Subarea Transportation Study (1981).
76. Owen, F., Northtown (T.H. 610) "Windowing" Technique, Transportation Analysis. Minnesota Dept. of Transp. (Oct. 1979).
77. Pedutó, F., Cioffi, G. and Albertin, R., Documentation of Selected Programs of the NYSDOT's Simulation System. N. Y. Dept. of Transp. (June 1977).
78. Petersen, W. B., User's Guide for HIWAY-2, A Highway Air Pollution Model. U. S. Environmental Protection Agency (May 1980).
79. Pignataro, L. J. et al., "Traffic Control in Oversaturated Street Networks." NCHRP Report 194 (1978) 152 pp.
80. R. H. Pratt Assoc., Inc., Low Cost Urban Transportation Alternatives: A Study of Ways to Increase the Effectiveness of Existing Transportation Facilities. Prepared for Dept. of Transp., Washington, D. C. (Jan. 1973).
81. Rosas, B. et al., Measuring and Modeling Carbon Monoxide at a High Volume Intersection. Minnesota Dept. of Transp., Federal Highway Admin. (Dec. 1980).
82. Rowan, N. J., Woods, D. L., and Stover, V., Alternatives for Improving Urban Transportation: A Management Overview. U. S. Dept. of Transp., Federal Highway Admin., Washington, D. C. (Oct. 1977).
83. Rudder, F. F., Jr., Lam, D. F. and Chueng, P., User's Manual: FHWA Level 2 Highway Traffic Noise Prediction Model, Stamina 1.0. Science Applications, Inc., prepared for U. S. Dept. of Transp., Washington, D. C. (May 1979).
84. Sanders, D. B. and Reynen, T. A., Characteristics of Urban Transportation Systems - A Handbook for Transportation Planners. Deleuw, Cather & Co., prepared for U. S. Dept. of Transp., Washington, D.C. (May 1974).
85. Shallal, L. and Khan, A. M., "Predicting Peak-Hour Traffic." Traffic Quarterly, Vol. 34:1 (Jan. 1980).
86. Simons, N. et al., Urban Goods Movement Program Design. Battelle, prepared for U. S. Dept. of Transp., Washington, D. C. (June 1972).
87. Skinner, L. E., Comparative Costs of Urban Transportation Systems. U. S. Dept. of Transp., Federal Highway Admin., Washington, D. C. (June 1978).
88. Sosslau, A. B. et al., "Quick-Response Urban Travel Estimation Techniques and Transferable Parameters, User's Guide." NCHRP Report 187 (1978) 229 pp.
89. Sosslau, A. B. et al., "Travel Estimation Procedures for Quick Response to Urban Policy Issues." NCHRP Report 186 (1978) 70 pp.
90. Stanford Research Institute, A Manual on User Benefit Analysis of Highway and Bus-Transit Improvements. American Assn. of State Highway and Transportation Officials (1977).
91. Stanford Research Institute, "Procedures for Estimating Highway Users Costs, Air Pollution, and Noise Effects". NCHRP Report 133 (1972) 127 pp.
92. Texas State Dept. of Highways and Public Transp., Traffic Engineering Programs: Passer II-80 (1980).
93. Tittemore, L. H. et al., An Analysis of Urban Travel By Time of Day. Peat, Marwick, Mitchell & Co., prepared for Dept. of Transp., Washington, D. C. (Jan. 1972).
94. U. S. Dept. of Commerce, "Documentation of Nine Methods for Estimating Transit Usage." Modal Split, Vol. 20, App. 48. (Dec. 1966).
95. U. S. Dept. of Commerce, Traffic Assignment Manual. Bureau of Public Roads, Office of Planning (June 1964).
96. U. S. Dept. of Transp., An Introduction To Urban Development Models and Guidelines For Their Use In Urban Transportation Planning. Federal Highway Admin., Washington, D. C. (Oct. 1975).
97. U. S. Dept. of Transp., An Introduction to Urban Travel Demand Forecasting - A Self-Instructional Text, (User-Oriented Materials for UTPS). Federal Highway Admin., Washington, D. C. (1977).
98. U. S. Dept. of Transp., Applications of New Travel Demand Forecasting Techniques to Transportation Planning: A Study of Individual Choice Models. Federal Highway Admin., Washington, D. C. (Mar. 1977).
99. U. S. Dept. of Transp., Calibrating & Testing a GRAVITY MODEL for Any Size Urban Area. Federal Highway Admin., Washington, D. C. (Oct. 1973).
100. U. S. Dept. of Transp., Caline 3 - A Graphical Solution Procedure for Estimating Carbon Monoxide (CO) Concentrates Near Roadways. Federal Highway Admin., Washington, D. C. (Dec. 1980).
101. U. S. Dept. of Transp., Energy Considerations in Transportation Planning. Federal Highway Admin., Washington, D. C. (Mar. 1979).
102. U. S. Dept. of Transp., Energy Requirements for Transportation Systems. Workshop notes, sponsored by Federal Highway Admin., Office of Environmental Policy, Washington, D. C. (June 1980).
103. U. S. Dept. of Transp., Evaluating Urban Transportation System Alternatives. Asst. Secretary for Policy and International Affairs, Office of Transp. Economic Analysis, Washington, D. C. (Nov. 1978).
104. U. S. Dept. of Transp., FHWA Computer Programs for Urban Transportation Planning. Federal Highway Admin., Washington, D. C. (July 1974).
105. U. S. Dept. of Transp., FHWA Traffic Noise Model (Computer Printout). Federal Highway Admin., Washington, D. C. (Nov. 1981).
106. U. S. Dept. of Transp., General Information.

Federal Highway Admin., Washington, D.C. (Apr. 1977) pp. 289-291.
107. U. S. Dept. of Transp., Urban Transportation Planning. Federal Highway Admin., Washington, D.C. (Mar. 1972) Chap. VII.
108. U. S. Dept. of Transp., Highway Travel Forecasts; Population, Economics, Fuel, Travel. Federal Highway Admin., Washington, D. C. (Nov. 1974).
109. U. S. Dept. of Transp., How to Prepare the Transportation Portion of Your State Air Quality Implementation Plan. Federal Highway Admin. with cooperation of U.S. Environmental Protection Agency, Washington, D. C. (Nov. 1978).
110. U. S. Dept. of Transp., "Interim Project Development Guidelines, 5-4-1." Noise Analysis Considerations - FHWA Standards, (Transmittal No. 17). Development Division, Federal Highway Admin., Washington, D. C. (July 16, 1976).
111. U. S. Dept. of Transp., PLANPAC/BACKPAC: General Information. Federal Highway Admin., Washington, D. C. (1977).
112. U. S. Dept. of Transp., Signal Operations Analysis Package (SOAP - Five Volumes). Federal Highway Admin., Washington, D. C. (1979).
113. U. S. Dept. of Transp., Trip Generation Analysis. Federal Highway Admin., Washington, D. C. (Aug. 1975).
114. U. S. Dept. of Transp., Urban Origin-Destination Surveys. Federal Highway Admin., Washington, D. C. (1973).
115. U. S. Dept. of Transp., Urban Transportation Planning System (UTPS), Reference Manual. Federal Highway Admin., Washington, D. C. (Apr. 1977).
116. U. S. Dept. of Transp., Urban Trip Distribution Friction Factors. Federal Highway Admin., Washington, D. C. (1974).
117. U. S. Environmental Protection Agency, Carbon Monoxide Hot Spot Guidelines, Vols. I - VI. Office of Air Quality Planning and Standards, Research Triangle Park, N. C. (Aug. 1978).
118. U. S. Enviromental Protection Agency, Guidelines for Air Quality Maintenance Planning and Analysis, Volume 9: Evaluating Indirect Sources. Office of Air and Waste Management, Research Triangle Park, N. C. (Jan. 1975).
119. Wang, J. J., Route Analysis Fundamentals. Washington Dept. of Highways, (n.d.)
120. Webster, F. V. et al., "Traffic Signals." Road Research Technical Paper No. 56. Road Research Laboratory, England (1966).
121. Westinghouse Electric Corp., Westinghouse Street Lighting Engineering Guide, B-5460 (n.d.).
122. Wisconsin Dept. of Transp., North Fitchburg Transportation Study. Division of Planning, Madison, Wisc. (Dec. 1975).
123. Wisconsin Dept. of Transp., "Revisions to Chapter 25: Socioeconomic Factors." Facilities Development Manual, (Transmittal No. 45). Division of Transp. Districts, Madison, Wisconsin (Nov. 16, 1981).
124. Yale Bureau of Highway Traffic, "Comparative Analysis of Traffic Assignment Techniques With Actual Highway Use." NCHRP Report 58, HRB (1968) 85 pp.
125. Yupo, C., A Review of Operational Urban Transportation Models. Peat, Marwick, Mitchell and Co., prepared for U. S. Dept. of Transp., Washington, D. C. (Apr. 1973).

APPENDIX
USERS GUIDE

CONTENTS

CONTENTS (Continued)

CHAPTER ONE
INTRODUCTION

GENERAL

This user's manual is a guide to aid in the development of traffic data for use in highway project planning and design. It is designed to serve as a state-of-the-art presentation of procedures that can be used to refine and detail traffic data obtained from computerized traffic assignment processes. The material is directed toward federal, state, regional, and local agency personnel involved in traffic planning, highway design, environmental studies, and related disciplines.

It is not the intent of this manual to produce a strictly "cookbook" approach to traffic forecast refinements. It should be clearly understood that there is a considerable amount of judgment involved in traffic forecasting. Professional judgment will always be an integral part of any transportation planning process. The objective is therefore not to eliminate enlightened judgment but to reduce the risks associated with poor judgment. This is accomplished by developing systematic procedures that can be applied in the appropriate circumstances and provide results that can be replicated within a reasonable range. As a result, increased confidence can be put into these forecasts by decision-makers and highway project planners and designers who must use the data.

The user's manual was developed as an appendix to the NCHRP research study entitled "Development of Highway Traffic Data for Project Planning and Design in Urban Areas." The manual contains a composite of several procedures identified and evaluated as part of the research study. In many cases, new or modified procedures are included to respond to the need for techniques which are applicable on a national scale.

ORGANIZATION OF THE MANUAL

This manual covers various procedures for developing traffic data for highway planning and design. The procedures can be used singularly or in combination, depending on the analysis to be undertaken. In many cases alternative procedures are presented, each appropriate to specific situations.

The user's manual is divided into 16 chapters. The following two initial chapters relate to the use of traffic data and preliminary checks that should be performed prior to using any specific procedure:

Chapter 2. Use of Traffic Data in Highway Project Planning and Design
Chapter 3. Preliminary Checks of Computerized Traffic Volume Forecasts

Each specific category of procedures is then described in a separate chapter:

Chapter 4. Refinement of Computerized Traffic Volume Forecasts
Chapter 5. Traffic Data for Alternative Network Assumptions
Chapter 6. Traffic Data for More Detailed Networks
Chapter 7. Traffic Data for Different Forecast Years
Chapter 8. Turning Movement Procedures
Chapter 9. Design Hour Volume and Other Time-of-day Procedures
Chapter 10. Directional Distribution Procedures
Chapter 11. Vehicle Classification Procedures
Chapter 12. Speed, Delay, and Queue Length Procedures
Chapter 13. Traffic Data for Design of Highway Pavements

The 10 chapters of procedures cover a wide range of techniques that are applicable singularly or in combination to various situations. The presentation of each procedure includes a discussion of its features and limitations, applicability, basis for development, input data requirements, directions for use, and worked-through examples in most cases. Extensive use of tables, graphics and appropriate worksheets is made in order to clarify these aspects to the analyst. In most cases, manually applied procedures have been described, although computer-aided techniques are presented where appropriate.

A series of three case studies has also been prepared to illustrate how these procedures can be combined into more comprehensive analyses:

Chapter 14. Case Study: Use of Refinement Procedures for Upgrading a Limited Access Highway
Chapter 15. Case Study: Use of Windowing Procedures for Evaluating an Arterial Improvement
Chapter 16. Case Study: Application of Procedures to Highway Design

An extensive bibliography is included at the end of the manual. This bibliography covers all documents that were directly used in the development of the procedures. Several other related references are included to provide additional information on selected topic areas.

APPLICABILITY OF PROCEDURES

The procedures presented in this manual are applicable over a wide range of analyses. The principal types of applications include the following:

1. Systems planning.
2. Corridor or subarea studies.
3. Evaluation of alternative plans.
4. Traffic operations studies.
5. Highway design.
6. Environmental studies.

Table A-1 depicts the applicability of each procedure to the foregoing six types of analyses. Most of the procedures in Chapters 4 through 10 can be applied both to systems-level or detailed studies.

The procedures in Chapters 11 through 13 were developed for application to specific types of detailed studies. In many cases these procedures should be used together to produce the desired traffic data output.

Table A-1. Applicability of procedures.

Chapter	Procedure	Application[1/]					
		Systems Planning	Corridor/ Subarea Studies	Evaluation Studies	Traffic Operations Studies	Highway Design	Environmental Studies
4	Refinement of Computerized Traffic Volume Forecasts						
	. Screenline Refinement Procedure	X	X	X	--	X	X
	. Select Link/Zonal Tree Analysis	X	X	X	--	X	X
5	Traffic Data for Alternative Network Assumptions						
	. Modified Screenline Procedure	X	X	X	--	X	X
	. Modified Select Link/ Zonal Tree Analysis	X	X	X	--	X	X
6	Traffic Data for More Detailed Networks						
	. Subarea Focussing/ Windowing Procedure	--	X	X	X	X	X
7	Traffic Data for Different Forecast Years						
	. Interpolation Method	X	X	X	--	X	X
	. Extrapolation Method	X	X	X	--	X	X
8	Turning Movement Procedures						
	. Factoring Procedures	--	X	X	X	X	X
	. Iterative Procedures	--	X	X	X	X	X
	. "T" Intersection Procedures	--	X	X	X	X	X
9	Design Hour Volume and Time-of-Day Procedures						
	. Typical Urban Facilities	X	X	X	X	X	X
	. Atypical Urban Facilities	X	X	X	X	X	X
10	Directional Distribution Procedures						
	. Modification of Base Year Data	--	X	X	X	X	X
	. Use of Anticipated Future Conditions	--	X	X	X	X	X
11	Vehicle Classification Procedures	--	--	--	X	X	X
12	Speed, Delay, and Queue Length Procedures						
	. Under-Capacity Conditions	--	--	X	X	X	X
	. Over-Capacity Conditions	--	--	X	X	X	X
13	Design of Highway Pavements	--	--	--	--	X	--

1/ X = Procedure is applicable to study type.

-- = Procedure is not applicable to study type.

CHAPTER TWO
USE OF TRAFFIC DATA IN HIGHWAY PROJECT PLANNING AND DESIGN

GENERAL

The use of traffic data in highway project planning and design varies considerably among urban areas. This variance is due to differences in decision-making processes, the scale and type of projects being considered, the amount of controversy surrounding each project, and the capabilities of technical staffs to produce the desired traffic data. Despite these differences, however, traffic data play an extremely important role in virtually all urban highway project planning and design studies. This role will be reviewed in this chapter.

Although the relationship between multimodal systems planning and highway project planning varies among agencies, in most cases the identification of the need for a highway improvement occurs during systems planning prior to the beginning of a highway project planning study. Project planning involves the analysis and evaluation of the feasibility, costs, benefits, and environmental impacts of a number of highway improvement alternatives designed to meet the identified need. Project planning normally terminates with a decision regarding the implementation status of a highway improvement project. Typically, when federal funds are involved this will include a decision on the part of the Federal Highway Administration to grant location approval for the project.

The amount of design that takes place during project planning as opposed to during a separate design phase also varies considerably among agencies and from project to project. Typically, project planning will involve what is usually termed preliminary or functional engineering. Preliminary engineering is designed to provide enough information to ensure that all significant impacts and accurate cost estimates can be determined, but it does not involve the consideration of design details. However, projects that are particularly controversal or environmentally sensitive, or projects for which both location and design approval are being simultaneously sought, will require a great amount of detailed design to take place during project planning. Detailed highway design involves the preparation of all engineering information that is necessary for a project to be implemented.

Despite differences in the project planning processes and variations in the specific information that is produced, virtually all urban areas in the United States use traffic data for three major purposes: (1) evaluation of alternatives, (2) input to environmental impact analyses, and (3) input to highway design. The remainder of this chapter is divided into discussions of each of these three categories of use.

EVALUATION OF ALTERNATIVES

The greatest variation in the highway project planning and design process occurs in the evaluation of alternatives. The method of evaluation varies depending on the decision-making process in an urban area, local area objectives, the type of project being considered, the critical issues associated with any given project, and the analysis procedures used.

Project planning studies develop information for all alternatives on the basis of a set of predetermined evaluation criteria that are designed to measure the impacts of each alternative. The evaluation criteria may include a number of cost-effectiveness measures that show impacts on a unit cost basis.

Some form of benefit-cost analysis is performed on most highway project planning studies. The most widely used guide for performing analyses of benefits and costs is the AASHTO manual, A Manual on User Benefit Analyses of Highway and Bus-Transit Improvements (90). Various traffic data are required to perform user benefit analyses following the AASHTO manual. For each highway link the following data are required:

- Representative directional hourly traffic volumes during peak and off-peak hours for each year during the design life of the facility.
- Percentage of vehicles by type:
  - Autos
  - Single unit trucks
  - Tractor-semitrailer combination trucks
- Link capacity.
- Operating speed.
- Accident rates.

For intersections the following information is needed:

- Green-to-cycle time ratio.
- Saturation flow.
- Capacity.
- Degree of saturation.
- Approach speed.

The foregoing data are used to determine user benefits through the calculation of reductions in travel time and delay and number of accidents by type. These data are then translated into vehicular operating cost savings.

Although many agencies perform some type of user benefit analyses for highway project planning studies, most have adopted simplified versions of the detailed procedures contained in the AASHTO manual. These procedures usually make simplifying assumptions for much of the input traffic data. For example, average daily traffic (ADT) data are often developed for only the build and design years for a project. ADT's are interpolated for all intermediate years. Standardized time-of-day, directional distribution, and vehicle classification percentages are then used to calculate hourly data by vehicle type.

In addition to performing benefit-cost analyses, project planners usually develop traffic data to be included in evaluation matrices designed to display key differences among alternatives. Although the data contained in these matrices vary considerably among and within urban areas, a number of key evaluation traffic data are normally developed. These include the following:

- Traffic volumes (link-specific or total screenline crossings) (24-hour or peak hour).
- Levels of service/volume-capacity ratios.
- Speed/travel time/delay.
- Vehicle miles of travel (VMT).
- Vehicle hours of travel (VHT).
- Number of accidents.
- Environmental data (i.e., air quality, energy consumption, noise).

These data may be displayed as absolute totals or relative to a no-build alternative.

Typically, the majority of these data are developed by traffic forecasting computer programs, although considerable refinement of the results may be necessary, particularly at the individual link level. For instance, problems are often encountered using link level-of-service data on arterial streets, since intersection capacity rather than link capacity normally controls. Average travel time and speed data must also be refined to account for intersection delays and to reflect differences in peak versus off-peak operating speeds.

As a result, biases will be introduced into the analyses unless these data are developed for the entire network of highways that are affected by the various alternatives. Except for relatively minor roadway improvement alternatives, it is difficult to estimate changes in these data without first performing a computer forecast. Even then, traffic diversion onto or off of minor roadways that are not coded into the network are not properly accounted for. Where computer traffic forecasts are not available, these specific data are frequently not developed.

Each of the procedures in this manual is applicable to the evaluation of alternatives. In most cases, a system-level traffic forecast will be refined and detailed at the link level using the procedures in Chapters 4 through 7. Specific turning movements or directional hourly volumes are then developed in Chapters 8, 9, and 10. As needed, the determination of vehicle classification or speed, delay, and queuing data can be made using procedures in Chapters 11 and 12 respectively. Evaluation of specific intersection/interchange designs may require the procedures in Chapter 13.

ENVIRONMENTAL ANALYSES

In most highway project planning studies, detailed analyses must be performed to estimate the impact of each highway improvement alternative on air quality, energy consumption, and noise. For highway improvement projects involving federal funding, procedures accepted by the Federal Highway Administration must be used. These procedures generally require very detailed traffic data inputs for which considerable development effort is usually required on the part of the analyst.

Table A-2 summarizes the key traffic and roadway input data requirements for several widely used environmental models. It is noted that data requirements can vary considerably among models; therefore, the traffic analyst must be familiar with the data needs for the specific model(s) being used in a particular area.

Environmental analyses will utilize data developed from several of the procedures presented in this manual. Link-refined 24-hour traffic volumes obtained in Chapters 4 through 7 are primary input to air quality, energy, and noise studies. Other specific data that are required for most of the models (see Table A-2) include time-of-day distributions (Chapter 9), vehicle type classifications (Chapter 11), and various forms of speed, delay, and queuing data (Chapter 12). Each of these elements must be closely examined in order to produce realistic environmental impact estimates.

Air Quality

The level of air quality impact analysis in most urban areas is dependent on background ambient air pollutant concentrations and the scope of the highway improvement project being studied. Current federal regulations require microscale air quality analyses for most highway projects. Mesoscale analyses are no longer normally performed for highway project planning studies because they are not sensitive enough to assess the relative impact of project level alternatives.

There are three pollutants typically analyzed in highway planning studies--carbon monoxide (CO), hydrocarbons (HC), and nitrogen oxides (NO_X). Because of the similarity of traffic data input requirements to estimate each of these pollutants, the following discussion will focus on CO as a practical example.

A two-step process is used to estimate CO and other pollutants (109). First, the emissions, or actual amount of pollutant, is determined. Second, the concentrations, or relative amount of pollutant, is calculated. Both estimates are important, although concentrations are more readily comparable among alternatives that have different roadway and traffic characteristics.

Carbon monoxide emissions are calculated using emission factors contained in the MOBILE Emission Factor Tables (33) or the EPA Modal Model (53). Other techniques typically used include the EPA Volume 9 guidelines (118) and the Carbon Monoxide Hot Spot Guidelines (36, 81, 117), both of which can be manually applied. Emissions at the intersection level can be simulated using the Intersection Midblock Model - IMM (117). The effects from various indirect sources, such as shopping centers, sports stadiums, and parking lots, can be modeled using the Indirect Source Model for Air Pollution - ISMAP (117), while applications to a network of links can be handled with the APRAC -1A or APRAC - 2 computer models (59).

Concentrations are calculated using a dispersion model, in most cases either the HIWAY-2 (78) or CALINE-3 (12, 100) model. The CO Hot Spot Guidelines and the APRAC, ISMAP, and IMM models can also compute concentrations.

Maximum 1-hour and consecutive 8-hour carbon monoxide concentrations are estimated at a number of "sensitive receptors" in the immediate vicinity of the highway project (e.g., residences, businesses, schools, parks, etc.). Concentrations will normally be estimated for both the expected year of opening and for the design year for the facility. Although traffic volumes usually are forecasted to be higher in the design year, emission rates per vehicle are expected to gradually decrease over time as a higher percentage of vehicles on the road are equipped with emission controls. Therefore, it is not always clear whether the highest CO concentrations will occur during the year of opening, the design year, or some year in between (66).

The primary inputs to CO and other pollutant emissions and concentration calculations are meteorological and traffic data, as given in Table A-2. Design hour or peak hour directional traffic volumes by vehicle class are usually used as traffic input to estimate 1-hour concentrations. These data are combined with estimates of vehicular average running or operating speeds and distribution of engine operating mode (i.e., percent of vehicles in the cold start, hot start, and hot stabilized modes). Although some agencies use the peak hour traffic volumes as input rather than the design hourly volume, in most cases this provides for a more conservative analysis than is necessary, since maximum CO concentrations in urban areas normally occur on cold winter days when peak hour volumes do not exceed design hour volumes. The principal exception to this situation will be in the vicinity of shopping centers where peak hourly volumes prior to the Christmas holidays may exceed design hour volumes (91). These hourly relationships are described further in Chapter Nine.

The peak 8-hour calculations should be for a consecutive time period which would produce the highest 8-hour CO emissions. Normally this period corresponds with the highest volume 8-hours (e.g., 11 AM to 7 PM). However, because CO emissions are greater at cooler temperatures, an 8-hour period during the morning may produce more CO emissions than a higher volume 8-hour period in the afternoon.

Vehicle classification input data consist of the percentages of vehicles that are autos (i.e., light duty vehicles) and light, medium, and heavy duty trucks. The heavy duty truck percentages are further subdivided into gasoline or diesel powered vehicles. This last breakdown is important to CO analyses in particular because diesel engines emit very little carbon monoxide.

Speed is a particularly important input variable, especially lower speeds. Below operating speeds of about 30-35 miles per hour, air pollution emissions increase significantly as operating speeds become lower. In the vicinity of intersections, more complex analyses are required to

Table A-2. Input data requirements for environmental models.

Input Data	AIR QUALITY[1]									ENERGY	NOISE		
	MOBILE (E)	Modal Model (E)	EPA VOL 9 (E)	Co Hot Spot Guide-lines (E,D)	APRAC (E,D)	ISMAP (E,D)	IMM (E,D)	CALINE (D)	HIWAY (D)	FHWA	STAMINA	SNAP	FHWA (Manual Method)
. Volume													
- 24 hr			X	X	X	X	X	From emissions models	From emissions models	X			
- Peak hr/design hr	X	X	X	X			X			X	X (at LOS=C)	X (at LOS=C)	X (at LOS=C)
- 8 hr	X	X	X			Interzonal Trips							
- Other	VMT												
. Capacity			X	X (for each approach)		X (for each approach)	X			X			X
. V/C Ratio							X			X (Density)			X (Density)
. Speed													
- Average running Speed	X	X	X		X (by facility type)	X		From emissions models	From emissions models	X			X
- Operating Speed			Also design speed	X			X			X	X	X	X
. Idle Time			X	X			X			X			X
. Stops			X	X			X			X			X
. Queue Length			X	X			X						
. Traffic Signals													
- Phasing			X	X		X	X						
- Cycle Length			X	X			X						
- G/C Ratio			X	X			X						
- Gap Acceptance							X						
. Diurnal Distributions					X (Includes weekends)	X	X						

Table A-2. Continued

Input Data	AIR QUALITY[1/]									ENERGY	NOISE		
	MOBILE (E)	Modal Model (E)	EPA VOL 9 (E)	Co Hot Spot Guide-lines (E,D)	APRAC (E,D)	ISMAP (E,D)	IMM (E,D)	CALINE (D)	HIWAY (D)	FHWA	STAMINA	SNAP	FHWA (Manual Method)
. Vehicle-Age Distribution	X	X	X	X	X	X	X			X			
. Vehicle Type Classification													
- Auto	X	X	X	X	X	X	X	From emissions model	From emissions model	X	X	X	X
- Light Trucks	2 types	2 types	X	X	2 types	2 types	2 types			X	X	X	X
- Medium Trucks			X								X	X	X
- Heavy Trucks			X										
- Gas	X	X		X	X	X	X			X			
- Diesel	X	X		X	X	X	X			X			
- Motorcycles	X	X		X	X	X	X						
- Other			Buses; Age mix							Bus	New vehicles		
. Percent Hot/Cold Starts	X	X		X		X	X			X			
. Roadway													
- Number of lanes			X (Plus ROW)	X	X (Width)	X	X (Width)	X (Width)	X (Width)		X		X
- Segment Length				X	X	X	X	X	X	X			X
- Surface Condition								X					X
- Grades										X	X	X	
. Receptors													
- Distance to Road				X	X	X		X	X		X	X	X
- Height					X			X					
- Angle of Observation								X			X	X	X
- Other											Barriers	Barriers	Barriers

1/ Air Quality Models
(E) = Emissions Model
(D) = Dispersion (Concentrations) Model

account for variations in emissions due to deceleration, acceleration, and idling at traffic signals or stop signs. Additional traffic data required in the vicinity of intersections include estimates of phasing, cycle lengths, and green time to cycle time (g/c) ratios. Queue lengths are required to determine if the receptors are affected by the queued vehicles. In order to calculate 8-hour CO concentrations, estimates are required of directional traffic volumes, speeds, and vehicle classification stratified for each of the peak consecutive 8-hours on an average weekday (91).

For both 1- and 8-hour concentration calculations, estimates must be made of the percentage of vehicles in the cold start, hot start, and hot stabilized modes. If a vehicle has not been used for some time it produces more CO during its initial phase of operation than if the engine is warm. The vehicle is considered to be in a cold start mode during the first 505 seconds of its operation if it has not been used during the previous 3 hours. It is difficult to estimate with any degree certainty what proportion of vehicles on a roadway will be in the cold start mode during a given time period, so most analysts use default values provided by the U.S. Environmental Protection Agency (12, 78). However, these default values are generalized for traffic on all facility types; as a result, the percentage of cold start vehicles on freeways and principal arterials are often conservatively overestimated.

The final set of traffic data required for air quality analyses are vehicle age distribution data. These data are necessary because newer cars have lower pollutant emissions. Although data are normally available regarding the number of vehicles registered from each model year, these data do not accurately reflect the vehicle age mix for vehicles on the road since newer cars and trucks tend to be driven more than older vehicles. As a result, in many urban areas national average age distribution data are used, even though considerable variation in these data can be found from one urban area to another.

Energy Consumption

The second category of environmental analysis requiring traffic data for input is energy consumption. The most widely used procedures for calculating highway energy consumption impacts are those prepared by the California Department of Transportation and contained in the "Energy Factor Handbook," which is published as an appendix to the notes for the Federal Highway Administration's workshop Energy Requirements for Transportation Systems (102). Several states have developed computerized versions of the procedures contained in the handbook (26, 101).

Energy consumption calculations are made for each vehicle type traveling over a segment of roadway during each year of the design life of a facility. The calculations are made using data that relate per-vehicle fuel consumption to operating speed, roadway grades and curvature, and pavement conditions. For each year during the design life of the facility, traffic data are required in the following categories:

- Volumes.
- Traffic density.
- Speed.
- Vehicle type classifications.
- Vehicle stops.

These data are specified further in Table A-2.

If adequate data are available, adjustments can also be made to account for the percentage of vehicles in the cold start mode, variances in vehicle age distribution from the national average, and variances in the percentage of gasoline versus diesel trucks from the national average. Where buses constitute a significant percentage of the traffic flow, separate calculations for buses can be made (101).

Noise

Noise models also require relatively specific input traffic data. As with air quality analyses, noise analyses are performed at a number of "sensitive receptors," which may be affected by noise from the proposed highway improvement. However, in contrast to most air quality pollutants, maximum noise levels do not necessarily occur at times of peak traffic volumes.

Two relationships must be considered. First, noise levels increase with speed on a per-vehicle basis. Second, noise levels increase with traffic volume. The result of this interplay is that total noise levels can actually be lower during congested conditions than during periods of lower traffic volumes. The point at which maximum noise levels occur, all other conditions being equal, is under level-of-service "C" traffic flow. Noise levels are also dependent on the number of trucks in the traffic flow passing by a sensitive receptor.

The standard model used to predict noise impacts is the FHWA Highway Traffic Noise Prediction Method (10, 105, 110), which was developed partly as a result of previous NCHRP research studies (32, 52, 91). Most agencies use the computerized versions of this model, either SNAP 1.1 (Simplified Noise Analysis Program) (2, 14) or STAMINA 1.0 (STAndard Method In Noise Analysis) (83). Other procedures in use in several agencies are generally similar in structure to the foregoing models and require the same input traffic data (15, 25).

In order to estimate noise levels, the several basic traffic data are required as input to the models. Table A-2 itemizes these data, which are summarized below.

Automobile Volumes

These volumes equal the lesser of the design hourly volume (reduced for truck traffic) or the maximum volume that can be handled under level-of-service C conditions. For automobiles, level-of-service C is considered to be the combination of speed and volume which creates the worst noise conditions. Alternatively, the average hourly volume for the highest 3 hours on an average day for the design year may be used for those highway sections where the above conditions are not anticipated to occur on a regular basis during the design year.

Truck Volumes

The design hourly truck volumes (for medium and heavy duty trucks) are used for those cases in which either the design hourly volume or level of service C volume was used for the automobile volume (see above). If the average hourly volume for the highest 3 hours on an average day was used for forecasting automobile traffic, comparable truck volumes should be used.

Operating Speeds

The operating speed should correspond to the traffic volumes chosen above.

In certain cases the above combination of traffic characteristics will not result in the most adverse noise conditions; if so, alternative traffic data should be developed. For example, on some roads truck volumes may be higher during off-peak hours than during peak hours, so using design hourly truck volumes may result in lower forecasted noise levels than would actually occur. In terms of impacts on noise levels, trucks contribute 20 to 30 times as much noise as automobiles, so it becomes essential that the input data accurately reflect the auto/truck traffic mix on the roadway during maximum noise periods. For new facilities, these periods can be estimated from noise readings taken near roadways that have traffic characteristics similar to those forecasted for the facility being studied.

The manual method of the FHWA noise model (10) includes some adjustments to account for noise occurring in interrupted flow (i.e., stop-and-go) conditions. To apply these adjustments, a value of average speed should be substituted for operating speed. The average speed value assumes the influence of traffic signal operations or other factors contributing to the interrupted flow. The truck noise factors are also increased to better replicate accelerating conditions.

HIGHWAY DESIGN

One of the most critical uses of traffic data is to perform preliminary and detailed engineering. Examples exist throughout the United States where the use of poor traffic forecast data has resulted in highway designs that were not appropriate for the level of traffic which ultimately used a facility after an improvement was made. Highway design in urban areas is also complicated by the fact that it is not always possible to design facilities that can provide adequate capacity to meet minimal design standards, because of fiscal or environmental impact considerations.

The two primary uses for traffic data in highway design are for capacity analyses and pavement design. To a lesser extent traffic data may also be used to determine lighting, shoulder, and lane width requirements, as well as distance requirements for offsets to trees, poles, guardrails, and other obstructions.

Capacity Analyses

In order to design highway facilities that will operate at an acceptable level of service, detailed capacity analyses are usually performed. These analyses are divided into three major types: (1) roadway segments, (2) interchanges, and (3) intersections.

Analyses are normally performed for forecasted traffic volumes during a design hour. AASHTO standards (6) call for the design hour to be the thirtieth highest hourly traffic volume expected during the design year, which in most cases is 20 years after the date of expected completion of the facility. In many urban areas traffic volumes during the thirtieth highest hour are approximated through the use of an average weekday peak hour volume. This topic is discussed further in Chapter 9.

AASHTO design standards require level of service C conditions on freeways and level-of-service D conditions on arterials in urban areas during the design hour. However, because of limitations in available fiscal resources, most states currently design for level-of-service D conditions in both freeways and arterials in urban areas. Even this level of service cannot be attained in certain cases.

Capacity analyses are performed using various methods available in such documents as the 1965 Highway Capacity Manual (38) and the TRB Circular 212, "Interim Materials on Highway Capacity" (45). In most cases the following design hour traffic data are required to perform capacity analyses:

- Directional traffic volumes.
- Merging, diverging, and weaving volumes in interchange and weaving areas on freeways.
- Intersection turning movements.
- Percent trucks and buses.
- Peak-hour factors.

In addition, information on roadway geometrics and intersection signal phasing is required.

The capacity analysis calculations require specific traffic data to be forecasted for a design hour which is normally 25 to 30 years in the future. As difficult as it is to forecast traffic volumes that far into the future, it is important that these data be reasonably accurate. The traffic data are used to determine the number of lanes required on the main line of both freeways and arterials, the type and number of lanes on ramps in interchange areas, the lengths of weaving sections on freeways, the number of approach lanes at intersections, the number and length of turning lanes at intersections, and the signalization requirements at intersections. Variations in design-hour traffic volumes of as little as 10 to 20 percent can result in substantial changes in design requirements, particularly at interchanges and intersections. Therefore, it is important that high quality traffic data be forecasted.

The refinement and detailing procedures presented in Chapters 4 through 7, combined with the time-of-day and directional distribution procedures in Chapters 9 and 10, enable the analyst to reduce the expected link traffic variations to reasonable ranges. The procedures can also be used to adjust interchange ramp and weaving volumes. The procedures in Chapter 8 will assist the analyst in producing realistic turning movements for use in conducting intersection capacity analyses. The vehicle classification procedures (Chapter 11) will provide the needed truck percentage factors.

Pavement Design

The second major use of traffic data for engineering is in pavement design. In designing pavements, the key input traffic parameter is the number of "equivalent" 18,000-pound single-axle loads that are expected during the design life of the pavement. The AASHTO Interim Guide for Design of Pavement Structures (5) has developed a series of "equivalence factors" for converting axle weight group traffic volumes to 18-kip equivalent loads. In many agencies these conversions are made through the use of computer programs that require as traffic inputs the average annual daily traffic (AADT), the percentage of trucks, and truck axle loading characteristics (obtained from loadometer stations) during each year of the design life of the pavement (usually 20 years). Experience in a number of agencies has shown that pavements have often been underdesigned because of the under-forecasting of truck volumes. As a result, the pavements have deteriorated more rapidly than originally anticipated.

The development of traffic data for pavement design is presented in Chapter 13. The vehicle classification procedures in Chapter 11 can be used to help estimate the percentage of trucks, although truck axle loadings should be determined from local or state loadometer station data.

OTHER USES

Traffic data are also used in establishing lighting, shoulder, and lane width requirements, as well as to set offset requirements to roadside obstructions, such as trees, poles, and guardrails. Design of these features is often dependent on the ADT range within which the traffic falls for the facility under consideration. Although not described in this manual, procedures for using traffic data for these purposes may be found in several documents (5, 6, 8, 38, 41, 121).

TRAFFIC PROJECTION REQUEST FORM
P-S-30 10-77

State of Wisconsin / Department of Transportation

TO: DOT – Planning
Planning Methods and Forecast Section
Hill Farms

From: District ______	Date:
Received:	Scheduled:
Assigned to:	Date out:

Project Description

Project ID: ______

Location: ______

Route: ______ County: ______

Forecast Year(s):

Estimated Time of Completion: ______

ETC + 10 years: ______

ETC + 20 years: ______

Other: ______

Design Data Requested *(Check those items required)*

- ☐ Mainline Volumes
- ☐ Truck Classification for Pavement Design and Noise Analysis
- ☐ K (% ADT in DHV); D (% DHV in predominate direction of travel); T (DHV) (% trucks in DHV)
- ☐ Turning Movements (Provide sketch indicating locations desired)
- ☐ P (% ADT in Peak Hour); T (PHV) (% Trucks in Peak Hour)
- ☐ K_8 (% ADT occurring in the average of the 8 highest consecutive hours of traffic on an average day)
- ☐ Truck Classification adjusted for Air Quality Analysis.

Supplemental Attachments *(Check those items provided)*

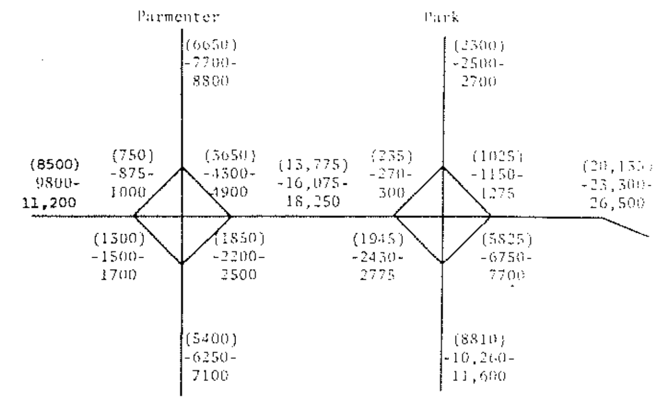
- ☐ County or Municipality map showing projection location. (To accompany all requests).
- ☐ A sketch showing existing or anticipated land development affecting this projection.
- ☐ Turning Movement Count(s).
- ☐ Other pertinent data: ______

Special Counts

TYPE	DATE	LOCATION	VOLUME

Remarks:

Figure A-1. Example of traffic request form.



Heavy Duty Truck Classification for Noise, Pavement Design and Air Quality (HDD & HDG)

Type	% ADT	Fuel Usage			
		% Diesel		% Gasoline	
		1985	2005	1985	2005
2D /1	2.4	10	75	90	25
3Ax	1.5	75	95	25	5
2-S1	0.1	75	98	25	2
2-S2	0.2	92	99	8	1
3-S2+	3.4	99	100	1	0
Total	7.7				

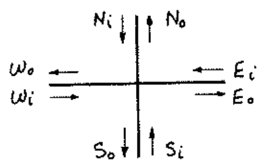
/1 Considered as "MT" for Noise Analysis

K* = 10.0%
P(ADT) = 12.0%
T(DHV) = 5.4%
T(PHV) = 4.5%
D(DHV) = 55-45
K_8 = 6.2%
T(ASHV) = 7.7%

*K is the highest hour percentage in the average weekday traffic.

Figure A-2. Example of data produced in response to traffic request form.

Table A-49. Balanced hourly directional volumes.



Time	NI_i^*	NO_i^*	SI_i^*	SO_i^*	EI_i^*	EO_i^*	WI_i^*	WO_i^*	I_i^*
12-1 Am	562	696	531	406	305	379	356	273	1754
1-2	341	489	379	242	194	257	245	171	1159
2-3	187	232	176	135	102	126	119	91	584
3-4	213	213	157	157	116	116	106	106	592
4-5	247	184	132	189	125	110	97	118	601
5-6	1124	625	428	899	277	200	169	274	1998
6-7	3376	1452	972	2769	1037	646	534	1052	5919
7-8	4771	2220	1511	3839	2083	1507	1272	2071	9637
8-9	2903	1865	1317	2256	1555	1280	1118	1492	6893
9-10	2146	1726	1249	1628	1083	1023	915	1016	5393
10-11	1992	1855	1362	1489	1203	1120	1017	1110	5574
11-12	1991	1852	1362	1487	1157	1164	1059	1066	5569
12-1 Pm	1914	1914	1419	1419	1157	1157	1059	1059	5549
1-2	2213	2059	1514	1653	1157	1165	1059	1066	5943
2-3	2435	2265	1666	1818	1272	1281	1165	1174	6538
3-4	2656	2854	2135	1952	1554	1670	1543	1412	7888
4-5	3395	4533	3480	2433	1731	2128	2019	1531	10,625
5-6	2604	4367	3438	1821	1457	1897	1845	1259	9344
6-7	2249	2792	2123	1626	1277	1463	1373	1141	7022
7-8	1918	1914	1421	1418	1221	1322	1212	1118	5772
8-9	1432	1538	1150	1052	999	1075	992	908	4573
9-10	1432	1536	1150	1050	851	989	915	773	4348
10-11	1022	1099	821	753	694	689	635	631	3172
11-12	783	907	683	572	463	456	424	418	2353
Total	43,906	41,187	30,576	33,063	23,070	23,220	21,248	21,330	118,800

2-Way Total	85,093	63,639	46,290	42,578
Forcasted ADT	85,400	63,300	46,400	42,500
Difference	-307 (0.4%)	+ 339 (0.5%)	- 110 (0.2%)	+78 (0.2%)
Comparison	OK	OK	OK	OK

Table A-50. Peak hourly vehicle classifications for each link.

NORTH LINK

Time	Inbound				
	Auto	Light	Med	Heavy	Avg. Running Speed
11A-12P	1768	54	121	48	31
12-1P	1711	56	103	44	31
1-2P	1932	69	148	64	30
2-3P	2084	83	192	75	30
3-4P	2268	85	212	88	29
4-5P	3039	122	166	68	28
5-6P	2406	89	81	28	29
6-7P	2101	65	58	25	30
	Outbound				
11A-12P	1644	50	113	45	31
12-1P	1711	56	103	44	31
1-2P	1798	64	138	60	30
2-3P	1939	77	179	70	30
3-4P	2437	91	228	94	29
4-5P	4057	163	222	91	25
5-6P	4035	148	135	48	26
6-7P	2608	81	73	31	29

EAST LINK

Time	Inbound				
	Auto	Light	Med	Heavy	Avg. Running Speed
11A-12P	1027	31	71	28	32
12-1P	1034	34	62	27	32
1-2P	1010	36	78	34	31
2-3P	1089	43	100	39	31
3-4P	1327	50	124	51	30
4-5P	1549	62	85	35	31
5-6P	1346	50	45	16	31
6-7P	1193	37	33	14	31
	Outbound				
11A-12P	1034	31	71	28	32
12-1P	1034	34	62	27	32
1-2P	1017	36	78	34	31
2-3P	1097	44	101	39	31
3-4P	1426	53	134	55	30
4-5P	1905	77	104	43	29
5-6P	1753	64	59	21	29
6-7P	1366	42	38	16	30

SOUTH LINK

Time	Inbound				
	Auto	Light	Med	Heavy	Avg. Running Speed
11A-12P	1209	37	83	33	32
12-1P	1269	40	77	33	32
1-2P	1322	47	101	44	31
2-3P	1426	57	132	52	31
3-4P	1823	68	170	70	30
4-5P	3115	125	171	70	28
5-6P	3177	117	107	38	28
6-7P	1983	62	55	23	30
	Outbound				
11A-12P	1320	40	91	36	32
12-1P	1269	41	76	33	32
1-2P	1443	51	111	48	31
2-3P	1556	62	144	56	31
3-4P	1667	62	156	64	30
4-5P	2178	88	119	48	30
5-6P	1683	62	56	20	31
6-7P	1519	47	42	18	31

WEST LINK

Time	Inbound				
	Auto	Light	Med	Heavy	Avg. Running Speed
11A-12P	940	29	65	25	32
12-1P	947	31	57	24	32
1-2P	925	33	71	31	31
2-3P	997	40	92	36	31
3-4P	1318	49	123	51	30
4-5P	1807	73	99	40	29
5-6P	1705	63	57	20	29
6-7P	1282	40	36	15	30
	Outbound				
11A-12P	952	31	58	25	32
12-1P	947	31	57	24	32
1-2P	931	33	71	31	31
2-3P	1005	40	93	36	31
3-4P	1206	45	113	47	30
4-5P	1370	55	75	31	30
5-6P	1163	43	39	14	31
6-7P	1066	33	30	13	31

include time to format the traffic data for specific environmental models; however, this effort would be minmal.

This time is divided by steps, as follows:

	Person-hours
Step 1: Develop hourly directional volumes	8
Step 2: Determine turning movements	1
Step 3: Perform capacity analysis	2
Step 4: Determine traffic data for environmental analysis	2
Step 5: Determine traffic data for pavement design	3
Total	16

The largest single effort is to develop the hourly directional volumes and to balance the inbound and outbound intersection movements (Step 1). The remaining steps require minimal time. However, if additional design alternatives are to be analyzed, the time requirements for Steps 3, 4, and 5 would increase roughly by a factor equal to the number of alternatives. Therefore, for three alternatives, the capacity analysis may require 3 x 2 hr = 6 hr.

In summary, these manual procedures can be isolated highway design options in a cost-efficient manner, such that sufficient data are provided for evaluation, environmental analyses, and for pavement design.

DATA FORMAT

Data for highway project planning and design should be requested using a standard format to reduce misunderstandings between the producers and users of the data. In turn, the developed data should be presented in a consistent, straightforward manner.

A typical example of a form used in Wisconsin is shown in Figure A-1. This form clearly requests the following information:

- Specific location of highway segment including a map (location, route, county).
- The forecast year(s).
- The traffic data requested (e.g., link volumes, vehicle classifications separate for design, noise, and air quality studies, hourly and directional distributions, turning movements including a sketch).
- Anticipated land-use development (shown on a sketch).
- Turning movement counts.
- Special counts specified by date and location.

An example of data received using this form is shown in Figure A-2. Similar forms can be developed to match local needs.

CHAPTER THREE
PRELIMINARY CHECKS OF COMPUTERIZED TRAFFIC VOLUME FORECASTS

GENERAL

In most urban areas in the United States some form of computerized travel demand forecasting process has been developed which serves as the basis for producing system-level traffic forecasts. The manner in which these forecasts are used for deriving project planning and design traffic data varies considerably among urban areas. In some cases computerized traffic forecasts are directly used with little or no refinement. In a few cases formalized step-by-step procedures have been developed for refining computer forecasts. Usually, however, link-level traffic volume forecasts are adjusted using considerable professional judgment to account for limitations in the traffic assignment modeling process. As a result, the "procedures" that are used to refine computer forecast data are being "documented" only in the minds of the analysts who perform the refinements.

Despite the wide variance in refinement processes, it is important to recognize that the refinement of system-level traffic forecasts is one of the most critical tasks in performing highway project planning and design studies. The development of accurate traffic forecasts often can determine the ultimate cost-effectiveness of project planning and design decisions. Refinement of system-level traffic forecasts requires a review and modification of computer model results and considerable knowledge of the limitations of the computerized modeling process.

The refinement process can be divided into two major elements: (1) checking the results of a computer assignment for accuracy and reasonableness, and (2) adjusting computer-generated link volumes to account for limitations in the assignment process. This chapter describes a number of preliminary checks of system level forecasts that should be performed to ensure the overall accuracy and reasonableness of results. Subsequent chapters provide documentation of procedures that can be used to adjust computer generated volumes to produce refined facility level traffic volumes for use in highway project planning and design.

The preliminary checks are used to identify and correct any errors that may have occurred during the system modeling process. These errors can occur during several stages of the forecasting process, including the following:

- Network coding (link capacities, speeds, length, etc.).
- Trip generation.
- Trip distribution.
- Modal split.
- Trip assignment.

There are several straightforward checks that can be used to determine whether or not a traffic forecast is suitable for further refinement. These checks should be performed as part of any traffic forecasting process, regardless of the ultimate use of the traffic data. These checks should routinely be performed as the first step for all system-level planning activities. Therefore, the analyst may only need to verify that suitable checks had been made during previous planning efforts. If considerable time has lapsed since the system-level planning activities, it is useful for the analyst to review all of these checks to ensure that the forecasts are still valid for use in conducting facility-level analyses. Obviously, the traffic refinement procedures presented in later chapters can only produce realistic results if the original system-level traffic forecast is reasonably accurate.

An analyst should begin the check of system-level traffic forecasts by examining base year and future year socioeconomic data on a zone-by-zone basis to gain an overall understanding of probable changes in travel patterns within the study area. Total trip generation by zone should be compared with land-use data to ensure that logical relationships exist. The computer highway network should be examined to check for errors in link definition. Where available, base year traffic assignments should be compared with actual base year traffic count data to ensure that existing traffic patterns are being adequately simulated. Finally, the forecasted traffic growth between the base year and future year assignments should be compared with historical trends for reasonableness.

To the extent possible, these preliminary checks should be performed at a regional or subregional level to ensure that the models are operating correctly. At a minimum, these checks should be conducted in the subareas or corridors for which subsequent traffic refinements will be required. It should be recognized, however, that decisions regarding the accuracy and reasonableness of the system-level traffic assignment can rarely be made by analyzing only a small portion of the network.

Prior to performing the preliminary checks the analyst should determine the format in which the traffic data are reported. Typical formats include the following:

- Directional--Volumes and capacities are specified for each direction of travel on a link and for each arrival and departure leg of an intersection (i.e., node).
- Nondirectional--Volumes and capacities are combined for both directions of travel on a link (except for one-way links) and for both the arrival and departure legs of an intersection (i.e., node).
- 24-Hour--Volumes and capacities are presented in terms of 24-hour values. Volumes typically relate to average daily traffic (ADT), while capacities represent a multiplier of peak-hour capacities. Typically, 24-hour capacities are assumed to be equal to ten times the peak-hour capacity. For example:

Peak-hour capacity	=	2,000 vph
Factor for 24 hours	=	10
24-hour capacity	=	20,000 vph

This factor assumes that 10 percent of the 24-hour traffic occurs during the peak hour.

- Peak Hour--Volumes and capacities are presented in terms of a single peak-hour of an average weekday. Peak hour forecasts can be produced for AM and/or PM conditions.
- Peak Period--Volumes and capacities are presented in terms of a series of hours during either the AM or PM peak period. Typically, a 2- to 3-hour time period bracketing the peak hour is used to represent the peak period.

These formats must be ascertained in order to adequately conduct the following preliminary checks and to refine or detail the traffic on specific facilities, as presented in subsequent chapters.

The following preliminary checks are based on the availability of a base year traffic assignment. The first check involves reviewing the input socioeconomic data for reasonableness for the base and forecast years. The second, third, and fourth checks are checks only of the base year assignments and include base year trip end summary simulated VMT and simulated link traffic volume checks. The fifth check is the check of the forecast year assignment and includes trip end summary and VMT checks for the forecast year.

CHECK 1—EXAMINE LAND-USE DATA ASSUMPTIONS

Prior to performing any other checks the analyst should become familiar with the amount and type of existing and forecasted land use in the traffic shed area of the facility for which traffic data are being produced. The assumed level of land-use development is the single most critical variable in forecasting the number of trips generated within a study area. Problems in simulating base year traffic volumes can often be traced to problems with zonal-level land-use data. Similarly, discrepancies between the base year and future year traffic forecasts can often be attributed to errors or inconsistencies in expected land-use changes. Therefore, it is important that these land-use data be closely examined and understood by the analyst. This check is especially important in situations where considerable time has lapsed since the system-level forecasts were made. In such cases, the future year land-use assumptions should be carefully reviewed to make sure they are still valid.

CHECK 2—COMPARE TRIP END SUMMARIES TO LAND-USE

Data contained in the computer-generated trip end summaries should be compared with the input land-use data for each study year. Trip end summaries provide data on either total productions and attractions or origins and destinations for each traffic analysis zone. These data can be displayed as total trips by time period and are frequently subdivided by trip purpose. The origin is always the starting point of a trip and the destination the ending point of a trip. For home-based trips the home end of the trip is always the production end and the nonhome end the attraction end. For nonhome-based trips, the origin end is the production end and the destination end is the attraction end of the trip.

Total trip ends for the zones of interest should first be compared with the corresponding land-use data. Emphasis should be placed on identifying extreme values (e.g., high or low) of either trip ends or land use. For further specificity, trips stratified by purpose and/or by mode should be individually examined. In all cases, comparisons between zonal trip ends and land use should be made for the base year and future year forecasts.

Several situations should flag the analyst's attention. For example, a particular zone may exhibit a very high number of work trip productions despite having a relatively low number of households. Similarly, a zone may show a very low number of nonwork trip attractions despite having a high level of retail employment. These situations would justify further checking of input data assumptions. In many situations, separate traffic forecasts are performed using different land-use assumptions during the same forecast year. In such cases, the trip end summaries can be compared among alternatives to determine if the differences in trip-making are commensurate with the changes in land use. If they are not, logical explanations should be closely examined (e.g., a network change may have occurred, or the modal split between alternatives may be different).

Any problems may be the result of computer errors, incorrect trip generation rates applied to the zones, or characteristics unique to that zone. Assessment of the first two factors can be accomplished by checking the input parameters or the computer software; however, assessment of the latter factor requires an intimate knowledge of travel and land-use characteristics in the zone.

CHECK 3—EXAMINE HIGHWAY NETWORK

Preliminary checks can help identify network coding errors on specific highway links. Typical errors occur in defining link distances, link capacities, link impedances (i.e., speed or time), and locations of centroid connections.

Extreme traffic volumes (i.e., high or low) assigned to a link(s) usually point to a coding problem. In particular, centroid connectors often show extreme values because their impedances and distances are somewhat arbitrary. This type of visual inspection can isolate many such problems, especially once the analyst begins to examine links within a specific study.

Several agencies use zonal tree data to trace minimum time paths between selected zones. The zonal tree procedure, described more fully in Chapter 4, enables the analyst to quickly identify travel paths that are unreasonable based on the analyst's knowledge of the study area. Using this information, the links can be modified as needed prior to the final system-level forecast. Additional highway network coding problems can be identified as part of Chapter 4.

CHECK 4—COMPARE BASE YEAR TRAFFIC DATA

Prior to attempting to perform manual refinements to a computerized traffic assignment the analyst should make a number of comparisons between simulated and actual base year traffic data. These comparisons will often indicate where specific network coding changes should be made (see Check 3) in order that study area link volumes are better simulated. Changes in the location of zone connectors, changes in link impedances or capacities, or additional links that should be included in the network may be identified during this review.

Figure A-3 has been developed to aid in determining the acceptability of the base year assignment on specific network links. The figure is based on the assumption that the maximum desirable traffic assignment deviation should not result in a design deviation of more than one highway travel lane. Therefore, the "acceptable" deviation is higher on low volume roads where a large percentage deviation will not have major design implications. The converse is true on higher volume facilities.

For example, data for the following two links are given:

	(1) Actual Traffic Count (ADT)	(2) Assigned Volume	(3) Deviation (Col. 2/Col. 1)	(4) Percent Deviation (Col. 3/Col. 6)
Link A	10,000	12,000	+ 3,000	+30%
Link B	70,000	53,000	-17,000	-24%

Although the percent deviation is less for Link B than for Link A, it is seen in Figure A-3 that the assignment for Link A falls within the acceptable range while that for Link B does not. This is reasonable because the absolute volume deviation of 17,000 ADT on Link B has considerably greater design implications than the 3,000 ADT difference on Link A.

Figure A-3 uses a scale based on 24-hour volume totals (ADT). Peak hour or peak period assignments can be examined by factoring the 24-hour scale by the appropriate percentage of daily traffic occurring during those time periods. For instance, if the peak hour contains 11 percent of the 24-hour traffic based on base year counts, the ADT's shown in Figure A-3 can be factored by 0.11 to produce a peak hour scale. Although no specific rules exist as to when an assignment should be considered acceptable, the vast majority of links should have assigned traffic volumes that fall within the maximum desirable deviation shown in Figure A-3. Further checks of total base year screenline volumes are discussed as part of the refinement procedure documentation contained in Chapter 4.

A related check involves comparing the base year simulated vehicle miles of travel (VMT) within the study area with the base year VMT obtained from actual traffic count data. Most computer assignments can provide VMT on a zonal basis, often by facility type. User-provided actual VMT would then be used to compare the values. It is essential that the actual VMT has been measured on the same roadways as those simulated by the computer model in order to ensure that

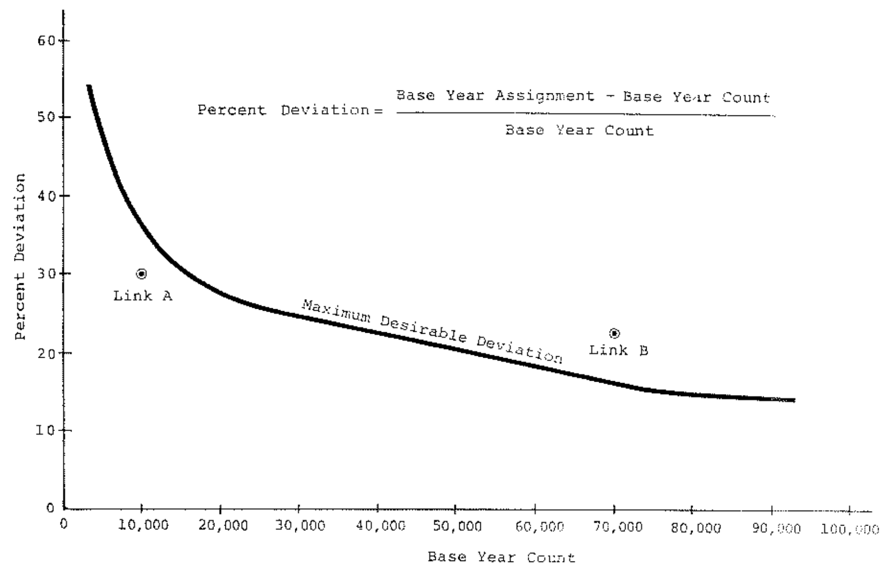


Figure A-3. Maximum desirable error for link volumes.

the comparison is valid. If large discrepancies become evident by comparing these results, a review of VMT by facility type may reveal the source of the error. If no actual VMT figures are available for the base year, values may be extrapolated from other years using VMT growth trends.

CHECK 5—COMPARE GROWTH TRENDS

This check tests the reasonableness of the forecasted traffic growth compared with historical growth trends. The following data are required:

- Future year traffic data (link volumes, trip ends, and/or VMT).
- Base year actual or simulated traffic data (link volumes, trip ends, and/or VMT).
- Historical record of one or more of the following:
    - Link Volumes
    - VMT
    - Population
    - Employment
    - Households

Typically these data are available on either a zonal, district, or regional level. If possible this check should focus on the data for the selected study area; however, comparisons of regional data can often help determine the overall reasonableness of the future year forecast.

To the extent possible, the base year data should reflect actual conditions rather than simulated conditions. For instance, actual base year VMT counts should be compared to the future year VMT forecasts. Such analyses will enable the future year forecasted data to be compared directly with actual base year data without the biases from the simulated base year assignment. Of course, through applying preliminary Checks 1 through 4, the simulated base year assignment should also accurately reflect actual conditions and therefore may be used with minimal error. The base year assignment data have the advantage of being compatible in format (e.g., VMT, trip-ends) with that of the future year data.

For comparison purposes, an average annual traffic growth rate should be computed for the period between the base year and the future year. The average annual growth rate, described in more detail with examples in Chapter 7 of this user's manual, can be readily computed for various link volumes, zonal trip ends, or VMT values.

This growth rate should then be compared with data from one or more of the following historical trends:

- Growth rate in VMT.
- Growth rate in population.
- Growth rate in households.
- Growth rate in employment.

These comparisons of growth rates are not intended to produce exact matches, but should provide a check of the reasonableness of the future year forecasts.

The analyst must decide at this point whether the forecasted growth rates are acceptable relative to historical growth trends. If unsatisfactory results are obtained from this check, it may be necessary to make computer input modifications and rerun the future year forecast.

One option to rerunning the models is to manually factor the future year volumes up or down on the basis of a more realistic growth rate determined by the analyst. This method may be reasonable for small area studies where relatively few links and zones are involved; however, this procedure generally will not produce satisfactory results if applied to a larger corridor or region. In such cases, the traffic forecasting models should be corrected and rerun. Related procedures for modifying a traffic forecast based on capacity and/or land-use constraints are presented in Chapters 4 and 7.

Each of these preliminary checks should be used to determine the overall accuracy of the traffic forecasts prior to applying any project-level refinement or detailing procedure. At the same time, these checks serve the purpose of fully familiarizing the analyst with the highway network and the trip assignments. This knowledge will aid the analyst in making judgments during the application of the refinement procedures presented in later chapters.

CHAPTER FOUR

REFINEMENT OF COMPUTERIZED TRAFFIC VOLUME FORECASTS

GENERAL

Nearly all computerized system level traffic assignments require that further refinement take place prior to their being used for highway project planning and design. This refinement step is one of the most critical steps in the highway project planning and design traffic forecasting process. The purpose of this chapter is to document procedures that will allow for this refinement to take place in a rational and consistent manner.

An immediate word of caution must be expressed, however, in order to prevent the procedures from being misapplied. As with any procedure that attempts to simulate something as complex as the travel patterns of an entire urban area, not all factors determining traffic volumes can be taken into account through application of a mathematical procedure. Therefore, although the procedures attempt to logically refine the results of the computerized traffic simulation process by taking into account factors that cannot be adequately incorporated in the computer process, it must be realized by users of this refinement process that considerable professional judgment must be applied both during and following application of the procedures.

Two types of procedures are presented. The first is a screenline refinement process (46, 77). This procedure uses relationships between base year traffic counts and future year capacities to adjust traffic crossing a prespecified screenline. It is most useful for analyzing corridor traffic movements or traffic assigned to an activity center that has a well-defined network structure.

The second procedure uses computer-generated data for selected network links or zones to help identify origin-destination trip patterns (104, 111, 115). These techniques, entitled select link and zonal tree analyses, provide the analyst with sufficient information to manually reassign traffic from one link to another in order to produce a refined assignment. This procedure is applicable for refining traffic movements within a small to medium sized network and along highway corridors. Detailed studies of freeway ramp movements can also be performed.

Therefore, the procedures presented in this chapter are applicable for refining volumes using various levels of network detail and types of assignment (e.g., all-or-nothing, capacity restrained.) Obviously, the refinement requirements for a detailed highway network are more vigorous than for a sketch planning corridor-level refinement. Similarly, capacity restrained highway assignments generally require fewer refinements than do all-or-nothing assignments. On the other hand, manual refinements are more straightforward with an all-or-nothing assignment, because interzonal travel movements are clearly defined.

The refinement procedures can be used to analyze these and other situations. The difference in their applications will be largely related to the amount of judgment that must be used. Therefore, emphasis is placed on the basic refinement techniques, followed by a section on special considerations. Illustrative examples also provide some insights into how the procedures can be applied to particular settings. Additional uses for these procedures are documented in Chapters 5 and 6, which address specific traffic refinement topics.

PRELIMINARY DATA BASE DEVELOPMENT

Prior to the actual application of the refinement procedures presented in this chapter a data base must be established. For the most part, the following data development steps are common to the screenline and select link/zonal tree procedures:

1. Define study area boundaries.
2. Define base year and future year.
3. Identify link and/or node characteristics.
4. Record base year traffic counts.
5. Record base and future year traffic assignments.

Each task is described below.

Step 1—Define Study Area Boundaries

The study area should be defined so that all the facilities under study are included. It is recommended that additional facilities also be included that could be expected to directly influence the traffic patterns on the facilities under study. All links from the study area portion of the network should be copied onto a separate sheet of paper at a large enough scale that the map can be used for analysis purposes. Centroids, centroid connectors, and nodes should also be detailed on the sheet. A sample format is shown in Figure A-4.

Step 2—Define the Base Year and Future Year

The specific years for which refinements or detailing are desired should be defined. Usually these years will correspond with the years for which computerized forecasts are available. However, in some cases traffic data may be desired for intermediate or extended years. Procedures for adjusting traffic forecasts to correspond with different future year assumptions are described in Chapter 7. Generally the computerized forecasts should first be refined for the years for which they were performed prior to applying the procedures in Chapter 7.

Step 3—Identify Network Characteristics

Each link in the study area should have the following characteristics listed for both the base year and the forecast year:

- Type of facility (e.g., freeway, surface arterial).
- Number of lanes.
- Length.
- Orientation (i.e., one-way; two-way).
- Type of traffic control (e.g., signalized, grade separated).
- Adjacent land-use characteristics.

For cases where turning movements will be required, characteristics of the nodes should include the following:

- Basic approach lane configuration (e.g., number of lanes, availability of turn lanes).
- Traffic control (e.g., unsignalized, signalized, green time, cycle length).
- Restricted movements if any (e.g., no left turns).

These link and/or node characteristics should be displayed in tabular form (Table A-3) and/or on the map prepared in Step 1 (Fig. A-4).

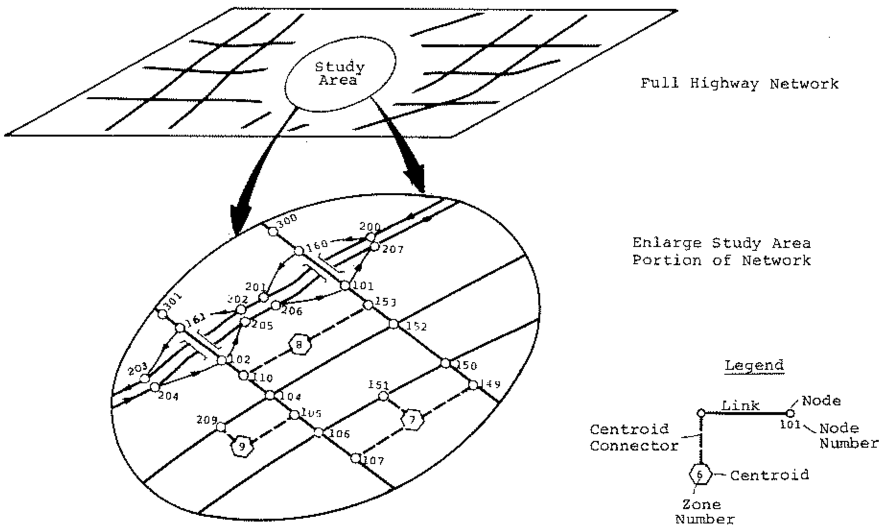


Figure A-4. Study area network format.

Table A-3. Typical format for display of network characteristics.1/

Link	Facility Type	Lanes: Orientation	Lanes: Number	Length (mi)	Traffic Control	Other
205-206	Freeway	One-Way (NB)	3	0.6	Grade Separated	Industry
201-202	Freeway	One-Way (SB)	3	0.6	Grade Separated	Industry
103-110	Arterial	Two-Way	4	0.3	Signals	Commercial
106-107	Arterial	Two-Way	6	0.5	Grade Separated	Industry
151-150	Arterial	Two-Way	4	0.8	Signals	Residential
Continued						

Node	Approach	Configuration2/	Traffic Control	Other
160	N	Ramp - 2 lane	Stop	One-way link
	S	-- No South Approach - 1 way SB --		
	E	3T, 1L	No Stop	No right turn
	W	3T	No Stop	No left turn
150	N	2T	Signal (g/c = 0.4)	-
	S	2T	Signal (g/c = 0.4)	-
	E	2T, 1L	Signal (g/c = 0.6)	-
	W	3T	Signal (g/c = 0.6)	No left turn
Continued				

1/ Refer to Figure A-4 for diagram of network.

2/ T = through lanes, L = left-turn lanes, R = right-turn lanes

Step 4—Record Base Year Traffic Counts

Using the base map developed in Step 1, all available base year traffic data for links within the study area should be plotted. The data should be gathered from state, county, or local government agencies responsible for traffic counts, from special studies, or from other sources as required. Special care must be taken to ensure that these traffic counts cover the same hours as those used in the computer assignments. This is especially critical if comparisons will be made with peak hour or peak period computer assignments, because AM and PM peak traffic volumes can often be significantly different.

Directional traffic counts should be plotted if these are compatible with the format of the computer assignments. If nondirectional assignments will be used, directional traffic counts must be summed together along a link. In other cases where counts are available for only part of a day, expansion factors should be used to convert to the same time period for which the assignment was made. Care should also be taken to apply appropriate seasonal and day-of-week factors to convert individual traffic counts to average day values.

If no base year traffic counts are available on a particular link(s), it may be necessary to interpolate values from adjacent links or to expand intersection turning movement counts into link volumes. As with other steps in this process, local knowledge of the traffic characteristics of the roadways and adjacent land-uses should be used to "fine tune" any traffic count estimates.

Step 5—Record Base and Future Year Traffic Assignment

Record the volumes and capacities for the base year and future year computer assignments. These values should be recorded directly from the computer printout onto the map prepared during Step 1.

The link capacities used for the traffic assignment may need to be adjusted at this point. Two specific situations are possible:

1. A capacity was artificially adjusted during the calibration in order to increase or decrease the link impedance. In this case, the capacity should be adjusted back to its original value so as to be compatible with the capacities on the other links.

2. Generalized capacities were used for the assignment. Several computer models employ capacities that are specific only to facility type. If possible, these simplified capacities should be adjusted on each link to better reflect actual (base year) or forecasted (future year) conditions. This adjustment is most likely to be necessary on base year assignments where roadway widths, types of traffic control, and pavement conditions vary widely.

Once this data base has been prepared, various traffic data refinement or detailing procedures can be pursued.

SCREENLINE REFINEMENT PROCEDURE

The purpose of the screenline refinement procedure is to improve upon the link-by-link traffic forecasts produced by computer models. Future year link volumes are adjusted by the procedure across a screenline based on relationships between base year traffic counts, base year assignments, and future year link capacities. Generally the base year should be the latest year for which both traffic count data and computerized traffic link assignments are available. Since most traffic assignments are made on an all-day basis, the traffic data should ideally be average weekday daily traffic (AWDT) or average daily traffic (ADT).

After a screenline is selected the base year traffic assignment (if available) is compared with actual base year traffic counts. The magnitude of deviation between these two values enables the analyst to decide whether or not to make an initial future year link adjustment. This adjustment is the average of two methods—one which calculates the ratio between the base year forecast and the actual base year traffic count, and one which calculates the numerical difference between these values. A subsequent adjustment is then made which combines the effects of future year capacity changes with the stabilizing effects of actual base year traffic patterns.

The screenline procedure, therefore, considers several factors that are critical to the preparation of realistic traffic assignments. The most accurate results are obtained if the inputs include reasonably good base year traffic counts, a base year assignment, and a future year forecast. The procedure has less validity if base year data are not available; however, refinement of link volumes can still be performed using the modifications discussed under "special considerations." The procedure is valid using all-or-nothing or capacity restrained assignments.

The screenline procedure typically adjusts all volumes crossing the screenline. Therefore, it is not always suitable for use in situations where only one or two link volumes are in need of refinement. The procedure is also limited to situations where reasonable screenlines can be constructed across parallel facilities. Accuracy is lost when nonparallel facilities (e.g., diagonal roads) are introduced into the screenline.

Once the input data are assembled and checked, the screenline computations can be performed quickly. For instance, a medium sized network involving 10 screenlines could be analyzed using the worksheet in 1 to 2 person-days. Final checks and adjustments to specific link volumes would require an additional 4 to 8 person-hours.

Basis for Development

The screenline refinement procedure is the combination of a procedure developed by the New York State Department of Transportation (77) and one developed for the Maryland Department of Transportation (46, 47) by JHK & Associates. The New York State DOT procedure is the basis for the initial screenline adjustment to account for discrepancies between the base year assignment and actual traffic counts. The Maryland DOT procedure incorporates the final adjustments for relative base year traffic counts and future year capacities. These adjustments have been combined into a comprehensive refinement procedure using a worksheet approach.

Input Data Requirements

The following data are used as input to the screenline refinement procedure:

- Highway network (base and future year) with historical record (i.e., type of facility, number of lanes, orientation, type of traffic control).
- Base year traffic counts.
- Base year assignment.
- Base year link capacities.
- Future year forecast.
- Future year link capacities.
- Land-use growth trends (optional).

These data should be available from the preliminary data base development and will be used either directly in the worksheet computations or for making reasonableness checks.

Directions for Use

The screenline refinement procedure includes four sequential steps, as follows:

Step 1: Select screenlines.

Step 2: Check base year volumes.

Step 3: Perform computations.

Step 4: Conduct final checks.

These steps and related substeps are diagrammed in Figure A-5. The following sections describe the procedure steps in detail.

Step 1--Select Screenlines

The first step in the procedure is to select one or more screenlines that will be used to adjust link volumes. It is important that a screenline crosses each of the facilities whose volumes are to be refined.

Selecting the screenlines for analysis is not always a straightforward process. In areas where roadways parallel one another for several miles or where geographic boundaries clearly define alternative routes (e.g., river crossings), screenlines are fairly easy to select. However, there will be a number of instances where these situations do not occur in any study area. Screenlines should, therefore, be based on judgment and a familiarity with the roadway network. It is suggested that the following guidelines be used in developing screenlines.

1. Determine the context with which the screenlines will be used. Generally one or more of the following situations will apply:

- Small area analysis.
- Wide corridor analysis.
- Regional analysis.

The scale of the analysis will dictate both the length of the screenline and the number of screenlines to be analyzed.

2. A screenline should intersect roadways that represent likely alternatives for directional traffic within a corridor. In some areas, the screenlines should be curved to follow a natural barrier such as a river or hill. However, meandering or diagonal-type roadways should be avoided, as shown in Figure A-6. In this example, "A," "B," and "C" Streets carry parallel traffic in the east-west direction. "D" Avenue is a diagonal facility that carries traffic in all directions. Therefore, for refining assignments in the east-west corridor, "D" Avenue should not be included in the screenline.

3. In most cases, zone connectors that are crossed by a screenline should not be included in the analysis. Special cases in which zone connectors are considered are discussed under "Special Considerations."

4. A screenline should cross a minimum of 3 roadways and preferably no more than 7 roadways. For computational simplicity, a practical maximum is 10 roadways.

5. Screenlines should be no longer than necessary. Figure A-7 provides a guide for selecting screenline length based on link density. For instance, in densely developed areas with many roadways, a practical limit of 2 miles is suggested, while in outlying, less dense areas, 4 to 5 miles would represent a reasonable screenline length. Special considerations are discussed later in this chapter.

6. Separate screenlines should be constructed midway between major roadway crossings or every 2 miles--whichever is less. This is important because link traffic volumes along a facility can change considerably within a short distance, especially on either side of a major intersection or interchange. Comparisons of results from parallel screenlines will be a major check of reasonableness of the refinement procedure.

Examples of screenlines on a corridor network are shown in Figure A-8. Screenlines A, B, and C are appropriate for balancing traffic assignments along north-south routes in the corridor. Screenlines D, E, and F are oriented toward east-west roadways.

Step 2--Check Base Year Volumes

In order to determine if the screenline assignment is a reasonable representation of corridor traffic, total traffic crossing the screenline should be compared between the base year assignment and the actual base year traffic counts. The volumes on each link crossed by the screenline should be added together for this analysis.

The percent deviation of these screenline total volumes should be calculated. An example of this analysis is given in Table A-4, using the screenlines depicted in Figure A-8. Figure A-9 has been developed to help estimate the maximum desirable screenline volume deviation. The rationale used to develop this figure is that the maximum permissible deviation of a screenline traffic estimate should be such that a highway design would not vary by more than one roadway lane. The dividing line shown in Figure A-9 should be used as an analysis guide rather than as an absolute cutoff level.

At lower screenline volumes, the permitted volume deviation is quite large, since such deviations would not result in significant design differences. Conversely, at higher screenline volumes, a lower deviation is desired in order to be confident that any design decisions would be valid. Figure A-9 was developed for use with 24-hour volumes. Peak hour or peak period screenline volumes could also be used if the horizontal scale of Figure A-9 were proportioned accordingly (e.g., if peak hour = 10 percent of daily, divide scale by 10).

The total screenline traffic count and the percent of base year assignment deviations should be plotted on the graph shown in Figure A-9. Screenlines A, D, E, and F fall within the acceptable range, while Screenline C exceeds the maximum desirable deviation for its given volume level. Because the Screenline B deviation lies immediately adjacent to the dividing line in Figure A-9, the analyst must judge its acceptability based on the screenline's relative location and importance within the study area, and based on the desired degree of refinement accuracy.

If the screenline totals are within the maximum desirable deviation, the subsequent worksheet computations can proceed. If the base year volumes exceed the maximum desirable deviation, however, several possible actions are possible, including the following:

1. For large discrepancies, correct deficiencies in the modeling process and rerun appropriate models. Such situations would include major errors in trip generation, trip distribution, modal split, or network coding. If the preliminary checks presented in Chapter 3 are conducted, major deviations will not normally occur.

2. Extend the screenline length to include additional facilities. This action tends to reduce the deviation across the screenline. Care must be taken that the added facilities represent realistic travel alternatives.

3. Manually factor the screenline volumes up or down by the amount which the base year assignment differs from the actual base year traffic counts. A method to perform this adjustment is presented in Step 3-2.

The "Special Considerations" section of this chapter describes a modified screenline procedure to follow when specific base year data are not available.

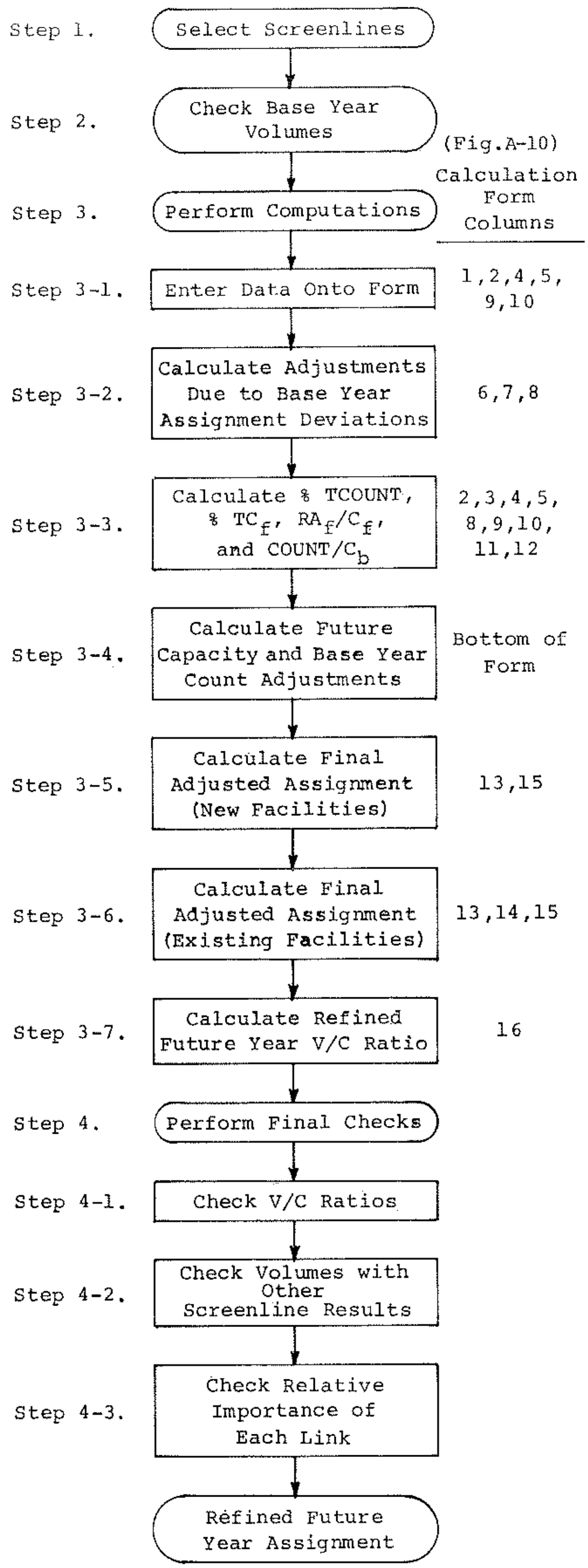


Figure A-5. Screenline refinement procedure.

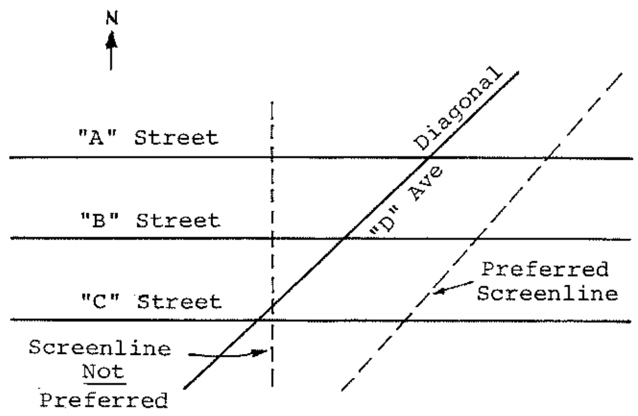


Figure A-6. Screenline selection.

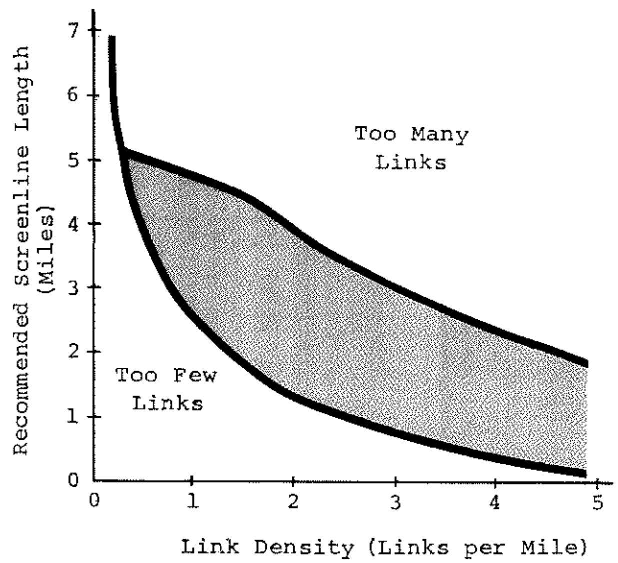


Figure A-7. Recommended screenline length.

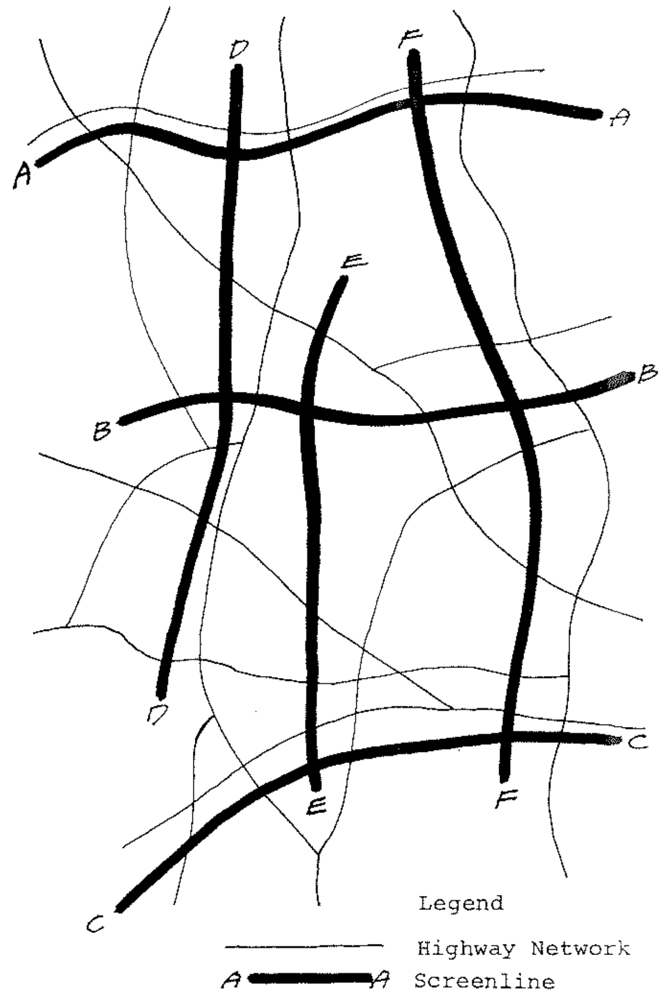


Figure A-8. Corridor screenlines.

Table A-4. Comparison of base year counts and assignments.

(1) Screenline	(2) Base Year Assignment1/	(3) Base Year Count1/	(4) Deviation (Col. 2-Col. 3)	(5) Percent Deviation2/ (Col. 4-Col. 3)*100	(6) Exceeds Allowable Deviation3/
A	124,800	135,400	-10,600	- 7.8	
B	107,600	83,100	+24,500	+29.5	Judgment
C	147,900	117,700	+30,200	+25.6	X
D	66,900	58,700	+ 8,200	+13.9	
E	43,800	46,400	- 2,400	- 5.1	
F	37,400	50,200	-12,800	-25.5	

1/ 24-Hour Volumes.

2/ Percent difference is relative to Base Year Count (Col. 3).

3/ See Figure A-9.

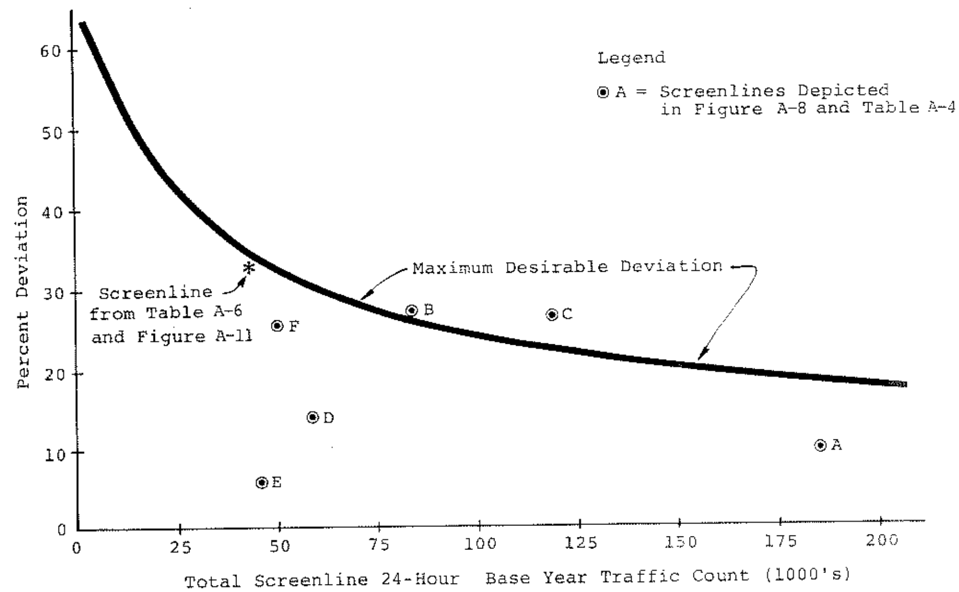


Figure A-9. Maximum desirable deviation in total screenline volumes.

Step 3--Perform Computations

The refinement procedure includes two types of adjustments. The first type adjusts the future year link volumes according to the amount of deviation between the actual base year traffic count and the base year assignment. The second type of adjustment is based on separate relationships between base year traffic counts and between future year link capacities.

Both of these adjustments are not necessary for all analyses. Indeed, in cases where base year data are not available, the second type of adjustment will be the only one possible. However, by combining these adjustments the link refinements will usually produce the most realistic results.

The calculations make use of a form, as shown in Figure A-10. For convenience, the column numbers on the form are referred to in subsequent steps. The columns are also specified in Figure A-5.

The definitions of the columns are listed in Table A-5. The volumes and capacities used in the procedure can be expressed in terms of 24-hour, peak-hour, or peak-period values as long as consistency is maintained.

Step 3-1--Enter Available Data Onto the Calculation Form. These entries are made on the calculation form (Figure A-10) as follows:

1. Enter names of each roadway (link) and node numbers of each link crossed by the screenline--(Col. 1).

2. Enter the base year traffic counts (COUNT)--(Col. 2). Sum this column to compute TCOUNT--(Col. 2).

3. Enter the base year assignment (A_b) and capacity (C_b)--(Cols. 4 and 9). Sum these columns to compute TA_b and TC_b.

4. Enter the future year forecast (A_f) and capacity (C_f)--(Cols. 5 and 10). Sum these columns to compute TA_f and TC_f.

It is important that these data are comparable in terms of time period. Peak-hour and 24-hour volumes and capacities should not be combined on this form. It should be noted that entries in columns 2, 4, and 9 will be left blank for roadway links that do not exist during the base year. If base year data are not available, refer to the "Special Considerations" section of this chapter.

An example problem is summarized in Figure A-11 and Table A-6. The data for this example are shown in Figure A-12 as they would be entered onto this form.

Step 3-2--Calculate Adjustments Due to Base Year Assignment Deviations. The purpose of these computations is to adjust the future year link assignments to account for probable assignment errors. The underlying assumption used is that errors occurring in a base year assignment will continue to occur proportionally in any future year forecasts.

This adjustment does not need to be applied in all situations. If the results of Step 2 indicate that the base year screenline values fall within the desirable range of deviation shown in Figure A-9, this adjustment can be omitted with negligible loss of accuracy (e.g., screenlines A, D, E, and F from Table A-4; Fig. A-9). In cases where the desirable limit is exceeded, however, it is suggested that this adjustment be performed (e.g., screenlines B and C from Table A-4; Fig. A-9).

The adjustment technique is based on a methodology developed by the New York State Department of Transportation (77). A future year link volume is adjusted using two factors--the ratio of the actual base year traffic count to the base year assignment and the numerical difference between the actual base year traffic count and the base year assignment. These two factors are then applied to the future year forecasted volumes according to the following equations:

RATIO Adjustment:

$$\text{RATIO} = (\text{COUNT}/A_b) * A_f \quad \text{(Col. 6)} \qquad \text{(A-1)}$$

DIFFERENCE Adjustment:

$$\text{DIFFERENCE} = (\text{COUNT} - A_b) + A_f \quad \text{(Col. 7)} \qquad \text{(A-2)}$$

where:

COUNT = actual base year traffic count (Col. 2);
A_b = base year traffic assignment (Col. 4);
A_f = future year traffic forecast (Col. 5);
RATIO = ratio adjusted future year link forecast (Col. 6); and
DIFFERENCE = difference adjusted future year link forecast (Col. 7).

The value for DIFFERENCE can be either positive or negative; the value for RATIO can be greater or less than one, but always positive.

The adjusted future year traffic forecast, RA_f, is then the average of these two results, as follows:

$$RA_f = (\text{RATIO} + \text{DIFFERENCE})/2 \qquad \text{(A-3)}$$

RA_f is placed in column 8 of the calculation form. Sum column 8 to compute TRA_f. As an example, consider the following link data:

COUNT = 5,000
A_b = 3,500
A_f = 7,300

Then:

RATIO = (5,000/3,500) * 7,300 = 10,400
DIFFERENCE = (5,000 - 3,500) + 7,300 = 8,800
RA_f = (10,400 + 8,800)/2 = 9,600

Two specific problems may occur with either RATIO or DIFFERENCE when applying this method. First, RA_f could assume an impossible negative value if DIFFERENCE is less than zero and if the absolute value of DIFFERENCE is greater than the absolute value of RATIO.

For example, given the following data:

COUNT = 1,000
A_b = 2,000
A_f = 500

Then:

RATIO = (1,000/2,000) * 500 = 250
DIFFERENCE = (1,000 - 2,000) + 500 = -500
RA_f = (250 - 500)/2 = -125

In this situation, it is suggested that RATIO only be used. Therefore, RATIO = RA_f = 250.

A second problem can occur if COUNT is significantly greater than A_b. In this case, a very high ratio factor may be compiled, resulting in excessively high values of RATIO and RA_f. For example, given the following data:

COUNT = 1,000
A_b = 10
A_f = 200

Study Area ______________________

Screenline ______________________

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
Facility (Nodes)	COUNT	% TCOUNT	A_b	A_f	Adjustment		RA_f	C_b	C_f	% TC_f	$\frac{RA_f}{C_f}$	Adjustment		FA_f	$\frac{FA_f}{C_f}$	$\frac{COUNT}{C_b}$
					RATIO	DIFFER-ENCE						CAPA-CITY	BASE COUNT			
TOTALS	TCOUNT		TA_b	TA_f			TRA_f	TC_b	TC_f		$\frac{TRA_f}{TC_f}$	See Fig. A-14	FCAP FCOUNT	TFA_f	$\frac{TFA_f}{TC_f}$	$\frac{TCOUNT}{TC_b}$

Figure A-10. Calculation form.

Table A-5. Definitions of screenline procedure terms.

Column	Variable	Definition
(1)	Facility (Nodes)	The name and/or route number of each facility bisected by the screenline is listed in sequence along with the identifying node numbers used in the highway network
(2)	COUNT TCOUNT	Actual base year traffic count Screenline total of actual base year traffic counts
(3)	%TCOUNT	Proportion (in decimals) of total screenline base year traffic count occurring on a particular link (COUNT/TCOUNT)
(4)	A_b TA_b	Base year traffic assignment Screenline total base year traffic assignment
(5)	A_f TA_f	Future year traffic forecast Screenline total future year traffic forecast
(6)	RATIO Adjustment	$(COUNT/A_b) * A_f$ = (Col. 2/Col. 4) * Col. 5
(7)	DIFFERENCE Adjustment	$(COUNT - A_b) + A_f$ = (Col. 2 - Col.4) + Col. 5
(8)	RA_f	Adjusted future year traffic forecast. Equals average of RATIO and DIFFERENCE adjustments--(Col. 6 + Col. 7) /2
	TRA_f	Screenline total adjusted future year traffic forecast
(9)	C_b TC_b	Base year capacity (at level-of-service E) Screenline total base year capacity (at level-of-service E)
(10)	C_f TC_f	Future year capacity (at level-of-service E) Screenline total future year capacity (at Level of Service E)
(11)	$\%TC_f$	Proportion (in decimals) of total screenline future year capacity occurring on a particular link (C_f/TC_f)
(12)	RA_f/C_f	Ratio of the adjusted future year traffic forecast to the future year capacity--(Col. 8/Col. 10)
	TRA_f/TC_f	Ratio of total screenline adjusted future year traffic forecast to total future year screenline capacity
(13)	CAPACITY Adjustment	Portion of a link's final refined future year traffic forecast resulting from its proportional future year capacity
(14)	BASECOUNT Adjustment	Portion of a link's final refined future year traffic forecast resulting from its proportional base year traffic count
(15)	FA_f	Final refined future year traffic forecast
	TFA_f	Screenline total final refined future year traffic forecast
(16)	FA_f/C_f	Ratio of the final refined future year traffic forecast to future year capacity--(Col. 15/Col. 10)
	TFA_f/TC_f	Ratio of total screenline refined future year traffic forecast to total future year screenline capacity
(17)	$COUNT/C_b$	Ratio of actual base year traffic count to base year capacity--(Col. 2/Col. 9)
	$TCOUNT/TC_b$	Ratio of total screenline actual base year traffic counts to total screenline base year capacity

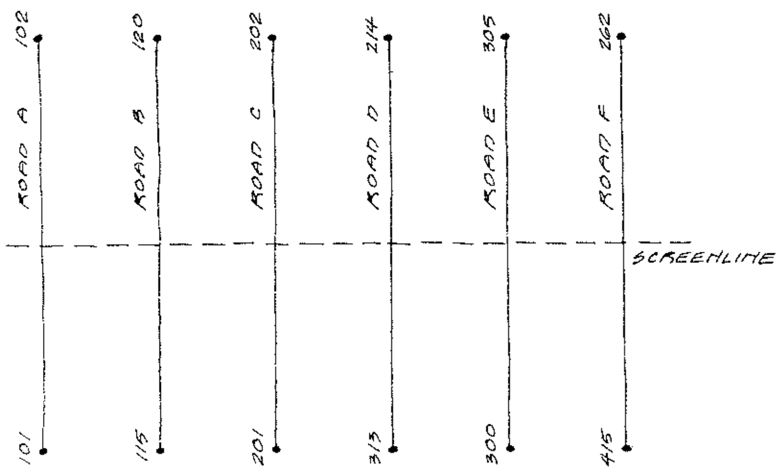


Figure A-11. Example screenline.

Table A-6. Example screenline characteristics.

Roadway (Nodes)	Base Year: Traffic Count (COUNT)	Base Year: Traffic Assignment A_b	Base Year: Capacity C_b	Future Year: Traffic Forecast A_f	Future Year: Capacity C_f
Road A (101-102)	2,500	900	13,500	1,300	13,500
Road B (115-120)	4,300	12,400	14,900	13,100	14,900
Road C (201-202)	12,350	3,400	12,200	2,000	13,500
Road D (313-214)	Does not exist in Base Year			107,100	129,600
Road E (300-305)	12,400	6,000	9,500	23,300	32,400
Road F (415-262)	11,800	6,700	13,500	900	13,500
Total	43,350	29,400	63,600	147,700	217,400

Study Area EXAMPLE

Screenline EXAMPLE

(1) Facility (Nodes)	(2) COUNT	(3) % TCOUNT	(4) A_b	(5) A_f	(6) Adjustment: RATIO	(7) Adjustment: DIFFERENCE	(8) RA_f	(9) C_b	(10) C_f	(11) % TC_f	(12) $\frac{RA_f}{C_f}$	(13) Adjustment: CAPACITY	(14) Adjustment: BASE COUNT	(15) FA_f	(16) $\frac{FA_f}{C_f}$	(17) $\frac{COUNT}{C_b}$
Road A (101-102)	2,500		900	1,300				13,500	13,500							
Road B (115-120)	4,300		12,400	13,100				14,900	14,900							
Road C (201-202)	12,350		3,400	2,000				12,200	13,500							
Road D (313-214)	—		—	107,100				—	129,600							
Road E (300-305)	12,400		6,000	23,300				9,500	32,400							
Road F (415-262)	11,800		6,700	900				13,500	13,500							
TOTALS	43,350 TCOUNT		29,400 TA_b	147,700 TA_f			TRA_f	63,600 TC_b	217,400 TC_f		$\frac{TRA_f}{TC_f}$	See Fig. A-14	FCAP FCOUNT	TFA_f	$\frac{TFA_f}{TC_f}$	$\frac{TCOUNT}{TC_b}$

Figure A-12. Calculation form with input data.

Then:

RATIO = (1,000/10) * 200 = 20,000

DIFFERENCE = (1,000 - 10) + 200 = 1,190

RA_f = (20,000 + 1,190)/2 = 10,600

In this situation, it is suggested that only DIFFERENCE be used. Therefore, DIFFERENCE = RA_f = 1,190.

These adjustments should be applied only to those links that will not be experiencing a significant capacity change in the future year. Where major capacity changes will occur on a link (i.e., greater than 25 percent), there are usually too many extraneous factors (e.g., land-use changes, major route diversions) implicit in the future year link assignment to reasonably expect that the base year assignment errors will carry over to the future. The analyst must use considerable judgment in this decision.

Using the data from Table A-6 and Figure A-11, the results from Step 2 are reviewed to determine if this adjustment is necessary. The deviation between the total screenline base year assignment and base year counts is found to equal 29,400 - 43,350 = -13,950. The percent deviation equals -13,950/43,350 = -32.2 percent. Using Figure A-9, the analyst determines that the assignment falls just within the maximum desirable deviation. Because the deviation barely falls into the "acceptable" range, and because the absolute deviation is over 30 percent, the decision is now made to use the RATIO and DIFFERENCE adjustments to reduce the impact of probable assignment errors.

Each link is examined for capacity changes. Apart from new road D, the only facility experiencing a major capacity change is road E, which will have over a threefold increase. Road C experiences a minor capacity increase of 11 percent. Based on this analysis, roads D and E are excluded from this adjustment.

The RATIO and DIFFERENCE adjustments for roads A, B, C, and F are shown in Figure A-13 (Cols. 6 and 7). The adjusted traffic forecast, RA_f, for all links is shown in column 8. Note that RA_f = A_f for roads D and E, which were not adjusted.

In this example, the adjusted screenline volume total, TRA_f (151,300), is greater than the original total, TA_f (147,700). This shift is expected, because the total base year assignment TA_b was less than the total screenline traffic counts, TCOUNT. Therefore, the adjustment seems reasonable.

<u>Step 3-3--Calculate % TCOUNT, % TC_f, RA_f/C_f, and $COUNT/C_b$.</u> The following computations are made:

1. Calculate

$$\% \text{ TCOUNT} = \text{COUNT}/\text{TCOUNT} \quad \text{(A-4)}$$

This calculation is performed for each of the links existing during the base year. The % TCOUNT is entered in column 3.

2. Calculate

$$\% \, TC_f = C_f/TC_f \quad \text{(A-5)}$$

The % TC_f is entered in column 11.

3. The ratio between the adjusted future year traffic forecast assignment (RA_f) and future year capacity (C_f) is calculated and entered in column 12. Note that RA_f = A_f for those links that were not adjusted in Step 3-2.

4. Calculate the base year volume (COUNT) to capacity (C_b) ratios and enter this value in column 17. The total base year screenline volume/capacity ratio is computed by taking the ratio of TCOUNT (Col. 2) and TC_b (Col. 8). These values will be used in making final checks of the forecast. Figure A-13 shows the form completed to this stage for the given example.

Study Area EXAMPLE

Screenline EXAMPLE

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
Facility (Nodes)	COUNT	% TCOUNT	A_b	A_f	Adjustment RATIO	Adjustment DIFFERENCE	RA_f	C_b	C_f	% TC_f	RA_f/C_f	Adjustment CAPACITY	Adjustment BASE COUNT	FA_f	FA_f/C_f	$COUNT/C_b$
Road A (101-102)	2,500	0.06	900	1,300	3,600	2,900	3,250	13,500	13,500	0.06	0.24					
Road B (115-120)	4,300	0.10	12,400	13,100	4,500	5,000	4,750	14,900	14,900	0.07	0.32					
Road C (201-202)	12,350	0.28	3,400	2,000	7,300	10,950	9,100	12,200	13,500	0.06	0.67					
Road D (313-214)	—	—	—	107,100	—	—	107,100	—	129,600	0.60	0.83					
Road E (300-305)	12,400	0.29	6,000	23,300	—	—	23,300	9,500	32,400	0.15	0.72					
Road F (415-262)	11,800	0.27	6,700	900	1,600	6,000	3,800	13,500	13,500	0.06	0.28					
TOTALS	43,350 TCOUNT		29,400 TA_b	147,700 TA_f			151,300 TRA_f	63,600 TC_b	217,400 TC_f		0.69 TRA_f/TC_f	See Fig. A-14	0.48 FCAP; 0.52 FCOUNT	TFA_f	TFA_f/TC_f	$TCOUNT/TC_b$

Figure A-13. Calculation form completed through step 3-3.

Example: Road B (115-120)

TCOUNT = 43,350

% TCOUNT = 4,300/43,350 = 0.10 (Col. 3)

TC_f = 217,400

$\%TC_f$ = 14,900/217,400 = 0.07 (Col. 11)

RA_f/C_f = 4,750/14,900 = 0.32 (Col. 12)

$COUNT/C_b$ = 4,300/14,900 = 0.29 (Col. 17)

Step 3-4--Calculate Future Capacity and Base Year Count Adjustments. These adjustments are based on the assumption that future year volumes are influenced by actual base year traffic patterns, by the addition, deletion, or modification of roadway capacity, and by the level of overall congestion that will occur. All other factors being equal, where negligible capacity changes or capacity constraints are expected to occur across a screenline, the future year assignment should closely replicate the actual base year traffic patterns. At the other extreme where significant roadway capacity changes are anticipated, the future year assignment is expected to be altered accordingly.

The amount of congestion, or level of service, along the screenline will also affect the future traffic patterns. Generally, as congestion worsens across a series of facilities (i.e., screenline), traffic will tend to distribute itself more evenly along all facilities in search of less congested routes. In such cases, future roadway capacity exerts a greater influence on traffic assignment than do the base year traffic counts.

Two adjustment factors are used--FCOUNT and FCAP. FCOUNT is the relative weight given to the base year traffic count distribution, while FCAP represents the weight given to the future year distribution of roadway capacity. Both factors are expressed in terms of a fraction, the sum of which equals 1.00 (100 percent).

The following computations are performed:

1. Calculate the total screenline volume/capacity ratio for the future year--TRA_f (Col. 8)/TC_f (Col. 10)--and place this value at the bottom of column 12. If TRA_f/TC_f is greater than 1.0, refer to the "Special Considerations" section of this procedure.
2. Enter the TRA_f/TC_f ratio onto the horizontal axis of Figure A-14 (point A).
3. Draw a line straight up from this point until it intersects with the turning line (point B).
4. Draw a horizontal line to the left from point B until it intersects with the vertical axis (point C). This value is FCOUNT.
5. Draw a horizontal line to the right from point B until it intersects with the vertical axis (point D). This value is FCAP (Note: FCAP can also be calculated directly as FCAP = 100 - FCOUNT)

Example: Using the same data,

$$V/C = \frac{TRA_f}{TC_f} = \frac{151,300}{217,400} = 0.69 \text{ (Point A)} \qquad \text{(A-6)}$$

FCOUNT = 0.52 (point C)

FCAP = 0.48 (point D) or 1.00 - 0.52 = 0.48

Note that only one calculation of FCOUNT and FCAP will be required for the screenline.

Step 3-5--Calculate Final Adjusted Assignment (FA_f) for NEW Facilities. On NEW facilities the final refined assignment is proportioned only to its relative capacity. Therefore, the following

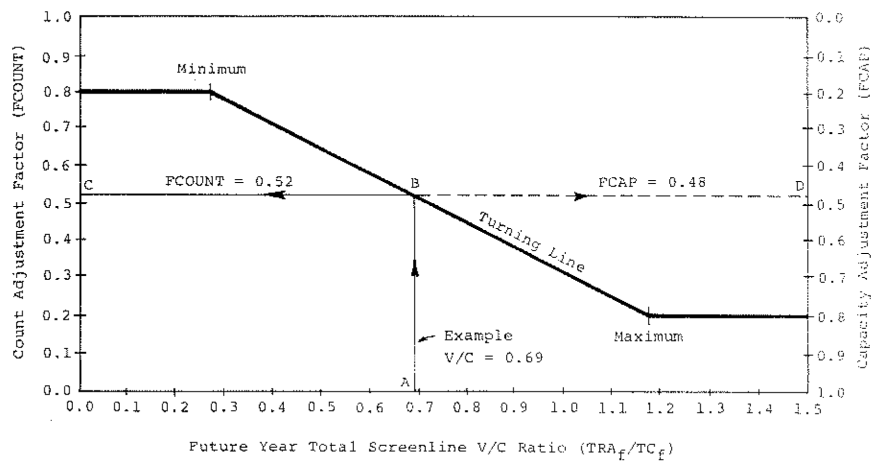


Figure A-14. CAPACITY and BASECOUNT adjustments.

computation can be performed:

1. Calculate $$\text{CAPACITY Adjustment} = \%TC_f * TRA_f \qquad \text{(A-7)}$$

and enter this value in column 13.

2. Calculate $$FA_f = \text{CAPACITY Adjustment} \qquad \text{(A-8)}$$

and enter this value in column 15.

Note that no adjustment for base year counts is made for new facilities because the facility did not exist during the base year.

Example: Road D (313-214)

CAPACITY Adjustment = 0.60 * 151,300 = 90,800

FA_f = 90,800

It is important that FA_f be calculated for all new facilities prior to starting Step 3-6.

Step 3-6--Calculate Final Adjusted Assignment for EXISTING Facilities. On EXISTING facilities the final adjusted assignment is proportioned based on its relative future year capacity and base year traffic count. The adjustment factors (FCOUNT and FCAP) from Step 3-4 are used to perform this tradeoff. The following computations are involved:

1. Calculate $$\text{CAPACITY Adjustment} = \%TC_f * (FCAP * TRA_f) \qquad \text{(A-9)}$$

and enter in column 13.

2. Calculate $$\text{BASECOUNT Adjustment} = \%TCOUNT * FCOUNT * (TRA_f - \Sigma\ FA_{fnew}) \qquad \text{(A-10)}$$

where FA_{fnew} = sum of final adjusted assignments for all new facilities computed during Step 3-5. Enter this value in column 14.

Therefore, the base year traffic count adjustment factor (FCOUNT) is applied only to the future screenline traffic which remains after the refined traffic volumes on all new facilities have been computed.

3. Calculate $$FA_f = \text{CAPACITY Adjustment} + \text{BASECOUNT Adjustment} \qquad \text{(A-11)}$$

and enter values in columns 15, 13, and 14 respectively.

4. Calculate $$TFA_f = \Sigma\ FA_f \qquad \text{(A-12)}$$

and enter value in bottom of column 15.

Example: Road B (115-120)

CAPACITY Adjustment = 0.07 * (0.48 * 151,300) = 5,100

BASECOUNT Adjustment = 0.10 * 0.52 * (151,300 - 90,800) = 3,100

FA_f = 5,100 + 3,100 = 8,200

TFA_f = FA_f = 151,400

and enter value in bottom of column 15.

Compare TFA_f with TRA_f (Col. 8). These totals should be approximately equal, considering that all assignments are typically rounded to the nearest 50 or 100 vehicles. Large differences should be rechecked in Steps 3-4 through 3-6. In this example, TFA_f = 151,400 and TRA_f = 151,300, an acceptable comparison.

Step 3-7--Calculate Refined Future Year Volume/Capacity Ratios. In column 16, compute the future year refined volume (FA_f) to capacity (C_f) ratio for each link. The total future year screenline refined V/C ratio is computed by taking the ratio of TFA_f (Col. 15) and TC_f (Col. 10). These values will be used for checking and verifying the refined assignments.

Example: Road B (115-120)

FA_f/C_f = 8,200/14,900 = 0.55

Figure A-15 shows the screenline calculation form completely filled out for this example.

Step 4--Perform Final Checks

The refined forecast that has been computed should now be checked for general reasonableness before being used in further planning or design studies. These reasonableness checks will include a review of the volume/capacity ratios for each link on the screenline and a check of the link assignments with those of other screenlines that may have intersected the same link. An important guide to the analyst should be to assess the refined volumes based on engineering judgment and familiarity with the roadways. Should problems develop, the screenline may have to be redrawn and the computations redone.

Step 4-1--Check Volume/Capacity Ratios. If the refinement procedure has been successful, the range of refined volume/capacity ratios for the links on the screenline should have been narrowed. This check is made by comparing the original V/C ratios (A_f/C_f = Col. 5/Col.10) with FA_f/C_f (Col. 16). For the example shown in Figure A-15, the range of FA_f/C_f ratios in column 16 is between 0.47 and 0.98, as compared to a range before of 0.07 to 0.88 by dividing A_f/C_f.

There may be instances where FA_f/C_f ratios are significantly higher or lower than the original A_f/C_f ratios. In these cases, a check of the base year $COUNT/C_b$ ratio (Col. 17) should be made. For example, for road C, FA_f/C_f = 0.98 and A_f/C_f = 0.15 for the refined and original forecasts respectively. This indicates a substantial shift due to the refinement process. Checking the base year $COUNT/C_b$ ratio, a value of 12,350/12,200 = 1.01 is obtained. Thus, the refined forecast (FA_f) is shown to be reasonable in this case. Overcapacity conditions are discussed in the "Special Considerations" section of this procedure.

Step 4-2--Check Volumes with Other Screenline Results. The refined traffic volumes (FA_f) should be checked with those from adjacent screenline computations wherever possible and practical. Links common to two or more screenlines should be examined first to make sure that the volumes are compatible between screenlines. Where significant differences occur, one or more of the screenlines may need to be restructured. Generally, however, these differences can be adjusted using knowledge of expected conditions on the facilities.

Step 4-3--Check the Relative Importance of Each Link. A comparison of the $\%TC_f$ (Col. 11) and %TCOUNT (Col. 3) entries on the calculation form can be a useful check, particularly when new links are part of the screenline. The %TCOUNT for each link can be interpreted as the intensity of relative use, while $\%TC_f$ can be roughly interpreted as potential of relative use. A comparison can be made to understand how new facilities can affect the redistribution of future volumes.

In the example used previously, the introduction of a facility such as road D changes the relative importance of the existing links. In the base year, roads C and F carry 55 percent of the volume (%TCOUNT), but in the future year they would have only 12 percent of the screenline capacity (%TC_f). Thus, their relative importance would tend to diminish, as indicated by the fact that they would carry only 17 percent of the final adjusted screenline volume (derived from Col. 15).

Special Considerations

During the course of applying the screenline refinement procedure, professional judgment must be used to reflect specific local conditions. The following sections present several special considerations that are likely to occur.

Zone Connectors and Screenlines

Zone connectors pose a special problem for establishing screenlines. In most cases, the zone connectors crossed by a screenline are those that feed traffic onto roadways that are oriented perpendicular to the screenline links. This situation is shown in Figure A-16. A screenline is constructed running in the north-south direction crossing links Z, T, W, and zone connector A. Zone connector A actually represents traffic generated by development located along links X and Y, which carry traffic volumes perpendicular to the traffic volumes being refined by the screenline. Therefore, zone connector A should not be included in this screenline.

Generally, the only situation in which zone connector A would logically be included in this screenline would be if it represented an important facility that had not been coded into the network (e.g., a major link into an industrial park depicted by zone 1). This could occur either by error or in the case where additional facilities could not be coded due to budgetary or network size constraints. In such cases, judgment should be used to allocate all or part of zone connector A volume to the screenline.

It is more correct to consider the traffic using zone connectors B and C, which deliver traffic onto R-W and S-T respectively. In order to accurately reflect the volume along these links which is due to the zone connector, the assigned volumes on links R-W and on links S-T could be averaged. This average volume could then be used in all subsequent refinement steps. For example, the following volumes are given:

	Volume	
Link R	1,500	Screenline does not cross
Link W	2,000	Screenline does cross

The difference between these volumes is attributable to the volume on zone connector B. Therefore, by taking the average of these volumes ((2,000 + 1,500)/2 = 1,750), the effects of the zone connector are spread across the two links. Thus, the value of 1,750 would be substituted for 2,000 in the screenline procedure, thereby more accurately representing the average volume along those sections of roadway.

Screenline Length

As discussed in Step 1, screenlines should be no longer than necessary. As a general rule, screenlines extending beyond the limits shown in Figure A-7 are of questionable value because parallel roadways spaced over those distances would not usually serve as alternative route choices. Even in regional level analyses, the preference has been to construct a series of screenlines across various corridors rather than attempt to produce one very long screenline for the entire region.

Study Area EXAMPLE
Screenline EXAMPLE

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
					Adjustment							Adjustment				
Facility (Nodes)	COUNT	% TCOUNT	A_b	A_f	RATIO	DIFFERENCE	RA_f	C_b	C_f	% TC_f	$\frac{RA_f}{C_f}$	CAPACITY	BASE COUNT	FA_f	$\frac{FA_f}{C_f}$	$\frac{COUNT}{C_b}$
Road A (101-102)	2,500	0.06	900	1,300	3,600	2,900	3,250	13,500	13,500	0.06	0.24	4,400	1,900	6,300	0.47	0.19
Road B (115-120)	4,300	0.10	12,400	13,100	4,500	5,000	4,750	14,900	14,900	0.07	0.32	5,100	3,100	8,200	0.55	0.29
Road C (201-202)	12,350	0.28	3,400	2,000	7,300	10,950	9,100	12,200	13,500	0.06	0.67	4,400	8,800	13,200	0.98	1.01
Road D (313-214)	—	—	—	107,100	—	—	107,100	—	129,600	0.60	0.83	90,800	—	90,800	0.70	—
Road E (300-305)	12,100	0.29	6,000	23,300	—	—	23,300	9,500	32,400	0.15	0.72	10,900	9,100	20,000	0.62	1.31
Road F (415-262)	11,800	0.27	6,700	900	1,600	6,000	3,800	13,500	13,500	0.06	0.28	4,400	8,500	12,900	0.96	0.87
TOTALS	43,350 TCOUNT		29,400 TA_b	147,700 TA_f			151,300 TRA_f	63,600 TC_b	217,400 TC_f		0.69 $\frac{TRA_f}{TC_f}$	See Fig. A-14	0.48 FCAP 0.52 FCOUNT	151,400 TFA_f	0.70 $\frac{TFA_f}{C_f}$	0.68 $\frac{TCOUNT}{TC_b}$

Figure A-15. Completed calculation form.

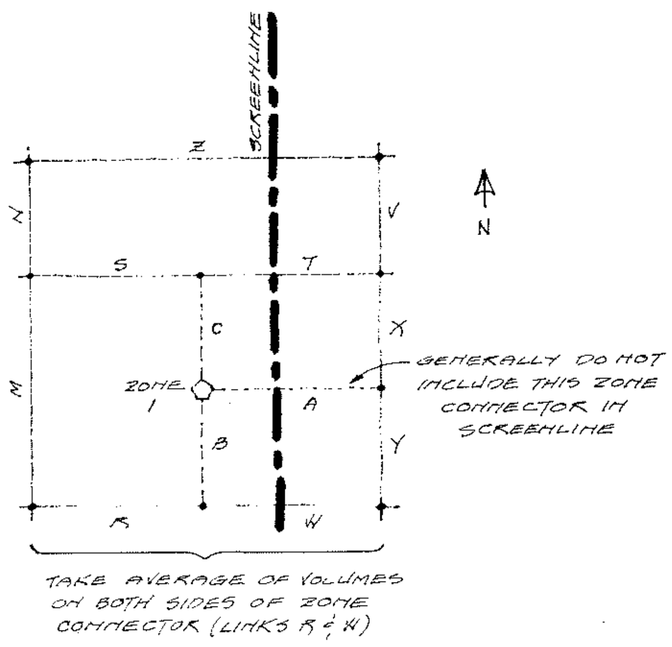


Figure A-16. Zone connectors and screenline.

Where major new facilities are planned for implementation, the choice of screenline length is especially critical. In these cases, the new facility may carry regional or interstate trips that previously did not exist in the corridor. The new roadway may result in a doubling or tripling of the capacity across the screenline. As a result, the screenline may have to be extended over a larger number of roadways to reduce the impact of the new road on the refinement procedure. This decision must also be reviewed after the completion of the calculations when checking the reasonableness of the results.

Lack of Base Year Data

In some situations one or more pieces of base year data may be unavailable. The screenline procedure must then be modified to accommodate these changes. Table A-7 indicates the procedural steps that would need modification for lack of base year counts, assignment, or capacities. Three primary situations are described.

Situation A--Lack of Base Year Traffic Counts (COUNT). Actual base year traffic counts are the most important base year data for use in the screenline procedure. Without these counts there can be no adjustment for probable assignment errors (Step 3-2). However, the procedure can still be used to adjust the future year volumes ($RA_f = A_f$) based on relative future year capacities (% TC_f) on each facility. No other screenline adjustments would be made except as needed during final checks (Step 4). The assumption used is that future traffic will distribute itself according to available roadway capacity. Note that FCOUNT and FCAP (Step 3-4) do not need to be calculated.

As discussed previously (Step 3-4), this adjustment is most valid for screenlines along which significant future changes in capacity are expected. Where this is not the case, a screenline adjustment based on future capacity alone may not result in a more realistic assignment. However, because the staff time requirements are small to perform the calculations, this adjustment can readily be made and compared with the original computer forecast. In most cases, the analyst should have at least some knowledge of relative base year traffic volumes (ADT or peak hour) such that a reasonable assessment of the future forecast can be made.

The base year assignment (A_b) and capacities (C_b) are not directly used in the screenline procedure without the base year counts (COUNT). Because the base year (A_b) and future year (A_f) assignments generally use similar networks, trip tables, and assumptions, several biases of the future year forecast are also likely to be evident in the base year assignment as well. Therefore, using A_b alone in the screenline procedure without any actual counts (COUNT) may in some cases perpetuate the model biases rather than compensate for them. The base year capacity (C_b) can still be used as a reasonableness check against future capacities (C_f). However, this check will rarely resolve major discrepancies between the base year and future year assignments.

The example presented in Figure A-11 and Table A-6 highlights this problem. From Table A-6, the following data are shown for roads B and C:

	Base Year Traffic Count (COUNT)	Base Year Assignment A_b	Future Year Forecast A_f	Base Year Capacity C_b	Future Year Capacity C_f
Road B	4,300	12,400	13,100	11,000	11,000
Road C	12,350	3,400	2,000	9,000	10,000
Total	16,650	15,800	15,100	20,000	21,000

Table A-7. Modification of screenline procedure due to lack of base year data.

Situation	Base Year Data Available? Traffic Counts (COUNT) Col. 2	Traffic Assignment (A_b) Col. 4	Capacity (C_b) Col. 9	Required Modification(s)1/
A	No	Yes or No	Yes or No	• Omit Columns 2, 3, 4, 6, 7, and 9 • RA_f (Col. 8) = A_f (Col. 5) • Omit Step 3-4 • Perform Step 3-5 for all facilities (assumes adjustment only due to relative capacities) FA_f = Capacity Adjustment (Cols. 13 and 15). • Omit Step 3-6 • Omit Step 4-1 comparison with base year V/C ratios
B	Yes	No	No	• Omit Columns 4 and 9 • Omit Step 3-2 (Cols. 6 and 7) • RA_f (Col. 8) = A_f (Col. 5) • Perform Steps 3-4, 3-5 and 3-6 as usual • Omit Step 4-1 comparison with base year V/C ratios
C	Yes	No	Yes	• Omit Column 4 (A_b) • Omit Step 3-2 (Cols. 6 and 7) • RA_f (Col. 8) = A_f (Col. 5) • Perform Steps 3-4, 3-5, 3-6 and Step 4 as usual

1/ Columns refer to those used in Figure A-10.

Study Area EXAMPLE

Screenline EXAMPLE WITH LACK OF BASE YEAR COUNTS

(1) Facility (Nodes)	(2) COUNT	(3) % TCOUNT	(4) A_b	(5) A_f	(6) Adjustment RATIO	(7) Adjustment DIFFERENCE	(8) RA_f	(9) C_b	(10) C_f	(11) % TC_f	(12) RA_f/C_f	(13) Adjustment CAPACITY	(14) Adjustment BASE COUNT	(15) FA_f	(16) A_f/C_f	(17) $COUNT/C_b$
Road A (101-102)	NA	NA	NU	1,300	NU	NU	1,300	13,500	13,500	0.06	0.10	8,900	—	8,900	0.65	NU
Road B (115-120)	NA	NA	NU	13,100	NU	NU	13,100	14,900	14,900	0.07	0.88	10,300	—	10,300	0.69	NU
Road C (201-202)	NA	NA	NU	2,000	NU	NU	2,000	12,200	13,500	0.06	0.15	8,900	—	8,900	0.65	NU
Road D (313-214)	NA	NA	NU	107,100	NU	NU	107,100	—	129,600	0.60	0.83	88,600	—	88,600	0.68	NU
Road E (300-305)	NA	NA	NU	23,300	NU	NU	23,300	9,500	32,400	0.15	0.72	22,100	—	22,100	0.68	NU
Road F (415-262)	NA	NA	NU	900	NU	NU	900	13,500	13,500	0.06	0.07	8,900	—	8,900	0.65	NU
TOTALS	NA TCOUNT		NU TA_b	147,700 TA_f			147,700 TRA_f	63,600 COLUMN NOT NECESSARY TC_b	217,400 TC_f		0.68 TRA_f/TC_f	See Fig. A-14	NU FCAP; NU FCOUNT	147,700 TFA_f	0.68 TFA_f/TC_f	NU $TCOUNT/TC_b$

NA = Not Available
NU = Not Used

Figure A-17. Screenline example with lack of base year counts.

It can be seen that the base year counts (COUNT) indicate actual volumes on the roads that are almost opposite to those of the base year assignment (A_b). The future year forecast (A_f) continues the same trend as A_b. In this case, if A_b were used in the absence of COUNT to adjust A_f, the road B would continue to show a much higher volume than road C, a situation that is not borne out by actual base year counts. Further, this discrepancy would not be rectified by adjusting for changes between the base year capacity (C_b) and forecast year capacity (C_f), which are very close.

Figure A-17 presents an example of this situation using the basic data from Figure 9 and Table 5. Note that only the CAPACITY adjustment is made (CAPACITY = FA_f). The volume/capacity ratios (FA_f/C_f), therefore are all very close to each other, a circumstance rarely found in real life. Special care must be taken in these cases to compare the FA_f with the original A_f to establish the reasonableness of the results.

Situation B--Lack of Base Year Assignment (A_b) and Capacities (C_b). In this situation the adjustment for probable assignment errors (Step 3-2) must be omitted along with any final check of base year volume/capacity ratios. However, the remaining steps can proceed as usual with the future year assignment ($A_f = RA_f$) modified by the CAPACITY and COUNT adjustments (Steps 3-5 and 3-6).

An example of this analysis is shown in Figure A-18. The COUNT adjustment helps retain some of the variation in V/C ratios which are evident in the base year. The primary difference between this result (Fig. A-18) and the results of the full refinement procedure (Fig. A-15) is the total screenline volume (TFA_f). Situation B does not permit an initial adjustment for probable assignment errors, yielding in this case a slightly lower screenline total.

Situation C--Lack of Base Year Assignment (A_b). This is a common situation arising where base year assignments are not typically run or where the assignment was not run for the correct base year (e.g., computer 1978 run but 1982 base year). Without the base year assignment (A_b), the refinement procedure can be performed in a similar manner as in situation B because base year counts (COUNT) are available. Therefore, the computations will be identical to those shown in Figure A-18. Because the adjustment for probable assignment errors is not performed (Step 3-2), the unadjusted year assignment ($A_f = RA_f$) is used directly in the CAPACITY and COUNT adjustments Steps 3-5, and 3-6. The availability of base year capacities (C_b) in this situation permits any necessary checks of base year volume/capacity ratios to be made in Step 4-1.

Applicability of Select Link or Zonal Tree Analysis. Select link and zonal tree analysis, described later in this chapter, can often be effectively used to refine forecasts when base year data are lacking. These procedures enable volumes to be manually reassigned from one link to another based on knowledge of the origin-to-destination movements on a particular link or from a specific zone. These analyses are not entirely dependent on base year data to the extent that the future volumes are adjusted based on reasonable travel paths within the study area. Therefore, select link or zonal tree analysis can be used separately or in conjunction with the screenline procedure to help compensate for the lack of sufficient base year data.

Overcapacity Conditions

Overcapacity conditions (at level-of-service E) can occur along the entire screenline or on selected links. In either case, the future year assignment may require manual adjustment.

Study Area EXAMPLE

Screenline EXAMPLE WITH LACK OF BASE YEAR ASSIGNMENTS AND CAPACITIES

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
Facility (Nodes)	COUNT	% TCOUNT	A_b	A_f	Adjustment RATIO	Adjustment DIFFERENCE	RA_f	C_b	C_f	% TC_f	RA_f/C_f	Adjustment CAPACITY	Adjustment BASE COUNT	FA_f	A_f/C_f	COUNT/C_b
Road A (101-102)	2,500	0.06	NA	1,300	NU	NU	1,300	NA	13,500	0.06	0.10	4,200	1,900	6,100	0.45	NU
Road B (115-120)	4,300	0.10	NA	13,100	NU	NU	13,100	NA	14,900	0.07	0.88	4,900	3,100	8,000	0.54	NU
Road C (201-202)	12,350	0.28	NA	2,000	NU	NU	2,000	NA	13,500	0.06	0.15	4,200	8,800	13,000	0.96	NU
Road D (313-214)	—	—	NA	107,100	NU	NU	107,100	NA	129,600	0.60	0.83	88,600	—	88,600	0.68	NU
Road E (300-305)	12,400	0.29	NA	23,300	NU	NU	23,300	NA	32,400	0.15	0.72	10,400	9,100	19,500	0.60	NU
Road F (415-262)	11,800	0.27	NA	900	NU	NU	900	NA	13,500	0.06	0.07	4,200	8,400	12,600	0.93	NU
TOTALS	43,350 TCOUNT		NA TA_b	147,700 TA_f			147,700 TRA_f	NA TC_b	217,400 TC_f		0.68 TRA_f/TC_f	See Fig. A-14	0.47 FCAP; 0.53 FCOUNT	147,800 TFA_f	0.68 TFA_f/TC_f	NU TCOUNT/TC_b

NA = Not Available
NU = Not Used

Figure A-18. Screenline example with lack of base year assignments and capacities.

Two primary overcapacity scenarios are presented in Table A-8 along with some possible actions that could be taken. In all cases, the analyst must make the final decision whether or not to make these adjustments.

Scenario A. The most common scenario is when selected links show overcapacity conditions while there is sufficient capacity elsewhere along the screenline. First, the analyst should verify that the capacities used are correct. If they are, in most cases a portion of the overcapacity volumes should be reassigned to parallel facilities that are operating under capacity. If select link or zonal tree analyses are available, this task is made easier because the origins and destinations for trips on an overcapacity link are available. The trips most likely to use alternative routes are identified and can be manually reassigned. Select link and zonal tree analysis is described later in this chapter.

If one link is severely overcapacity (e.g., V/C is greater than 1.25), chances are that all of the excess volume cannot be realistically diverted to other links. The analyst should therefore not reduce the link volume such that a congestion problem would be totally eliminated; rather, the magnitude of the problem should be reduced to realistic levels. Situations where traffic volumes regularly exceed computed capacity by as much as 10 to 15 percent are frequently observed in congested corridors, and therefore it is often not desirable to reduce all volumes to the computed capacity represented in the network or the screenline. In some cases the excess traffic will "spill-over" onto local streets which are not shown in the network. If these local streets are deemed to be viable alternative routes, they should be added to the network and some of the excess volume assigned to those facilities. This adjustment is usually judgmentally made given the analyst's knowledge of the local area. Some practitioners have assigned capacities to these local streets and have included them in a revised screenline analysis.

A second factor that should be taken into account is a phenomenon commonly referred to as "the spreading of the peaks." If attractive alternative routes are not available, some travellers who would, under capacity unrestrained situations, travel during the peak hour, instead will change the time when they make their trip. This will result in a lowering of the percentage of travel during the peak hour and will in effect increase the "24-hour capacity" of a link.

Scenario B. The second scenario presented in Table A-8 occurs when the total screenline volume exceeds total screenline capacity. In the extreme case where all links are overcapacity, the land-use and/or trip generation factors as well as the percentage of daily travel assumed to occur in the peak hour should be reviewed carefully. If necessary, these input values should be scaled back and the computer models rerun. Changes in land-use or trip generation can affect trip distribution and modal shift, as well as trip assignment. The only way in which these factors can be totally accounted for is by rerunning the models.

A simplified technique used by some analysts is to scale down all of the screenline link volumes by a factor reflecting a roadway capacity or land-use constraint. Other socioeconomic data, such as population or employment, can occasionally be used as constraints in place of land-use data. The assumption used is that because the traffic volumes forecasted to cross the screenline are unrealistically high, only a portion of the projected land-use development and thus projected trip-making would actually occur. This rationale permits screenline volumes to be reduced using either a total screenline factor or a factor specific to each link.

Table A-8. Overcapacity conditions.

	Scenario		
	Screenline Total Volume	Individual Link Volumes	Possible Actions
A	Undercapacity	Some links overcapacity	. Check capacities to verify reasonableness. If necessary, redo screenline procedure with revised capacities. . Reassign volumes from overcapacity links to undercapacity links. Use select link or zonal tree analysis if available to divert trips. Use local streets not in network if necessary.
B	Overcapacity	Some or all links overcapacity	. Check capacities to verify reasonableness. If capacities are too low, revise accordingly and redo screenline procedure. . Lengthen screenline to take in undercapacity links. Rerun screenline procedure. . Reassign volumes from overcapacity links to local streets not shown in network. Add local streets to screenline or use select link or zonal tree analysis to manually divert trips. . Revise screenline and link volumes to match available capacity. . Rerun computer forecast using scaled-back land use or trip generation rates.

This technique can be applied using the following steps:

o **Step 1: Determine Constraint Factor**

For a capacity constraint:

$$\text{Capacity Contraint Factor} = 1 - \frac{\text{Volume Assigned} - \text{Volume Constrained}}{\text{Volume Assigned}} \qquad \text{(A-13)}$$

where:

Volume Assigned = link volume or total screenline volume initially assigned or refined using screenline procedure;

Volume Constrained = maximum reasonable link or total screenline volume due to capacity constraints;

$0 <$ Capacity Constraint Factor < 1.0.

For a land-use constraint:

$$\text{Land-Use Constraint Factor} = 1 - \frac{\text{Input Land-Use} - \text{Constrained Land-Use}}{\text{Input Land-Use}} \qquad \text{(A-14)}$$

where:

Input Land-Use = zonal land-use input to the model for zones in vicinity of link or screenline;

Constrained Land Use = maximum reasonable land-use for zones in vicinity of link or screenline. Based usually on capacity limitations or available developable land;

$0 <$ Land-Use Constraint Factor < 1.0.

o **Step 2: Apply Constraint Factor to Screenline Volumes**

Multiply the capacity or land-use constraint factor times the link volumes initially assigned or refined using the screenline procedure.

$$\text{Adjusted Volume} = \text{Constraint Factor} * \text{Volume Assigned} \qquad \text{(A-15)}$$

where:

Adjusted Volume = link volume reduced due to capacity or land-use constraints;

Constraint Factor = capacity or land-use constraint factor from Step 1;

Volume Assigned = link volume initially assigned or refined using screenline procedure.

For example, the following screenline data are available:

	Volume Assigned (FA_f)	Capacity (C_f)	V/C
Street A	1,000	800	1.25
Street B	1,500	1,300	1.15
Street C	5,000	4,000	1.25
TOTAL	7,500	6,100	1.23

The decision is made that the volumes assigned are unrealistically high given the available capacity. A screenline capacity constraint factor is desired because each link is the same percentage. The capacity constraint is calculated as follows: 1.0 - (7,500 - 6,100/7,500) = 0.81. This yields the following adjusted volumes:

	Calculations	Adjusted Volume	Adjusted V/C
Street A	0.81 * 1,000	810	1.01
Street B	0.81 * 1,500	1,215	0.93
Street C	0.81 * 5,000	4,050	1.01
		6,075	1.00

The primary limitation of this technique is that it somewhat arbitrarily subtracts volumes across a screenline without adding the volume back to the network somewhere else within the corridor or subarea. It also assumes that trip distribution will not change due to the reduction in trip making; as a result, relative link volumes may be in error. For these reasons it is suggested that this technique only be applied for preliminary planning activities.

In less extreme situations, the overcapacity problem may be because one or more key links were omitted from the screenline. In such cases, the screenline can be lengthened to include additional links which could possibly provide enough capacity to accommodate the excess volumes.

Similarly, volumes can sometimes be diverted to local streets not shown in the network, thus providing additional capacity. As discussed in the previous scenario, the analyst must usually judgmentally decide how many trips to divert to local streets based on local knowledge of traffic patterns. Select link or zonal tree analysis can be useful in this exercise.

SELECT LINK ANALYSIS

A frequently used computer-aided traffic refinement procedure is select link analysis. Its primary use is in providing the analyst with origin-destination patterns of some or all zonal trips using a specific link or group of links in the network.

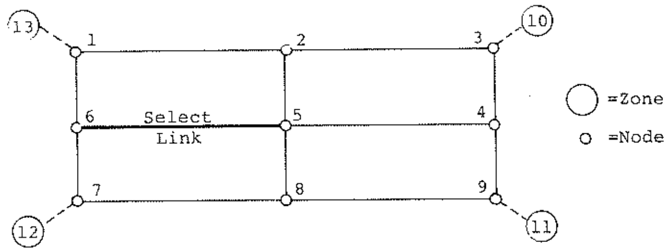
The computer is used to print out the desired select link data. The analyst then manually adjusts the traffic assignment by reviewing the origin-destination patterns. Features of select link computer programs include some or all of the following, as diagrammed in Figure A-19.

- A listing of zonal trip interchanges that pass through the selected link(s) (Fig. A-19B).
- An assignment to the network of all origin-destination trips using a specified link(s) (Fig. A-19B).
- A listing of trip interchanges between two or more specified links (Fig. A-19C).

Using select link analysis, the analyst can identify which origin-destination trip interchanges from the trip table pass through a given link. These specific trips can then be assigned to the network, giving a clear picture of trip movements in the vicinity of the link. Finally, some select link programs allow the analyst to determine which origin-destination trips are common to two or more links. This latter feature, often called point-to-point analysis, is especially useful for analyzing freeway weaving movements and key trip movements within a subarea. An example of a point-to-point data analysis is presented at the end of this section.

The advantages of this procedure include the ability of the computer to provide the analyst with a clear picture of desired trip movements. Once the program is operational, several links can be quickly examined with few computer parameter changes and at a modest cost. The programs print out the desired data either in a tabular or graphical format that can be readily used in the refinement process. In addition to helping refine basic computer assignments, select link analysis

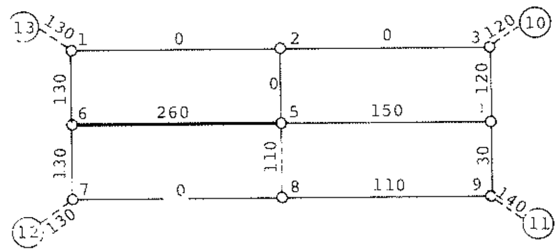
(A) Trip Interchange Data



Link No. 5-6

From Zone	To Zone	2-Way Trips
10	11	0
10	12	100
10	13	20
11	12	30
11	13	110
12	13	0

(B) Loaded Select Link



(C) Trip Interchange Between Two Links (Point-to-Point)

ORIGIN LINK	DESTINATION LINK	TRIPS
5-6	4-5	150
5-6	8-9	110
5-6	6-7	130

Figure A-19. Types of select link analysis.

can be used to modify the assignment to account for network changes such as increasing the capacity of a link, changing the alignment of a facility, and adding or deleting links within the network. Similarly, it can aid analysts in performing manual reassignments of traffic to a more detailed highway network. These latter applications of select link analyses are described in Chapters 5 and 6.

The primary limitation of this procedure is the need to have an available select link computer program that is compatible with the other planning models that are used by planning agencies in the urban area (e.g., UTPS, FHWA). Any select link analysis output needs to be compared with base year traffic counts or origin-destination studies in order to establish its reasonableness and validity. Therefore, the usefulness of the procedure is diminished if adequate base year data are not available. In most cases, select link analyses do not explicitly consider link capacities on a network. Volumes are redistributed based on reasonable travel paths rather than on the basis of available capacity. This limitation must be realized by the analyst, and special efforts should be taken to check the resulting volumes against capacity. Finally, because the computer network typically does not include all roadway links or connectors, the results of the select link analysis must frequently be judgmentally adjusted to reflect actual vehicle movements. This limitation is most apparent for analyzing freeway weaving movements where ramps are often coded together within the network. As a result, the select link analysis will not be able to specifically identify weaving movements on the actual ramps. Additional network specificity in the vicinity of key freeway interchanges should therefore be considered prior to running the select link programs.

Basis for Development

Various computer subroutines for conducting select link analysis have been developed by agencies throughout the country. One of the most widely documented is the selected line plot program used by the New York State Department of Transportation (77). This program provides the analyst with traffic flows for a given link along with district-level trip interchanges.

The FHWA PLANPAC (104, 111) computer battery includes the program LINKUSE, a multipurpose program that performs select link analysis among other capabilities. The UTPS program UROAD also includes a select link analysis function. These widely used programs can produce each of the select link products described previously in a tabular format. Some agencies have adapted these programs to automatically plot the data onto a network map. The following select link refinement procedure has been largely synthesized from analysis techiques used by the Minnesota, Ohio, and Maryland Departments of Transportation.

Input Data Requirements

The required input for select link analysis include the following:

- Historical record network and trip table.
- Specified link(s) for which select link data are to be generated.
- Type of select link analysis desired (e.g., zonal interchange listing; loaded assignment; point-to-point movements).

For most select link computer programs, the data entry parameters are simple and can be quickly coded.

Directions for Use

The following directions will enable the analyst to obtain a select link computer output and perform basic manual refinements to the computer assignment.

Step 1--Determine Key Links within the Study Area

The network should be closely analyzed to determine which links are most important to include in a select link analysis. This decision will vary depending on the type of refinement desired. For example, if the purpose is to refine major traffic movements within the study area, a variety of links should be identified along the entry points to the study area and along primary roadways within the study area.

The analysis may focus on refinement of traffic movements in the vicinity of a freeway interchange. In this case, the key links would be those in the immediate vicinity of the interchange with fewer links chosen elsewhere in the network.

Many times the key links cannot be readily identified without a traffic assignment. Most of the select link programs can be run separately from the full assignment; therefore, the analyst can have the benefit of the assigned volumes prior to choosing links for further analysis. The traffic assignment aids the selection of links in the following ways:

- It identifies links that are heavily over- or under-assigned.
- It identifies link pairs that show a large imbalance in traffic.
- It identifies variances in zone connector volumes.

The assignment therefore offers the analyst a clearer view of which traffic movements should be more carefully analyzed using select link data.

Once the links have been determined, they should be marked in color on the network map. This exercise allows the analyst to see if there are any areas of the study area in which additional or fewer links should be chosen.

Step 2--Determine the Type of Select Link Analysis to be Performed

The type of select link analysis chosen will depend on the purpose of the refinement and to a large degree on the capabilities of the computer program. In the case of LINKUSE, UROAD, and other versatile programs, the analyst must decide between a full origin-destination listing for each link, a loaded link assignment, or a point-to-point analysis.

The full origin-destination listing or loaded link assignment is most valuable for refining traffic on one or more links within a medium-to-large study area. The point-to-point analysis is useful for conducting more detailed studies of trip movements in a small study area. In all cases, the analyst must work within the confines of the available program's capabilities.

Step 3--Prepare Input Data, Run Program, and Check Output

The required link input data and parameters should be prepared in the format specified by the program. The program should be run and the output data immediately checked for reasonableness. An example of LINKUSE and UROAD deck setups is shown in Figure A-21.

Step 4--Place Output into Refinement Format

The output from the select link program should be formatted in a manner that will permit the traffic refinement to proceed in a logical manner. In some cases the select link data may need to be renumbered because of slight changes in the assumed network. For example, a proposed freeway design may be altered such that the computer-coded freeway access points do not exactly coincide. Therefore, certain select link data from the original network may require reformatting prior to actually performing the traffic refinement. Analysts writing new select link programs in-house should be very conscious of the output format so that a minimal amount of data transposition will be required.

Step 5--Identify Inconsistencies and Errors

The resulting select link data should be carefully analyzed. In most cases the analysis will focus on identifying inconsistencies and possible network errors. Some potential trouble signs include the following:

- Extremely large or small trip interchanges or link assignments.
- Large variances between directional origin-destination volumes (e.g., trips from A to B are significantly different than trips from B to A across the link).
- Discrepancies between volumes using parallel routes.

Although many of these problems will be readily apparent to the analyst by scanning the select link data, other data sources should also be used for making comparisons. If possible, the select link data for the future year should be compared with similar data for the base year. The base year data may include actual traffic counts, base year computer assignments or base year origin-destination survey results. The familiarity of the analyst with actual base year traffic flows will often be sufficient to identify problems in the future year forecast.

Another technique is to run the same select link analysis for both base year and future year assignments. The analyst can then compare the results to determine if the differences in origin-destination patterns in the future on a particular link are reasonable given the land-use or roadway changes that were forecasted to occur in the interim. Zonal tree data (see "Special Considerations") can also assist in locating the sources of inconsistencies or errors.

Step 6--Make Refinements to Traffic Assignment

The traffic assignment should be adjusted as necessary. Errors should be resolved by making manual adjustments or by rerunning the computer programs using correct data. Inconsistencies should be resolved using judgment and knowledge of the study area to reassign trips where needed.

The adjustment process does not follow any standard equation or worksheet. Rather, by closely examining the select link data (Step 5), the analyst is provided with sufficient background with which to logically perform the traffic assignment refinement. Attention will focus on the problem areas identified during Step 5. The subsequent example will offer some insight into the logic employed for a particular refinement process.

After progressing into the refinement step, it is probable that the analyst will identify additional links on which he or she will wish to obtain select link data. Other types of select link data may also be sought. For instance, if point-to-point data are initially obtained, the analyst may decide that more specific origin-destination or loaded link data were desirable. These subsequent

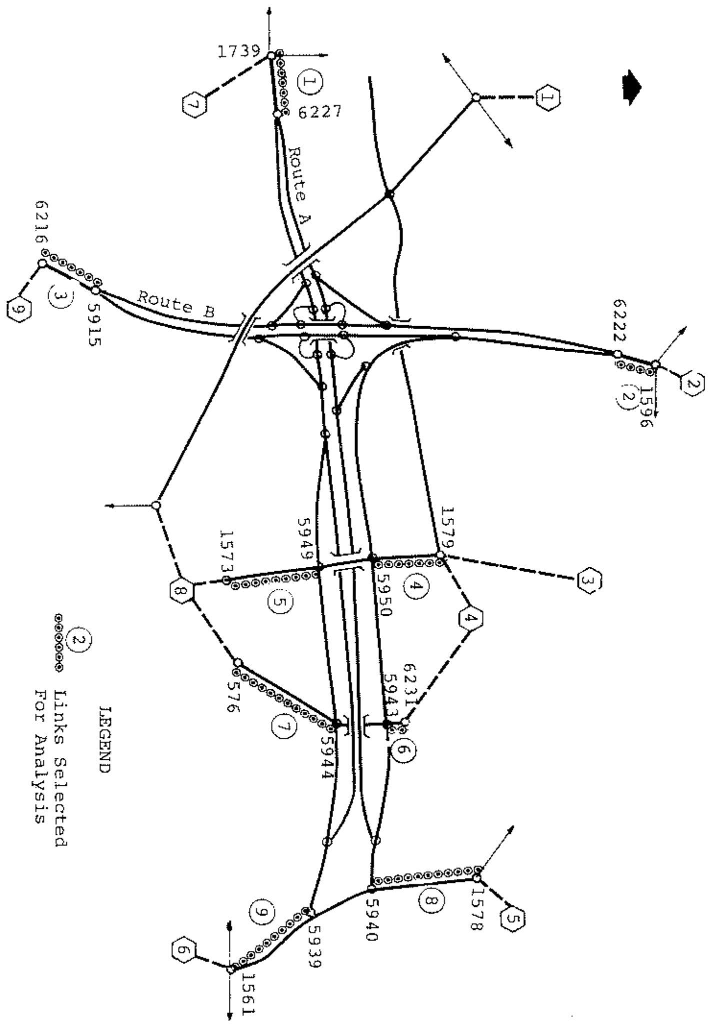


Figure A-20. Example network for select link analysis.

computer runs could then be performed with the capabilities of the program. In this manner, data specific to the analysis at hand will be generated without superfluous data.

The final refined assignment should be compared with the link capacities. If certain links are shown to be over capacity because of the adjustments made, additional reallocation of trips may be warranted. In such cases, the analyst should return to the select link data to identify trip movements that could logically be shifted onto links with adequate capacity. If an entire corridor or subarea is over capacity, the analyst should review the land-use and trip generation data, and if necessary rerun the models using revised data.

Time Requirements

If a select link program is operational, the procedure can be performed expeditiously. The entire procedure could generally be applied to a small-to-moderate sized network in 1 to 3 person-days. If software development is required, this time could easily double or triple for the initial application. Most in-house select link subroutines have been on the order of 100 lines of computer code.

Example Using Select Link Data

This example illustrates the use of select link analysis to refine a traffic assignment for a study area surrounding a proposed freeway facility. The example is adapted from information provided by the Minnesota Department of Transportation.

Given: Coded network (Fig. A-20).
24-hour trip table (not shown).
Future year traffic assignment (not shown)

Objective: Refine the traffic in the vicinity of the freeway interchange of routes A and B.

Procedure:

Step 1--The key links are identified as 1 through 9 on Figure A-20. These represent all of the entry points to the freeway system. No other internal links were considered to be critical for this analysis.

Step 2--The primary select link analysis chosen was the point-to-point capability provided by LINKUSE (104). This permitted trips made between specified network links to be isolated. Loaded select link volumes were available from a previous computer run.

Step 3--The input computer deck setup for LINKUSE is shown in Figure A-21. A partial setup for the program UROAD is also shown for comparison purposes. The LINKUSE program was run and the point-to-point output was obtained. This output was checked for reasonableness.

Step 4--The originial LINKUSE output, as shown in Figure A-21, displayed the data in a format that did not allow for quick comparisons to be made. Therefore, a square matrix was manually constructed (Figure A-22) which enabled various trip interchanges to be readily compared.

Step 5--Various problems were evident by observing the select link data. These include the following, indicated by circled cells in Figure A-22:

- Large imbalances occurred between trip movements 1-2 and 2-1 and between 1-3 and 3-1. These were determined to be caused by the decision to code a cloverleaf interchange into the network with the result that fewer trips were assigned to the longer and slower cloverleafs than to the outer ramps.
- The movements between 6 and 7 were too high since there was actually no roadway connecting link 6 with zone 4 on the north.

LINKUSE deck set-up for point-to-point analysis among 9 links →

```
* CARD FOR A GO
ID,2000 ON 1990 ALT IC
SELECT-01,1739-6227
SELFCT-02,6222-1596
SELECT-03,6216-5915
SELECT-04,5950-1579
SELECT-05,5949-1573
SELECT-06,5943-6231
SELECT-07,5944-576
SELECT-08,5940-1578
SELECT-09,5939-1561
PAR,GROUP*FAVE=ALL,LINKWEAVE=NO
PAR,USELEVEL=2
GO
```

UROAD deck set-up for point-to-point analysis among 9 links ↓

```
& PARAM THRU = 196, CTIME = 0.60, CDIST = 0.01, TABLES = 101, THETA = 0.000,
VFIELD = 0  & End
& SELECT REPORT = 4, ALNK9 = 1739, 6227, 6222, 1596, 6216, 5915,
5950, 1579, 5949, 1573, 5943, 6231, 5944, 576, 5940, 1578,
5939, 1561  & END
& DATA .
```

LINKUSE partial printout for point-to-point analysis

IN ZONE 1 GROUP TOTAL = 29412

DEST	0	1	2	3	4	5	6	7	8	9
0		—	1700	2033	1413	1811	—	2995	—	—
10	1501	734	1305	1417	163	1293	1085	—	11962	

IN ZONE 2 GROUP TOTAL = 25646

DEST	0	1	2	3	4	5	6	7	8	9
0		4503	—	20044	—	—	—	1099		

IN ZONE 3 GROUP TOTAL = 32831

DEST	0	1	2	3	4	5	6	7	8	9
0		326	20891	—	854	—	—	1976	—	—
10	735	152	642	61	53	599	663	—	5877	

IN ZONE 4 GROUP TOTAL = 2092

DEST	0	1	2	3	4	5	6	7	8	9
0		1442	—	451-[illegible]	—	199				

IN ZONE 5 GROUP TOTAL = 1912

DEST	0	1	2	3	4	5	6	7	8	9
0		1682	33	—	197					

IN ZONE 6 GROUP TOTAL = 1243

DEST	0	1	2	3	4	5	6	7	8	9
0		—	—	—	—	—	—	1243		

Figure A-21. Select link computer setup and data output.

To / From (LINK)	LINK 1	2	3	4	5	6	7	8	9
1	—	1,700	2,033	1,413	1,811	0	2,995	1,300	750
2	4,503	—	20,044	0	0	0	1,399	0	0
3	328	20,091	—	750	0	0	1,950	550	100
4	1,442	0	650	—	199	0	0	0	0
5	1,682	33	0	197	—	0	0	0	0
6	0	0	0	0	0	—	1,243	0	0
7	3,292	1,458	0	0	0	1,243	—	400	200
8	1,300	0	550	0	0	0	400	—	1,600
9	750	0	100	0	0	0	200	1,600	—

Figure A-22. Reformatted select link output.

To / From (LINK)	LINK 1	2	3	4	5	6	7	8	9
1	—	3,500 (+1,800)	1,200 (-823)	1,413	1,811	—	2,995	1,300	750
2	3,500 (-1,000)	—	20,044	0	0	0	1,399	0	0
3	1,200 (+872)	20,091	—	750	0	0	0 (-1,950)	550	100
4	1,442	0	650	—	1,442 (+1,243)	0	0	0	0
5	1,682	33	0	1,440 (+1,243)	—	0	0	0	0
6	0	0	0	0	0	—	0 (-1,243)	0	0
7	3,292	1,458	0	0	0	0 (-1,243)	—	400	200
8	1,300	0	550	0	0	0	400	—	1,600
9	750	0	100	0	0	0	200	1,600	—

Figure A-23. Revised "point-to-point" trip interchanges.

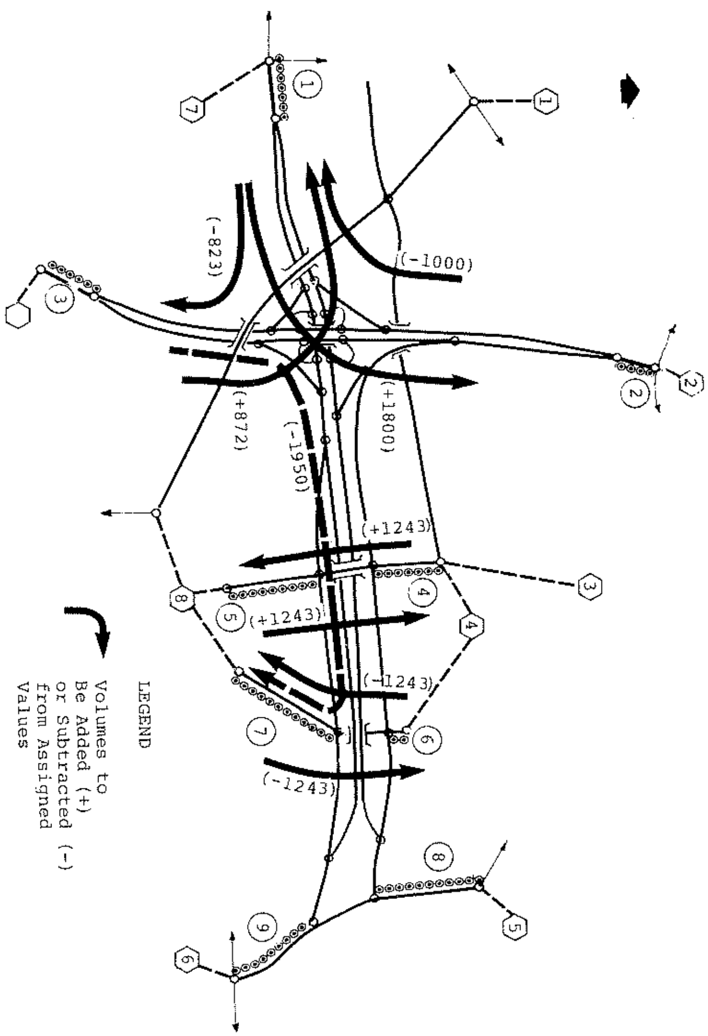


Figure A-24. Volume revisions using select link analysis.

- The movements between 4 and 5 were low when compared with base year counts.
- The movements between 3 and 7 (i.e., between zones 9 & 8) were not considered reasonable since an alternative route to the south (off the network shown) actually provided a more direct route between zones 9 and 8.
- The inconsistencies between other directional movements were considered to be minor.

Step 6--The traffic assignment was manually adjusted to account for the problems observed in Step 5. These actions included the following:

- The 24-hour directional trip imbalance between link 1 and links 2 and 3 was analyzed using travel times, review of land-uses and review of "load select link" data. Accounting for the miscoded cloverleaf problem, the difference in travel times by direction was not significant enough to justify such a large trip imbalance. A review of land-uses and trip generation in the vicinity indicated that the 24-hour directional volumes should be approximately equal for these movements.

In the case of movements 1-3 and 3-1, the maximum volume (2033 from 1 to 3) was not considered large enough to warrant more detailed analysis. Therefore, the two movements were added together (2033 + 328 = 2361), divided by two (2361 ÷ 2 = 1180), and rounded off to 1200 trips in each direction.

In the case of movements 1-2 and 2-1, a review of "load select link" data (not shown) for links 1 and 2 indicated that these movements should also be roughly equal and that the total (both directions) volume (1700 + 4503 = 6203) was low by around 800 vehicles. Therefore, the 1800 trips were added to movement 1-2 and 1000 trips were removed from movement 2-1, yielding 3500 in each direction.

- The volumes between links 6 and 7 were removed and added to the movement between 4 and 5 to account for the lack of a road connection between link 6 and zone 4. This also brought the volumes between 4 and 5 into proper scale with base year counts.
- The volume between link 3 and 7 was removed from the network and assumed to use the alternative route to the south.

The above changes were compiled together and displayed in a revised matrix (Fig. A-23). These changes were then systematically applied to the original future year assignment using a manual assignment process. These changes are diagrammed in Figure A-24. As a final check, the revised link volumes should be compared with the link capacities to ensure that the refined assignment is reasonable and workable.

Special Considerations

There are two special considerations relating to select link analysis, as discussed below.

Combining Select Link and Screenline Analyses

Select link analysis can be performed together with the screenline refinement procedure. This may be accomplished using two alternative approaches as follows:

1. Perform both the screenline analysis and select link analysis separately for the same study area. Compare the results of these methods noting major discrepancies. Perform a final refinement using some proportion (e.g., average) of the traffic volumes from each procedure.
2. First perform the select link analysis to refine the study area traffic assignment. Use the results of this refinement as input to the screenline procedure. Therefore, the traffic volumes obtained from the select link analysis would be considered the Adjusted Future Year Assignment

(RA_f) to insert in column 8 of Figure A-10. Columns 5, 6, and 7 of Figure A-10 would be left blank because the select link procedure substitutes for the initial screenline adjustment. All subsequent screenline refinement steps would be performed, with the resulting volumes (FA_f) serving as the final refined values.

Combining these two procedures has the advantage that the select link analysis provides a better adjustment of specific link inconsistencies, while the screenline procedure explicitly considers the relative base year counts and future year capacities.

Use of Zonal Tree Analysis

Select link computer programs are not currently available in every agency or for every type of forecasting model (e.g., UTPS). However, virtually all traffic assignment models include subroutines to produce a record showing the shortest route from a given zone centroid to all nodes in the highway network. This record is call a <u>zonal tree</u>. A <u>tree trace</u> is a printout showing the sequence of nodes which defines the minimum time paths between zone centroids. Travel times to each node are generally obtained as an option. A <u>loaded tree</u> is the minimum time path from a given zone to all other zones with the trips originating in that zone assigned to it. The loaded tree is identical to performing a load select link analysis on the link(s) connecting the given zone to the highway network (i.e., the zone connector). An example of a tree trace and a loaded tree is depicted in Figures A-25 and A-26, respectively.

Zonal tree analyses produce a logical assessment of trip patterns and can aid the analyst in identifying traffic flows as well as network coding problems (e.g., improper impedances). Such problems are more likely to be significant in all-or-nothing assignments where certain links can become oversaturated if one impedance is set slightly high or low.

On large networks, zonal tree analysis can become time consuming and require trees to be produced for several zones. The analyst must be experienced in manually tracing trees and in selecting additional paths to investigate.

Zonal tree data can be used in much the same manner as select link data. The big advantage of select link analysis is that all of the trips using a link can be produced simultaneously, rather than running separate tree traces. A select link analysis would identify not only the trips from one zone to all other zones using a link but also the origins and destinations of all other trips using the link.

Zonal tree analyses should be used to assist in the refinement process. However, where possible, the trees should be combined with select link analyses and/or the screenline procedure in order to produce realistic results in a reasonable time frame.

Example Using Zonal Tree Data

Given the zonal tree data for the network example depicted in Figures A-25 and A-26, the analyst can study traffic patterns from zone 1 to all other points in the network. Assume that the total assigned volume on link 26-25 (bottom right corner of network) is considered by the analyst to be too high based on previous comparisons with base year counts, land-uses, or with volumes on parallel facilities. The task at hand, therefore, is to reduce the volume on link 26-25 in a logical manner.

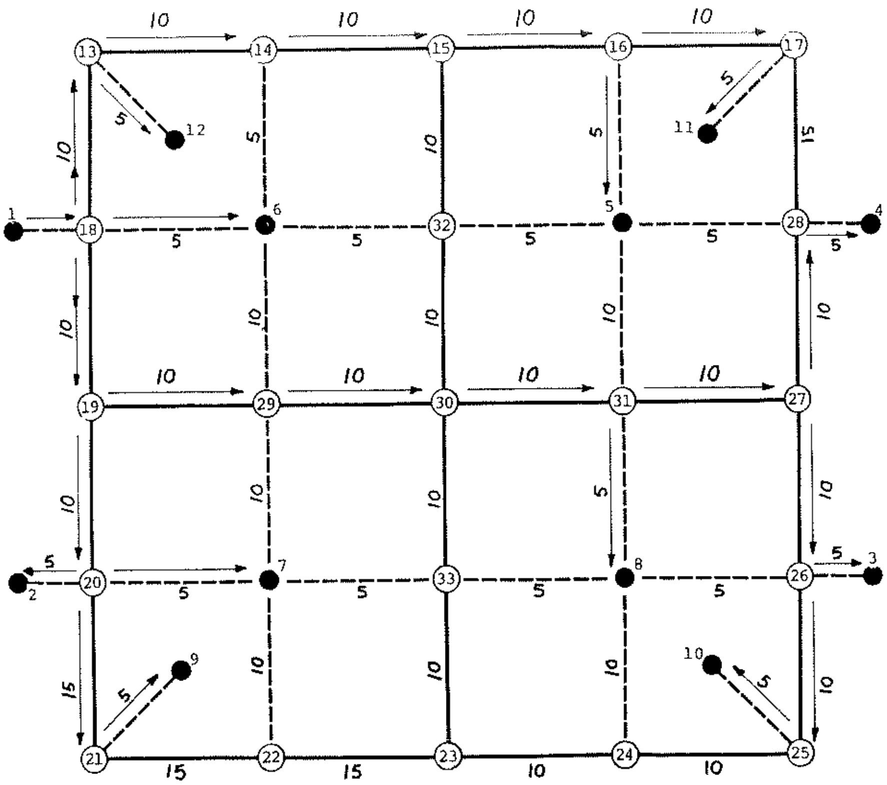
From the tree trace (Fig. A-25) the minimum time path from zone 1 to zone 10 is found to follow the sequence of nodes 1-18-19-29-30-31-27-26-25-10 with a total impedance of 60 minutes. Therefore, the link trace has identified that zone 1 traffic utilizes link 26-25. The loaded tree data from Figure A-26 identifies the magnitude of the zone 1 traffic on each link (e.g., on link 26-25 zone 1 volume = 100). In this example the 100 trips could also be identified by reviewing the trip table for trips between zones 1 and 10. However, using the trip table without loaded trees is manually feasible only if the paths between each zone can be clearly identified.

Given this tree and impedance information, the analyst hopes to divert at least some of the 100 trips away from link 26-25 in order to reduce the total link volume. Upon reviewing the impedances on alternative routes, it is determined that route 1-18-19-29-30-33-23-24-25-10 has an identical impedance of 60 minutes, while route 1-18-19-20-21-22-23-24-25-10 has an impedance of 65 minutes. Because the first two routes have identical impedances, the decision is made to split the 100 trips equally between the two routes, as shown in Figure A-27.

It is also evident from the shape of the network and from the tree traces from zone 1 that the trips between zone 10 and zones 5 and 6 also pass through link 26-25. At this point, additional trees from zones 5 and 6 could be produced, or the analyst could obtain the magnitude of the trip interchanges (i.e., 5 to 10; 6 to 10) from the trip table. In the latter case, the analyst must use judgment and an analysis of link impedances to manually trace the interzonal trips. For instance, the trips from zone 5 to zone 10 could use either path 5-28-27-26-25-10 or 5-31-27-26-25-10, depending on impedances. Since links 27-26-25-10 are the same for both paths, only the impedances for 5-28-27 and 5-31-27 must be compared to determine the shortest path. (e.g., path 5-28-27, impedance = 15; path 5-31-27, impedance = 20).

Once the pattern of trips using link 26-25 is estimated using the tree traces, trips can be manually reassigned to other links. In this example, trips from zone 6 to zone 10 were able to be reasonably diverted to an alternate route (Fig. A-27), while trips from zone 5 to zone 10 did not have a good alternative to link 26-25.

As a result of using zonal tree data, the analyst was able to identify the magnitude and origin-destination of trips using link 26-25. By calculating the link impedances for alternative paths, a logical redistribution of trips between zones 1-10 and 6-10 was possible.

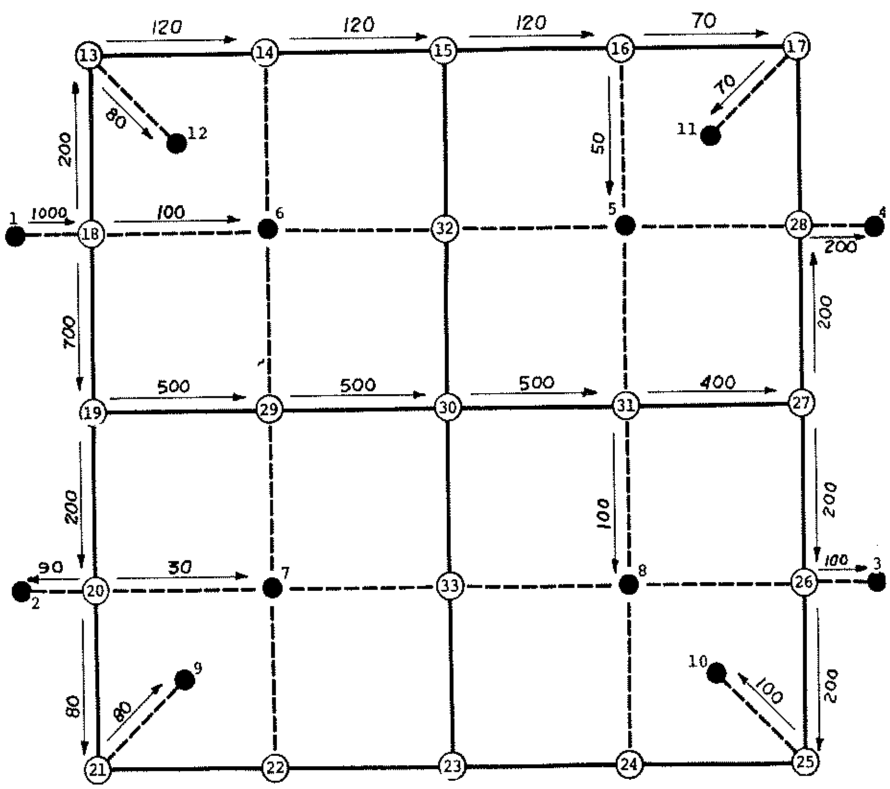


Legend

→ Tree Trace (Zone 1)

10 Link Impedance

Figure A-25. Zonal tree trace.



Legend

100→ Zone 1 Trips

Figure A-26. Loaded zonal tree.

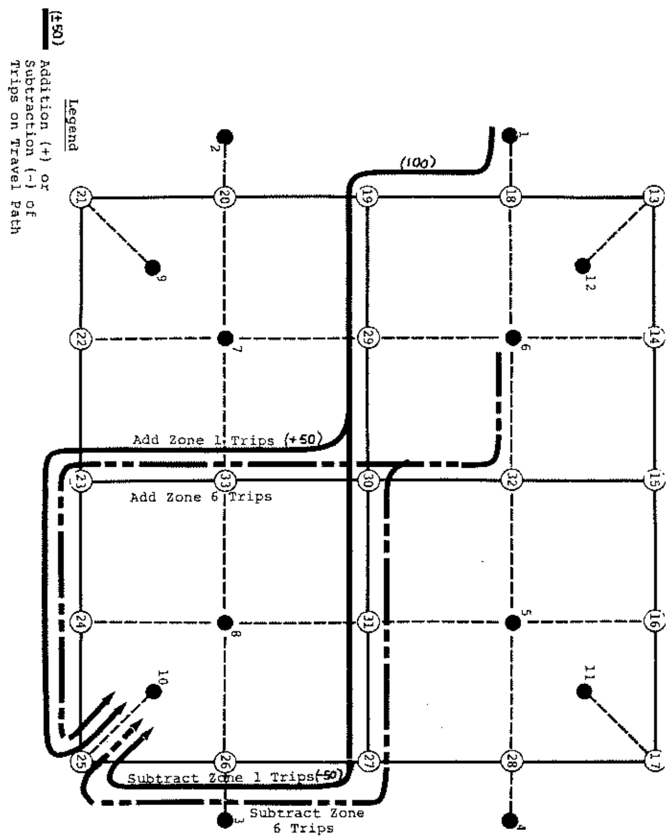


Figure A-27. Volume revisions using zonal tree data.

CHAPTER FIVE
TRAFFIC DATA FOR ALTERNATIVE NETWORK ASSUMPTIONS

GENERAL

One of the major uses of traffic data analysis is to enable alternative highway projects to be effectively evaluated. The alternatives being studied may include major changes to the highway network, such as the construction of a parallel freeway, or may involve more modest actions, such as the upgrading of an existing facility or changing a roadway alignment.

In conducting alternatives analyses, the traffic forecaster must produce adequate traffic assignments for each assumed highway network. Often all of these alternatives have not been reflected in the computer assignments, and in many cases neither time nor funds are available to produce separate computer traffic forecasts for each alternative. Therefore, the analyst must rely on reasonable manual techniques with which to modify available computer forecasts.

The future year highway network assumes various roadway configurations and capacities. However, the timing and magnitude of new construction often cannot be accurately specified. Therefore, the traffic analyst is typically requested to provide traffic forecasts for various levels of roadway improvements over time. These improvements are usually represented by changes in capacity or alignment applied to various network links.

The high relative computer costs of conducting separate traffic forecasts for small network changes may place the analyst in the position of making manual traffic adjustments to one or more basic forecasts. This can occur most frequently if a period of time has lapsed since the original computer forecast was made.

The basic tools available for refining and detailing traffic data are described in Chapters 4 and 5 of this manual. The purpose of this chapter is to describe procedures for adapting these tools to the analysis of alternative networks.

The procedures are applicable to the following situations:

- Change in Roadway Capacity--An existing or planned roadway is upgraded by adding lanes or by improving roadway geometrics. Decreases in roadway capacity (e.g., reduce number of lanes; lower facility classification) can also be examined.
- Construction of Parallel Roadways--A new facility is constructed in a corridor or subarea. Includes addition of minor arterials or short sections of major facilities (e.g., bypass of activity centers). Procedures are not generally applicable for construction of extended facilities passing through study area (e.g., interurban expressway).
- Change in Roadway Alignment--Alternative roadway alignments can be considered. Procedures are not generally applicable for major realignments of roadway within the study area (e.g., shift of proposed roadway from one side of an activity center to another).
- Addition or Deletion of Roadway Links--Short segments of roadway are added to or deleted from the network. Typical applications include the extension of a roadway being constructed in stages or the completion or termination of a minor arterial passing through a residential or commercial area.

The procedures are presented in such a manner that one or more of these situations could be analyzed for any given alternative. Because each highway alternative has unique characteristics, it was impossible to develop procedures that could be rigidly applied in all circumstances. The selected procedures address the key issues to be considered and therefore can be widely applied. In all cases, there is a heavy reliance on a mixture of numerical computations and judgment.

BASIS FOR DEVELOPMENT

The procedures recommended to analyze alternative highway networks are largely based on the following techniques:

1. Screenline Refinement Procedure--This procedure, described in Chapter 4, was adapted from techniques used by the Maryland Department of Transportation (46) and the New York State Department of Transportation (77). This is most useful for analyzing changes in roadway capacity or construction of parallel facilities.

2. Select Link/Zonal Tree Analysis--These analyses, described in Chapter 4, use various supplemental traffic assignment data generated by the computer to identify travel patterns within the network. These travel patterns then form the background for adjusting traffic volumes on specific links. Zonal tree data are available with most transportation modeling packages (104, 111, 115) (e.g., UTPS, FHWA). Select link computer programs were largely the outgrowth of the FHWA program LINKUSE (104), although various adaptions are in use.

For detailed network analyses using select link/zonal tree data, a tabular accounting procedure is recommended. This procedure, entitled "Manual Traffic-Assignment Methodology for Small Networks," was developed as part of NCHRP Report 187 (88). The select link/zonal tree procedures can be used to analyze most of the network changes described previously. They have particular applicability for analyzing impacts due to construction of parallel roadways, change in roadway alignments, and addition or deletion of links.

Each of the procedures permits manual applications, although the accessibility to specific computer output greatly enhances the resulting accuracy.

FEATURES AND LIMITATIONS

The advantage of the screenline procedure is that the effects of network changes can be spread over several parallel facilities. The select link/zonal tree analysis is most useful for adjusting traffic in the immediate vicinity of the anticipated change(s). The manual traffic assignment methodology permits logical reassignment of traffic through a small network which has experienced several changes in the network configuration.

All of these procedures have the limitation that they do not assume any change in trip generation, trip distribution, or total corridor traffic as the result of the network change. This limitation is not a significant problem for analyzing minor to moderate changes, such as the addition of a lane to a roadway or the realignment of a facility; however, a major change, such as the construction of a new facility or the upgrading of an arterial to a freeway, could result in travel pattern shifts which cannot readily be handled using manual techniques. If a major network change is envisioned, serious consideration should be given to making adjustments to land-use projections or trip generation rates and rerunning the entire model sequence.

In applying the procedures to the above situations, the analyst must use judgment to determine whether the proposed network change is significant enough or whether the network is congested enough to justify a reassignment of traffic volumes. In general, the greater the level of congestion on the original network, the greater will the anticipated traffic shift due to a network change. Table A-9 depicts four probable scenarios. For example, increases in roadway capacity generally result in traffic being diverted from parallel facilities to the facility that is being upgraded. Traffic will react to capacity changes most dramatically if the corridor or subarea is already experiencing congestion. Where the traffic congestion is low (e.g., level-of-service A, B, or C), a moderate increase in capacity on a facility (e.g., change from 4 to 6 lanes) will likely create only a traffic diversion. However, large increases in capacity or a functional roadway change (e.g. upgrading a 4-lane arterial to a 6-lane freeway) will create enough of a time incentive to divert traffic even if congestion is not a problem. The same trend in reverse is apparent for cases where roadway capacity is decreased.

Table A-9. Traffic response to network changes.

Extent of Network Change	Level of Network Congestion 1/	Probable Traffic Response
Moderate-Major	High	Large shift expected
Moderate-Major	Low	Moderate shift expected
Minor	High	Small ot Moderate shift exected
Minor	Low	Small shift expected

1/ Congestion level in original (prechange) network.

The following sections present methodologies for modifying the screenline and select link/zonal tree procedures developed in Chapter 4. Emphasis will be placed on providing illustrative examples for each type of network change situation.

MODIFIED SCREENLINE PROCEDURE

The screenline procedure can be easily modified to permit the analyst to examine selected changes in the assumed highway network. It is most applicable for examining changes in roadway capacity and construction of parallel facilities.

Directions for Use

Reference is made to the worksheet (Figure A-10) from Chapter 4 in the following directions.

Step 1--Apply Screenline Procedure for Original Future Year Network

The traffic assignment should first be refined using the same highway network assumed for the original traffic forecast. The worksheet should be fully completed, assuming no change in the network. This step provides a refined assignment from which the impacts of the network change can be measured. In many cases, this step may have been completed during previous studies. Figure A-15 from Chapter 4 presents an example of a completed worksheet.

Step 2--Repeat Procedure Using Revised Network Data

The worksheet is completed in the same manner as in Step 1, substituting the revised network data as appropriate. For a roadway capacity change, the new future capacity (C_f) is placed into column 10 keeping the information in columns 1 through 9 the same. If a parallel roadway is constructed, the new roadway link is added to the bottom of the screenline (Col. 1) along with the future capacity (C_f) inserted into column 10. Columns 2 through 9 are left blank. Columns 11 through 16 are completed based on the revised information. The change in future screenline and link capacities will modify the relative total capacity (% TC_f - Col. 15) applied to each link. This in turn will change in CAPACITY (Col. 13) and COUNT (Col. 14) adjustments and the final refined assignment (FA_f in Col. 15).

Step 3--Compare Assignments

The worksheets from Step 1 and Step 2 are compared for differences in roadway link assignments. These differences constitute the traffic shifts that would occur due to the network change.

Step 4--Perform Reasonableness Checks

The results of the Step 3 comparisons should be checked for reasonableness. The traffic shift due to the network should be in scale with the magnitude and type of the modification and with the level of traffic congestion experienced in the original network. These checks should be performed on a link-by-link basis to ensure that the calculated changes on one link are in line with changes experienced on parallel facilities. It is probable that the reasonableness checks will reveal some discrepancies that must be further checked.

Step 5--Make Final Adjustments

Once the screenline results have been compared and checked for reasonableness, the analyst should make final adjustments using judgment. These adjustments will involve manually reassigning selected traffic among links. In most cases the screenline procedure will provide the analyst with at least an order-of-magnitude assessment of traffic that would be expected to shift in response to the network change. If a series of screenline analyses are being performed, the analyst must be careful to compare the results among screenlines to make sure that the final adjustments are in scale.

This modification of the screenline procedure will provide reasonable adjustments of a traffic assignment due to network changes. In most cases the effect will be to spread out the impact over several links rather than to concentrate the shift onto one or two facilities. If the analyst feels that the impacts of the capacity change will be more isolated, the select link/zonal tree analysis may be of greater use.

Example—Change in Roadway Capacity

Using the same example, depicted in Figure A-11 and Table A-6 of Chapter 4, assume that the following capacity changes (C_f) would occur:

	Original C_f	Modified C_f	Reason
Road A	13,500	20,000	Minor upgrading
Road E	32,400	55,000	Moderate upgrading

Figure A-28 depicts the changes in the worksheet that would occur (Step 2). As a result of the increases in capacity on roads A and E, the relative percentage of traffic increases on those facilities, as shown in the final assignment FA_f (Col. 15).

Next, the two worksheet results must be compared (Step 3). In this example, the FA_f appear as follows:

	Original FA_f (Step 1-Fig. A-15)	Modified FA_f (Step 2-Fig. A-28)	Change
Road A	6,300	8,100	+1,800 (+29%)
Road B	8,200	7,900	-300 (-4%)
Road C	13,200	14,800	+1,600 (+12%)
Road D	90,800	80,200	-10,600 (-12%)
Road E	20,000	25,800	+5,800 (+29%)
Road F	12,900	14,400	+1,500 (+12%)
	151,400	151,200	-200 (Rounding Difference)

Roads A and E show the largest increase in traffic. The volumes on roads C and F, which carried significant portions of base year traffic (see Col. 3-% TCOUNT), also increased. New road D (freeway) shows a sizeable decrease.

Using this information, reasonableness checks were made (Step 4). In this example, both of the capacity increases (roads A and E) were small to moderate in magnitude, and neither facility was upgraded to freeway standards. However, road E runs adjacent to new road D, which is a freeway. Therefore, it was reasonable to expect a moderate improvement in road E to cause a diversion of traffic from the freeway, although perhaps not as a much as indicated by the procedure (-12%). The absolute change in road A volume (+1,800) was modest, even though the percentage change was quite high (+29%).

The next reasonableness check was aimed at the volume/capacity (V/C) ratios. The original screenline V/C ratio (TFA_f/TC_f) was 0.70 (from Figure A-15, Col. 16), indicating a moderate level of congestion. On a link basis, there were certain facilities (i.e., links C and F) showing near capacity conditions in the original network. Therefore, it was reasonable to expect that capacity improvements on roads A and E could cause some change in travel patterns.

The net effect of the capacity adjustment was a spreading-out of volumes across the screenline. Overall, the screenline volume/capacity ratio (FA_f/C_f) was decreased (0.61 after vs. 0.70 before). Individual link V/C ratios also changed. In the case of roads C and F, overcapacity conditions appeared. This situation was not reasonable in light of the overall tendency to spread traffic across the facilities. Therefore, these volumes were in need of some final adjustments.

Some final adjustments were required (Step 5). In the example, all of the revised volumes appeared to be reasonable except for the magnitude of the changes on roads C, D, and F. Based on the foregoing discussion, the decision was made to reduce the volumes on roads C and F to achieve a V/C ratio of 1.0, and to increase the volume on road D by the same amount. Refer to Chapter 4 for additional discussions of overcapacity conditions.

Therefore the final adjustments appear as follows:

	FA_f (Adjusted)	Adjustments
Road A	8,100	None
Road B	7,900	None
Road C	13,500	(V/C * C_f = 1.00 * 13,500)
Road D	82,400	80,200 + (14,800-13,500) + (14,400-13,500)
Road E	25,800	None
Road F	13,500	(V/C * C_f = 1.00 * 13,500)
	151,200	

The total analysis for one screenline was performed in approximately 1 person-hour. Similar levels of effort would be expected for additional screenlines.

MODIFIED SELECT LINK/ZONAL TREE ANALYSIS

Select link or zonal tree analyses can be used to adjust assignments based on network changes. In general, the objective is to use the computer-generated origin-destination data for selected links or zones to divert trips to/from links that are being changed.

The select link/zonal tree procedures are most applicable for analyzing different roadway alignments, construction of parallel facilities, or the addition/subtraction of roadway links. Because the procedures as presented in Chapter 4 do not explicitly consider roadway capacity, special assumptions and modifications are necessary to adequately handle capacity changes.

Directions for Use

The following directions provide basic guidance for analyzing network changes using select link/zonal tree procedures.

Step 1--Refine Assignment for Original Future Year Network

The traffic assignment should first be refined for the highway network assumed for the original traffic forecast. This step may be the result of previous refinement efforts using screenline, select link, or other procedures. However, it is important that the resulting network volumes are compatible with those that will be part of the select link/zonal tree analysis.

Step 2--Estimate Magnitude of Network Change

Before performing any select link/zonal tree analysis, the analyst should determine the magnitude of the network change. As described previously, small changes on an uncongested network are not likely to produce much of a traffic shift. As the magnitude of the network change increases, traffic would be expected to be diverted to/from an increasing number of facilities in the network.

The refined traffic forecast determined in Step 1 should be compared with the forecasted capacity on key links to determine at least an order-of-magnitude level of congestion. Once this is done, the analyst can decide if or how much the network change will be expected to impact the network assignment. If the expected impact is minimal, subsequent detailed analyses may be necessary. If a moderate major network change is expected, the analyst now has a working

Study Area EXAMPLE
Screenline EXAMPLE WITH REVISED CAPACITIES

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
Facility (Nodes)	COUNT	% TCOUNT	A_b	A_f	Adjustment RATIO	Adjustment DIFFERENCE	RA_f	C_b	C_f	% TC_f	$\frac{RA_f}{C_f}$	Adjustment CAPACITY	Adjustment BASE COUNT	FA_f	$\frac{FA_f}{C_f}$	$\frac{COUNT}{C_b}$
Road A (101-102)	2,500	0.06	900	1,300	3,600	2,900	3,250	13,500	20,000	0.09	0.16	5,600	2,500	8,100	0.41	0.19
Road B (115-120)	4,300	0.10	12,400	13,100	4,500	5,000	4,750	14,900	14,900	0.06	0.32	3,700	4,200	7,900	0.53	0.29
Road C (201-202)	12,350	0.28	3,400	2,000	7,300	10,950	9,100	12,200	13,500	0.05	0.67	3,100	11,700	14,800	1.10	1.01
Road D (313-214)	—	—	—	107,100	—	—	107,100	—	129,600	0.53	0.83	80,200	—	80,200	0.62	—
Road E (300-305)	12,400	0.29	6,000	23,300	—	—	23,300	9,500	55,000	0.22	0.42	13,600	12,200	25,800	0.47	1.31
Road F (415-262)	11,800	0.27	6,700	900	1,600	6,000	3,800	13,500	13,500	0.05	0.28	3,100	11,300	14,400	1.07	0.87
TOTALS	43,350 TCOUNT		29,400 TA_b	147,700 TA_f			151,300 TRA_f	63,600 TC_b	246,500 TC_f		0.61 $\frac{TRA_f}{TC_f}$	See Fig. A-14	0.41 FCAP; 0.59 FCOUNT	151,200 TFA_f	0.61 $\frac{FA_f}{f}$	0.68 $\frac{TCOUNT}{TC_b}$

Figure A-28. Example screenline procedure using revised capacities.

knowledge of which links in the network should be examined more closely using the select link/zonal tree analyses.

Step 3--Determine Links or Zones for Analyzing Network Change

The locations of proposed network changes should be identified on the network map, an example of which is shown in Figure A-29. These links are prime candidates for select link analysis, along with links leading into the study area. If zonal trees are being used in the analysis, zones should be selected that are situated in the vicinity of the modified links (Figure A-29).

Many of these links or zones may have been previously used as part of the refinement of the original network assignment (see Step 1) and therefore have data already available. In such cases, this step will only require the analyst to select any additional links or zones for analysis purposes.

Step 4--Perform Appropriate Computer Runs

The type of select link/zonal tree analysis selected for use will largely be determined by the type of available programs (see Chapter 4). If possible, both point-to-point and link origin-destination data are valuable from select link analysis, while link impedances and loaded zonal trees are also very useful. Keep in mind that the primary purpose of this exercise is to logically reassign traffic among facilities based upon changes in capacity. Therefore, a full knowledge of traffic patterns is desirable.

Step 5--Identify Competing Paths and Compute New Travel Times

The select link or zonal tree data should be used to help identify competing paths from/to which traffic may be diverted. The results of Step 2 will enable the analyst to identify the probable breadth of the network change impact.

A network change will generally result in a change in travel time (impedance) for various travel paths. Zonal tree data can be used to identify link impedances for several competing paths. The analyst can then estimate revised impedances for traffic which would possibly take advantage of the network change.

This computation can be performed as follows:

- For added links (e.g., parallel or extended roadway) the link impedance is computed manually in the same manner as done by the computer assignment model. For all-or-nothing assignments, the impedance equals distance divided by assumed speed. For capacity restrained assignments, an equation relating volume to capacity is used. Typically, the impedance used for zonal tree analysis is calculated assuming an unconstrained network, yielding a result similar to that of an all-or-nothing assignment. The individual link(s) impedances can then be manually added along selected paths to determine whether the added link will have a time advantage compared to existing paths.
- For deleted links, no new link computations are required. The traffic previously using that path will be forced to choose between competing paths, for which impedances are available from the computer zonal tree data.
- For different alignments, the previous link impedance for that facility is proportioned up/down by the amount that the modified facility is longer/shorter. The assumption usually made is that the impedance per unit of distance (e.g. minutes/mile) stays the same for the modified

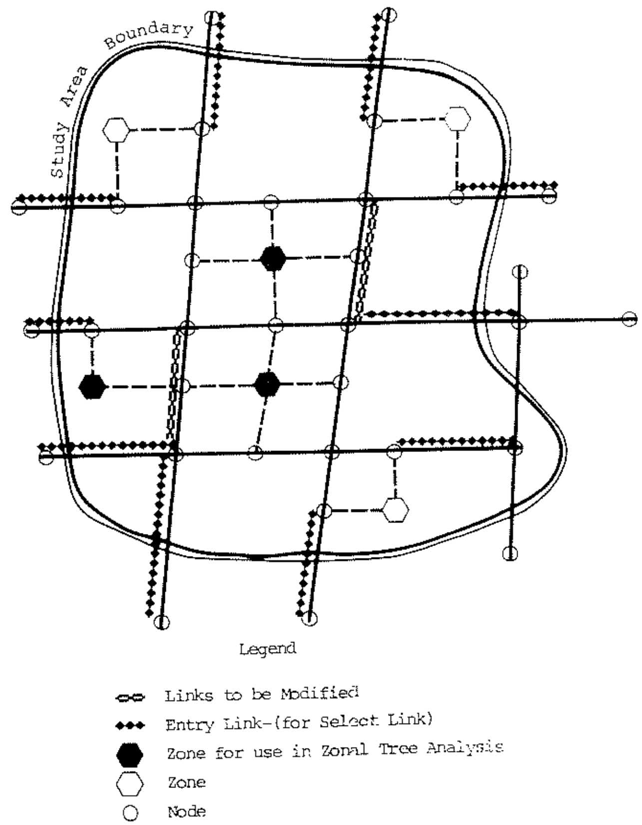


Figure A-29. Selection of links and zones for analyzing network changes.

alignment. Once this new link impedance is calculated, various path impedances can be manually computed.

- For different capacities, the previous link impedance is generally proportioned up/down by some fraction of the amount which the capacity has decreased/increased. Judgment must be used to determine an appropriate change. One source of information is the rationale used to assign speeds to the original network links. Many coders use prespecified speeds for various functional classifications and roadway widths. Given these data, the analyst could assign a new speed to the revised link and calculate a change of impedance by subtracting travel times, as follows:

$$\frac{\text{Link distance}}{\text{Speed After}} - \frac{\text{Link distance}}{\text{Speed Before}} = \frac{\text{Change in Travel}}{\text{Time (Impedance)}} \qquad \text{(A-16)}$$

Assuming a 5-mile link with a change of speed due to a capacity increase from 30 mph (before) to 40 mph (after), the following computations would occur:

$$5/40 - 5/30 = -0.04 \text{ hours} = -2.5 \text{ minutes}$$

The negative sign indicates a decrease in impedance. The advantage of this technique is that the resulting impedance will be comparable with the unloaded impedances known on the other links.

This technique does not, however, consider the volume/capacity ratio. Therefore, the use of unloaded impedances may understate the travel time change that would occur in a congested facility due to a capacity increase. In such cases, the analyst should compute speed changes from the curves (Figs. A-82 and A-83) in Chapter 12. Volume/capacity ratios can be computed for the before-and-after case using the volumes assigned to the link in the original refined assignment (Step 1). Given the V/C ratios, before-and-after speeds can be estimated from the appropriate curves. Equation A-16 would then be used to compute a change in travel time. Care must be taken to ensure that the "before" speed taken from the appropriate curve is in scale with the unloaded speed assumed for that link in the original network. If a functional change has occurred (e.g., upgrade from arterial to expressway), two different curves may need to be used for the before-and-after case, since the functional change will involve a change of design speed and speed limit.

The magnitude of the impedance changes calculated in this step will establish the relative attractiveness of various travel paths within the network. By knowing these relationships, the analyst can manually reassign traffic among various competing paths.

Step 6--Perform Volume Adjustments

In this step, the traffic assigned to the original network (Step 1) is diverted to/from links depending on the network change. The magnitude of this diversion is related to the following factors: (1) magnitude of the network change (see Step 2), and (2) number and type of competing paths (see Step 5).

The analyst should determine a reasonable proportion of the candidate trips to reassign. The select link or zonal tree analysis will be particularly useful in locating the magnitude and paths of trips that are candidates for diversion. Keep in mind, however, that these adjustments will be made to the refined traffic volumes from Step 1. The refined volumes on a particular link, therefore, will not always match the link volumes indicated by the select link/zonal tree data, which are based on the unrefined computer assignment. As a result, the magnitude of trips shown on a certain path by, say, a load select link printout should be considered as approximate for the purposes of adjusting for the network change.

The actual traffic adjustment will involve considerable judgment on the part the analyst. For small network changes, the adjustment will usually be confined to one or two competing paths, while larger changes may involve reallocation of trips across several facilities. In the latter case, the analyst will find the accounting methods used as part of the NCHRP Report 187 manual assignment procedure (88) to be of assistance in keeping track of various trip interchanges.

Step 7--Make Final Check of Volume/Capacity Ratios

Once the traffic has been reassigned based on the network change, the new link volumes should be compared with the link capacities. If a volume/capacity ratio occurs greater than 1.0, alternative paths should be investigated. As with the screenline procedure, the select link/zonal tree analysis of network changes should attempt to balance any diverted traffic along reasonable travel paths. Therefore, overcapacity conditions are generally undersirable on the final assignment. Should overcapacity conditions prevail, serious consideration should be given to making land-use or other assignment modifications as discussed in Chapter 4.

Example--Construction of Parallel Facility

This example uses select link analysis to shift traffic to a newly constructed parallel arterial. The parallel facility, not included in the computer forecast, represents a bypass of a small activity center through which the existing facilities pass. A diagram of the study area network is shown in Figure A-30. The new facility consists of links 4-5 and 4-6.

The refined assignment for the original network is also shown on Figure A-30. These volumes were refined using a combination of screenline and select link procedures (see Step 1). The proposed network change is significant (Step 2). A review of V/C ratios for the links on the original network indicates severe congestion problems.

Link 1-2 was chosen as the key segment for which to perform select link analysis (Step 3). This link accommodates the highest volume of traffic and would experience the worst congestion. After reviewing the data for this link, a decision will be made whether to perform additional select link runs.

An origin-destination select link analysis was performed on link 1-2 (Step 4). Point-to-point and loaded link data were not available to the analyst. After some manual reformatting, the select link data were displayed as shown in Figure A-31.

The analyst identified origin-destination movements that could reasonably be expected to use the new facility (Step 5). Link travel times were reviewed to help identify the competing paths. As seen below, several origin-destination movements would exhibit faster travel times using the new facility, while some would be faster remaining on the original network paths.

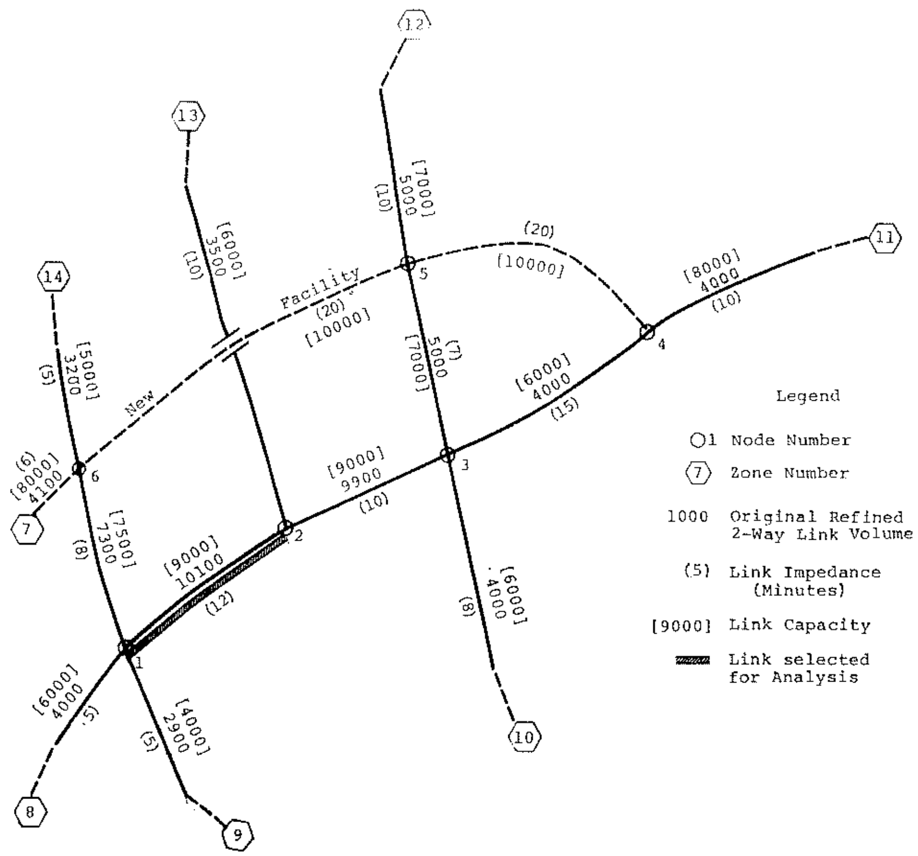


Figure A-30. Example network for construction of a parallel facility.

Zone \ Zone	7	8	9	10	11	12	13	14
7	—	0	0	500	1,000	600	400	0
8		—	0	600	900	800	600	0
9			—	300	800	600	200	0
10				—	0	0	0	1,500
11					—	0	0	1,000
12						—	0	200
13							—	100
14								—

Note: Two-way volumes only in this example.

Figure A-31. Select link data for example link 1-2.

Origin-Destination Movement	Path A Original Network Links	(Impedance)	Path B Revised Network Links	(Impedance)	Impedance Difference (B-A)
7-10	7-6-1-2-3-10	(44)	7-6-5-3-10	(41)	(-3)
7-11	7-6-1-2-3-4-11	(61)	7-6-5-4-11	(56)	(-5)
7-12	7-6-1-2-3-5-12	(51)	7-6-5-12	(36)	(-15)
8-11	8-1-2-3-4-11	(52)	8-1-6-5-4-11	(63)	(+11)
8-12	8-1-2-3-5-12	(43)	8-1-6-5-12	(43)	(0)
9-11	9-1-2-3-4-11	(52)	9-1-6-5-4-11	(63)	(+11)
9-12	9-1-2-3-5-12	(43)	9-1-6-5-12	(43)	(0)
10-14	10-3-2-1-6-14	(43)	10-3-5-6-14	(40)	(-3)
11-14	11-4-3-2-1-6-14	(60)	10-4-5-6-14	(55)	(-5)
12-14	12-5-3-2-1-6-14	(52)	12-5-6-14	(35)	(-17)

Using visual inspection, movements 7-13, 8-10, 8-13, 9-10, 9-13, and 13-14 were not considered candidates for using the new facility. Therefore, O-D travel times were not computed for these movements. For larger networks the analyst need only compute changes in travel time expected between the competing paths, in order to save duplicative computation time. If one were comparing the zone 11 to zone 14 movements, for example, the true competing travel times are for paths 4-5-6 versus 4-3-2-1-6. The travel times for links 6-14 and 4-11 are common to both competing paths and therefore can usually be omitted.

Using the select link and impedance data, several trip movements are identified that are likely to shift to the parallel facility (Step 6). Movements 7-10, 7-11, 7-12, 10-14, 11-14, and 12-14 are all expected to benefit from the new facility, while movements 8-12 and 9-12 have equivalent travel times on either path. The analyst decides to reassign the following traffic to the new facility (path B).

Movement	% Traffic	X	O-D Traffic	=	Total Traffic Shifted
7-10	100%		500		500
7-11	100%		1000		1000
7-12	100%		600		600
10-14	100%		1500		1500
11-14	100%		1000		1000
12-14	100%		200		200
8-12	50%		800		400
9-12	50%		600		300
			TOTAL		5,500

These changes are diagrammed in Figure A-32, along with the revised assignment. The analyst also identified from the link impedance data that zonal movement 11-12 would likely shift from path 11-4-3-5-12 to 11-4-5-12 using the new facility. From the trip table, it was found that movement 11-12 contains 200 trips. These trips were also reassigned and included in the value-sharing in Figure A-32. Similar data could have been obtained by running a select link analysis on link 3-4; however, for this small network these patterns could be visually observed.

The adjusted link traffic was compared with the available capacity as a final check (Step 7). The volumes on the new facility (links 6-5-4) are well within capacity limits, while the volumes on links 1-2-3 were reduced to an acceptable level. Link 1-6, which was operating near capacity, also showed a decrease in volume. The net addition of 300 vehicles onto link 3-5 was not significant enough to cause a problem (i.e., V/C = 5300/7000 = 0.76). Therefore, the volume adjustment appears to be reasonable given the available capacity. The relative balance of traffic using the new facility (links 4-5-6) and the original parallel path (links 1-2-3-4) is also realistic. The reduction in traffic on links 1-2-3-4 will likely create a slight decrease in travel time impedance along that path, resulting in a form of equilibrium with the new facility (links 4-5-6). This type of order-of-magnitude comparison can be conducted even if travel time changes due to traffic demand shifts are not explicitly calculated.

Example—Change in Roadway Alignment

This example assumes that a computer forecast has been conducted for the highway network shown in Figure A-33. Included in this figure are the original refined assignment (Step 1) and link capacities. A change in the roadway alignment has been proposed for links 12-13-8. The revised alignment is depicted in Figure A-33 as links 12-13A-8A. The analyst must manually adjust traffic volumes as necessary.

In order to determine the magnitude of the network change (Step 2), the analyst first looks at the overall level of congestion on the original network. The V/C's calculated range from 0.40 to 1.00, indicating that several links are operating near or over capacity. In particular, links 9-10 and 9-13 are problem areas. Given these conditions, the changed alignment is likely to significantly affect travel times for several paths.

The analyst decides to use zonal tree analysis to perform the traffic adjustment because select link data are not available. Zones 1 and 2 are chosen for analysis given their proximity and orientation to the revised alignment (Step 2).

A tree trace is run on the computer for zones 1 and 2 (Step 4). The results are shown in Figure A-34 along with the original link impedances. Revised link impedances due to the changed alignment are also shown in Figure A-34 (Step 5). The computations for the new roadway alignment are shown below:

- Link 12-13A is 15% longer than Link 12-13. Therefore, Impedance (12 to 13A) = 1.15 x 12 = 14, rounded up to nearest integer.
- Link 13A-8A is 45% shorter than Link 13-8. Therefore, Impedance (13A to 8A) = (1 - 0.45) x 12 = 7, rounded up to nearest integer.

The impedances for divided links 13-13A, 13A - 9, 9 - 8A, and 8A - A are proportioned according to their relative lengths. The total impedance for each original link (e.g., 13-9) remains the same as before.

Several competing paths are identified (Step 5) from the tree traces and revised impedances. Traces for zone 1 indicate that the trips from zone 5 originally follow path 5-11-10-9-8-1 in preference to path 5-11-12-13-8-1, which has a slightly higher impedance. The revised roadway alignment, however, results in equal impedances for these paths (i.e., 5-11-10-9-8A-1 vs. 5-11-12-13A-8A-1). This indicates that some shift in trips might occur, especially since trips using link 10-9 experiences congestion problems and would be likely candidates to seek alternate paths. No other tree traces from zone 1 are observed to be affected by the new alignment.

The zone 2 tree traces reveal a preference of trips from zone 6 to use path 6-12-11-10-9-2 rather than 6-12-13-9-2. The changed alignment significantly reduces the impedance for path

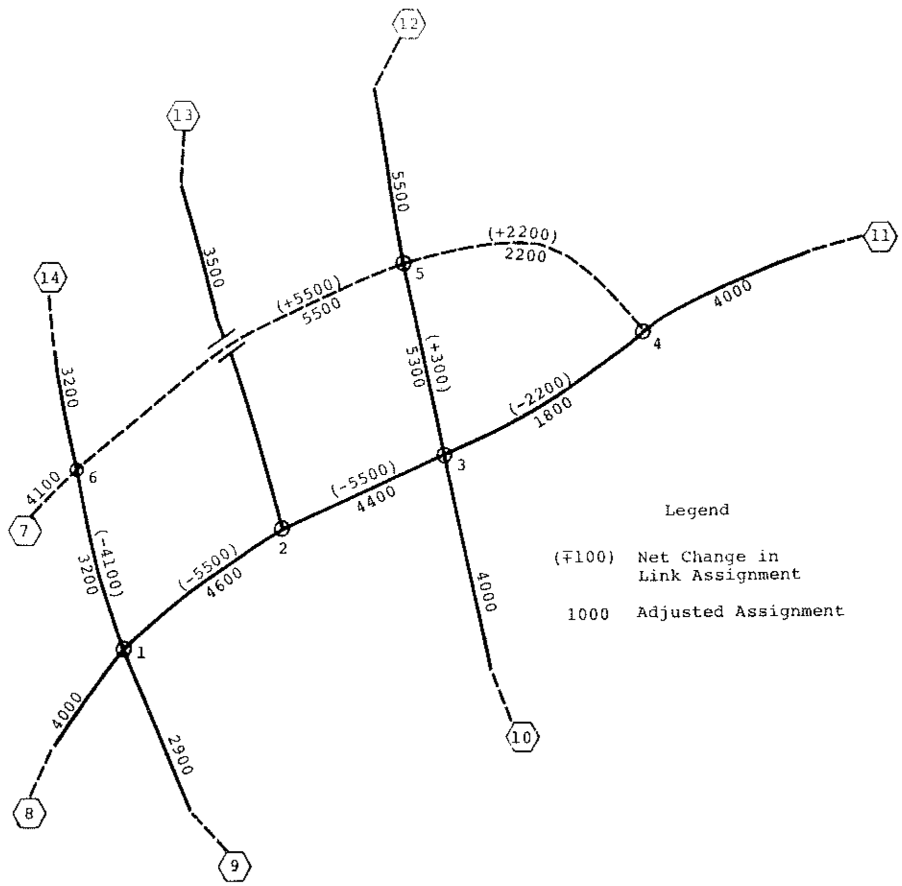


Figure A-32. Revised assignment with parallel facility.

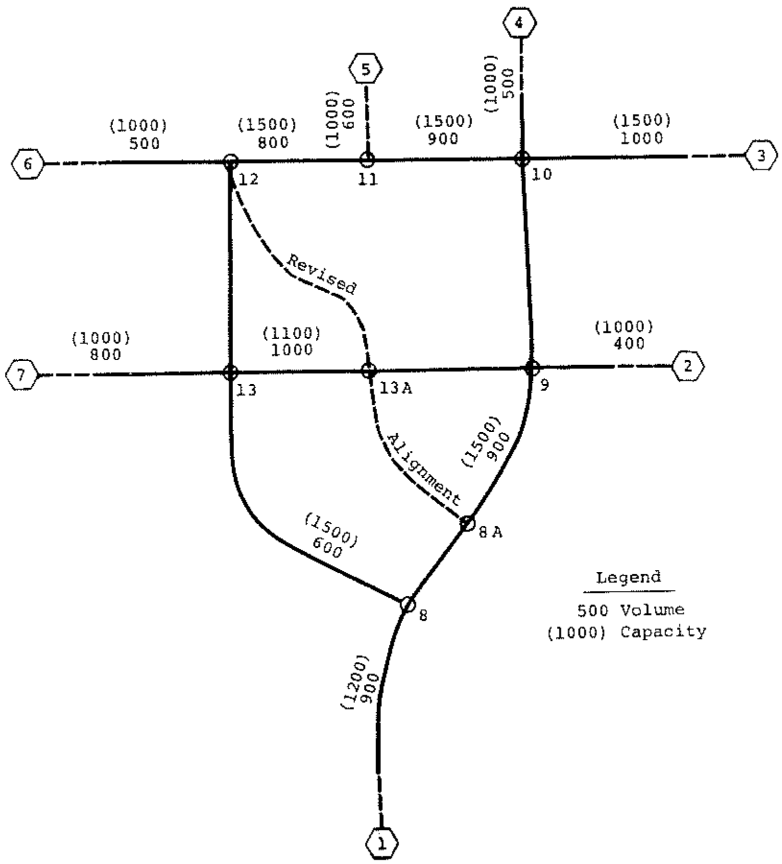


Figure A-33. Example network for change in roadway alignment.

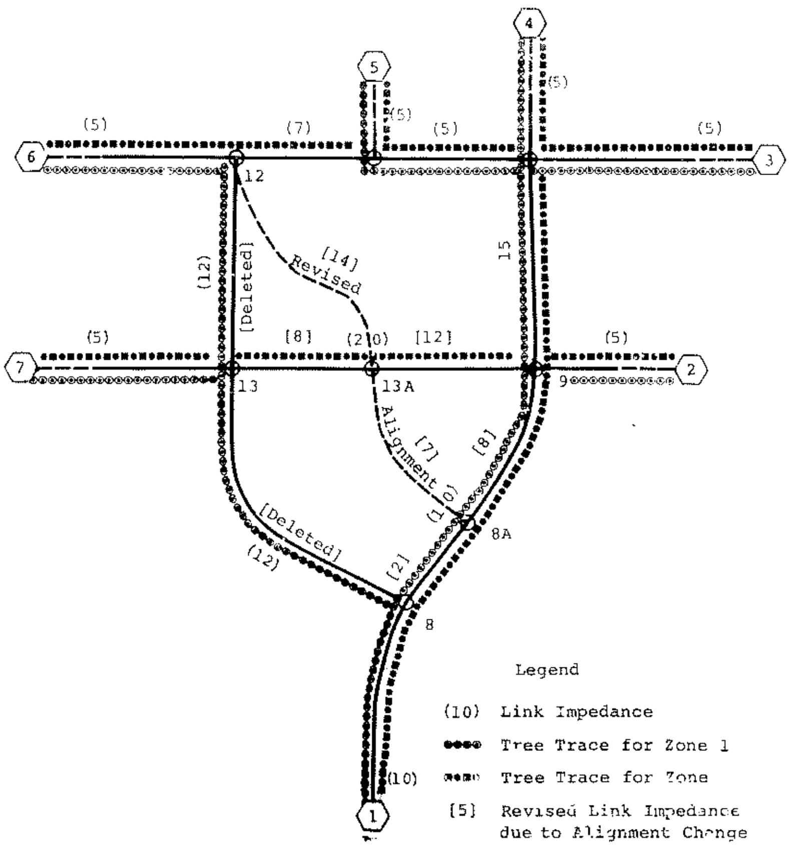


Figure A-34. Tree traces for example zones 1 and 2.

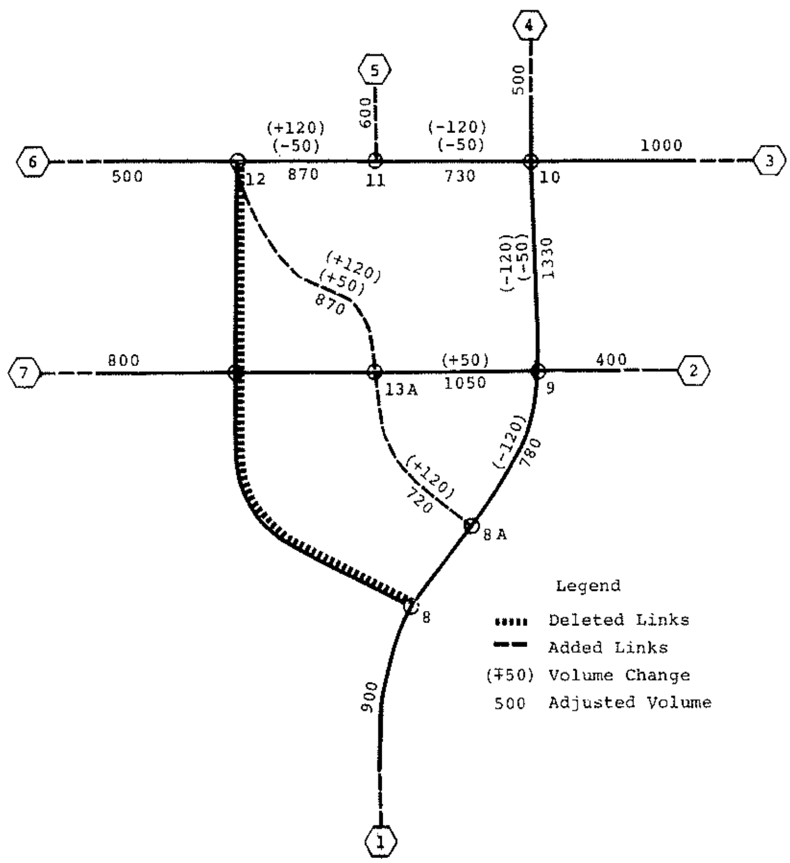


Figure A-35. Revised assignment with realigned roadway.

6-12-13A-9-2 such that it becomes the lowest time path for those trips. No other paths are affected.

These two tree traces provide a good visual picture of trip patterns in the vicinity of the revised alignment; therefore, no additional traces are required. In more complex networks, the analyst may request additional tree traces to be performed once some basic travel trends have been established.

Using this information, the volumes on the revised network can be adjusted (Step 6). From the trip table, the following volumes for zonal movements 5-1 and 6-2 are obtained:

Interzonal Movement	Volume
5-1	200
6-2	100

Since movement 5-1 now has two equal time options, the volume is proportioned between the competing paths. Since link 10-9 is operating at capacity, it is logical to assume a greater proportion of the traffic will use the new realigned facility that bypasses this congested link. Therefore, the analyst assumes 60% (120 trips) will use path 5-11-12-13A-8A-1 and 40% (80 trips) will remain on path 5-11-10-9-8A-1. This change is depicted in Figure A-35.

Based upon the impedances alone, all of the 100 trips for movement 6-2 would logically be diverted onto the realigned roadway path 6-12-13A-9-2. However, since link 9-13 (and hence 9-13A) is originally operating near capacity, it is unrealistic to expect a total shift to this path. The reduction of 120 trips on link 9-10 from movement 5-1 (see previous discussion) makes the original path 6-12-11-10-9-2 somewhat more attractive again. Therefore, the decision is made to equally split the 100 trips for movement 6-2 between the two competing paths, as shown in Figure A-35.

As a final check, new volume/capacity ratios are computed for the revised link assignments. It is found that no link exceeds capacity and that the relative magnitude of the link volumes is reasonable. Comparing Figures A-33 and A-35, it is found that the new facility carries higher volumes than the previous alignment due to a small decrease in link travel time and orientation. The zonal tree and impedance data proved very useful in identifying both the type and magnitude of the change to be expected.

CHAPTER SIX
TRAFFIC DATA FOR MORE DETAILED NETWORKS

GENERAL

Often the analyst is required to produce traffic assignments on highway networks which are more detailed than those used in the system-level forecast. Several different assignments may be requested using alternative network assumptions within a small-to-moderate sized study area.

A procedure is presented which enables the analyst to perform these analyses using either a computer-based or a manual approach. Emphasis in this chapter is placed on the manual application in order to understand the logic that is required. Reference is made to appropriate computer programs that can be used where staff time and resources permit.

The purpose of this procedure is to produce a traffic assignment on a detailed highway network using data available from a systems-level forecast. Two related methods are presented--subarea windowing and subarea focussing. Subarea windowing involves isolating the study area with a cordon and then detailing the trips and the network for this area only. Outside of the study area, all vehicle trips are treated as "external" trips and, therefore, are not subject to change. Because the subarea window is extracted from the network, subsequent analyses can usually be performed manually, although computer techniques are most appropriate if several alternatives are being tested, or if the study area is moderate to large in size. A manual traffic assignment methodology is included in the windowing technique.

Subarea focussing retains the entire regional or subregional highway network and trip table; however, zonal and network data are detailed or aggregated in varying degrees. Within the study area, the network and zonal definition is increased to include specific smaller arterials and/or collector streets and smaller zones specified by the analysis. Away from the study area the highway links and zones are progressively aggregated as the distance from the study area increases. Since the basic regional network remains intact, all trip distributions are subject to change with the introduction of detailed facilities within the study area. As a result, subarea focussing generally requires the use of computerized models.

Subarea windowing and focussing can be applied to subarea or corridor studies where only a system-level traffic assignment has been made on a regional or subregional network. The procedure is most valid in cases where base year traffic counts are available for the facilities on which the more detailed assignment is desired. When base year counts are not available, additional assumptions regarding travel patterns within the subarea or corridor must be made. The procedure is useful for conducting alternative analyses on various subarea highway options or for obtaining detailed assignments for design purposes on collector or small arterial streets.

Detailed networks can be analyzed in a less rigorous manner using the refinement procedures presented in Chapter 5. In particular, network changes due to link addition or subtraction and roadway realignment are of particular relevance to the construction of a more detailed network. Select link and zonal tree analysis procedures are used extensively to perform the traffic forecast refinement for these situations. A modified screenline refinement procedure is also suggested for possible use. Special considerations involving the use of these procedures to analyze a more detailed network are presented at the end of this chapter.

BASIS FOR DEVELOPMENT

The procedures presented in this chapter represent a combination of techniques found in the literature and through personal interviews. Two primary analysis methods--focussing and windowing, are presended to adapt regional network assignment methods to smaller area analyses. Basic information regarding these techniques is provided in Federal Highway Administration literature (18, 106, 107). Subarea focussing was documented more completely by the North Central Texas Council of Governments (75) and the Maricopa Association of Governments (61). These documents were supplemented by discussions with personnel in various agencies that use the focussing method.

Extensive background for the subarea windowing method was provided by the Minnesota Department of Transportation (71, 76). The primary use of the Minnesota documentation was to provide a logical framework from which to construct a subarea window using either manual or computer techniques. Data specific to available windowing computer programs were obtained from documentation for FHWA (104) and UTPS (115) computer batteries. Additional windowing information was provided by the Ohio Department of Transportation and the Washington State Department of Highways (119).

The manual traffic assignment methodology used in the windowing technique was developed as part of NCHRP Report 187 (88). The screenline refinement procedure and select link/zonal tree analysis are documented completely in Chapter 4, while the techniques used to examine highway network changes are presented in Chapter 5.

SUBAREA FOCUSSING/WINDOWING PROCEDURE

Subarea focussing and subarea windowing are two related methodologies used to produce traffic forecasts on a more detailed highway network than was used in the system-level forecast. Special emphasis is given to the windowing method because many, if not all, of the analyses can often be performed manually.

The decision to use the focussing or windowing method is dependent on several factors, as follows:

1. Subarea focussing is preferred if the detailed network alternatives within the study area are expected to create the following situations:

   a. A change in travel demand resulting from changes in trip distribution or modal split.

   b. A major shift in route choice which would affect the locations of trips crossing the study area boundary. Subarea windowing assumes that the external travel demand and route choice remain fixed for all changes within the study area.

2. For computer applications, focussing generally requires less software development time because the only major changes are to the network coding and internal trip table. Windowing usually requires supplemental programs to be run in order to reformat the trip table and network to match the study area boundaries. The use of available windowing programs (e.g., NAG, DONUT) can help reduce these software costs.

3. The preference for focussing increases as the size of the study area increases. Larger networks involve more complex trip patterns and influences which may be cumbersome to perform with any manual windowing method. Also, the larger network increases the probability that external trips will be influenced (see item (1)).

4. Windowing becomes more cost-effective as the number of alternatives to be tested increases. Because a small network is being modeled the lower per-run computer costs of windowing soon offsets the initial fixed software development costs; conversely, focussing computer costs remain on par with those of the system-level forecasts, yielding much higher costs as the number of alternatives increases. For this reason, windowing permits more sensitivity analysis (i.e., bracketing) to be performed using various network or land use assumptions.

These factors must be considered very carefully by the analyst prior to applying either the focussing or windowing method. In many cases, precedence within the agency will indicate a preferred methodology; however, most computer program batteries are flexible enough to permit the use of either method.

Input Data Requirements

The required input data for either the focussing or windowing methods are the following:

- System-level historical record:
  - Trip table
  - Network
  - Zonal land use
  - Select link or zonal tree data (for manual applications)
- Description of network changes to occur within study area
- Appropriate computer software (for computer applications)

The historical record data should be available from the system-level forecast. The study area network description is defined by the user based on the alternative to be tested. The computer software must be independently obtained or developed to match the system-level modeling formats.

Directions For Use

The procedures for subarea windowing and subarea focussing basically follow the same steps:

1. Define study area.
2. Define new zone system and highway network.
3. Define trip table for revised network.
4. Assign trips to revised network.
5. Refine trip assignment within study area.

Each of these steps is described below.

Step 1--Define Study Area

The first step is to specify the study area in which the detailed traffic forecast is desired. For computational purposes this area should be kept to a minimum; however, it should include that portion of the network in which any link changes (e.g., addition, deletion, upgrading) will be proposed. Greater specificity is required in this step for the windowing method since the study area will be totally extracted from the regional network.

The study area should be defined by reviewing the following sources:

- Area maps
- Aerial photography
- Jurisdictional boundaries
- Natural boundaries (e.g., rivers, mountains)

- Field inspections

Initial emphasis should be placed on identifying actual streets and land uses to be included in the detailed network. The analyst should then overlay these features onto the system-level network map and compare the actual and simulated roadways (i.e., links) and land uses (i.e., zones) for consistency. This will permit a close examination of specific network links, nodes, zones, and zone connectors that should be included within the study area. The final effort is to define the study area boundaries on the network map as a cordon line outside of which detailed roadway assignments are not required.

The following guidelines are offered in selecting the study area: (See Fig. A-36)

1. Choose the study area to cover not only the facilities being analyzed, but also the zones that might affect the use of those facilities. Special care should be taken to include all zone connectors and adjacent facilities that could serve as alternate routes to/from the zone.

2. Choose the study area boundary lines to coincide with the system-level transportation analysis zone (TAZ) boundaries.

3. Plot all directional turning movements and link volumes from the base year or future year assignments on an enlarged network map showing highway system, node numbers, and centroid numbers covering the local area. Examine the plotted traffic volumes closely to see which analysis zones should logically be included in the study area.

4. For windowing, care must be taken to include all internal circulation roadways within the study area. This means that all trips originating and terminating solely within the study area must be accommodated by facilities that are situated within the study area. Zonal tree data can be used to determine minimum-time paths between zones.

The UTPS computer program NAG (115) offers another means of defining the study area. Given a focal node, say, in the center of the expected study area, plus a maximum trip length (in minutes) from that focal node, the program selects all links lying within that time limit. As a result, the study area thus defined can differ greatly from one that is defined on the basis of geographic distance or other factors. A study area defined in terms of travel time is especially useful for analyzing the impacts of concentrated developments such as shopping centers, office or industrial parks, and high-density residential areas.

Step 2--Define New Zone System and Highway Network

This step involves detailing the characteristics within the study area and aggregating the characteristics in the remainder of the network. There are two aspects to consider--traffic analysis zones and highway network.

Zone changes will be required as part of either windowing or focussing. Within the study area, the system-level TAZ's are usually so large in size that their use in detailed route analysis will produce unreasonable results in terms of traffic assignments. For example, one TAZ might encompass several intersections and/or interchanges on a route under study. Using minimum time techniques, trips entering the TAZ will be assigned through intersections and/or interchanges based on their proximity to a particular zone connector. This can result in large trip imbalances at certain locations. To overcome this problem, the zone should be subzoned in such a way that there will be more frequent zone connectors, each with fewer trips, thereby more accurately simulating actual street loadings. Determination of subzone boundaries is based on minimum travel time from each subzone to available access points. This topic is also addressed in Step 3B.

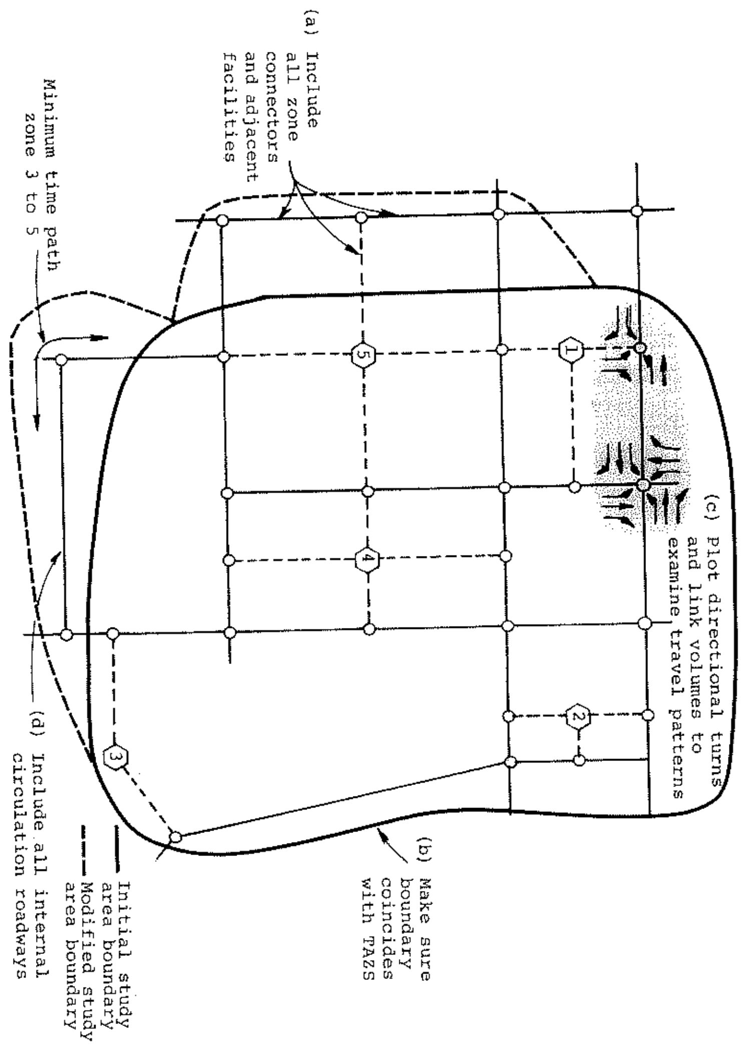


Figure A-36. Selection of study area for windowing/focussing procedures.

The State of Washington (119) uses two means of subzoning, as follows:

1. Directional subzoning--The boundary of the subzone is determined for trips made in a specific direction. Therefore, different sets of subzones may be needed for different directions. Figure A-37 shows an example with Route A and Capital Boulevard as arterials, and 1st Street and 5th Avenue as collectors. Traffic volumes on Capital Boulevard and Route A and turning movements at the intersections of Route A and 1st Street and at Route A and Capital Boulevard are requested. Directional subzones are shown for north-south trips on Route A and for east-west trips on Capital Boulevard. For more complicated cases, more than two sets of directional subzoning may be necessary.

2. Nondirectional subzoning--The boundaries of each subzone are largely determined by the uniformity and geographical features of the zone. Trips can be apportioned to subzones regardless of the direction of the trip. Figure A-38 shows the same zone as in Figure A-37, but with nondirectional subzoning. Note that each subzone has only one zone connector. This construction simplifies the trip assignment and produces reasonable results if the subzones are made small enough. Larger subzones may require additional zone connectors.

Nondirectional subzoning can be used in both manual and computer assignments. However, the number of subzones tends to become so large that the number of movements to be assigned becomes too large to efficiently assign manually. Directional subzoning can reduce the number of subzones used, but cannot be used in computer assignments. It can also become confusing to determine the direction each subzone represents.

Outside of the study area, several zonal changes will also be required. In windowing, the system-level zones are aggregated into a series of new external zones encircling the study area. The new zones connect to each roadway that extends past the study area boundary. These external zones will serve as the origins/destinations of all trips made to/from locations outside of the study area. This concept is shown in Figure A-39. Zones 1 through 5 represent the new external zones (or centroids) that must be added and sequentially numbered around the study area. Internal zones 6 and 7 are also renumbered from the system-level network for ease in analysis. Zones 6 and 7 could subsequently be subzoned as discussed previously.

In focussing, zones can be aggregated in order to reduce computer costs and to conserve the number of zones that are in the network. Often the system-level network includes the maximum permitted number of zones (and nodes); therefore, as subzones are added within the study area, other zones must be removed outside of the study area.

The zonal structure should probably remain intact in the vicinity immediately adjacent to the study area. Patterns of trips traveling through or into the study area might otherwise be disrupted as the zones are aggregated and the zone connectors are changed to match aggregated facilities. In outlying portions of the network, zonal aggregation with reasonably placed zone connectors usually has a minimum effect on trip patterns within the study area. Several agencies have developed special computer software to automatically rebuild zone connectors and approach links for aggregated zones and highway networks.

Network changes will also be required. Within the study area, it is essential that all highways that affect the routing of the traffic be included in the revised network. Using the system-level network as a starting point, additional local, city, county, and state routes within the study area are added to develop a detailed network. Because the network is established for a specific design year, it is necessary to include proposed new facilities and any planned improvements on existing highways. Using the same basic window network as shown previously in Figure A-39, Figure A-40

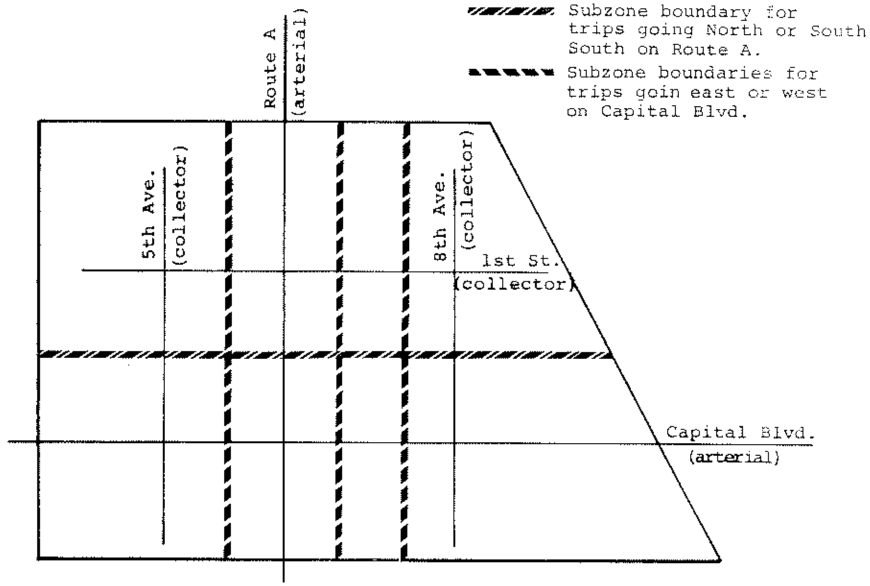


Figure A-37. Directional subzoning.

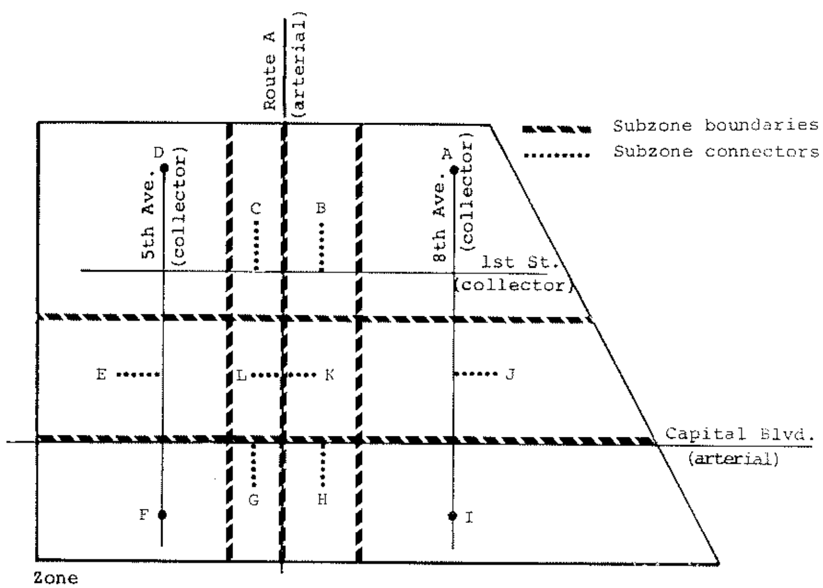
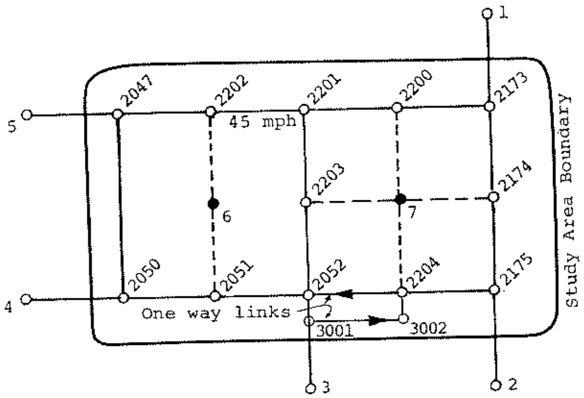
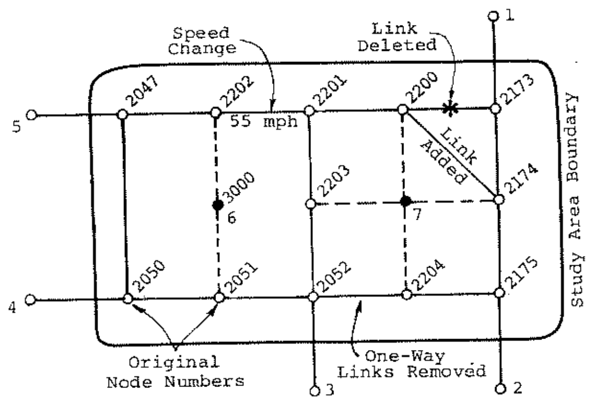


Figure A-38. Nondirectional subzoning.



Legend
O_1 New External Zones (Centroids)
$\bullet_6$ New Window Zone Numbers

Figure A-39. Windowed network.



Legend
O_1 New External Zones (Centroids)
$\bullet_6$ New Window Zone Number

Figure A-40. Windowed network modifications.

presents an example of typical highway network modifications. A link is added between nodes 2174 and 2200, while link 2173-2200 has been deleted. Link 2201-2202 has had its speed increased from 45 to 55 mph. One-way links 2052-2204 and 3001-3002 have also been replaced by the single two-way link, 2052-2204. The replacement of parallel one-way links with an equivalent two-way link has been used by some agencies to avoid traffic assignment problems that have occurred in some small windowed networks. Note that the original network node numbers have been retained, although the zones were renumbered. If desired, windowed networks may use renumbered nodes. If focussing is used, extreme care must be taken that internally renumbered nodes (or zones) do not duplicate numbers used elsewhere in the network. For all of the new street systems in the study area, link distances and impedances are assigned for use in the analysis.

Outside of the study area, the network revisions depend on whether windowing or focussing will be performed. Windowing requires that all network links external to the study area boundary be eliminated and replaced by a series of dummy links connected to new external zones. This task is automatically performed by the FHWA program DONUT (104) and the UTPS program NAG (115).

Various levels of network aggregation are typically required for focussing. In the vicinity of the study area, the original system-level network detail is usually left intact so that trips entering the study area are correctly assigned. In locations increasingly further from the study area, links are combined or eliminated. Major arterials and freeways form the primary elements of the outlying study network. This aggregation of links must be conducted simultaneously with the aggregation of zones so that new centroids are properly connected to the revised network. In order to decrease computer costs the number of links deleted outside the study area should exceed the number of new links added within the study area.

In summary, performance of this step will produce a refined traffic network and zone structure within the study area. Outside of the study area, the links and zones will either be condensed into a series of external zones connected to study area links (i.e., windowing), or else the links and zones will be progressively aggregated (i.e., focussing). The resulting network is suitable for conducting subsequent detailed analyses within the study area.

Step 3--Define Trip Table for Revised Network

This step involves creating a trip table that corresponds to the revised network. Within the study area, the effort made is usually to apportion the trips generated by a zone into two or more subzones. Outside of the study area, aggregation of zonal trips is necessary.

Windowing offers the greatest challenge for reconstructing a trip table, and therefore the following discussions pertain primarily to that method. The focussing method requires many of the same considerations, although the trip table does not require drastic restructuring as is the case of windowing.

It should be noted that the UTPS program NAG automatically defines a revised trip table for a windowed study area using minimum time path trees to determine the proper trip movements to be allocated to the new external zones. Subzonal trips, however, must be allocated manually or by the use of other computer software.

Step 3A--Identify Zonal Interchanges. The purpose of this task is to determine the pattern of trip movements through and within the study area. There are three trip components that must be considered (Figure A-41):

1. External-external (EE) trips--these are trips passing through the study area between two external zones.

2. Internal-external (IE) trips--these trips have one end within the study area and one end outside the study area.

3. Internal-internal (II) trips--these trips originate and terminate solely within the study area and do not cross the study area boundary.

There are several sources of information that can be used to determine these travel patterns, as follows: (1) trip table from system level assignment, (2) link impedances, (3) select link analysis (see Chapter 4), and (4) zonal tree analysis (see Chapter 4).

The original trip table will provide a listing of all trips made among the zones. These trip interchanges will provide an excellent indication of trip patterns in the vicinity of the study area. The trip table should be used to establish the overall distribution of trips, rather than trying to determine specific trip movements. Internal-internal and internal-external trip paths can usually be closely determined from the trip table interchanges if the study area is relatively small. As the study area increases in size, however, these paths cannot be so easily inferred from the trip table.

External-external trips can also be identified using the trip table; however, the extent to which these trips pass through the study area cannot be determined without more detailed analyses. Zonal tree analysis can be used to locate trip patterns from specific zones. Select link analysis can also be very useful in identifying the extent to which specific zonal trips load onto a particular link. Select link analysis can be conducted on several links within the study area in order to determine how many trips have originated in and are destined to zones both inside and outside of the study area. These analyses enable estimates to be made of external-external trips passing through the study area.

Volumes assigned to internal zone connectors can also be analyzed to determine the basic direction in which trips are distributed from a particular zone. This can be very useful in analyzing specific generators such as shopping centers, industrial plants, or airports.

Step 3B--Allocate Total Trips to Subzones. Once the subzone boundaries and trip patterns have been determined, the trips must be allocated to the subzones. This allocation will depend on whether a directional or nondirectional subzoning system has been established (see Step 2). However, in both cases the basic criteria to be used are land-use intensity and distribution of land use within the zone.

The existing and proposed land uses within the zone should be investigated. One way to perform this analysis is to overlay the subzone boundaries onto a land-use map or aerial photograph as shown in Figure A-42. A visual inspection of the subzone land uses will provide an initial basis for suballocating the trips generated by that zone. Other factors that should be considered are the following: (1) location of major generators such as shopping centers or office parks, (2) locations of access points from the zone onto major collectors or arterials, and (3) intensity of land uses in various portions of the zone. The extent to which these factors are quantified for use in splitting the trips associated with the zone will depend on the desired level of detail and the type of subzoning used.

In cases where the land uses are uniformly distributed within the zone, the subzonal trips can be allocated according to area size. For example, given the following zone with four subzones, the trip allocation could be easily accomplished:

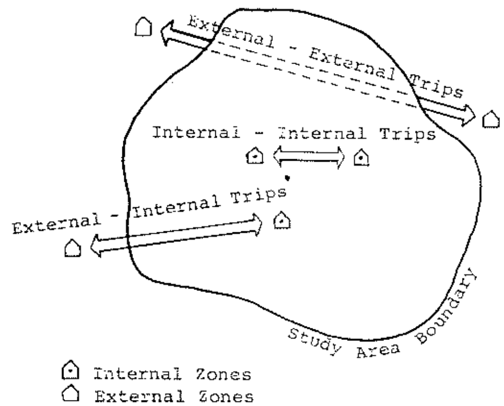


Figure A-41. Distinction between internal and external trips.

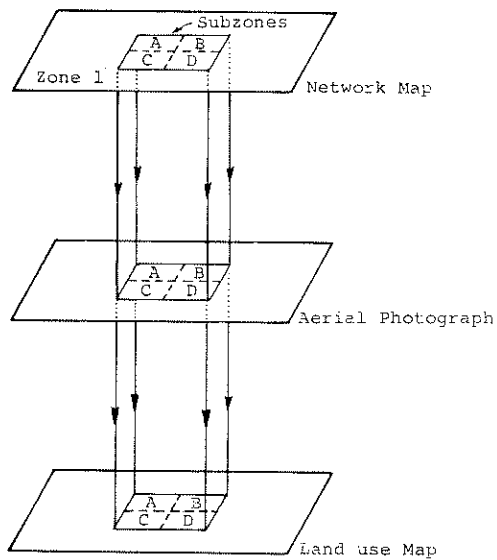


Figure A-42 Allocation of trips to subzones through overlay of information.

Zone	Subzone	Subzone Area (Sq Ft)	Proportion of Trips Allocated
1	A	96,000	96,000/551,000 = 0.17
	B	150,000	150,000/551,000 = 0.28
	C	45,000	45,000/551,000 = 0.08
	D	260,000	260,000/551,000 = 0.48
	Total Area of Zone 1	551,000	1.00(1.00%)

In cases where each subzone contains a different intensity of land use, similar calculations could be performed by comparing square footage of land uses, number of dwelling units, number of employees, or other appropriate units.

Once these proportions are established, the zonal trips can be allocated to the subzones. When nondirectional subzoning is used, the trips in each subzone are geographically distributed in the same proportion as were the trips in the original zone. Using the above example for zone 1 with four subzones, the allocations would occur as in Figure A-43.

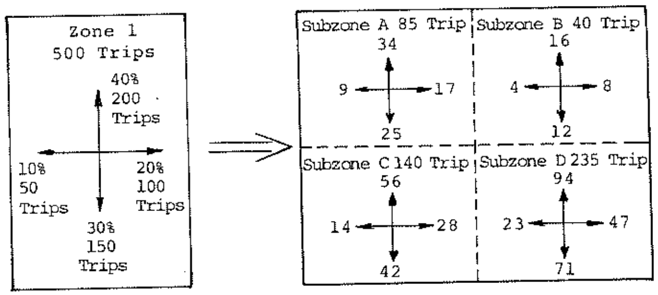


Figure A-43. Example of nondirectional subzoning.

Directional subzoning requires that only those trips oriented in a particular direction are allocated to a subzone. Since different subzones are established for different directions of travel, the trip allocation percentage will change. Using the same basic example, assume that the directional subzones were established as in Figure A-44.

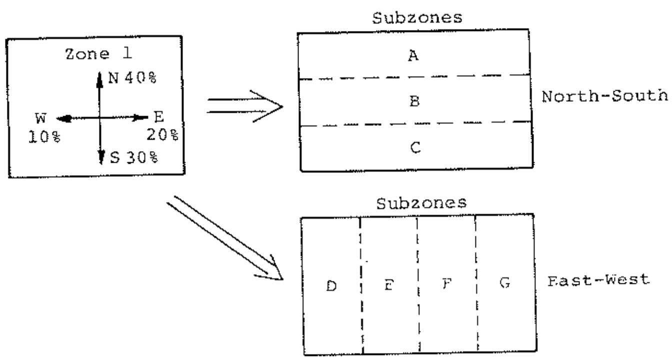


Figure A-44. Example of directional subzoning.

The trip allocation percentages assumed are as follows:

Zone	Direction	Subzone	Percentage of Directional Trips	Percentage of Total Trips
1	N-S	A	20%	-
		B	30%	-
		C	50%	-
			100%	70%
	E-W	D	20%	-
		E	20%	-
		F	30%	-
		G	30%	-
				30%
				100%

Using the total of 500 trips with the distribution as shown above, the directional trips are calculated for each subzone, shown in Figure A-45.

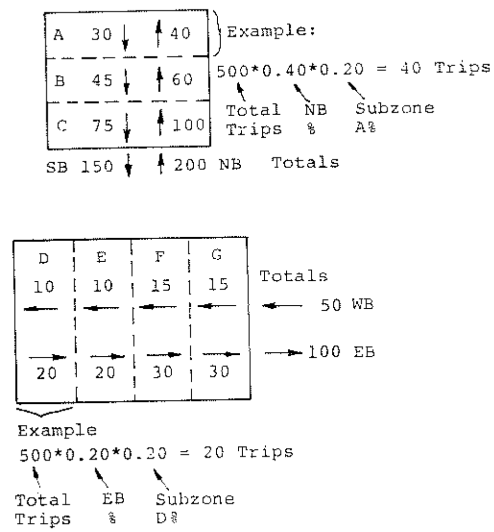


Figure A-45. Directional trip allocation to subzones.

Once the trips are calculated for each subzone, they should be inspected for reasonableness. One important check is to make sure that access points would exist from each subzone to the primary facilities that would be carrying the directional volumes. The relative magnitude of allocated trips in each direction should also be checked for each subzone.

Step 3C--Allocate Total Trips to External Zones. The newly created external zones located around a windowed study area must also have trips allocated. These external zones represent a composite of all zones in the remainder of the network. The effort, therefore, is to allocate appropriate trips from the original network zones to the new external zones. The new external zones will include internal-external trips and external-external trips, but will <u>not</u> include other regional trips that do not pass through the study area.

In most cases, the external zones will be located among major facilities penetrating the study area. Therefore, the volumes assigned to these facilities in the system-level forecast will in most cases become the number of trips allocated to the new external zone. This concept is shown in Figure A-46. In situations where additional external zones are created around the windowed network, the trips allocated to these additional zones must be proportioned from adjacent zones. In doing so, the total number of trips entering and leaving the study area must remain constant. The process of apportioning trips among external zones should consider such factors as the capacity and base year volume on the roadways which they represent, as well as the internal zones which they serve.

For example, in Figure A-47 an external zone C is added between two major facilities, that are already represented by external zones A and B. External zone C represents a minor arterial leading into the study area with a destination to a major industrial park, represented by subzones D and E. This minor arterial had not been included in the system-level forecast. In this example, external zones A and B would handle most of the IE and EE trips oriented in this direction. The new external zone C primarily serves internal subzones D and E and is not expected to accommodate a high volume of EE trips or IE trips between other zones. Therefore, the trips allocated to external zone C should not greatly exceed those trips generated by the industrial park (i.e., zones D and E) made in that direction. Since the facility represented by external zone C is a minor arterial, its capacity would also limit the number of trips which it could handle. The trips finally allocated to external zone C should be proportionally subtracted from the trips originally allocated to zones A and B. This will ensure that the total number of trips does not change.

<u>Step 3D--Calculate Zonal Trip Interchanges</u>. The next task is to construct new trip tables consisting of trips allocated to study area subzones and external zones. Separate trip tables should be produced for internal-internal (II), internal-external (IE), and external-external (EE) trips. These three trip tables can then be combined together prior to trip assignment.

Some basic mathematical relationships that should be understood prior to constructing the trip tables are the following:

$$\text{Total External Zone Trip Totals} = (IE + EE) \quad \text{(A-17)}$$

$$\text{Total Internal Zone Trip Totals} = (IE + II) \quad \text{(A-18)}$$

$$\text{Total Highway Assignment} = (II + IE + EE) \quad \text{(A-19)}$$

Given these relationships, it is possible to perform several checks during the preparation of the trip table. Each component will be discussed separately.

Internal - Internal (II) Trips. The total II trip movements among study area zones are obtained from the system-level trip table. Where subzones have been constructed, the II trip table should be adjusted on a percentage basis.

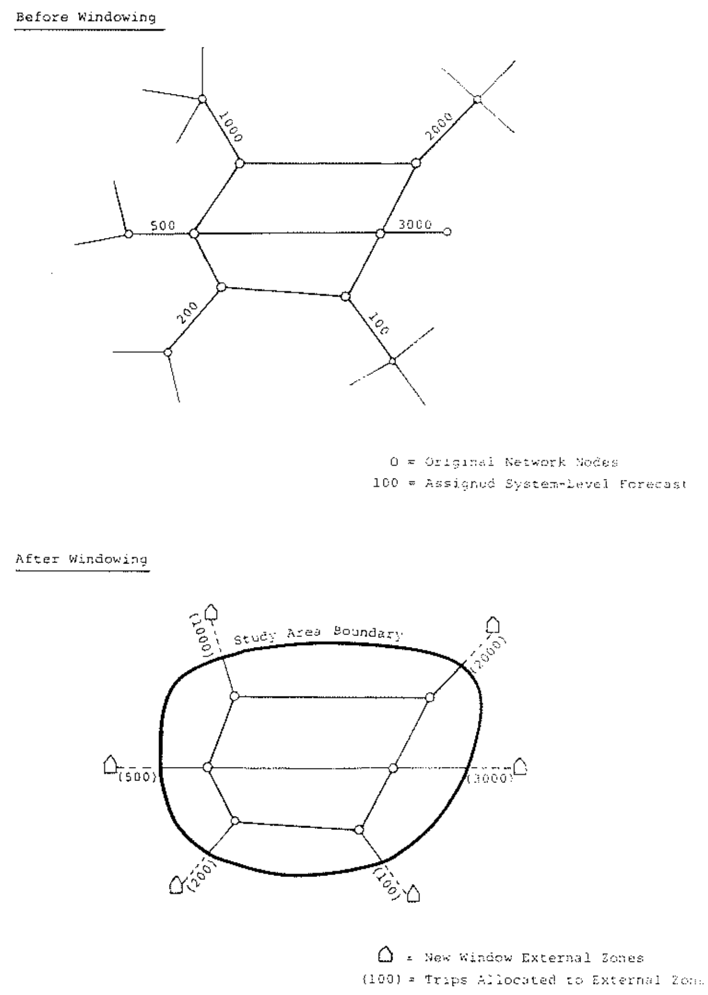


Figure A-46. External zone definition.

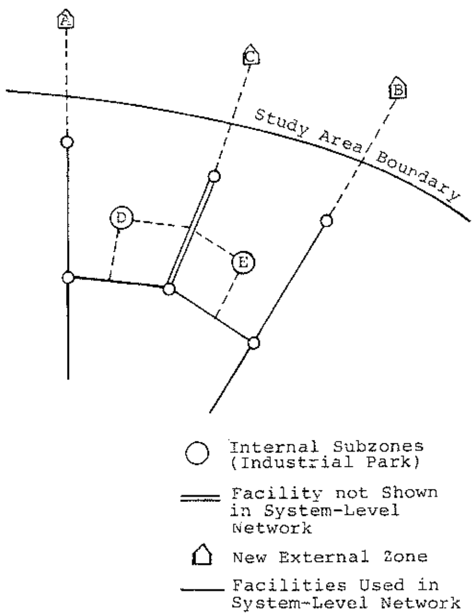


Figure A-47. Addition of external zone to windowed network.

For example, the nondirectional subzoning has occurred for zones 1 and 2, as shown in Figure A-48.

Figure A-48. Example subzoning for zones 1 and 2.

If the II movement between zones 1 and 2 were 100 trips, the following I/I subzonal movements could be estimated:

Subzone 1A to 2 A	100 * 0.40 * 0.70 =	28 Trips
Subzone 1A to 2B	100 * 0.40 * 0.30 =	12 Trips
Subzone 1B to 2A	100 * 0.60 * 0.70 =	42 Trips
Subzone 1B to 2B	100 * 0.60 * 0.30 =	18 Trips
	Total =	100 Trips

II Trips can therefore be distributed according to Eq. A-20:

$$T_{A\text{-}B} = S_A * S_B \times T_G \qquad \text{(A-20)}$$

where:

$T_{A\text{-}B}$ = total trips between subzones A and B.

S_A, S_B = trip percentage for subzones A and B, respectively; and

T_G = total II trips in the original trip table.

The percentages of each subzone for the II trips may be different from those used for IE trips if local land-use developments warrant the change. For example, a subzone that contains a local shopping center is likely to attract a larger percentage of II trips than a subzone consisting of a major industry or office park. Knowledge of the land uses in each subzone will enable these percentages to be judgmentally adjusted. Should such an adjustment be made, it is important to adjust the IE trip total for that subzone such that the total internal zone trips (i.e., II + IE) for that subzone remain constant. An example of an adjustment is depicted in Figure A-49.

Internal-External (IE) Trips. The total IE trips for each zone or subzone can be obtained by subtracting the II trips from the total internal zonal or subzonal trips derived from the system-level trip table. However, these trips must next be distributed to the new external zones.

Each trip interchange between the internal zones (or subzones) and the original system-level regional zones must be allocated to one or more of the new external zones. This amounts to a traffic assignment, in that each IE trip will follow a minimum time path. The travel patterns identified in Step 3A should be reviewed to help determine the distribution of IE trips. To aid in manually constructing the new trip interchanges between the internal zones and the new external zones, the following method can be used:

1. Overlay the windowed study area onto the regional network map (see Fig. A-42).

2. Draw "spheres of influence" for each new external zone. An external zone effectively "captures" a portion of the IE trips associated with the original regional zones outside of the study area. As exemplified in Figure A-50, new external zone 1 captures the IE trips between the internal study area zones and regional zones 8, 12, 13 and 20.

In constructing spheres of influence, consideration must be given to the type of highway network outside the study area as well as to the linkages between the new external zones and the internal zones. For instance, external zones representing freeways or arterials penetrating the study area would likely capture the trips from a wider range of regional zones, and thus have a larger sphere of influence. The trips associated with each external zone (Step 3C) should be constantly reviewed to ensure that reasonable spheres of influence are constructed. A table should be prepared listing which regional zones are associated with which new external zone. Regional zones that fall under the influence of two or more external zones should be specially flagged.

The spheres of influence that are developed should be treated only as guidelines for distributing IE trips to external zones. For small study areas the distribution of trips may be straightforward; however, as the study area increases, there exists a wider variety of locations at which IE trips will enter the study area. As an extreme example, given the situation in Figure A-51, assume that the study area (e.g., CBD) is encircled by a freeway. A sphere of influence has been constructed for external zone 1, encompassing regional zones 8, 9, and 10. The following trip interchanges exist from the system level assignment for internal zones 6 and 7:

Internal Zone	External Zone		Trips
6	8		100
6	9		150
6	10		50
		Total	300
7	8		200
7	9		250
7	10		150
		Total	600

Using the sphere of influence technique, the analyst would construct a new trip table aggregating zones 8, 9, and 10 into new external zone 1, yielding the following:

Internal Zone	External Zone		Trips
6	1		300
7	1		600
		Total	900

This IE trip table seems reasonable for internal zone 7, given its proximity to external zone 1. However, for internal zone 6 it is unrealistic to presume that the entire 300 trips would enter the study area at external zone 1 and then travel through the study area to reach internal zone 6. In this case, some of these trips would be expected to use the circumferential freeway to enter the study area at external zone 2. Therefore, a more reasonable trip table might be:

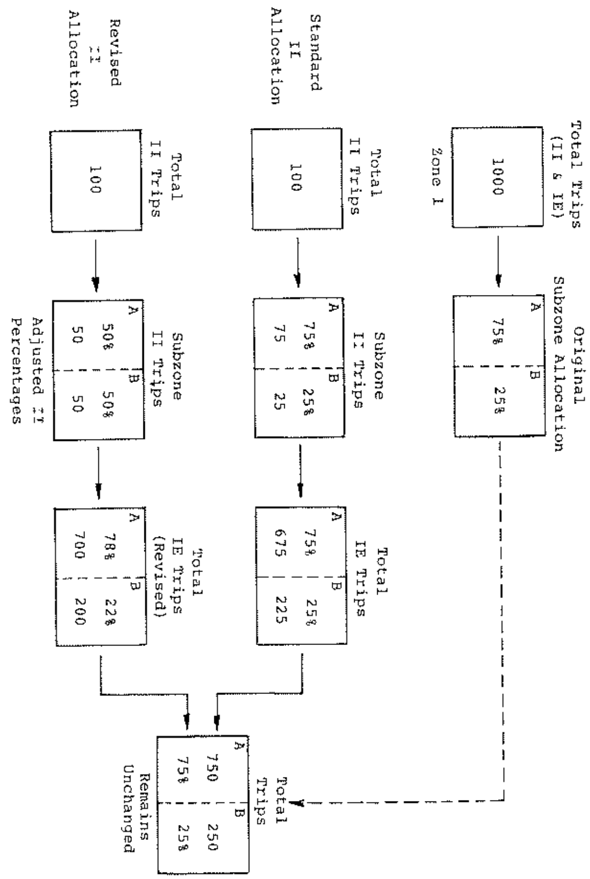


Figure A-49. Example of adjusted II trip percentages.

Internal Zone	External Zone		Trips
6	1		150
6	2		150
7	1		600
		Total	900

Zonal tree traces run on the system level network will provide an excellent indication of travel paths that may not be intuitively obvious to the analyst.

Select link analyses can be used in lieu of manually constructing and analyzing the spheres of influence. If origin-destination select link data are obtained for each system-level network link penetrating the study area (i.e., new external zone locations), a very accurate picture can be obtained of IE, as well as EE trips. Additional point-to-point analyses among these links will identify movements of EE trips through the study area. Therefore, select link analyses can help produce a valid window trip table in an efficient manner.

IE trip interchanges for new subzones should be allocated according to the proportion of the total zone which each subzone represents. This proportion must reflect any changes made during the calculation of II trips.

External-External (EE) Trips. The trips passing through the study area are assigned to a pair of external zones. The results of zonal tree or select link analyses during Step 3A will enable the analyst to obtain a good estimate of the major external-external trips. Base year traffic counts and system-level traffic assignments should also be inspected to identify these patterns. Many of the remaining "minor" trips can usually be omitted for analysis purposes; however, knowledge of the local area will usually enable the analyst to determine the extent and magnitude of many of these trips. This effort is straightforward in most cases, because a significant number of regional zones typically will contribute very few or no EE trips passing through the study area. These regional zones can be omitted from further consideration.

Because the external-external trips will not be readily identified from the trip table as are the other trip components, it is likely that the resulting EE trips will need to be adjusted so that external zone trip totals are maintained. As a check, the total EE trips allocated to an external zone can be estimated by subtracting the total IE trips from the external zone trip total determined during Step 3C using Eq. A-17. For example, if external zone "X" is allocated 5,000 total trips, of which 3,600 are estimated to be IE trips (from the IE trip table), the total E-E trips must equal 5,000 -3,600 = 1,400. These 1,400 EE trips for zone "X" can then be compared to the trips estimated for zone "X" using the EE trip table developed in Step 3D. If the EE trip totals do not match, further adjustments are required either to the external zone trip totals (Step 3C) or to the IE or EE trip tables (Step 3D).

Combined Trip Table. The II, IE, and EE trip tables should be combined prior to trip assignment. Because the format of each trip table is identical, the combination becomes a simple additive process.

Once the total trip table is completed, trip end summaries for the internal and external zones should be compared with those available from the system-level forecast. Discrepancies should be checked in each of the separate II, IE, or EE trip tables. Reasonableness checks should also be made against ground counts and facility capacities.

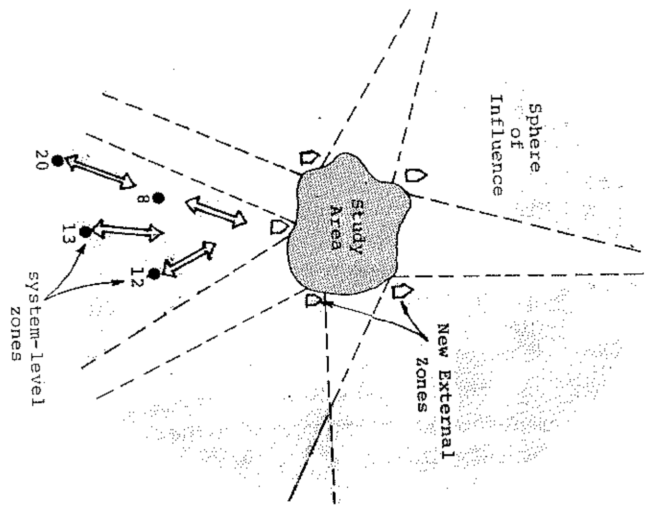


Figure A-50. Sphere of influence.

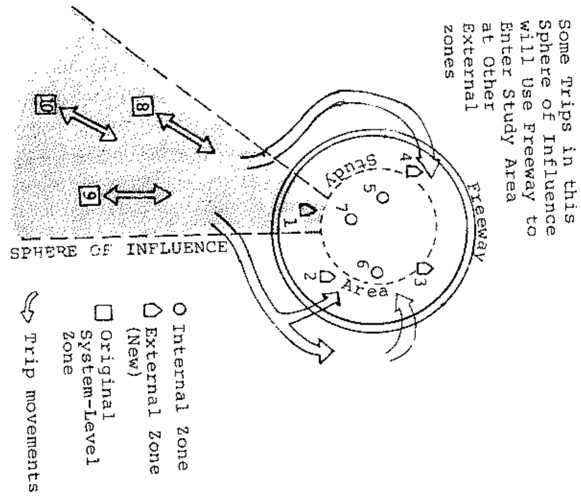


Figure A-51. Trip allocation to external zones.

Step 4--Assign Trips to Revised Network

The basic methodology for assigning traffic to the detailed study area is to determine a logical path for each trip interchange and then to accumulate the number of trips on each street segment along the path (19, 88). Subarea focussing, because of its large network size, usually requires a computer to efficiently assign trips to the revised highway network and zonal system. Most agencies use standard FHWA or UTPS traffic assignment programs for this purpose. Special care must be taken to ensure that the revised network links are compatible in format with those used for the system-level forecasts. The trip table used must also have been formatted to include the changes in the zone system due to aggregation (outside the study area) or disaggregation into subzones (inside the study area).

Traffic assignments for windowed subareas can also be performed using standard computer programs, provided that the new (smaller) network and trip table is properly defined and formatted. In most cases, the assignment program will be the same as was used for the system-level forecasts.

If the windowed network size is not excessively large, the traffic assignment can often be manually performed. The basic principle of a manual assignment is to select the path for each zonal (or subzonal) movement manually and then to record the movement in a tabular or graphical format. The movements are then summed together to produce the total assignment.

The recommended methodology was developed as part of NCHRP Report 187 (88). This methodology includes nine basic steps. The first six (6) produce link traffic volumes, while the last three are used to produce intersection turning movements. Although the NCHRP method pertains to a study area with a single internal zone, the logic used is easily extended to the analysis of more complex internal networks.

The nine steps have been modified into seven steps as described below. The analyst is referred to NCHRP Report 187 (88) for further details and an example. A case study application of this method to a windowed subarea is presented in Chapter 15.

1. **Prepare Map of Study Area**--A map is layed out showing the study area and external entry points. The network prepared as part of the window method will be sufficient for this purpose.

2. **Determine Trip Interchanges**--Trip interchanges are prepared for all internal and external trips. The trip table already prepared for the window will provide these data. Steps 2 and 5 from the NCHRP Report 187 are combined for the window method.

3. **Identify Highway Paths**--The "most reasonable" highway paths are identified for each zonal (or subzonal) interchange. Experienced judgment is primarily used to identify such paths because the methodology makes no specific provision for minimum time path selection. The results of zonal tree and/or select link analyses will often provide the analyst with adequate quantitative information with which to select the "most reasonable" paths without having to perform detailed impedance calculations.

4. **Load Network With Trips**--The interzonal trips are assigned to the network along the identified highway paths using an all-or-nothing logic. Steps 4 and 6 from the NCHRP Report 187 are combined for the window method. Two methods are described. The graphical method assigns trips directly onto the network map. Directional arrows and different colored pencils are often used to distinguish the trip movements. This method is applicable only for small networks with few zones.

The tabular method first depicts each zone and highway link in matrix form. If directional link volumes are being derived, each link direction should be entered separately into the matrix (e.g., for a link between nodes 201 and 202, list separate entries for 201-202 and for 202-201 by direction). Trips are listed in the correct cells of the matrix corresponding to the links (by direction if desired) traversed along each interzonal path. The trip totals for each link are subsequently added together (separately for each direction) to yield the total directional link volume.

The tabular method also becomes cumbersome as the network size increases, because a separate table must be prepared for each zone and for each direction of travel. For example, a network with 8 zones (or subzones) would require 32 tables (8 zones times 2 directions of movement) to describe all directional link movements. If only two-way link volumes are desired using a nondirectional trip table (e.g., use total trips between zones 1 and 2 rather than using separate trip movements from zone 1 to 2 and from zone 2 to 1), then the number of tables could be reduced to 8.

5. **Review Trip Interchanges**--The trip interchanges have been previously defined as part of Step 2. They will subsequently be used for developing turning movements at selected intersections.

6. **Number Intersections and Turning Movements**--Each intersection to be analyzed is designated with a unique number. The node number used in the windowed network is sufficient for this task. Next, all possible intersection turning movements are uniquely numbered. Usually for a 4-legged intersection, for instance, these movements total to 12. Therefore, if three 4-legged intersections will be analyzed, say nodes 101, 102 and 103, node 101 would have turning movements 1 through 12, node 102 would have 13 through 24, and node 103 would have 25 through 36.

7. **Load Turning Movements**--The final step is to assign the turning movements. This effort is similar to that used in Step 4 for the link assignment, except that now the trips are loaded onto the numbered turning movements. A tabular format is suggested to systematically account for turning movements associated with each trip interchange. As with the link assignment method, however, the manual applicability of this turning movement method is limited by the number of zones and the number of required intersection turns. For instance, the network with 8 zones (or subzones) with 5 required intersection analyses (each with 12 turns) would need 16 tables (8 zones times 2 directions of interzonal travel) of dimensions 8 X 60 (8 zones by 60 turning movements) to fully describe all directional movements. Nondirectional turning movements (i.e., total both directions) could be obtained with half of the number of tables and calculations.

Step 5--Refine Trip Assignment within Study Area

The resulting detailed network assignment should be refined in a similar manner to the system-level assignment. Procedures presented in Chapter 4 should be used to compare the subarea assignment with base year traffic counts, future land-use development patterns, and volume/capacity ratios. Subarea screenline checks and comparisons with select link or zonal tree data should be made where possible to ensure that the subarea traffic assignment is a reasonable representation of facility volumes and traffic patterns. Judgment must be exercised to make any final adjustments to the link and/or turning movement volumes based on these checks.

SPECIAL CONSIDERATION--APPLICABILITY OF SCREENLINE, SELECT LINK, AND ZONAL TREE PROCEDURES

Several procedures were presented in Chapter 5 to develop traffic data for alternative network assumptions. In particular, the effects of network changes such as changing alignments, addition or deletion of links, change in capacity, and construction of parallel roadways were examined. These techniques are applicable to the examination of more detailed networks as well.

The analysis procedures in Chapter 5 rely heavily on modifications to the screenline refinement method and to select link/zonal tree analyses developed in Chapter 4, plus the addition of the manual traffic assignment methodology described in this chapter. In situations where a detailed network to be examined is small in scale or contains well-defined parallel links (e.g., a grid collector street system), these procedures can often be employed without having to perform a focussing or windowing analysis.

As discussed previously in this chapter, select link and zonal tree analysis can be of asssitance in identifying traffic patterns within a study area. Often the analyst can visually inspect the detailed network and reassign trips to the new links using the knowledge gained from these data. Similarly, trips can be reassigned to newly formed subzones based on zonal tree data available from the original undivided zone. As the detailed network becomes more complex, the need increases to use more structural focussing or windowing methods to supplement these somewhat simplified analyses.

The modified screenline refinement procedure is most appropriate in this context for analyzing the traffic impacts of adding a more detailed network of parallel streets, such as collectors. Where traffic patterns can be easily identified in each direction within the study area, screenlines can be constructed which cross both the system-level facilities and the newly detailed streets. Trips are then reallocated across the screenline.

The system-level traffic assignment allocates all system trips to the facilities included in the original network. As the network is detailed, some of these trips will shift to the newly added roadways; however, the total screenline volume would remain relatively constant.

Using this assumption, volumes on the detailed network links can be estimated using the modified screenline procedure from Chapter 5. Two situations are possible, depending on the data available for the detailed network:

1. Situation A--Base Year Counts and Future Year Capacities are Available--In this case, volumes are apportioned across the screenline using both the CAPACITY (Col. 13) and BASECOUNT (Col. 14) adjustments in the worksheet (Fig. A-10) from Chapter 4. Base year volumes for the detailed roadways (which exist in the base year) can sometimes be derived from developer traffic studies, from turning movement counts at intersections of these roadways with major facilities, or from interpolation of counts between parallel facilities. The base year counts are important because many smaller streets (e.g., collectors) included in the detailed network carry significantly different (higher or lower) base year volumes than would be estimated by looking at relative capacities only. This is because most smaller streets serve such variable local traffic volumes, whose magnitudes are not usually related to the street capacity. These relative base year trends could be assumed to stay somewhat stable in future years. Future year capacities on the added facilities are readily estimated by comparing the street widths and/or number of lanes with those of similar facilities already in the network for which capacities had previously been determined.

2. Situation B--Future Year Capacities Only are Available--The screenline volume is apportioned solely on the basis of relative capacity. Therefore, in the screenline worksheet (Fig. A-10), the CAPACITY adjustment (Col. 13) would be the only one performed. The future year capacities on the added facilities can be estimated as described in situation A. This technique is most valid if many or all of the facilities added to the detailed network did not exist during the base year. In such cases, future volumes would tend to be apportioned across a screenline more on the basis of relative capacity than on the basis of relative base year counts. This is especially true as the level of congestion increases within the study area. In general, however, this technique will tend to overassign traffic to the detailed facilities and underassign traffic to the original system-level facilities. Therefore, the analyst must carefully check the screenline results and make final adjustments where necessary using judgment.

CHAPTER SEVEN
TRAFFIC DATA FOR DIFFERENT FORECAST YEARS

GENERAL

The specification of a traffic forecast for a particular target year can be a difficult task. The analyst is often confronted with the need to produce traffic data for a target year that is different from that used in any computer forecasts. Even in localities where forecasts are frequently updated, there are often requests for forecasts for other years. In such cases, traffic must be estimated using available data.

APPLICABILITY

The procedures that are presented enable traffic forecasts to be modified based upon several factors, including the following: availability of land-use projections, patterns of land-use and traffic growth, staging of highway and transit facilities, available capacity of the roadway system, historical traffic trends, timetable of land-use development, and availability of future year forecasts

These factors determine whether the traffic growth will be linear (uniform) or nonlinear (nonuniform). They also determine the extent to which full land-use buildout is being realized in the corridor or subarea.

BASIS FOR PROCEDURES

The basis for all of the procedures is that a traffic volume trend can be established by analyzing land-use patterns and/or historical traffic counts. This trend can occur either in a linear (i.e., straight-line), or nonlinear (i.e., curved or stepped-line) fashion.

Linear Growth

Linear growth is exemplified by a straight-line function (Figure A-52) in which the growth rate is constant over time.

Nonlinear Growth

Nonlinear growth can occur in several ways, as shown in Figure A-53. Basically, the growth rate changes over time. A common nonlinear function is the exponential curve, shown in Figures A-53(A) and A-53(B). In Figure A-53(A), the growth rate increases over time, while in Figure A-53(B), the growth rate will decrease. Figure A-53(C) depicts a situation in which growth occurs in a stepped manner, reflecting discrete rather than continuous growth. Other combinations of these nonlinear curves can be constructed to reflect local conditions.

The procedures in this chapter use linear or nonlinear curves to either interpolate growth between two years or to extrapolate growth from a single time frame. Figure A-54 depicts the suitability of each method given a set of available traffic forecasts. Extrapolation is a necessity for making forecasts beyond the last available forecast year assignment. Extrapolation can also be used to make forecasts for a short period of time (e.g., 5 years) past the base year.

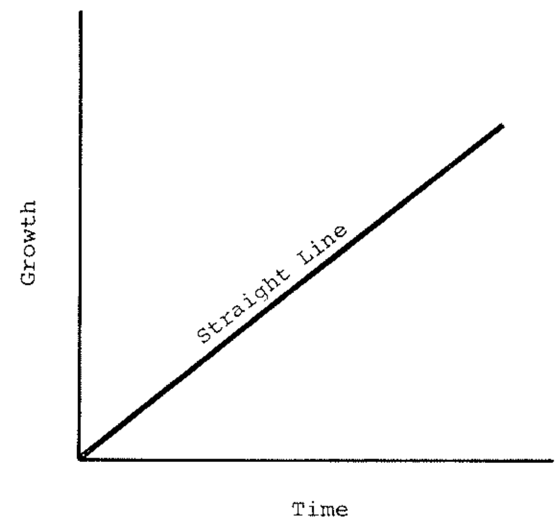
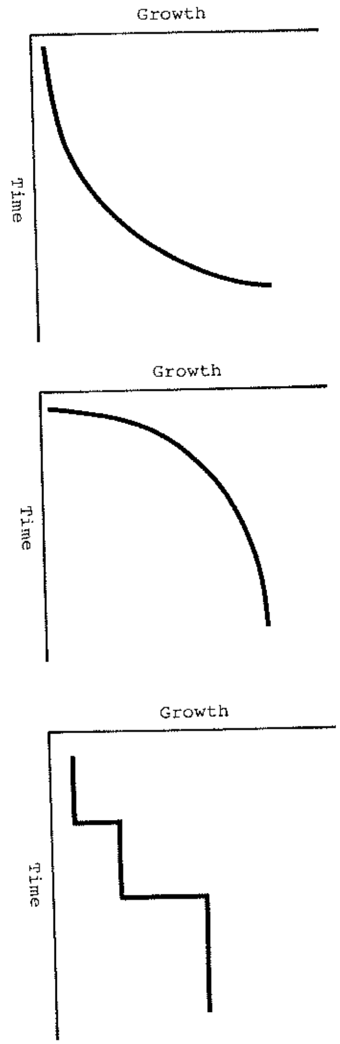


Figure A-52. Linear growth.



(A) Increasing Growth Rate

(B) Decreasing Growth Rate

(C) Stepped Growth

Figure A-53. Nonlinear growth.

I=Interpolation
E=Extrapolation

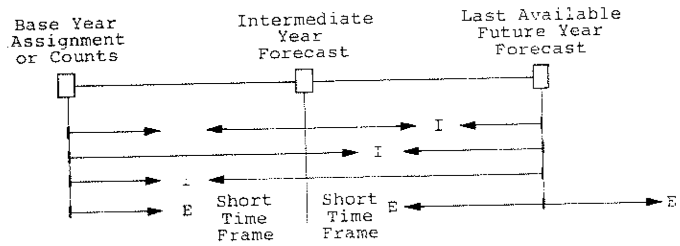


Figure A-54. Suitability of interpolation and extrapolation methods.

Interpolation requires two sets of known values between which data can be generated. Therefore, it is a suitable method for estimating traffic between two future year forecasts or between the base year and a future year assignment.

INTERPOLATION METHOD

Traffic forecasts for intermediate years can be estimated by interpolating between available computer assignments. The available information may include future year forecasts and/or base year assignments.

The advantage of the interpolation method is that the target year is situated between 2 years for which traffic data are available. Assuming that the available computer assignments are reasonable, then the target year data must fall somewhere in between. Therefore, a working range is established for the desired data.

Obviously, interpolation is most accurate in cases where the two computer assignments are close together in time. Care must be exercised in all cases to specify the network and land-use changes that have occurred between the two assignments so that a realistic growth curve can be developed.

Interpolation is extremely sensitive to the shape of the growth curve assumed to exist between two points in time. As depicted in Figures A-52 and A-53, there is a wide variety of growth curves that could be assumed based on knowledge of the study area. Therefore, the analyst must carefully select a curve that is most representative of the situation. Again, the margin for error is reduced if the time span between assignments is small.

Input Data Requirements

The following data are required for this method:

- Two computer assignments bracketing the target year. The two networks should be compatible with one another, taking into account specific link modifications, additions, or deletions.

• Data on land-use or demographic characteristic changes that are expected to occur betweeen the 2 years. Similar data specific to the target year are also required.

The analyst should also know the level of effort to be applied to this analysis.

Directions for Use

The interpolation method requires four (4) primary steps, as follows.

Step 1--Select Assignments to Bracket the Desired Year

Select two traffic assignments between which a reasonable interpolation can be made for the desired year. The following guidelines are offered:

1. Select forecasts that are as close together chronologically as possible to reduce the estimation error. For example, to estimate 1990 traffic, values could usually be interpolated more accurately between 1985 and 1995 forecasts than between 1985 and 2005 forecasts.

2. Select two different sets of computer assignments, if possible, in order to compare results. For instance, given assignments for 1982 (base year), 1987, 1995, and 2000, a 1990 forecast could be interpolated between the 1987 and 1995 forecasts. However, the 1982 base year assignment is likely to be more accurate than the 1987 forecast and could therefore provide a better base (along with the 1995 forecast) from which to interpolate 1990 values. The results of a 1982/1995 interpolation could then be compared with the results of interpolating between the 1987 and 1995 forecasts.

Step 2--Determine the Shape of the Growth Curve

The type of traffic growth expected to occur in the study area should be isolated. Using this knowledge, a growth curve can be constructed between the forecast years. Depending on the level of detail requested in the analysis, separate growth curves may be developed for areas around each roadway facility or for clusters of facilities. Table A-10 offers a listing of suggested factors to consider for various levels of analysis detail. As the required level of detail increases, so does the need to account for additional factors such as relationships between local traffic and specific land-use changes.

Low Level Analysis. Analyses dealing with sketch planning, or low levels of detail requirements, generally work with corridor-wide traffic values. For interpolation purposes, the primary factors would be the difference in forecast traffic volumes for the total corridor and the timing of any major highway or transit facilities. Such analyses should focus on linear interpolation of the corridor total traffic volume differences between the two forecast years.

Medium Level Analysis. For more detailed studies, emphasis is placed on establishing highway needs. The interpolation should use a linear or nonlinear function which considers zonal land-use growth trends and facility level forecast traffic volume differences for both local and through traffic. The shape of the nonlinear curve should correspond to the land-use growth trends.

High Level Analysis. A high level of traffic analysis would consist of detailed design studies for future facilities. The preferred procedure is a nonlinear interpolation of volumes taking into

Table A-10. Factors to consider for growth rate analysis.

	Level of Detail	Factors to Consider	Preferred Procedure(s)
(a)	Low level (e.g., sketch planning)	• Difference in corridor-level forecast traffic volumes (I) • Historical traffic trends (E) • Timing of highway/transit facilities	• Linear interpolation or extrapolation using corridor-level totals
(b)	Medium level (e.g., facility-level analysis of highway needs)	• The above (a), plus: • Zonal level land-use growth trends • Facility level forecast traffic volume differences (I) • Local vs. through traffic	• Linear or nonlinear interpolation or extrapolation using facility level totals
(c)	High level (e.g., design of facilities including intersection/interchange needs)	• The above (a,b), plus: • Specific tract level land-use patterns	• Nonlinear interpolation or extrapolation on specific facility segments

(I = Interpolation Only   E = Extrapolation Only) account specific proposals for land-use development in various tracts along the facility.

Land-use changes will form the major basis for selecting the shape of the curve. Table A-11 describes some basic considerations that can aid in developing an appropriate curve. The identified land-use trends are also related to typical growth curves shown in Figure A-55. Figure A-55 is described fully in Step 3.

Step 3--Calculate Interpolation Factor

Once the shape of the curve(s) has been selected, an interpolation factor, F, should be calculated. This factor may be derived from changes in land-use or socioeconomic data during the time frame or can be related to forecasted changes in traffic volumes or trip making. Figure A-55 presents several typical linear and nonlinear curves that represent types of continuous growth. The following symbols are used in Figure A-55:

F_i = interpolation factor; $(O < F_i < 1)$

N = time period (years) between computer assignments; and

n = time period (years) between target year and early year computer assignment.

To use these curves, the value n/N must first be calculated. For instance, if the target year is 1990 and the two computer assignments used are 1985 and 2000, N = 15 (2000 - 1985 = 15 years); n = 5 (1990 - 1985 = 5 years); and n/N = 0.333 (5/15). This point is located on the horizontal axis (n/N) of Figure A-55. A vertical line is drawn up until it intersects with the desired curve. In this example, if the decreasing nonlinear curve $(n/N)^{0.5}$ is chosen, a line is drawn to that point (Fig. A-55). A horizontal line is then drawn to the left until it intersects the vertical axis. This point is the interpolation factor F_i = 0.57. Typical values of (n/N) for various land-use trends are given in Table A-11.

A stepped curve is not shown since the magnitude of each "step" will vary in each situation. However, the analyst can use the same normalized scales as shown in Figure 55 to construct an appropriate stepped curve. As the number of "steps" increases within the time period, the stepped curve becomes similar to the continuous curves in Figure A-55. In all cases, the analyst may choose to combine these or other curves as desired to match local conditions.

Step 4--Perform Computations

Compute the target year traffic by interpolating between the computer assigned traffic volumes forecasted for the years on either side. The computations will vary according to the curve selected; however, the basic format will be the following:

Given:

V_T = volume in target year;

V_B = forecasted volume in year before target year;

V_A = forecasted volume in year after target year; and

F_i = interpolation factor (see Step 3).

Then:

if $V_A > V_B$: $(V_A - V_B) * F_i + V_B = V_T$ (A-21)

if $V_A < V_B$: $V_B - (V_A - V_B) * F_i = V_T$ (A-22)

Table A-11. Relationships between land use trends and traffic growth.

Growth Curve	Land-Use Trends
Linear	Constant land-use growth over time. More likely to occur in established, more densely developed areas. Often used for interpolating through traffic in slow-to-moderately growing regions. (n/N = 1.0 in Figure A-55)
Nonlinear - Increasing Growth Rate	Land-use growth will accelerate over time in a continuous fashion. Used for analyzing facilities in newly developing areas that will have a maximum amount of growth occurring in the latter years. (suggested n/N range 1.5 to 5.0)
Nonlinear - Decreasing Growth Rate	Land-use growth will decelerate over time in a continuous fashion. Typically used to analyze facilities in areas where development has peaked and is expected to decrease sharply in the short-term followed by a leveling off in growth. (suggested n/N range 0.2 to 0.5)
Nonlinear - Stepped	Land-use growth occurs in discrete groupings of development spaced at intervals throughout the time period. Typically used to analyze areas with staged land-use development occurring in clusters of intense development rather than in a continuous manner. Also used to forecast changes shortly after the opening of a major new or upgraded facility. (If there are more than 5 "steps" within the time period, consideration should be given to use of a nonlinear continuous curve.)

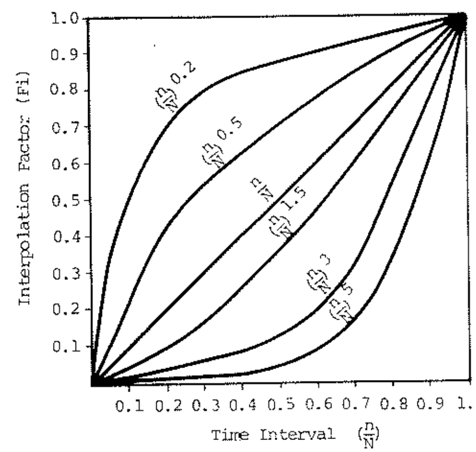


Figure A-55. Interpolation factor curves.

where

V_T must be > 0.

In Eq. A-21 the target year volume (V_T) is larger than the earlier volume (V_B) by a proportion of the amount by which the latter year volume (V_A) exceeds V_B. In Eq. A-22 V_T is smaller than V_B. The amount of the increase/decrease is totally dependent on the value of the interpolation factor, F_i, which has been defined by the type of curve selected by the analyst. The value F_i will range between 0 and 1.0.

The target year volume V_T may be determined on a corridor basis for sketch planning purposes or on a facility-by-facility basis for more detailed analyses. Once the target year volumes are computed, it is probable that further refinement of these volumes will be required using procedures described in Chapter 4.

In the previous example (Step 3), an interpolation factor $F_i = 0.57$ was obtained. Given the following assigned volumes, a target year volume can be calculated:

Given:

$V_B = 1{,}000$

$V_A = 2{,}200$

In this case:

$V_A > F_B$ therefore use Eq. A-21.

$V_T = (2{,}200 - 1{,}000) * 0.57 + 1{,}000 = 1{,}684$ Volume in Target Year

EXTRAPOLATION METHOD

The extrapolation method uses known or estimated growth trends to forecast traffic for a year situated either before or after an available computer assignment or base year count. It is most applicable in the following situations:

1. Traffic estimates for years beyond any available traffic forecast.
2. Traffic estimates for years within a short time frame from the base year.
3. Traffic estimates when only one adequate traffic forecast is available.

Extrapolation has the advantage that only one usable computer traffic assignment (or actual traffic counts if the base year is used) is required for analysis. It has the disadvantage that the analyst has no "bracket," or range of values between which the target year volumes should fall. As a result, it is feasible to extrapolate traffic volumes into the future which are higher than the highway system or land-use plan can accommodate. Extreme care must be taken if volumes are to be extrapolated past a reasonable timeframe, say, 5 years. This problem is minimized if realistic traffic growth curves are prepared.

Input Data Requirements

The following data are required for this method:

- One traffic assignment on either side of the target year. If the base year is used, actual traffic counts can be substituted for a traffic assignment.
- Data on land-use or demographic characteristic changes that are expected to occur between the target year and the year from which the volumes will be extrapolated.

The analyst should determine the level of detail that is expected for the analysis.

Directions for Use

The following four (4) basic steps are included in the method.

Step 1--Select Forecast

Select one traffic assignment (or counts) from which extrapolations can be made. When traffic estimates are desired beyond any available assignments, the latest and/or best available computer forecast should be selected wherever possible because extrapolations lose accuracy roughly in proportion to the length of time over which the estimates are to be made. For instance, given 1990 and 2000 traffic forecasts, an analyst wishing to estimate 2005 traffic would in most cases select the 2000 forecast as a base. Keep in mind that the later year forecast may not be very reliable, in which case the analyst may select a more reliable forecast from an earlier year. In the example above, one could extrapolate from 1990 to 2005 if the 2000 forecast were deemed to be unsatisfactory.

Extrapolating into the future from the base year is a common practice. This can be accomplished using either base year counts or a base year computer assignment. In most cases, using actual base year traffic counts will usually produce the most realistic estimate of travel in the study area. If later year computer forecasts are also available, target year traffic estimates can be made either by extrapolating from the base year or by interpolating between 2 years (see Fig. A-54). Extrapolation is usually most accurate over the short term (i.e., 1 to 5 years), while interpolation (see previous section) is suggested for later target years.

Occasionally suitable traffic forecasts are not available except for, say, the base year and/or a time 20 to 30 years distant. Extrapolation from the earliest suitable forecast will usually produce the best results in this situation.

The target year is usually later than the year from which the traffic will be extrapolated. In some cases, however, traffic can be extrapolated "backwards" to an earlier year. The unavailability of suitable forecasts may create a need for this reverse extrapolation rather than interpolation. For example, given 1990 and 2010 forecasts, traffic estimates for 2005 would most likely be made by interpolating between the 1990 and 2010 forecasts. However, if the 1990 forecast were found to be unsuitable, the analyst may need to extrapolate back from 2010 to 2005.

Step 2--Determine the Shape of the Growth Curve

A typical growth curve(s) should be determined for extrapolation purposes. The procedure to follow is the same as that described previously for interpolation. The complexity and number of the curves will vary according to the level of analysis to be performed. Table A-10 describes some of the factors and procedures to be considered.

Step 3--Calculate Extrapolation Factor

An extrapolation factor (F_e) should be calculated from the shape of the growth curve (Step 2) and from specific knowledge of trends in land-use, socioeconomic characteristics, and traffic counts. Because only one set of computer assignments (or base year counts) is used in the calculations, the extrapolation factor cannot be derived from changes between forecasted traffic volumes. The most common data used are historic or projected land-use or socioeconomic trends.

Land-use or socioeconomic (e.g., population, employment) changes are used as surrogates for changes in traffic volumes. These trends will have established the shape of the growth curve in Step 2 and can be used to calculate the extrapolation factor (F_e).

The extrapolation factor is usually derived by first determining an average annual growth rate. This growth rate can be approximated by looking at general traffic or land-use trends. However, it is preferable to calculate a more precise value by comparing data between two different years. For short term projections from the base year, historical traffic counts or land-use data can be used. For example, traffic counts for the time period 1975-1982 can establish a traffic growth trend for extrapolation of traffic from 1982 to 1985.

If land-use or other demographic data are available for the target year, these values can be compared with similar data from the year for which the computer assignment has been made. For instance, suppose that land-use projections have been made for the target year 1995, although no computer assignment is available. It is decided to extrapolate from a 1990 computer assignment for which compatible land-use data are also available. An annual growth rate for the 1990-1995 period can therefore be developed.

The annual traffic growth rate can be derived from either of the following equations.

$$g = (x/y)^{1/Z} - 1 \qquad \text{(A-23)}$$

$$g = e^{\left[\frac{(\ln(x) - \ln(y))}{Z}\right]} - 1 \qquad \text{(A-24)}$$

where:

- g = average annual growth rate;
- x = future (or base) year value (volume, land-use, population, etc.);
- y = earlier year value (volume, land-use, population, etc.);
- Z = number of years;
- e = exponential function; and
- $\ln$ = natural logarithm function.

Example:

- x = Future Year Population = 2,500
- y = Earlier Year Population = 1,000
- Z = 8 years

$g = (2{,}500/1{,}000)^{1/8} - 1 = 0.121$

or

$$g = e^{\left[\frac{(\ln 2{,}500 - \ln 1{,}000)}{8}\right]} - 1 = 0.121$$

Average Annual Population Growth rate = 12.1 percent

If possible, a separate growth rate should be calculated using various data trends (e.g., land use, population, employment) in order to determine which growth rate should be applied to the traffic volumes. In some cases, different growth rates can be applied to different groups of traffic (e.g., work trips, nonwork trips). This topic is discussed further later in this section. Once g is known, the growth rate can be extrapolated for any number of years given Eq. A-25:

$$F_e = (g + 1)^n \qquad \text{(A-25)}$$

where:

- F_e = extrapolation factor;
- g = annual growth rate; and
- n = number of years for extrapolation.

Using the same example, if the annual growth rate of 12.1 percent were assumed to hold into the future, the following extrapolation factor could be calculated for a 5-year period:

$$F_e = (0.121 + 1)^5 = (1.121)^5 = 1.77$$

indicating a 77 percent growth during the period. Note that F_e will always be greater than zero. If growth occurs, F_e will be greater than 1.0. A value of F_e of less than 1.0 indicates a decrease in traffic volume. Reverse extrapolation will generally yield a value of F_e less than 1.0.

Extrapolations of growth to years later than any available forecast will generally require knowledge of ultimate "build-out" land-use projections highway capacity. In many cases the extrapolation factor used in this case is an extension of the growth rate determined from interpolations between earlier forecasts.

For example, the analyst is given 1985 and 2000 forecasts with the task to estimate traffic for 2010. First, a growth curve can be established for the 1985-2000 timeframe using the interpolation method. This curve can then be extended or modified as desired to extrapolate from 2000 to 2010. Modifications would result by comparing land-use/socioeconomic estimates for the 2000-2010 and 1985-2000 time periods.

Step 4--Perform Computations

Compute the target year traffic by extrapolating from the selected computer traffic forecast (Step 1). The equation for extrapolation is as follows:

Given:

- V_T = volume in target year
- V_F = volume in selected forecast or base year
- F_e = extrapolation factor (from Step 3)

Then:

$$V_T = F_e * V_F \qquad \text{(A-26)}$$

where $V_T > 0$.

For example, using the previously determined extrapolation factor, F_e = 1.77, and given a computer forecasted link traffic volume of 1,250 vph, the following target year volume is derived:

$V_T = F_e * V_F = 1.77 * 1{,}250 = 2{,}213$ vph in the target year

V_T can be calculated on a corridor or facility level basis, depending on the level of analysis. It is probable that further refinement of these volumes will be required using procedures described in Chapter 4.

SPECIAL CONSIDERATIONS

Growth for Different Traffic Segments

The procedures described in this chapter permit different traffic segments to be separately analyzed. The most common segmentation is between "through" and "local" traffic. "Through" traffic is generated by land-uses external to the study area, while "local" traffic has originated or is destined to somewhere within the study area. Traditional traffic studies will separate these traffic strata in order to more accurately depict growth in traffic along a facility or corridor.

Typically the growth curves for through and local traffic are different. Therefore, assuming that the same growth curve for all traffic will likely result in inaccuracies while interpolating or extrapolating. Through traffic generally exhibits a more linear growth function, because the complexity of traffic origins and destinations tends to mask localized variations. Growth curves for local traffic, conversely, can vary widely, depending on the intensity and staging of development. This variation will be most evident at the tract or subzonal analysis level, with fewer differences at the zonal or subregional level. The level of analysis detail required will dictate the extent to which growth curves should be modified for local traffic.

Other traffic strata that could be separated include trip modes (e.g., single occupant auto, multi occupant auto, transit, etc.), trip purpose (e.g., work, nonwork (shopping, school, medical), and trip demographics (e.g., income, family size, race, etc.). Generally, these strata will not be accurately known for future years or the growth trends may not be separately defined. Therefore, this level of stratification will generally not produce significantly improved traffic estimates for intermediate or extended years.

Consideration of Land Use and Roadway Capacity

One of the primary dangers of extrapolating into the future is the possibility that the resulting traffic estimates will exceed the planned roadway capacity or that the traffic volumes will not be consistent with the ultimate land-use plan for the study area. The growth curve selected should reflect these constraints during all study years. This topic is presented by Memmott and Buffington (66).

In situations where the full "build-out" development level is known, this land-use value should be closely compared with both the base year land-use and the projected land-use for the target year (if known). In many cases the latest predicted land-use values will be for the forecast year assignment from which the traffic for the target year is being extrapolated. For instance, a target 2005 traffic estimate may be extrapolated from a 1995 forecast which utilized specific assumed land-uses. The land-use intensity in 1995 should be compared with the full build-out development to ensure that the growth extrapolated to 2005 does not exceed that build-out limit. As a rule of thumb, a target year growth that is up to 10 to 15 percent higher than the build-out limit has been considered reasonable by many agencies, given the probable errors in estimation. However, major differences should be examined and the extrapolated growth adjusted if necessary.

Another related factor to consider is the expected capacity of the highway system in the study area. To a large extent future traffic growth will be limited by available roadway capacity. This can occur in the short term (i.e., 5 to 10 years) and in the long term (i.e., 10 to 30 years). Any interpolation or extrapolation of traffic to alternate target years must specifically acknowledge this capacity. If roadway capacity is exceeded, a slowdown in growth can be expected within the study area. It is unrealistic to expect traffic growth to completely stop; rather, a slowdown will occur.

In order to check the capacity constraint, the following factors should be reviewed: total predicted traffic volume (ADT, peak hour), total available roadway (or corridor) capacity, calculated volume/capacity (V/C) ratios, and expected roadway improvements. If traffic is extrapolated, the foregoing factors should be reviewed both for the forecast (or base) year for which assignments are available and for the target year assuming no capacity constraint. If traffic is interpolated, these factors should be examined for the assignments on either side of the target year. If capacity has changed between these two years (e.g., new or improved roadway added), the capacity assumed for the target year must be closely examined so that it matches the type of traffic growth that is expected.

If the anticipated traffic growth exceeds capacity, the growth curve should be adjusted. As shown in Figure A-56, this adjustment would typically involve a leveling off of the growth curve to represent a reasonable fraction of the previously assumed rate. For linear curves (Fig. A-56(a)), the slope of the curve would be reduced until such time that additional roadway capacity were added. At this point, the slope may increase dramatically until a stable growth rate is achieved. For the nonlinear with increasing growth rate curve (Fig. A-56(b)), the high rate of growth may be significantly reduced to reflect the capacity constraint. After the constraint is removed, growth is likely to increase at a slightly less reduced rate than originally. The nonlinear curve with a decreasing growth rate (Fig. A-56(c)) reflects what typically happens as capacity is slowly reached. The rate of growth will slowly decrease. If capacity is reached during an interim year, this curve too may be altered. The stepped curve (Fig. A-56(d)) is ideally suited to accommodate severe capacity constraints. The growth stops at various points until capacity is increased, at which point sudden growth boosts occur. This type of haphazard growth may be found in many newly developing areas that experience periodic capacity crises.

Turning Movements

The interpolation or extrapolation of link traffic volumes can occur in a logical fashion using appropriate growth curves and factors. Turning movements, on the other hand, can change dramatically between time periods. Turning movements are primarily influenced by local traffic changes in that they are dependent on the magnitude and location of specific development parcels. Turning movements are less sensitive to changes in through traffic.

Turning movements should not merely be factored up or down in the same manner as are the link volumes. The analyst should review the link volume growth in connection with the location and cause of this growth. For instance, a link volume may increase by 20 percent, yet that growth may not be equally proportioned to all turning movements. If the growth is primarily due to new residential units, the turning movements oriented to those residential areas should be significantly increased, while intersection through traffic would increase at a lower rate.

The turning movement procedures and examples presented in Chapter 8 take into account changes in link volumes relative to other link volumes approaching an intersection. Using these procedures the turning movements will change to reflect the differential traffic growth in the area. In all cases, the analyst must closely check the resulting turning movements to determine their reasonableness relative to base year counts or to turning movements forecasted for other years.

Wide Zonal Variations

For detailed studies in study areas that will exhibit wide variations in zonal growth, simple interpolation or extrapolation of data may not be satisfactory. In such cases, the following technique adapted from those used by the Maricopa Association of Governments (61) could be of assistance:

1. Divide the study area into discrete components according to expected growth. For instance, given the zone structure shown in Figure A-57, it was determined that zones 1, 2, 5 and 6; 3, 7, and 8; and 4, 9 and 10 could be clustered together according to expected growth.

2. Analyze base year and available forecasted traffic volumes associated with each zone and zone cluster to determine trends. The forecasted volume trends for each cluster should correspond with the expected land-use or socioeconomic growth for those zones. If land-use/socioeconomic data are available for the target year, these trends should also be analyzed. The average annual growth rate, described in the extrapolation method and in Chapter 3, is a convenient unit for comparison of various growth trends. If these trends match, the interpolation/extrapolation of volumes to the target year can be conducted for each facility in the vicinity of a particular zonal cluster. Judgment must be used in many cases where facilities pass through two or more clusters.

3. Conduct "select link" computer assignments for base and future year assignments to check for changes in travel patterns. Select link assignments are described in Chapter 4. Computer zonal trees can also be examined to detect changes in origin-destination travel patterns for study area zones.

4. Check historical traffic and land-use/socioeconomic data to ensure that selected growth rates and travel patterns are reasonable. A particular situation in which this analysis would be useful is in producing traffic data at interim termination points for freeway construction. A review of changes that occurred at other freeway termination points along the same or similar roadways can be very useful in checking the reasonableness of the traffic estimates.

5. Using the appropriate interpolation or extrapolation procedure for each zone or zone cluster, assign traffic onto the target year network. In most cases, local and through traffic will be segmented. The resulting link assignment is refined as needed to account for changes at specific zones. Turning movements are separately analyzed using procedures described in the previous section and in Chapter 8.

This technique allows traffic forecasts to be transferred to other target years with a minimal loss of precision. It enables small study areas to be analyzed in detail to account for differential changes in zonal land-uses or socioeconomic characteristics. Chapter 5 provides additional information regarding studies of small area detailed highway networks. For less detailed analyses, one or two growth curves should be sufficient to factor the traffic in the entire study area.

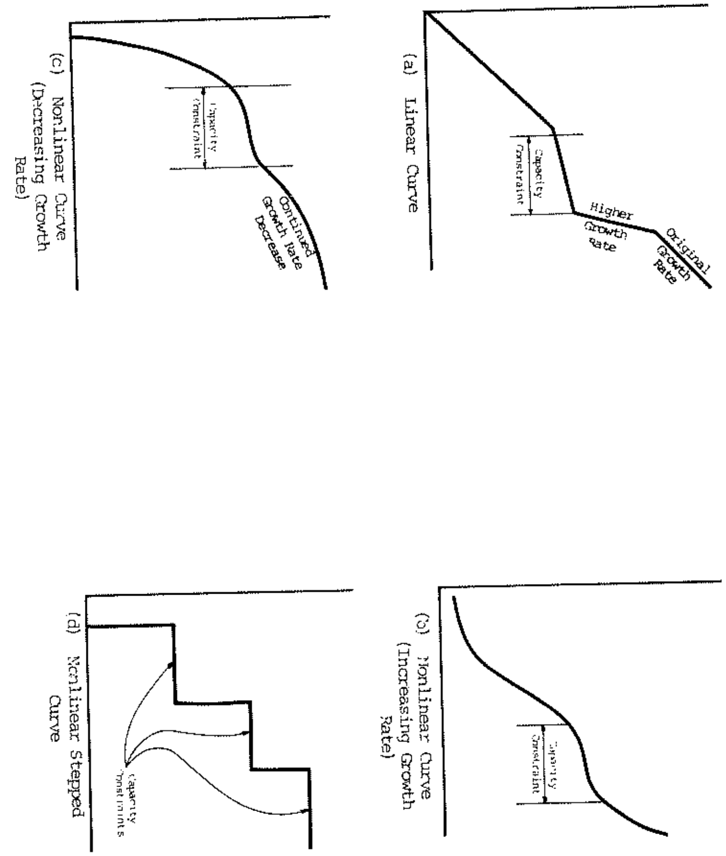


Figure A-56. Effects of capacity constraints on traffic growth.

CHAPTER EIGHT
TURNING MOVEMENT PROCEDURES

GENERAL

Turning movement data are often required for the planning and design of highway intersections and interchanges. Computerized traffic assignments rarely provide turning movement forecasts that can be directly used for these purposes, resulting in a need for significant refinement. Often the system-level forecasts do not provide any turning movement data. Therefore, procedures are presented that enable the analyst to develop these data from various sources and for various uses. These procedures can be used independently or to supplement the link refinement and detailing procedures documented in Chapters 4 through 7.

The appropriate procedure to use is dependent on several factors, including the availability or suitability of the following:

1. Future year turning volume forecasts.
2. Directional or nondirectional volume forecasts. Directional turning volumes are specific to each direction of travel. Nondirectional turning volumes represent two-way volumes passing between adjacent links. Figure A-58 illustrates these differences using a common four-way intersection.
3. Actual base year turning movement counts.
4. Base year turning movement assignments.
5. Desired time period (e.g., peak hour, 24-hour).
6. Number of intersection approaches.

In some cases the analyst must combine two or more procedures in order to arrive at a reasonable turning movement estimate.

Three sets of procedures are presented in this chapter. They are the following:

1. Factoring Procedures--includes use of either Ratio Method or Difference Method.
2. Iterative Procedures--includes separate Directional and Nondirectional Volume Methods.
3. "T" Intersection Procedures--includes separate Directional and Nondirectional Turning Movement Methods.

The primary feature of the factoring procedures is their computational simplicity. By the same token, their simplicity means that several potentially key factors have not been considered. The procedures also require actual base year turning movement counts as well as a base year turning movement assignment, thus limiting their applicability.

The iterative procedures are significantly different, depending on whether directional or nondirectional turning volumes are used as input. The directional volume method adjusts future year turning movements to match as closely as possible a predetermined estimate of turning percentages. It can be applied whether or not base year turning movements are known. The method can become time-consuming if a significant number of iteration calculations are required. The nondirectional volume method requires considerably more judgment on the part of the analyst. Typically these turns are derived only from a knowledge of nondirectional approach link volumes and an estimate of the total turn percentage at the intersection. Therefore, the task is to produce turning movements that appear to be reasonable based on the given approach volumes and the distribution of adjacent land uses. The results are not intended to be used for design purposes.

The "T" intersection procedures were developed to address the uniqueness of an intersection having only three approaches. A unique solution can be obtained for nondirectional turning volumes

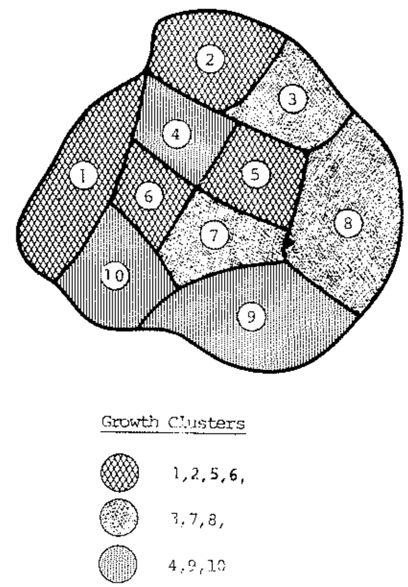


Figure A-57. Zones clustered by growth.

if the approach volumes are known. Directional turning volumes can also be directly computed if the directional approach volumes and one turning movement are known. Otherwise, reasonable estimates of directional turning volumes can be made from nondirectional volumes using some basic relationships characteristic of a "T" intersection.

These procedures are described in the following sections of this chapter.

FACTORING PROCEDURES

Future year turning movement forecasts are frequently based on the relationships between base year assignments and actual base year counts. The assumption used is that the discrepancy between a base year count and a base year assignment is likely to be of the same magnitude in the future year. Given this assumption, the future year turning movements can be modified by comparing the relative ratios or differences between base year link or turning volumes. The procedures are equally valid for producing directional or nondirectional turning movements.

Input Data Requirements

The following directional or nondirectional data are required for both the ratio or difference procedures:

1. Future year turning movement forecast.
2. Base year turning movement assignment.
3. Base year turning movement counts.

The first two data items are obtained from computer assignments, while the actual base year data (item 3) must be obtained from existing counting programs or from field studies.

Directions for Use

Similar computations are performed for the ratio and difference methods, as follows.

Ratio Method

Each turning movement in the future year assignment is factored by the ratio of the base year actual traffic count to the base year assignment.

$$V_{ri} = F_i * (B_{ci}/B_{ai}) \qquad \text{(A-27)}$$

where:

V_{ri} = ratio adjusted future year volume turning movement i;
F_i = future year forecasted volume for turning movement i;
B_{ci} = base year traffic count for turning movement i; and
B_{ai} = base year assigned volume for turning movement i.

Each turning volume is adjusted separately and then summed to produce an adjusted total approach volume.

For example, the numbered intersection as shown in Figure A-59 is provided.

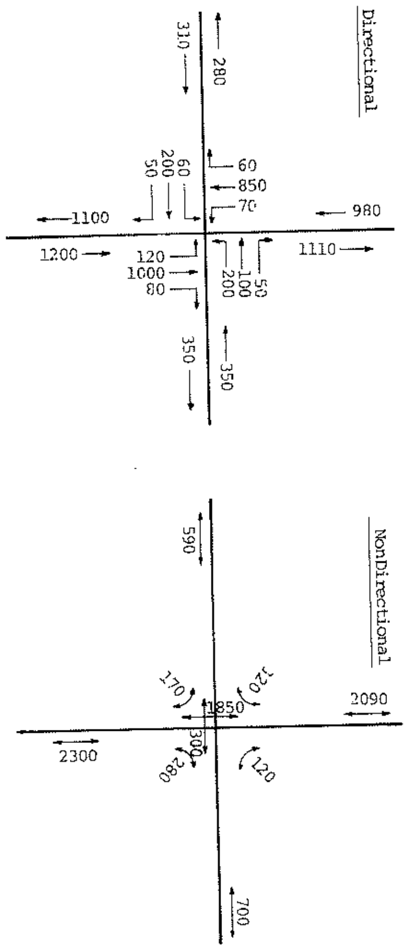


Figure A-58. Directional and nondirectional turning volumes.

Assume the following data for turning movement 3:

F_3 = 500

B_{c3} = 200

B_{a3} = 260

Then:

V_{r3} = 500* (200/260) = 385

Similar calculations would be performed for the other 11 turning movements.

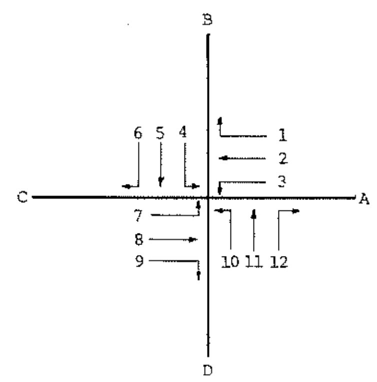


Figure A-59. Example intersection turning movements.

Difference Method

Each turning movement in the future year assignment is factored by the difference between the base year actual traffic count and the base year assignment.

$$V_{di} = F_i + (B_{ci} + B_{ai}) \qquad \text{(A-28)}$$

Where:

V_{di} = difference adjusted future year volume for turning movement i;

F_i = future year forecasted volume for turning movement i;

B_{ci} = base year traffic count for turning movement i; and

B_{ai} = base year assigned volume fo turning movement i.

Each turning volume is adjusted separately and then summed to produce an adjusted total approach volume.

Using the same example from above, the following calculations would occur for turning movement 3 (see Fig. A-59):

V_{d3} = 500 + (200-260) = 440

Similar calculations would be performed for the other 11 turning movements.

Both the ratio and the difference methods must be carefully applied to avoid extreme values. In the ratio method, if the base year count is significantly higher or lower than the base year assignment, the adjusted future year volume may be unrealistically high or low. Similarly, in the difference method, extreme discrepancies during the base year can significantly alter the future year volume. Negative values can also occur, which is a disadvantage of the difference method.

Combined Method

The two methods can be combined using a procedure similar to that developed for link volumes by the New York State Department of Transportation (77). The results of the ratio and difference methods are simply averaged to produce the final future adjusted turning volume.

$$(V_{ri} + V_{di})/2 = V_{fi} \qquad \text{(A-29)}$$

where:

V_{ri} = ratio adjusted future year volume for turning movement i;

V_{di} = difference adjusted future year volume for turning movement i; and

V_{fi} = final averaged future year volume for turning movement i.

Using the results from the ratio and difference adjustments, the final averaged future year volume for turning movement 3 would be:

V_{fi} = (385 + 440)/2 = 413

This averaging method tends to reduce the extremes experienced by the individual methods. Judgment must still be used, however, to assess whether the resulting turning volumes are realistic. In particular, this method may produce revised future year approach volumes that are significantly different from the future year volumes previously forecast. If desired, the turning movements can be further adjusted using the iterative method described in the next section. The iterative method is most useful when the analyst wishes to retain a specified future year link volume on each intersection approach.

Special Consideration—Lack of Base Year Turning Volumes

The base year volumes B_{ci} and B_{ai} should preferably represent the same turning movement i as that represented by the future volume F_i. Therefore, F_i would be adjusted based on the relative ratio between the actual and assigned base year volume for the specific turning movement i. If base year turning volumes are not available, however, approach link volumes may be substituted for B_{ci} and B_{ai} in the ratio method only. This substitution will result in each turning movement on an approach being adjusted by the same ratio. Obviously, this technique will not produce an adjustment that is as specific as that derived by comparing individual base year turning movements. However, it will account for major deviations between the actual and assigned volumes.

For example, on approach B to the intersection shown in Figure A-59, assume the following information:

Base year actual approach volume (link) = 500

Base year assigned approach volume (link) = 700

The future year forecasted volumes (F_i) for turning movements 4, 5, and 6 on approach B would then be adjusted as follows:

$V_{ri} = F_i * (500/700) = F_i * 0.71$ for i = 4, 5, 6

The adjustment would be different for each intersection approach.

Note that the difference method cannot be used with base year link volumes because the total difference between actual and assigned link volumes cannot be added (or subtracted) to each individual turning movement.

ITERATIVE PROCEDURES

This section contains procedures for producing either directional or non-directional turning volumes using an iterative approach. Iteration involves applying a technique repeatedly until the results converge to an acceptable result. Both procedures derive future year turning movements from prespecified link volumes and an initial estimate of turning percentages. Iteration is required to balance the volume of traffic entering and leaving the intersection. Therefore, the number of iterations necessary to produce an acceptable set of turning volumes is dependent on the ability of the analyst to make reasonable a priori estimates of turning percentages. These estimates can be made by analyzing base year counts at the same intersection, by reviewing turning movements at similar intersections, or by examining adjacent land use intensity and distribution.

Directional Volume Method

Starting with user-estimated turning percentages, the directional volume method proceeds through an iterative computational technique to produce a final set of future year turning volumes. The computations involve alternately balancing the rows (inflows) and the columns (outflows) of a turning movement matrix until an acceptable convergence is obtained. Future year link volumes are fixed using this method and the turning movements are adjusted to match.

This procedure is most applicable in cases where the future year turning volume forecasts are not expected to be radically different from either the base year conditions or from the initial user-supplied estimates of turning percentages. If large differences occur, several iterations may be required to reach convergence to the prespecified future year link volumes. Normally, however, six to ten iterations requiring one or two person-hours should suffice.

Basis for Development

The directional volume method is based on a basic iteration technique developed by Furness (30) and modified for intersection flows by Mekky (64). A similar but more complex formulation developed by Bacharach (7) involves input-output changes using a biproportional matrix method.

Apart from these iterative techniques, there also exists a noniterative method for generating intersection directional turning movements. This method, developed by Norman et al. (43, 73), may be substituted for the iterative approach in cases where the analyst has good initial estimates of the future year turning movements. However, the mathematical complexity of the formulation, plus the probability that negative numbers may result, indicates that the iterative method described in this chapter will produce the most consistent results in a reasonable time frame.

Input Data Requirements

The following input data are required:

- Future year directional link volumes.
- Either: Base year actual or assigned directional turning movements.
  Initial estimate of future year directional turning percentages.

The future year link volumes are obtained directly from the computer forecasts or from the results of a link refinement or detailing procedure (see Chapters 4 through 7). The base year data would preferably be actual turning movement counts, but turning data from a base year assignment could also be used. In lieu of base year data, the analyst must make an initial estimate of future year turning percentages based on an examination of adjacent land uses or the turning movements at similar intersections.

Directions for Use

The directional volume method consists of five steps, as diagrammed in Figure A-60. The following notations are used in the calculations:

- n = number of links emanating from the intersection;
- O_{ib} = base year (b) inflow to the intersection on link i (i=1...R);
- O_{if} = future year (f) inflow to the intersection on link i (i=1...n);
- D_{jb} = base year (b) outflow from the intersection on link j (j=1...n);
- D_{jf} = future year (f) outflow from the intersection on link j (j=1...n);
- T_{ijb} = base year (b) traffic flow entering through link i and leaving through link j;
- T_{ijf} = future year (f) traffic flow entering through link i and leaving through link j;
- P_{ijf} = future year (f) estimated percentage (expressed in decimal form) of traffic flow from link i to link j (use in place of T_{ijb}); and
- $*$ = represents adjusted values in each iteration.

These notations can be illustrated using the example intersection diagrammed in Figure A-61. In this case, the number of links is 4 (n=4). The base year and future year inflows O_{ib} and O_{if} are shown for each link, as are the corresponding outflows D_{jb} and D_{jf}. The base year and future year turning movements T_{ijb} and T_{ijf} are diagrammed for each of the 12 movements.

If the base year turns T_{ijb} were not known, estimated future year turn percentages (P_{ijf}) could be substituted, as illustrated for link 1. The P_{ijf} must total to 1.00 (or 100%) for each approach. Therefore, $P_{12f} + P_{13f} + P_{14f} = 1.00$ and so forth for each approach link.

The computational steps are described below, followed by an example.

Step 1--Construct Initial Turning Movement Matrix. The first step involves constructing an initial matrix of turning movements to be used in the iterations. The construction varies depending on whether or not base year turning volumes are available. In these and subsequent matrices, the diagonal elements (i=j) will always be equal to zero unless U-turns are permitted.

Step 1A--Base Year Turning Volumes Known. First construct a turning movement matrix of base year turning volumes (T_{ijb}). Next, insert the row and column totals. The row totals should represent inflows (O_{ib}) and the column totals should represent outflows (O_{jb}). This is shown below.

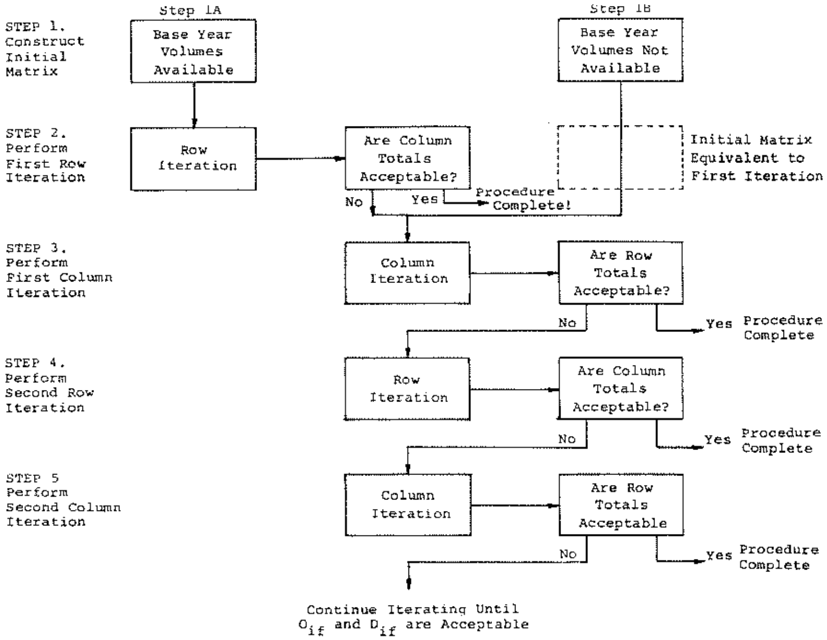


Figure A-60. Iterative procedure to compute directional turning volumes.

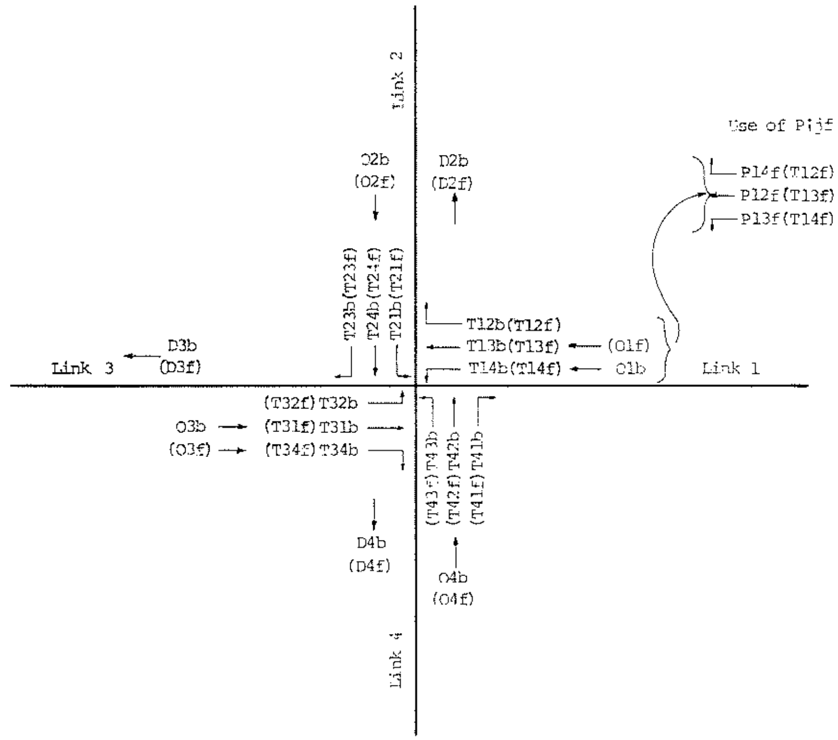


Figure A-61. Intersection notation used for directional iterative procedure.

	Columns -Out flows- Dib(Dif)				
Rows -Inflows- (Oi)Oib	x	x	x	x	Base Year Turn Movements
	x	Tijb		x	
	x			x	
	x	x	x	x	

The sum of the T_{ijb} across each row and down each column should be equal to the O_{ib} and D_{jb} respectively.

Finally, display the future year link volume inflows (O_{if}) and outflows (D_{jf}) in parentheses adjacent to the corresponding base year link volumes. Proceed to Step 2.

If base year turning volumes are not known, omit this step and proceed to Step 1B.

Step 1B--Base Year Turning Volumes Not Known. Construct an initial future year turning movement matrix using estimated turn percentages. The row totals should represent the future year assigned inflows (O_{if}). The column totals should represent outflows (D_{jf}). The individual turning movements (i.e., cells of the matrix) are calculated as follows:

$$T_{ijf}^* = O_{if} * P_{ijf} \qquad \text{(A-30)}$$

Note that the P_{ijf} are applied to the inflows (O_{if}), not to the outflows (D_{jf}). This computation is repeated for each cell of the matrix.

To aid in this computation, it is often helpful to construct a separate matrix of estimated turning percentages, as shown below. Matrix multiplication can then be performed.

	Column Totals Not Usually Equal to 1.00			
1.00	x	x	x	x
1.00	x	Pijf		x
1.00	x			x
1.00	x	x	x	x

The row totals must equal 1.00. Except by coincidence, the column totals of P_{ijf} will not equal 1.00.

At this point the inflows (O_{if}) are equal to the desired future year assigned link volumes. The adjusted outflows (D_{jf}*), on the other hand, must be calculated as the sum of all traffic volumes (T_{ijf}*) for the appropriate column (j). For instance, for link 2, D_{jf} * would equal the sum of all traffic turning onto Link 2 (e.g., $T_{12f}^* + T_{32f}^* + T_{42f}^* = D_{2f}^*$ for a four-legged intersection with no U-turns).

$$D_{jf}^* = \sum_{i=1}^{N} T_{ijf}^* \qquad \text{(A-31)}$$

Except in rare cases, this value of D_{jf}* will not equal the desired future year outflow D_{jf}. Therefore, iterations will be required to enable the D_{jf}* to converge onto the desired D_{jf} value. If the difference between these values is acceptable to the analyst, the procedure is complete. Typically, a difference of ± 10 percent is considered to be acceptable. The matrix at this stage is shown below.

	Djf*			
Oif	x	x	x	x
	x	Tijf*		x
	x			x
	x	x	x	x

Step 1B actually represents the first row iteration of the procedure, although derived in a slightly different fashion from the case where base year volumes are known. Therefore, if further iterations are required, the analyst should now skip to Step 3. These relationships are shown in Figure A-60.

Step 2--Perform the First Row Iteration. Perform this step only after Step 1A. In the matrix replace the base year inflows (O_{ib}) with the future year inflows (O_{if}). Then adjust each individual turning movement according to the following:

$$T_{ijf}^* = (O_{if}/O_{ib}) \times T_{ijb} \qquad \text{(A-32)}$$

where T_{ijf}* is the adjusted future volume for this iteration.

Construct a new matrix consisting of the T_{ijf*} and O_{if}. Now calculate the new D_{jf}* by summing the T_{ijf}* in each column j.

$$D_{jf}^* = \sum_{i=1}^{N} T_{ijf}^* \qquad \text{(A-33)}$$

The matrix at this stage is shown below.

	Djf*			
Oif	x	x	x	x
	x	Tijf*		x
	x			x
	x	x	x	x

The D_{jf}* should be compared with the desired D_{jf} from Step 1A. If the difference between these values is acceptable to the analyst, then the procedure is complete. Typically, a difference of ± 10 percent is considered to be acceptable. If a larger discrepancy is apparent, then a further iteration(s) is required.

Step 3--Perform the First Column Iteration. This step is performed on the adjusted turning movement matrix from Steps 2 or 1B. Replace the outflows (D_{jf*}) with the original D_{jf}. Adjust each individual movement according to the following:

$$T_{ijf}{*}NEW = (D_{jf}/D_{jf}{*})\, T_{ijf}{*}OLD \qquad \text{(A-34)}$$

where:

$T_{ijf}{*}OLD$ = $T_{ijf}{*}$ value in the matrix developed in Steps 2 or 1B; and
$T_{ijf}{*}NEW$ = Adjusted T_{ijf} after column iteration.

In subsequent iterations, $T_{ijf}{*}NEW$ becomes $T_{ijf}{*}OLD$ and so forth.

Construct a new matrix consisting of the $T_{ijf}{*}NEW$ and D_{jf}. Calculate the adjusted $O_{if}{*}$ by summing the $T_{ijf}{*}NEW$ in each row.

$$O_{if}{*} = \sum_{j=1}^{N} T_{ijf}{*}NEW \qquad \text{(A-35)}$$

The $O_{if}{*}$ should be compared with the original O_{if}. If the difference between these values is acceptable to the analyst, the procedure is complete. Typically, a value of $\pm$ 10 percent is considered to be acceptable. If a larger discrepancy is apparent, continue with a further iteration(s).

Step 4--Repeat Row Iteration. If needed, repeat the Step 2 procedure for row iterations. Calculate new values for $T_{ijf}{*}NEW$ and $D_{jf}{*}$. Compare $D_{jf}{*}$ with D_{jf}.

Step 5--Repeat Column Iteration. If needed after Step 4, repeat the Step 3 column iteration procedure. Calculate new values for $T_{ijf}{*}NEW$ and $O_{if}{*}$. Compare $O_{if}{*}$ with O_{if}.

The row and column iterations should be continued until acceptable values of $O_{if}{*}$ and $D_{jf}{*}$ are obtained. The $T_{ijf}{*}$ values in the final iteration matrix will represent the final adjusted directional turning and through movements. The $T_{ijf}{*}$ should be closely reviewed for reasonableness before using them in subsequent planning and design studies.

Example

Step 1A A four-link intersection has base year turning movements and future year link volumes as illustrated in Figure A-62 and displayed in matrix form in Figure A-63. For this example, Step 1B is not used, and the analysis moves to Step 2.

Step 2: First Row Iteration (Fig. A-64)

Step 3: First Column Iteration (Fig. A-65)

In this example, the differences in row totals are within 5 percent after the second iteration. If this difference is acceptable, select the $T_{ijf}{*}NEW$ from Step 3 as the final turning movement matrix, and subsequent iterations will not be required.

For comparison, after six iterations, the results in Figure A-66 could be obtained. Therefore, the additional iterations have reduced the differences further still.

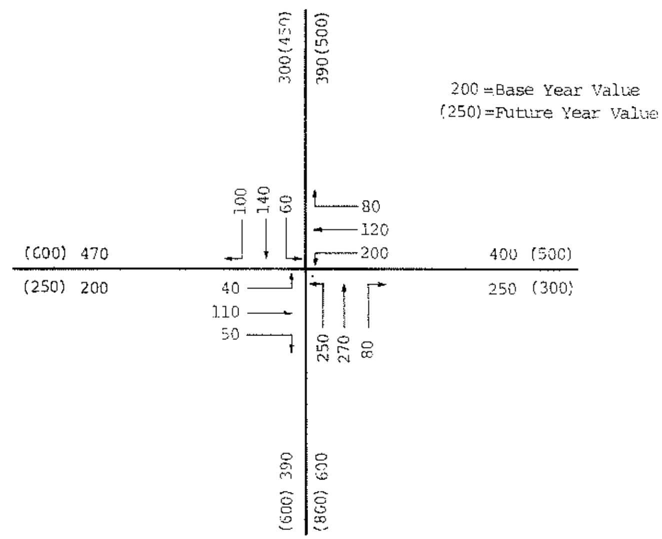


Figure A-62. Example of directional intersection volumes.

	(300) 250	(500) 390	(600) 470	(600) 390	(D_{jf}) D_{jb} Out-flows
(500) 400	0	80	120	200	T_{ijb}
(450) 300	60	0	100	140	
(250) 200	110	40	0	50	
(800) 600	80	270	250	0	
(O_{if}) O_{ib} Inflows					

Figure A-63. Intersection volumes displayed in matrix format.

O_{if} \ D^*_{jf}	335	510	633	522
500	0	100	150	250
450	90	0	150	210
250	138	50	0	62
800	107	360	333	0

$$D^*_{jf} = \sum_{i=1}^{4} T^*_{ijf} \qquad T^*_{ijf} = \frac{O_{if}}{O_{ib}} \times T_{ijf}$$

Compare	D^*_{jf} ≈	D_{jf}	Δ
j = 1	335	300	+12 %
j = 2	510	500	+ 2 %
j = 3	633	600	+ 6 %
j = 4	522	600	- 13%
Total	2000	2000 ✓	

Figure A-64. First row iteration.

O^*_{if} \ D_{jf}	300	500	600	600
527	0	98	142	287
464	81	0	142	241
244	123	49	0	72
765	96	353	316	0

$$T^{*NEW}_{ijf} = \frac{D_{jf}}{D^*_{jf}} \times T^{*OLD}_{ijf}$$

Compare	O^*_{if} ≈	O_{if}	Δ
i = 1	527	500	+5 %
i = 2	464	450	+3 %
i = 3	244	250	- 2 %
i = 4	765	800	-4 %
	2000	2000 ✓	

Figure A-65. First column iteration.

O^*_{if} \ D_{jf}	300	500	600	600
502	0	89	132	281
453	76	0	135	242
250	124	49	0	77
795	100	362	333	0

$$T^{*NEW}_{ijf} = \frac{D_{jf}}{D^*_{jf}} \times T^{*OLD}_{ijf}$$

Compare	O^*_{if} ≈	O_{if}	Δ
i = 1	502	500	0 %
i = 2	453	450	+1 %
i = 3	250	250	0 %
i = 4	795	800	-1 %
	2000	2000 ✓	

Figure A-66. Intersection volumes after 6 iterations.

Nondirectional Volume Method

The nondirectional volume method produces two-way turning volumes at an intersection given two-way link volumes and an estimate of the total vehicle turning percentage. The basic assumption used is that the volume of traffic on a given approach of an intersection is a surrogate for land-use attractions and production downstream. Turning movements at an intersection should therefore be some function of the attractions and productions each direction of travel offers.

The method provides a five-step sequence and may have to be performed iteratively to achieve a balanced distribution of turns and through movements. The number of iterations required will vary between intersections, depending on the number of intersection approaches and the volume of turns. Usually three to four iterations requiring one to three person-hours will be sufficient.

Basis for Development

This nondirectional volume method is adapted from an unpublished technique developed by Marshment at the Middle Rio Grande Council of Governments (63). It originally evolved through combining empirical analyses of actual intersection operations with local knowledge of characteristics specific to the intersection being studied. As such, there is no theoretical basis for the method, and there is no uniqiue solution. Rather the method produces a reasonable turning movement scenario using the assumptions described above.

Because of its sketch-planning nature, the method relies heavily on the judgment of the analyst to select reasonable total turn percentages and to make manual adjustments to the volumes after completion of the basic computations. However, its straightforward formulation provides a logical tool with which to analyze basic intersection turning movements in situations where only link volumes are known.

Input Data Requirements

Nondirectional link volumes (i.e., total both directions) on each approach are required input data for this method. The nondirectional link volumes are obtained directly from the computer assignment or from the results of link refinement or detailing procedures described in Chapters 4 through 7.

Directions for Use

The five-step methodology is described below, along with an illustrative example.

Step 1--Estimate Total Turning Percentage. The first step is to estimate the percentage of total inflowing traffic which turns (either right or left). The turning percentage value must normally be estimated based on the unique characteristics of the intersection and comparable intersections from other parts of the urban area. If the actual signal green time given to individual turning movements is known at the subject intersection, these values can be used instead of the estimated percentage for the entire intersection.

This turning movement percentage is estimated relative to the sum of only inflowing (i.e., one direction) volume. The inflowing volume equals one-half of the total nondirectional volume. Therefore, a turn percentage relative to the total non-directional volume would need to be doubled.

For example, Figure A-67 depicts a four-way intersection with nondirectional link volumes.

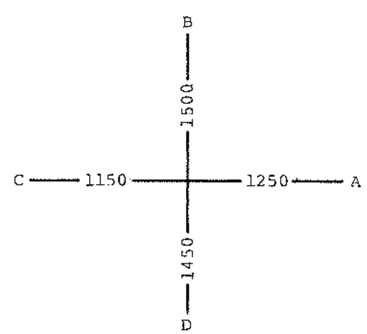


Figure A-67. Example of nondirectional intersection volumes.

The sum of the nondirectional link volumes is 1,250 + 1,500 + 1,150 + 1,450 = 5,350. Therefore, the total inflowing (equals outflowing) volume is 5,350/2 = 2,675. It is assumed on the basis of conditions at similar intersections that the total turn percentage would not exceed 20 percent of the inflowing traffic, or 0.20 * 2,675 = 535 turning vehicles. This value of 535 would equal the sum of all inflowing left turn and right turn volumes (V_{turns}). Because the 2,675 total represents the total inflowing volume, Eq. A-36 can be used to calculate total through volume ($V_{through}$):

$$V_{Turns} + V_{Through} = 2{,}675 \qquad \text{(A-36)}$$

Substituting for the turning volume yields:

$V_{Through}$ = 2,675 - 535 = 2,140 (sum of all approaches)

$V_{Through}$ can be checked for reasonableness against volumes on similar approaches with known directional link volumes.

Step 2--Calculate the Relative Weight of Each Intersection Approach. This step is best accomplished graphically. Draw a generalized schematic of the intersection. Sum all the nondirectional volumes on all the intersection approaches. Express the volume on a particular approach as a proportion of total volume. The proportions (or relative weights) on all approaches must sum to 1.00 (100%).

Using the example in Figure A-67, the relative weights for approaches A through D would be as follows:

Total Nondirectional Volumes = 5,350 (from Step 1)

Approach A:	1,250/5,350 = 0.23
Approach B:	1,500/5,350 = 0.28
Approach C:	1,150/5,350 = 0.22
Approach D:	1,450/5,350 = 0.27
	1.00

Step 3--Perform Initial Allocation of Turns. This step involves allocating the volume on each approach to the other intersection approaches. Multiply the total volume on an approach by the relative weights, as computed in Step 2 for the remaining approaches which involve turns. Straight-through volumes are not allocated at this time. This calculation should be performed for each intersection approach to produce turns to the other approaches. For this methodology, U-turns are assumed to be negligible and are not included. Continuing with the example, the following calculations would be made:

From Approach	To Approach	
A	B	1,250 * 0.28 = 350
	D	1,250 * 0.27 = 338
B	A	1,500 * 0.23 = 345
	C	1,500 * 0.22 = 330
C	B	1,150 * 0.28 = 310
	D	1,150 * 0.27 = 310
D	A	1,450 * 0.23 = 334
	C	1,450 * 0.22 = 319

At this point there will be two sets of two-way turn volumes for each interchange opportunity (e.g. A to B; B to A). To avoid double-counting, each pair of turn volumes for each potential turn should be averaged to produce one nondirectional turn volume for each potential interchange. These values will be further refined in subsequent steps. Figure A-68 shows this averaging calculation.

Step 4—Adjust Turning Volumes Based on Total Turning Percentage. The total volume of turns generated in Step 3 will typically exceed the likely volume of turns at the intersection. To adjust the Step 3 estimates, a turning percentage adjustment needs to be imposed. The adjustment involves the following computations:

(a) Write down the total inflowing volume (Step 1).

(b) Write down the total turn percentage (Step 1).

(c) Compute total expected volume of turns as (a) * (b).

(d) Sum the turning volumes calculated during Step 3.

(e) Adjust the individual turns from Step 3 using either a difference or a ratio method. For the difference method, subtract (d) from (c) to determine the total turn volume discrepancy. Divide this difference by the number of turning movements (e.g., for a four approach intersection there are four turning movements) and add/subtract to/from the turning movements. For the ratio method, divide (c) by (d) and multiply this value times each of the turning movements. Both of these methods produce satisfactory results in most cases; however, the difference method may result in negative numbers if the total estimated intersection turning percentage (Step 1) is too low.

At the end of this step, the total volume of turns at the intersection will be equal to the expected volume total from Step 1.

Using the example:

(a) Total inflowing volume = 2,675

(b) Total turn percentage = 20% (0.20)

(c) Total expected volume of turns = 2,675 * 0.20 = 535

(d) Sum of turns from Step 3 = 348 + 320 + 315 + 336 = 1,319

(e) Adjustment:

Difference Method: 535 - 1,319 = -784 - 84/4 = 196 to be subtracted from each turning volumes

Ratio Method: 35/1,319 = 0.41 to be multiplied by each turning volumes

Since no negative numbers would result in this example, the difference method is selected. The result is shown in Figure A-69.

<u>Step 5—Balance the Approach Volumes and Adjusted Turn Volumes</u>. Typically the preceding steps will yield a turning movement estimate that conforms to the estimated turning percentage established in Step 1. However, it is possible, even likely, that the method will not yield an intersection scenario that accounts for all traffic traversing the intersection. To test for this situation, take each approach of the intersection and do the following:

(a) Write down the total approach volume

(b) Subtract the turns made to/from that approach from cross streets

(c) Add the turns made to/from the approach on the opposite side of the intersection.

This computation should be performed independently for each intersection approach. If the intersection clears all traffic, the total volume on the opposing approach of the intersection should equal the volume estimated from the above test. If these volumes do not correspond, an adjustment needs to be made to out-of-balance numbers to bring the analysis into equilibrium, and thus to account for all of the intersection volume. The adjustment technique is not routine, but needs to be tailored to the specific results of the test.

In this example, the following calculations are performed for approach A:

Approach A: (a) 1,250

(b) 1,250 - 152 - 140 = 958

(c) 958 + 124 + 119 = 1,201 (compared with Approach C volume = 1,150)

Similar calculations are performed for the other approaches with the results shown in Figure A-70. It can be seen that the calculations yielded fairly close in agreement in this case. Comparing opposite approaches, Approaches A and C are under/overestimated by 51 respectively, while B and D are under/overestimated by 33 respectively.

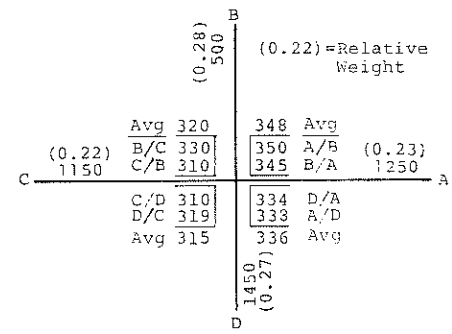


Figure A-68. Averaging of nondirectional turning volumes.

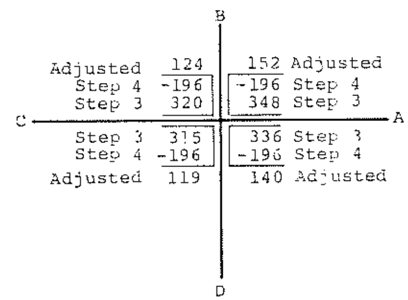


Figure A-69. Adjusted nondirectional turning volumes.

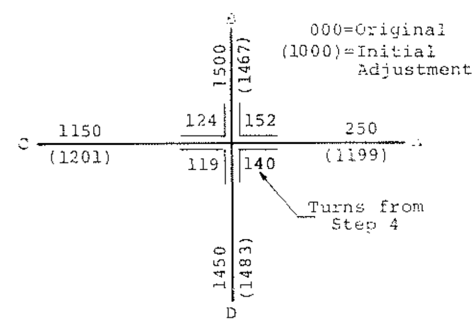


Figure A-70. Results of test for intersection volume clearance.

Two situations are normally encountered in this analysis:

1. The opposite intersection approaches show a greater difference in adjusted volume (Step 5) than was evident in the original volumes (Step 2).

2. The two opposing intersection approaches have adjusted volumes (Step 5) that are closer to each other than was evident in the original volumes (Step 2).

In the first situation, iterating the entire procedure from Step 2 using the new approach volumes will narrow the volume differences between two opposing intersection approaches. The nature of the procedure tends to reduce differences. Thus, when working with intersections with dramatically different volumes on each approach, the procedure will reduce the differences on opposing approaches, and if subjected to enough iterations, will ultimately yield the average of the two volumes on each opposing approach.

The second situation, in which the difference in volume on opposing approaches needs to be increased, is more complicated. The volumes in this example typify this discrepancy, as shown below:

	Original Volume Difference (Step 2)	Adjusted Volume Difference (Step 5)	Conclusion
Approaches A/C	1,250-1,150 = 100	1,201-1,199 = 2	Increase Difference
Approaches B/D	1,500-1,450 = 50	1,483-1,467 = 16	Increase Difference

This difference needs to be apportioned between the two approach volumes, keeping the turning volumes constant. The following computations will provide an adjustment on the first iteration which will increase the difference between the opposing volumes.

(a) Sum the volumes on the two opposing approaches using the original volumes input at the outset of the analysis (Step 2).

(b) Determine the proportion of this volume (a) represented by each of the two opposing approaches. This must sum to 1.00 (100%).

(c) Determine the approach volume difference between the adjusted (Step 5) and the original estimates. This absolute difference should be the same on each side of the intersection, although the sign will change.

(d) Multiply the proportions (b) by the volume difference (c). Add/subtract this number to/from the calculated volumes as appropriate.

The above adjustments should be applied to each intersection approach in order to ensure that the approach volumes are in scale relative to the completed turning volumes. Note that unless the proportions determined from (b) are split 50%-50%, then (d) will result in a change in the sum of the opposing approach volumes (a) and also of the total intersection volume.

These calculations are shown below for the example:

	Approaches A/C	Approaches B/D
(a)	1,250 + 1,150 = 2,400	1,500 + 1,450 = 2,950
(b)	A: 1,250/2,400 = 0.52	B: 1,500/2,950 = 0.51
	C: 1,150/2,400 = 0.48	D: 1,450/2,950 = 0.49
(c)	A: 1,250 - 1,199 = +51	B: 1,500 - 1,467 = +33
or	C: 1,150 - 1,201 = -51	or D: 1,450 - 1,483 = -33
(d)	A: 0.52 * 51 = 27 (Add)	B: 0.51 * 33 = 17 (Add)
	C: 0.48 * 51 = 24 (Subtract)	D: 0.49 * 33 = 14 (Subtract)

The final nondirectional intersection volumes are shown in Figure A-71.

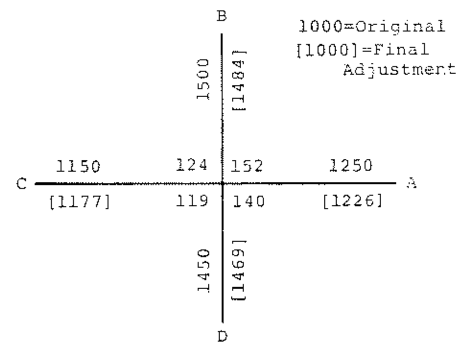


Figure A-71. Final nondirectional intersection volumes.

As a check, the total adjusted intersection volume is calculated to be 1,226 + 1,484 + 1,177 + 1,469 = 5,356, which is slightly higher than the original total of 5,350. This difference is not significant; however, in other examples additional manual link and/or turn adjustments may be required to ensure that the total intersection volume remains within a range acceptable to the analyst.

Special Situations

This method is intended for sketch planning purposes to determine the approximate nondirectional movements at an intersection. The method can also be applied with care to directional volumes, although the added complexity of this situation often leads to a time-consuming process. Therefore, directional volumes are better estimated using the preceeding iterative method.

Several applications of the method require special attention. At an intersection with five or more approaches, the approaches may have to be broken down into two or three partial intersections and then remerged to manually combine the results. This involves considerable judgment. Another difficult situation occurs when one of the intersection approaches is a one-way street. In such a situation two-way trip interchanges cannot occur. To treat this problem, the intersection must be broken into a set of one-way streets. One street would carry the flow into the intersection, and all of the other approaches would carry flow away from the intersection. At this point, the method proceeds as usual from Step 1 for each "set" of one-way streets. After all movements have been accounted for, the results should be merged and adjusted to ensure that all traffic can clear the intersection (Step 5).

A very common difficulty encountered with this method occurs where the intersection approaches carry radically different volumes. For example, a residential collector carrying a volume of 6,000 may cross a large principal arterial carrying 35,000 or more. Since the method tends to reduce the differences between link volumes on adjacent approaches the analyst in this situation will frequently be confronted with the need to increase the differences in volumes.

The method can be made to work with different turning movement percentages for different approach directions. An analyst familiar with local conditions can invoke this feature with good results. When signal green times for particular turning movements are available, these should be used instead of a total percentage for all turns (Step 1). In such cases, Step 4 can be modified to adjust each turning movement independently using the difference or ratio method. These results are then merged to produce a total intersection flow diagram for input to Step 5.

"T" INTERSECTION PROCEDURES

The turning movements on a three-legged, or "T" intersection can often be determined using simpler procedures. A unique solution can be obtained for nondirectional turning movements. Directional turning volumes can be directly computed from directional link volumes if only one intersection movement is available. Basic mathematical relationships among the link volumes can aid in estimating one of the turning movements for input to these computations.

Nondirectional Turning Movement Method

Nondirectional turn volumes can be easily computed if nondirectional link volumes on the three approaches are known. Note that directional link volumes must be summed together prior to application of the procedure.

Basis for Development

The nondirectional method is mathematically based on algebraic relationships. The two unknown turning volumes can be directly obtained from two independent equations. Therefore the solution is unique.

Input Data Requirements

Input data required for this method are nondirectional link volumes for each of the three approaches. These volumes can be obtained directly from the computer assignment or from the results of the refinement or detailing procedures presented in Chapters 4 through 7.

Directions for Use

Referring to Figure A-72 for notations, the following equations are used:

$$X = (A - B + C)/2 \quad \text{(A-37)}$$

$$Y = (C - A + B)/2 \quad \text{(A-38)}$$

Where A, B and C are link volumes and X and Y are the desired turning movements:

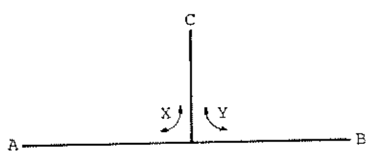


Figure A-72. "T" intersection with nondirectional turning movements.

For example, given the following link volumes:

A = 10,000
B = 12,000
C = 4,000

Then:

$$X = (10{,}000 - 12{,}000 + 4{,}000)/2 = 1{,}000$$

$$Y = (4{,}000 - 10{,}000 + 12{,}000)/2 = 3{,}000$$

As a check, the sum of X+Y must equal link volume C. Therefore, 1,000 + 3,000 = 4,000 = C (Check). Care must be taken to denote the movements as shown above. Otherwise, Eqs. A-37 and A-38 would require adjustment.

Directional Turning Movement Method

Directional volumes at "T" intersections cannot be uniquely determined from directional link volumes alone. However, knowledge of one directional volume will produce a unique solution for all other directional volumes.

Basis for Development

Because a "T" intersection has only six directional movements involved, simple mathematics can be used to derive equations to aid in the solution. A total of five independent equations are available to solve for six unknown volumes. Therefore, one movement must be known or estimated before the other five movements can be calculated. Some basic mathematical relationships can also be made among the six directional link volumes. These relationships can assist in estimating one of the turning volumes, from which the others can be directly computed as discussed above.

Input Data Requirements

Six directional link volumes are required for input to this method. In addition, one of the six turning volumes must be known or estimated.

The link volumes can be obtained from a directional computer assignment or from the results of a refinement or detailing procedure described in Chapters 4 through 7. The procedures in Chapter 10 can be used to derive directional link volumes from nondirectional link volumes. The one turning volume can be estimated from base year counts, turning volumes at similar intersections, or from known relationships among the link volumes. This latter source is described below.

Directions for Use

If one turning volume or one through movement is known or can be estimated, the analyst can calculate the remaining volumes. Five independent equations can be constructed. Figure A-73 shows a typical situation with unknown volumes A, B, C, D and E, while F is assumed to be known (F = 100), as are the link volumes 1 through 6. The following equations are possible:

$$E = \text{Volume 4} - F \text{ where F is known} \quad \text{(A-39)}$$

$$A = \text{Volume 5} - E \quad \text{(A-40)}$$

$$B = \text{Volume 2} - A \quad \text{(A-41)}$$

$$C = \text{Volume } 3 - B \qquad \text{(A-42)}$$

$$D = \text{Volume } 6 - C \qquad \text{(A-43)}$$

Substituting for the link volumes and for F, the volumes A through E are calculated sequentially as follows:

E = 400 - (100) = 300

Then:

A = 1000 - (300) = 700
B = 800 - (700) = 100
C = 300 - (100) = 200
D = 1200 - (200) = 1000

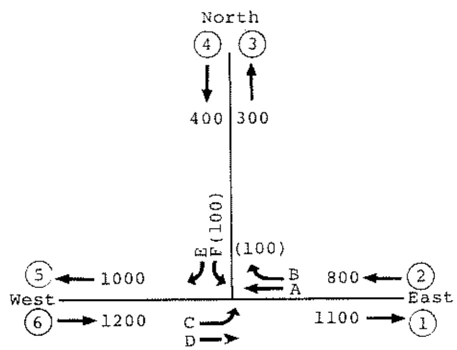


Figure A-73. "T" intersection with directional turning movements.

Other similar computations with different number pairs would produce the same unique results.

In most cases, the analyst will know or be able to estimate one of these directional movements. Straight-through movements are usually quite apparent from the directional link volumes. In the example in Figure A-73, the through volumes A and D would be expected to represent a high proportion of their respective approach volumes given the continuity of directional link volumes on both sides of the intersection (e.g., the link volumes on the west and east approaches are similar), and given the relatively small link volumes on the north approach. In cases where the cross street link volumes (e.g., on north approach) are high relative to the main street link volumes (e.g., on west and east approaches), then lower through volumes (e.g., movements A and D) would be expected. Once these relationships are established, one of the directional volumes can usually be estimated.

Another technique is to first compute the nondirectional turning volumes using the method described previously. Using Figure A-73, the nondirectional turning volumes will equal the sum of directional volumes (C + E) and (B + F). The task is then to determine each of these directional volumes, plus the through volumes A and D given the nondirectional turning volumes and the directional link volumes, which are known.

Some basic relationships can be established using the directional link volumes. For instance, as shown in Figure A-73, if link volume (1) is greater than (6), turning movement F must be greater than C. Similarly, if (5) is greater than (2), E must be greater than B. Finally, if (3) is greater than (4), (B + C) must be greater than (E + F). Because the link volumes 1 through 6 are known, the magnitudes of these inequalities are also known. The converse of these relationships is also true. Given this knowledge, the analyst can usually estimate at least a range for each of the turning volumes. Once a single turning volume is estimated within a tolerable range, the remaining directional volumes can then be computed directly as described previously.

Some of the above relationships can also be developed for four or five-legged intersections where specified movements (e.g., left turns) are prohibited. However, the added complexity of multi-leg intersections usually prevents the analyst from constructing meaningful mathematical relationships within a reasonable timeframe.

CHAPTER NINE

DESIGN HOUR VOLUME AND TIME-OF-DAY PROCEDURES

GENERAL

Two critical types of traffic data needed for highway project planning and design are design hour volumes and other time-of-day traffic data, including peak hour factors and the distribution of traffic by hour of day. On most highway facilities in an urban area, traffic during an average weekday varies substantially. Regular and repetitive peaking of traffic occurs during the morning and evening peak periods principally as a result of travel to and from work. Moreover, on most highway facilities substantial peaking of traffic occurs even within an individual peak hour. This results in the need for urban highway facility design to utilize an hourly volume as the basis for design and factors to account for further traffic peaking within that hour.

The sections of this chapter will describe procedures for forecasting design hour volumes and peak hour factors. Procedures for forecasting the distribution of average daily traffic for each hour of the day will also be described, as such hourly traffic data are necessary for some environmental impact analyses.

DESIGN HOUR VOLUME (DHV) CONSIDERATIONS

The primary objective in forecasting design hour volume (DHV) is to select a specific hour of future traffic volume that will be used as the basis for design. Standard engineering practice prescribes that the hour of future volume selected as the basis for design should be that hour at which the ratio of benefits to costs is maximized over the sum of the 8,760 hours occurring throughout the forecast year (6).

Therefore, the selection of the proper design hour requires an understanding of the variation in hourly traffic volumes throughout the forecast year. Fortunately, only two basic patterns of hourly traffic variation are generally present in urban areas. One of the patterns is typical of most facilities in most urban areas and is shown in Figure A-74. The peak hours of this pattern are dominated by the repetitive peaking of traffic during a morning and evening weekday peak hour. Each of these peak hours occurs about 250 times each year. As a result there is usually little difference between the 1st or 10th highest hour of traffic and the 30th highest hour, 100th highest hour, 250th highest hour, and in some cases even the 500th highest hour.

The other pattern of hourly traffic variation is found only on those facilities, or in those urban areas, where the greatest traffic peaks are a result of seasonal and/or weekend recreational travel. It may also be found on those facilities located in ex-urban or rural areas. The pattern of hourly traffic variation on such facilities as shown on Figure A-75 indicates that the highest hours of traffic are typically much greater than the 30th highest hour. There are subsequent differences between the 30th highest hour and the 100th and 200th highest hours of traffic, but they are not nearly as significant.

For each pattern of hourly traffic volume variation over a year, the design hour volume is the hour at which the slope of the traffic volume curve in Figures A-74 and A-75 changes most rapidly. It is at this hour that the ratio of benefits to costs of the facility design is usually maximized over the sum of all hours of the forecast year. It is particularly important for a facility with peaks defined by recreational or seasonal travel (Fig. 75) to utilize the proper DHV. In such cases, use of a much higher volume for design would be wasteful as the facility would have excess capacity which

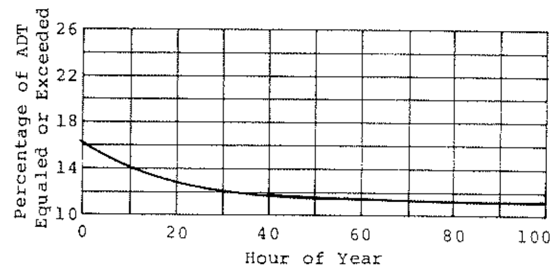


Figure A-74. Hourly traffic volume variations on typical urban facilities.

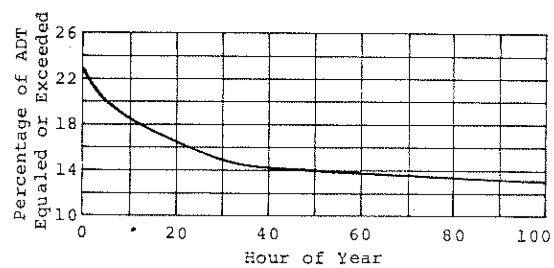


Figure A-75. Hourly traffic volume variations on a typical urban facilities with recreational or seasonal peaks.

would rarely be used. Conversely, use of a much lower volume would result in an inadequate design for many hours of the year. Because of the steepness of this curve at the highest volume end, a small increase in the capacity provided would permit the facility to be adequate for many additional hours.

It has generally been found that the hour which should be used as the basis for design on seasonal or recreational facilities is the 30th highest hour of volume (6). However, the 30th highest hour of volume should not be considered as the universal standard for highway design. A more flexible standard that should be adopted is to select the hour which maximizes the benefit-to-cost ratio.

With respect to the more typical urban facilities with peaks defined by weekday work travel (Fig. A-74), some highway designs continue to be based on the 30th highest hour, while other highway designs have been based on the average weekday peak hour. The difference between these two approaches, however, may not be significant (6). The 30th highest hour can be viewed as approximately representing the average of the highest weekly peak hour of traffic occurring during the year. This is typically the traffic averaged across all Friday afternoon peak hours for the year. The average weekday peak hour, on the other hand, can be viewed as approximately representing the 125th highest hour of traffic volume. It typically consists of the average of the 250 highest peak hours for 52 weeks, or the average traffic found during each afternoon peak hour (Monday through Friday) for a year. Thus, the difference between the two approaches is generally the difference between the 30th and the 125th highest hours of traffic, or the difference between the average Friday afternoon peak hour traffic volume and the average weekday peak hour traffic volume. The approach that should be used for designing a particular facility should be based on selecting that hour for design which has the best potential to maximize the benefit-to-cost ratio of the road improvement over a one year period.

It is generally accepted that for typical urban facilities with peaks defined by work travel the DHV comprises between 8 and 12 percent of the average daily traffic (ADT). For atypical facilities with peaks defined by recreational or seasonal travel, the DHV generally comprises between 12 and 18 percent of the ADT (6). Procedures for forecasting DHV for each of these situations are described in the following sections.

BASIS FOR DEVELOPMENT

The design hour volume and other time-of-day procedures draw heavily on the products of research conducted by Peat, Marwick, Mitchell & Co. (93) and adapted in NCHRP Report 187 (88). Specific topic areas as noted in the text were assimilated from studies by the Middle Rio Grande Council of Governments (70), Harmelink (35) and Shallal (85). Design issues relating to temporal distribution were obtained primarily from policies prepared by AASHTO (5, 6).

DHV FORECASTING PROCEDURES FOR TYPICAL URBAN FACILITIES

The procedures which have been developed for forecasting DHV on typical urban facilities assume the use of the average weekday peak hour of traffic (PHT) for design purposes as described above. There are two basic types of procedures. One can be applied under the situation where it may be assumed that the ratio between the base year PHT and the average daily traffic (ADT) will not change over time. It is typically applied only to analysis of base year facilities; as a result, only the measurement of base year PHT/ADT ratios is necessary to make the forecast. The other procedure assumes that the PHT/ADT ratio will change and uses a range of characteristics on the facility to forecast the future year traffic peaking. This procedure is applicable to both base year and new future facilities.

Procedure for DHV Forecasts Assuming No Change in PHT/ADT Ratio

This procedure forecasts future weekday peak hour traffic by comparing an actual or estimated base year PHT/ADT ratio with a forecasted ADT on a facility. The procedure requires the assumption that the base year PHT/ADT ratio will not change over time.

This procedure is applicable only if it can be assumed that the degree of peaking on the facility will not change. Three conditions should be met if this assumption of stability is to be made. First, the change in ADT on the facility between the base year and future year should not be substantial. Second, the change in the type of land-uses and trips served should not be substantial. Third, the degree of congestion on the facility and parallel facilities should not change over time.

The validity of this procedure is based on findings that PHT/ADT ratios can be stable over time, particularly in those cases where the facility is anticipated to meet the conditions outlined above. For this reason, the conditions should be carefully analyzed for each facility to ensure that the procedure remains valid.

Input Data Requirements

The following data are required as input to this procedure:

- Base year average weekday peak hour traffic (PHT-base).
- Base year average daily traffic (ADT-base).
- Future year forecasted average daily traffic (ADT-future).

The base year data should be derived from actual ground counts if possible. If necessary, the data can be interpolated or extrapolated from traffic data from adjacent years or parallel intersection facilities. The future year data can be taken directly from computerized traffic forecasts or from the results of the refinement and detailing procedures in Chapters 4 through 7.

Directions for Use

The following are step-by-step directions for the procedure:

Step 1--Compute the Base Year PHT/ADT. Divide the known or estimated PHT by the ADT on the same facility.

Step 2--Multiply the Base Year PHT/ADT by the Future Year ADT. The equation becomes:

$$(PHT/ADT)_{base} \times ADT_{future} = PHT_{future} \qquad (A-44)$$

This procedure does have potential to be applied to new facilities where it is necessary to assume that the base year PHT/ADT ratio will change. A revised PHT/ADT ratio is estimated for the new facility under analysis using base year data from nearby facilities that are judged to have characteristics similar to the proposed facility. This new value would be substituted for (PHT/ADT) base in Step 2 to compute the future PHT for the new facility. The next procedure describes this situation in greater detail.

Procedure for DHV Forecasts on New Facility or Assuming Change in PHT/ADT Ratio

This procedure forecasts future weekday peak hour traffic for a new facility, or for an existing facility where significant changes in the PHT/ADT ratio are anticipated. The procedure considers the future facility characteristics known to influence the PHT/ADT ratio.

The procedure may involve statistical analysis of the influence of each facility characteristic. The PHT/ADT estimate may be obtained through a cross-classification table of PHT/ADT ratios stratified by various facility characteristics known to have the greatest influence. A regression equation with PHT/ADT ratios as the dependent variables and facility characteristics as the independent variables may be developed instead. The advantage of a statistical approach is that it clearly quantifies the degree of influence of each facility characteristic.

The principal disadvantage of the statistical approach is its data requirements. A very large PHT and ADT counting program may be necessary for its proper development and maintenance. For example, a cross-classification table with five PHT/ADT ratios stratified by five different facility characteristics requires sufficient data for the calculation of 25 average DHV/ADT ratios. Special counting programs will be necessary to satisfy this data requirement for various roadway classifications. The other disadvantage of this approach is that it may be applied blindly without judgment. This is important in this case because the large data requirements of this approach may dictate that certain other factors which may marginally influence the PHT/ADT ratio should not be included in a model. Similarly, certain average PHT/ADT ratios established as part of the model may be based on very limited traffic counting.

The alternative to a statistical approach is what will be called the judgmental approach. It requires the person responsible for the PHT forecast to be aware of the factors that influence PHT/ADT ratios and their degree of influence. This knowledge must be obtained from a review of base year PHT/ADT ratios on similar facilities. Thus, a PHT and ADT counting program is required under this approach as well, but it may not need to be as extensive. Special counts would only be required as necessary. For example, if a forecast PHT/ADT ratio had to be established for a facility with certain characteristics, and if no base year PHT and ADT count data were available for such facilities, special counts could be performed on specific facilities with the appropriate characteristics. The disadvantage then of this approach is that the forecast is totally dependent on the judgment of the analyst.

This procedure can be applied either in a statistical or judgmental approach to any typical urban facility with traffic peaks dominated by work travel. The procedure is particularly useful for analyzing new facilities or existing facilities for which it is necessary to assume that the existing PHT/ADT ratio will change by the forecast year.

The basis of the validity of this procedure is that analyses of traffic count data have established that selected highway facility design, location, and use characteristics explain much of the variation in highway facility PHT/ADT ratios. Several of these characteristics are described in the directions for the procedure. Once these variations are known, accepted statistical or judgmental methods can be employed to produce reasonable PHT/ADT ratios for use in forecasting future DHV.

Input Data Requirements

The following data are used as input to this procedure:

- Future year forecasted average daily traffic (ADT_{future}).
- Base year estimate of PHT/ADT ratio on facility or on similar facilities.
- Estimated future year facility characteristics (e.g., type, location, orientation to CBD, adjacent land-uses, level of service).

The base year data should be derived if possible from actual ground counts, or estimated if necessary from data from other years or from other facilities. The future year ADT can be taken directly from computerized traffic forecasts or from the results of the refinement and detailing procedures in Chapters 4 through 7. The future year facility characteristics should be obtained from design plans, land-use projections, and estimates derived from base year conditions (e.g., level of service) in the area.

Directions for Use

The following are step-by-step directions for developing and applying this DHV forecasting procedure.

Step 1--Identify the Highway Facility Characteristics Which Influence the PHT/ADT Ratio and Quantify the Degree of Influence of Each Characteristic. The first step using either the statistical or judgmental approach is to identify the key characteristics of the facility that influence the PHT/ADT ratio. Once this is done, the degree of influence can be quantified. Five factors are typically considered to influence urban highway traffic peaking: facility type, facility location, facility orientation, adjacent land-uses, and facility level of service (i.e., congestion).

Facility type has generally been determined to be correlated with the PHT/ADT ratios. The typical stratifications used for arterial facility type are freeways/expressways, major arterials, and minor arterials. Particularly in the larger urban areas, it has been found that PHT/ADT ratios are lowest for the highest facility types. The higher facility types such as freeways and expressways are likely to carry more traffic in the off-peak, particularly with respect to truck and through traffic. The higher type facilities may also be the only facilities for which the amount of traffic carried in the peak hour may be restricted due to capacity.

Facility location within the urban area is also correlated with the PHT/ADT ratio. Typical stratifications used for urban facility location include the central business district (CBD), central city, and suburban (88, 93). Typically, central city facility PHT/ADT ratios are lower than those in either the central business district or the suburbs. The relatively low amount of evening traffic in the typical CBD and the lower level of congestion found in most suburban areas contribute to somewhat higher ratios in those locations. Conversely, the central city typically handles more uniform traffic throughout the day and experiences more congestion, leading to a lower PHT/ADT ratio.

The third facility characteristic which has been considered as being correlated with the PHT/ADT ratio is facility orientation with respect to the CBD. The typical stratifications employed are radial and crosstown. These stratifications only apply to facilities located outside of the CBD.

The fourth facility characteristic is adjacent land-use. The typical stratifications include commercial and noncommercial land-use. Adjacent commercial land-use generally implies either strip development along the facility or access to a major shopping center. Facilities that are considered as serving commercial land-uses typically have lower PHT/ADT ratios as they serve relatively more traffic during nonpeak hours.

The fifth factor which has been considered is level of service or congestion. It is the least used of all the characteristics. However, on a practical basis, accounting for the potential influence of congestion makes sense. First, much of the influence attributed to the other four characteristics is based on their ability to differentiate between facilities that do or do not experience congestion. And second, it is logical for a facility that is congested only during peak hours to carry less peak hour traffic as a proportion of total weekday traffic than an uncongested facility. Only on a congested facility is there a reason for peak hour traffic to be diverted in terms of path, time, mode, or even area of travel. Moreover, this potential for diversion may be present only during the peak hour.

Regardless of the desirability of considering congestion, there is a practical problem with its application. Generally, the computation of peak hour congestion requires previous knowledge of the PHT/ADT ratio. Therefore, a ratio must be initially assumed and then adjusted through an iterative process. In addition, the facility design must be known. One way this problem is resolved is to represent congestion through stratifications of ADT divided by the number of lanes provided.

Step 2—Select a PHT/ADT Ratio, Based on the Anticipated Characteristics of the Facility. Most current applications of this DHV forecasting procedure use the judgmental approach to combine the facility characteristics from Step 1 into an estimate of the PHT/ADT ratio. PHT/ADT ratios on similar facility types are also examined. One known application of the statistical approach through cross-classification methods investigated factors of adjacent land-use, location, and orientation and concluded that adjacent land-use was the key characteristic to consider (70). This analysis, however, was not solely limited to forecasting the proportion of weekday traffic that occurred during the peak hour, because it was to be used in forecasting the proportion of weekday traffic that occurred during each hour of the day. The other known application of the statistical approach based the PHT/ADT strictly on level of service (6). A series of regression equations was developed to predict this ratio and hence the DHV. A different equation expressing the ratio strictly as a function of ADT was established for a number of points along a range of values of ADT divided by the number of lanes provided.

An empirical approach can be used, based on data from nine urban areas (88, 93). These transferable data are intended to be applied for short-cut "sketch planning" estimates in other urban areas. This application used factors of location and orientation, as well as size of urban area population. Various tables were developed to relate hourly traffic volume distributions to such factors as size of urban area, location, and orientation. Tables A-13 through A-24 are reproduced as an addendum to this chapter.

Step 3--Multiply the PHT/ADT Ratio by the Future ADT. This step involves the use of a simple equation, as follows:

$$(PHT/ADT)_{estimate} * ADT_{future} = PHT_{future} \qquad \text{(A-45)}$$

The estimated future PHT/ADT ratio is obtained from Steps 1 and 2, while the future ADT is a data input. The output will be a forecasted PHT value for the future year.

Example Problem

The following is an example of the DHV forecasting procedure for a typical urban facility assuming a change in the PHT/ADT ratio. It follows the three steps described above.

Step 1--Identify Highway Facility Characteristics. It will be assumed here that the facility under analysis is a radial arterial in a suburban portion of a medium sized urban area (500,000 population). The base year facility is assumed to be a two lane arterial radially oriented to the downtown. Its traffic volume is expected to increase substantially from 5,000 to 15,000 vehicles per weekday as new residential development is expected to occur along the arterial. No major commercial development is expected to be located along or served by the arterial. As a result, the facility should be considered a noncommercial, suburban arterial. No level-of-service data are available.

Two approaches for performing this step will be shown. One approach was a statistical base as shown in Table A-12. It relates the PHT/ADT ratio to adjacent land-uses and location. The other approach uses the empirical data (88) shown in addendum Tables A-13 through A-24. Given the data in this example, addendum Table A-20 was selected as being most appropriate.

Step 2--Select a PHT/ADT Ratio. Table A-12 would indicate the PHT/ADT ratio on this facility to be 9.8 percent. Addendum Table A-20 yields a PHT/ADT value of 8.5 percent. These estimates should be checked in a number of ways. The difference between the facility's base year and forecasted PHT/ADT ratios should be reviewed. Also, the existing PHT/ADT ratios of facilities with a similar location (suburban), orientation (radial), adjacent land-use (noncommercial), and type (arterial) should be compared to the PHT/ADT ratios for reasonableness. In this example, a value of around 9 percent, midway between the two estimated ratios (i.e., 8.5 and 9.8 percent) was considered to be most reasonable to use.

Step 3--Multiply the PHT/ADT Ratio by the Future ADT.

$$\begin{aligned} PHT_{future} &= (PHT/ADT)_{estimate} * ADT_{future} \\ &= 0.09 \times 15{,}000 \\ &= 1{,}350 \text{ vehicles per hour} \end{aligned}$$

This volume can be used for subsequent planning and design studies.

DHV FORECASTING PROCEDURES FOR ATYPICAL URBAN FACILITIES

There are two basic situations or sets of circumstances under which the forecasting of design hour volume (DHV) for a typical urban facilities is undertaken. One situation assumes that the ratio of the base year design hour volume to the average daily traffic (DHV/ADT) of the facility in question will not change. The other situation assumes that the DHV/ADT ratio of the facility in question will change. Estimates of the forecast year DHV/ADT ratio are made by examining DHV/ADT ratios of facilities that are experiencing operations and peaking characteristics similar to those envisioned in the forecast year on the facility in question.

The assumption of no change in the base year DHV/ADT ratio is applicable only under a limited set of conditions (6). Specifically, all three of the following conditions must be met. First, the base year DHV/ADT ratio on the facility should not exceed the average DHV/ADT ratio on similar facilities in the area. Second, the forecasted change in ADT should not be substantial. Third, there should not be any significant change in the type of trips or land-uses served. Because a base year DHV/ADT ratio is required, this procedure is applicable only to analysis of facilities existing in the base year.

There are two DHV forecasting procedures that will be described in this section. Each procedure can be used under both of the DHV forecasting assumptions. The simpler of the two procedures involves identifying one or a small number of permanent count station (PCS) facilities that have operations and peaking characteristics similar to the future peaking and operations envisioned for the facility in question. If it is being assumed that there will not be a change in the DHV/ADT ratio on the facility under analysis, the PCS facilities selected should have traffic peaking and operations that are similar to the base year conditions on the facility under analysis. If it is being assumed that there will be some change in the DHV/ADT ratio or if the facility under

Table A-12. Hourly traffic volume distribution according to commercial development and geographic location.

Time of Day	Central City Arterials		Suburban Arterials	
	Commercial	Noncommercial	Commercial	Noncommercial
AM				
12:00 - 1:00	.8	1.0	1.0	.7
1:00 - 2:00	.5	.4	.6	.3
2:00 - 3:00	.3	.2	.3	.1
3:00 - 4:00	.2	.2	.2	.2
4:00 - 5:00	.2	.2	.2	.2
5:00 - 6:00	.5	.4	.4	.5
6:00 - 7:00	1.8	2.3	2.1	2.8
7:00 - 8:00	5.4	7.4	6.3	8.0
8:00 - 9:00	5.4	7.1	5.2	5.7
9:00 - 10:00	5.7	5.2	5.0	4.6
10:00 - 11:00	6.5	4.9	5.6	4.6
11:00 - 12:00	7.3	5.6	6.7	5.5
PM				
12:00 - 1:00	7.5	6.5	7.4	6.0
1:00 - 2:00	7.4	6.0	6.8	5.7
2:00 - 3:00	7.3	6.3	7.1	6.5
3:00 - 4:00	7.6	7.9	7.3	7.4
4:00 - 5:00	8.4*	9.2*	8.3*	8.6
5:00 - 6:00	7.8	8.9	8.1	9.8*
6:00 - 7:00	5.3	6.0	5.8	7.1
7:00 - 8:00	4.1	4.4	4.4	5.1
8:00 - 9:00	3.5	3.3	3.7	3.6
9:00 - 10:00	3.0	2.9	3.5	3.2
10:00 - 11:00	2.1	2.1	2.4	2.2
11:00 - 12:00	1.4	1.5	1.6	1.3
	100.0	99.9	100.0	99.7

*PHT/ADT Ratios

Source: Albuquerque, NM Data (70)

analysis is new, the PCS facilities selected should have base year traffic peaking and operations that are similar to the envisioned future conditions on the facility under analysis. The DHV/ADT ratios of the selected PCS station(s) are then transferred to the facility under analysis, usually as an average. The transferred DHV/ADT ratio is multiplied by the forecasted ADT of the facility to establish the future DHV.

The second procedure requires dividing all PCS facilities into groups of similar base year DHV/ADT ratios. The appropriate group to which the facility under analysis belongs must then be determined, based on its envisioned future peaking and operations. The DHV/ADT ratio of the selected group is then multiplied by the forecasted future ADT of the facility in question to establish its future design hour volume.

There is a third procedure that has been used in Canada (36), but will not be elaborated on in this chapter. It has input data requirements that are substantially greater than the two above procedures, therefore potentially delaying project planning. In addition, the approach is only valid under the situation in which the base year DHV/ADT ratio is assumed not to change. The basic data requirements include hourly counts for each day of nonholiday weekends during the months of July and August. The procedure involves grouping PCS facilities according to similar peaking characteristics. A relationship for each group is developed between DHV and the traffic volume of a ranked July-August nonholiday weekend hour. The facility under question is assigned to one of the PCS groups. Its DHV is then predicted from its ranked July-August nonholiday weekend hourly traffic and from the selected PCS group relationship between DHV and these July-August hourly traffic volumes.

Input Data Requirements

The required input data for the two procedures described in this section include the following:

- Base year DHV/ADT ratios.
- Future year ADT forecast.
- PCS program operational in base year.

The base year ADT data should be derived from actual ground counts. Using these data, the base year DHV can be estimated by examining the slope of the ADT hourly volume distribution (see Figs. A-74 and A-75). The future year ADT forecast should be obtained from a computer assignment or from the results of applying the link refinement or detailing procedures described in Chapters 4 through 7.

Procedure Using Transfer of Selected PCS DHV/ADT Ratios

This procedure involves the transfer of base year DHV/ADT ratios obtained from selected facilities to the future facility under analysis.

Directions for Use

The following are step-by-step directions for this procedure.

Step 1--Identify Those PCS Facilities Which Have Characteristics Similar to Those Envisioned in the Forecast Year for the Facility Under Analysis. The basic consideration in the selection of PCS facilities is that the types of trips and land-uses they serve should be the same as those envisioned in the future for the facility in question, so that their peaking characteristics and

DHV/ADT ratios will be the same. If it is being assumed that the DHV/ADT ratio of the facility under analysis will not change, to an extent, the proximity of the PCS facilities to that facility should be reviewed, because this may indicate a similarity of land-uses and trips served. Checks of consistency for seasonal and daily variation between the facility and the selected PCS stations may be conducted if data are available. Under all circumstances, specific data should be examined for the selected PCS facilities in order to confirm that their peaking characteristics will reasonably be the same as those envisioned in the future for the facility under analysis. These data may include variation of ADT by month, variation of ADT by day of week, and the pattern of hourly variation throughout the year.

Step 2--Identify and Select the Appropriate DHV/ADT Ratios of the Selected PCS Facilities. The base year DHV/ADT ratios will be established by reviewing the patterns of hourly variation throughout the year at each selected PCS. Typically, the 30th highest hour of volume should be the DHV in this case.

Step 3--Multiply the Selected DHV/ADT Ratio From the PCS by the Future ADT on the Facility. The following equation is used:

$$(DHV/ADT)_{PCS} \times ADT_{future} = DHV_{future} \qquad \text{(A-46)}$$

The DHV/ADT ratio is obtained from Steps 1 and 2, while the future facility ADT is a data input.

Procedure Using Transfer of Grouped PCS DHV/ADT Ratios

This procedure uses a more structured approach to compare DHV/ADT ratios within groups of facilities. An appropriate group with its average DHV/ADT is then selected for use in computing future year DHV on a specific facility.

Directions for Use

The following are step-by-step directions for this technique.

Step 1--Divide PCS Facilities Into Groups Having Similar Characteristics. This grouping should be made with considerations given to each PCS's monthly traffic variation, daily traffic variation, hourly traffic variation throughout the year, and DHV/ADT ratio.

Step 2--Establish the Relationship Between ADT and DHV for Each Group. This step can be done by calculating the average DHV/ADT ratio for each group or by developing a regression equation between DHV and ADT if a sufficient number of PCS facilities are in each group.

Step 3--Assign the Facility Under Analysis to the Most Appropriate PCS Group. Consideration must be given in choosing the group that has existing peaking characteristics most like those envisioned for the future for the facility under analysis. Consideration of the similarity of trips (e.g., recreational) and land-use served is important. Proximity of the facility to PCS locations may also be examined. If it is to be assumed that the DHV/ADT ratio of the facility will not change, and if seasonal counts are available for the facility, the variation between the seasonal average daily traffic and the total ADT for the facility should be compared to that of each PCS group.

Step 4--Establish the Future DHV by Applying the Appropriate DHV/ADT Ratio to the Future ADT on the Facility. The following equation is used:

$$(DHV/ADT)_{PCS} \times ADT_{future} = DHV\ future \qquad \text{(A-47)}$$

The appropriate DHV/ADT ratio is obtained from Step 3, while the future facility ADT is a data input.

Example Problem

The following is an example of the application of DHV forecasting procedures for a facility in a small urban area that attracts summer recreational traffic from a large metropolitan area. The small urban area is assumed to have a population about 25,000 and is projected to increase to 40,000 within 20 years. It has a number of summer resorts and a state park within, or in close proximity to its urban area boundaries. It is within 50 miles of one metropolitan area of over 1,000,000 in population, two metropolitan areas of over 200,000 in population, and one metropolitan area of several million in population. These aspects are shown in Figure A-76.

The facility under analysis is currently a two-lane highway and has an ADT of 7,000 vpd. This ADT has been forecast to increase by about 70 percent to 12,000 vpd within 20 years.

It will be assumed in this example problem that no organized PCS program exists for grouping PCS's according to similar characteristics. As a result, the procedure that is applied uses the transfer of selected PCS DHV/ADT ratios. The procedure uses the same three steps.

Step 1--Identify Similar PCS Facilities. The peaking and operation of the example facility is expected to change somewhat in the future. Specifically, it is expected to serve a greater amount of recreational traffic. This is anticipated because recreational traffic to and within the small urban area is expected to increase and the level of congestion on other facilities in the traffic corridor is expected to increase significantly.

As shown in Figure A-76, it is assumed that there are four permanent count stations in the vicinity of the example facility. The patterns of hourly variation of traffic for the PCS's are shown in Figure A-77.

PCS #1 is the station selected for use in forecasting DHV for the facility segment under analysis. Its present operation is most likely to be similar to the future operation of the desired facility. PCS #1 is in the same traffic corridor and currently carries most of the recreational traffic. In addition, of the four potential PCS's, it is the most similar faclity type (arterial) to the facility under analysis.

Step 2--Identify and Select the Appropriate DHV/ADT Ratio. The DHV/ADT ratio should be selected in order that it leads to the design of a facility that will maximize facility benefits compared to costs over the sum of all hours in the design year. In other words, the hour selected should not result in a facility design that will be greatly underutilized for most hours of the year, nor should it result in a facility design that will be inadequate for many hours of the year as a result of a small amount of capacity not provided.

The DHV/ADT ratio of 0.16 is selected for this example. This is close to the 30th highest hour of volume at PCS #1.

Step 3--Multiply the Selected DHV/ADT Ratio by the Future ADT.

$$\begin{aligned} DHV_f &= ADT_f \times (DHV/ADT)_{PCS} \\ &= 12{,}000 \text{ vehicles/day} \times 0.16 \frac{\text{vehicles/hour}}{\text{vehicles/day}} \\ &= 1{,}920 \text{ vehicles/hour} \end{aligned}$$

This volume can be used for subsequent planning or design studies.

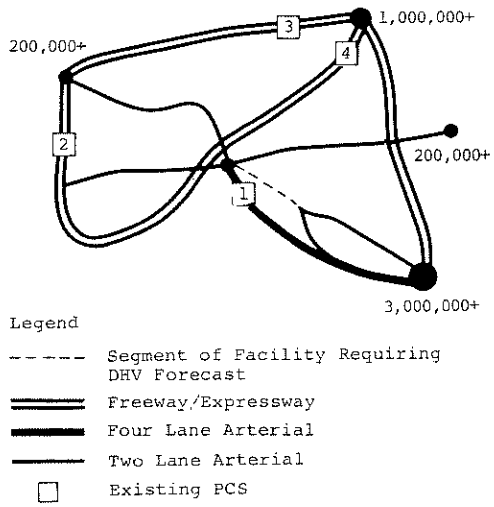


Figure A-76. Example facilities for DHV forecasting.

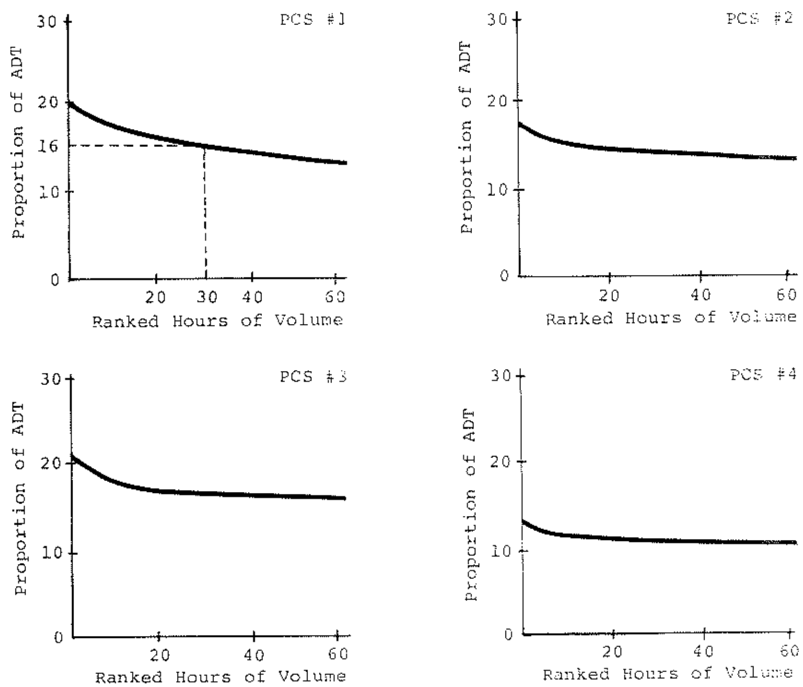


Figure A-77. Hourly traffic variations at example PCS's.

SPECIAL CONSIDERATIONS

The topic of DHV and other time-of-day traffic forecasting procedures includes several special considerations. Three primary considerations are treated in this section--the use of peak hour traffic assignments, the forecasting of hourly traffic data, and the forecasting of peak hour factors. Each of these subjects is relevant to the thorough application of these procedures to planning, environmental, and design analyses.

Peak Hour Traffic Assignment

It is important to understand that the need for peak hour traffic volume forecasting procedures results from the use of system assignments that forecast all day (24-hour) travel. All day traffic assignments are performed largely because it is much more difficult to predict the trip generation occurring during a single hour than trip generation occurring over an entire day.

One approach to developing peak hour traffic volumes, and for that matter, peak hour directional distributions (see Chapter 10) would be to use a peak hour traffic assignment. Four methods of performing such an assignment have been identified in NCHRP Report 58 (39). Two of the methods would develop peak hour assignments by factoring 24-hour trip data; one of these would factor trip-ends, and the other would factor trip interchanges. A third method would factor 24-hour work trip interchanges to produce peak hour trip totals. A fourth method would directly develop peak hour trip generation equations. However, very few practical applications of any of these methods can be identified and evaluated.

Hourly Traffic Data

As noted earlier, system level traffic assignments for average daily traffic often do not provide the detailed level of traffic data essential to conduct some necessary environmental analyses. Such analyses require traffic volumes to be forecasted for each of several hours of a typical weekday.

The procedures presented in this chapter can be modified to accommodate these environmental analysis requirements. Specifically, if it is reasonable to assume that the hourly distribution of trips will not change over time, an estimate of the base year hourly distribution on a facility may be used for forecasting purposes. Of course, the same caveats with respect to this assumption for DHV forecasting apply here, including stability of land-uses and trips served, no significant change in ADT, and no change in the degree of congestion.

Similarly, if the facility under analysis is a new facility or if it is necessary to assume that the hourly distribution of traffic may change, the future year hourly distribution should be based on hourly distributions from area facilities with similar characteristics. The only difference between the application of these procedures is that a statistical approach is probably necessary, with the cross-classification method being the most appropriate. This is because a volume forecast must be developed for several hours of a day. There are two known applications of this procedure for forecasting the hourly distribution of traffic, one displayed in Table A-12 for the Albuquerque region (70), and one developed for several urban areas, as displayed in Tables A-13 through 23 in the addendum to this chapter (88).

Peak Hour Factor

The peak hour factor (PHF) is another element of traffic data necessary for project planning that is often not provided by system level traffic assignments. The PHF is included in project planning considerations in order that the adequacy of preliminary highway designs can be evaluated throughout the entire design hour of volume. The PHF for freeways and expressways is the ratio of the traffic carried during the peak 5 minutes of the peak hour to the total traffic carried during the peak hour. The peak hour factor for all other arterials is the ratio of traffic carried during the peak 15 minutes of the peak hour to the total traffic carried during the peak hour. Thus, the PHF is a value always equal to or less than one. If the PHF is close to one, flow is fairly uniform throughout the peak hour. As the PHF decreases, the traffic volume peaks become steeper within the peak hour.

Typically, it is assumed that the forecasted PHF is the same as a base year PHF. The base year PHF is estimated in one of three ways. One method is to measure the PHF on the facility under analysis. Another method is the use of measurements of PHF's on similar facilities. The third method is to use overall average PHF's measured either for an entire urban area or estimated area based on urban area size from procedures in the Highway Capacity Manual (38).

ADDENDUM—HOURLY DISTRIBUTION OF TRAVEL

The following tables have been reproduced from NCHRP Report 187 (88) for convenient use by the analyst in applying the procedures described in this chapter. Additional discussion of these tables and related material is provided in the source document.

Table A-13. Hourly distribution of total travel on expressways/freeways: urbanized area population, 50,000-100,000.

HOUR	DISTRIBUTION & ORIENTATION BY SUBREGION						HOUR
	CBD & Central City				Suburb		
	Radial		X-Town		All Orientations		
	% ADT	DIR SPLT[b]	% ADT	DIR SPLT[b]	% ADT	DIR SPLT[b]	
24-1	1.0	26	1.5	48	1.5	60	24
1-2	0.5	28	1.0	46	1.0	66	1
2-3	0.5	34	1.0	46	0.5	52	2
3-4	0.5	38	1.0	44	0.5	54	3
4-5	0.5	54	1.0	44	0.5	34	4
5-6	2.5	62	2.0	48	1.5	24	5
6-7	5.5	60	4.0	52	3.5	26	6
7-8	7.0	56	7.0	62	6.5	40	7
8-9	5.5	56	5.0	48	5.0	52	8
9-10	5.5	50	5.0	46	5.0	58	9
10-11	5.5	48	5.5	48	5.0	62	10
11-12	5.5	50	5.5	48	5.0	62	11
12-13	5.0	50	5.5	48	5.0	56	12
13-14	5.5	54	5.0	48	6.5	56	13
14-15	6.0	54	5.5	48	6.5	54	14
15-16	6.5	56	7.0	46	7.0	54	15
16-17	8.5	40	7.5	42	8.0	50	16
17-18	7.0	40	7.0	38	8.5	50	17
18-19	6.0	42	5.5	44	7.0	36	18
19-20	4.5	44	4.5	42	5.0	40	19
20-21	3.5	48	4.0	44	3.5	42	20
21-22	3.0	48	3.5	46	3.0	46	21
22-23	2.5	46	3.0	50	2.5	44	22
23-24	2.0	34	2.5	52	2.0	54	23
	100.0		100.0		100.0		

a. Source: Reference (36) and nine urbanized area studies.
b. % in a.m. peak direction.

Table A-14. Hourly distribution of total travel on arterials: urbanized area population, 50,000- 100,000[a].

HOUR	DISTRIBUTION & ORIENTATION BY SUBREGION				HOUR
	Central City		Suburb		
	All Orientations		All Orientations		
	% ADT	DIR SPLT[b]	% ADT	DIR SPLT[b]	
24-1	0.5	38	1.0	52	24
1-2	0.5	40	1.0	48	1
2-3	0.0	34	1.0	46	2
3-4	0.0	42	0.5	58	3
4-5	0.0	54	1.0	42	4
5-6	0.5	66	2.0	46	5
6-7	1.5	78	3.0	60	6
7-8	7.0	70	6.0	70	7
8-9	2.5	58	4.5	56	8
9-10	1.5	52	4.0	56	9
10-11	1.0	52	5.0	62	10
11-12	2.0	50	5.0	46	11
12-13	2.0	50	5.0	50	12
13-14	2.0	52	6.0	46	13
14-15	4.5	38	6.5	44	14
15-16	15.5	34	7.0	48	15
16-17	20.0	46	9.0	46	16
17-18	13.0	46	8.5	40	17
18-19	7.5	48	6.5	48	18
19-20	11.0	48	5.5	44	19
20-21	4.0	46	4.0	42	20
21-22	1.5	52	3.5	42	21
22-23	1.5	56	2.5	46	22
23-24	0.5	40	2.0	52	23
	100.0		100.0		

a. Source: Reference (36) and nine urbanized area studies.
b. % in a.m. peak direction.

Table A-15. Hourly distribution of total travel on collectors: urbanized area population, 50,000- 100,000[a].

HOUR	DISTRIBUTION & ORIENTATION BY SUBREGION										HOUR
	CBD		Central City				Suburb				
	All Orientations		Radial		X-Town		Radial		X-Town		
	% ADT	DIR SPLT[b]	% ADT	DIR SPLT[b]	% ADT	DIR SPLT[b]	% ADT	DIR SPLT[b]	% ADT	DIR SPLT[b]	
24-1	1.0	50	1.0	50	1.0	46	1.0	54	1.0	56	24
1-2	0.5	50	0.5	52	0.5	42	0.5	60	0.5	66	1
2-3	0.5	50	0.5	54	0.5	34	0.5	52	0.5	62	2
3-4	0.5	-	0.0	50	0.0	56	0.5	52	0.0	52	3
4-5	0.5	54	0.5	56	0.0	58	0.5	52	0.5	46	4
5-6	1.0	58	1.0	58	1.0	62	1.0	52	1.0	44	5
6-7	2.5	60	3.5	58	3.0	58	3.5	56	3.5	66	6
7-8	6.0	62	7.0	58	6.5	60	6.5	56	8.0	54	7
8-9	6.0	64	4.5	56	4.0	54	4.5	54	5.0	50	8
9-10	7.0	60	4.5	54	4.0	50	4.5	52	4.5	44	9
10-11	6.0	54	4.5	52	4.5	48	5.0	50	5.0	48	10
11-12	6.0	56	5.0	50	5.0	46	5.0	50	5.0	52	11
12-13	6.0	56	5.5	50	5.5	48	6.0	52	5.0	50	12
13-14	6.0	52	5.5	50	5.5	50	6.0	52	5.5	44	13
14-15	6.5	52	6.0	50	6.0	48	6.0	50	6.0	48	14
15-16	6.5	50	6.5	46	7.0	46	6.0	48	7.0	52	15
16-17	6.5	44	8.0	48	8.5	44	8.0	46	9.0	50	16
17-18	6.0	42	7.5	46	7.5	44	7.5	46	7.5	46	17
18-19	5.5	50	7.0	50	7.0	50	6.5	52	6.5	46	18
19-20	5.5	52	6.0	50	7.5	50	6.0	54	5.5	54	19
20-21	4.5	48	5.0	48	6.0	46	5.0	50	4.5	44	20
21-22	4.5	46	4.5	44	4.5	48	4.0	50	4.0	50	21
22-23	3.5	50	3.5	48	3.0	52	3.5	50	3.0	56	22
23-24	2.0	50	2.5	48	2.0	46	2.5	52	2.0	58	23
	100.0		100.0		100.0		100.0		100.0		

a. Source: Reference (36) and nine urbanized area studies.
b. % in a.m. peak direction.

Table A-16. Hourly distribution of total travel on expressways/freeways: urbanized area population, 100,000-250,000[a].

HOUR	CBD All Orientations % ADT	CBD All Orientations DIR SPLT[b]	Central City Radial % ADT	Central City Radial DIR SPLT[b]	Central City X-Town % ADT	Central City X-Town DIR SPLT[b]	Suburb Radial % ADT	Suburb Radial DIR SPLT[b]	Suburb X-Town % ADT	Suburb X-Town DIR SPLT[b]	HOUR
24-1	1.5	46	1.0	26	1.5	48	2.0	52	2.0	50	24
1-2	1.0	50	0.5	28	1.0	46	1.5	50	1.5	48	1
2-3	1.0	50	0.5	34	1.0	46	1.0	44	0.5	44	2
3-4	1.0	54	0.5	38	1.0	44	1.0	48	0.5	48	3
4-5	1.0	56	0.5	54	1.0	44	1.0	50	0.5	52	4
5-6	3.0	66	2.5	62	2.0	48	2.0	54	1.0	64	5
6-7	5.5	62	5.5	60	4.0	52	3.5	58	5.5	64	6
7-8	7.5	64	7.0	56	7.0	62	5.5	64	10.0	56	7
8-9	6.0	64	5.5	56	5.0	48	6.0	60	6.0	64	8
9-10	5.0	60	5.5	50	5.0	46	5.5	54	4.5	54	9
10-11	5.0	56	5.5	48	5.5	48	6.0	54	4.0	52	10
11-12	4.5	54	5.5	50	5.5	48	6.0	50	4.0	50	11
12-13	4.5	54	5.0	50	5.5	48	6.0	50	4.0	50	12
13-14	4.5	56	5.5	50	5.0	48	6.0	50	4.0	50	13
14-15	5.5	52	6.0	54	5.5	48	6.0	50	4.5	54	14
15-16	7.0	50	6.5	46	7.0	46	6.0	54	7.5	50	15
16-17	8.5	46	8.5	40	7.5	42	7.0	44	10.0	46	16
17-18	7.5	44	7.0	40	7.0	38	7.0	40	9.0	42	17
18-19	5.0	52	6.0	42	5.5	44	6.0	40	5.5	48	18
19-20	4.5	54	4.5	44	4.5	42	4.0	48	4.5	48	19
20-21	3.5	52	3.5	48	4.0	44	3.5	46	3.5	50	20
21-22	3.0	48	3.0	48	3.5	46	3.0	48	3.0	50	21
22-23	2.5	50	2.5	46	3.0	50	2.5	52	2.5	50	22
23-24	2.0	48	2.0	34	2.5	52	2.0	54	2.0	50	23
	100.0		100.0		100.0		100.0		100.0		

a. Source: Reference (36) and nine urbanized area studies.
b. % in a.m. peak direction.

Table A-17. Hourly distribution of total travel on arterials: urbanized area population, 100,000- 250,000.

HOUR	CBD All Orientations % ADT	CBD All Orientations DIR SPLT[b]	Central City Radial % ADT	Central City Radial DIR SPLT[b]	Central City X-Town % ADT	Central City X-Town DIR SPLT[b]	Suburb Radial % ADT	Suburb Radial DIR SPLT[b]	Suburb X-Town % ADT	Suburb X-Town DIR SPLT[b]	HOUR
24-1	1.0	38	1.0	40	1.0	44	1.0	50	1.0	44	24
1-2	0.5	40	0.5	44	0.5	44	0.5	50	0.5	50	1
2-3	0.5	34	0.5	42	0.5	46	0.5	50	0.5	42	2
3-4	0.5	42	0.5	48	0.5	48	0.0	50	0.5	52	3
4-5	0.5	54	0.5	52	0.5	54	0.5	50	0.5	48	4
5-6	1.0	66	1.0	64	1.0	64	1.0	62	1.0	66	5
6-7	4.0	78	3.0	70	3.0	58	2.5	66	2.5	66	6
7-8	8.0	70	7.0	68	7.5	56	7.0	74	7.5	68	7
8-9	7.0	58	5.5	58	5.5	56	6.0	66	6.0	52	8
9-10	6.0	52	5.0	52	5.0	54	5.0	56	4.5	50	9
10-11	6.0	52	5.0	50	5.5	54	5.0	54	4.5	48	10
11-12	6.5	50	5.5	48	5.5	50	5.5	50	5.5	48	11
12-13	6.5	50	6.0	50	6.0	50	6.0	48	5.5	52	12
13-14	6.5	52	6.0	50	5.5	50	6.0	50	5.5	50	13
14-15	5.5	38	6.5	52	6.5	50	6.0	50	5.5	50	14
15-16	6.0	34	7.5	48	7.5	46	7.0	48	6.5	50	15
16-17	8.0	46	8.5	42	8.0	46	8.0	42	7.5	46	16
17-18	7.5	46	8.0	38	7.5	46	8.5	36	9.0	36	17
18-19	4.5	48	5.5	44	6.0	50	6.5	42	6.5	42	18
19-20	4.0	48	5.0	48	5.0	48	5.5	48	5.5	44	19
20-21	3.5	46	4.0	48	4.0	48	4.0	48	5.0	46	20
21-22	3.0	52	3.5	54	3.5	46	3.5	44	4.0	50	21
22-23	2.0	56	2.5	46	2.5	48	2.5	48	3.0	48	22
23-24	1.5	40	2.0	48	2.0	48	2.0	50	2.0	46	23
	100.0		100.0		100.0		100.0		100.0		

a. Source: Reference (36) and nine urbanized area studies.
b. % in a.m. peak direction.

Table A-18. Hourly distribution of total travel on collectors: urbanized area population, 100,000- 250,000.

HOUR	Central City All Orientations % ADT	Central City All Orientations DIR SPLT[b]	Suburb All Orientations % ADT	Suburb All Orientations DIR SPLT[b]	HOUR
24-1	0.5	38	1.0	50	24
1-2	0.5	40	0.5	50	1
2-3	0.0	34	0.5	50	2
3-4	0.0	42	0.0	50	3
4-5	0.0	54	0.0	50	4
5-6	0.5	66	1.0	20	5
6-7	1.5	78	3.5	24	6
7-8	7.0	70	10.5	40	7
8-9	2.5	58	7.5	66	8
9-10	1.5	52	4.5	58	9
10-11	1.0	52	4.5	50	10
11-12	2.0	50	5.0	46	11
12-13	2.0	50	6.0	48	12
13-14	2.0	52	6.0	42	13
14-15	4.5	38	4.5	46	14
15-16	15.5	34	3.5	20	15
16-17	20.0	46	9.0	54	16
17-18	13.0	46	6.5	48	17
18-19	7.5	48	65	48	18
19-20	11.0	48	6.0	48	19
20-21	4.0	46	4.5	50	20
21-22	1.5	52	4.0	50	21
22-23	1.5	56	3.0	50	22
23-24	0.5	40	2.0	50	23
	100.0		100.0		

a. Source: Reference (36) and nine urbanized area studies.
b. % in a.m. peak direction.

Table A-23. Hourly distribution of total travel on arterials: urbanized area population, 750-000- 2,000,000.

HOUR	DISTRIBUTION & ORIENTATION BY SUBREGION										HOUR
	CBD		Central City				Suburb				
	All Orientations		Radial		X-Town		Radial		X-Town		
	% ADT	DIR SPLT[b]	% ADT	DIR SPLT[b]	% ADT	DIR SPLT[b]	% ADT	DIR SPLT[b]	% ADT	DIR SPLT[b]	
24-1	1.5	46	1.5	46	1.5	40	1.5	44	1.5	44	24
1-2	1.0	46	1.0	48	1.0	44	1.0	40	1.0	42	1
2-3	0.5	44	0.5	48	0.5	48	0.5	44	0.5	44	2
3-4	0.5	42	0.5	50	0.5	42	0.5	50	0.5	50	3
4-5	1.0	54	0.5	56	0.5	54	0.5	58	0.5	54	4
5-6	2.0	58	1.5	62	1.5	64	2.0	66	1.0	60	5
6-7	4.0	60	5.0	63	5.0	68	5.5	72	3.5	64	6
7-8	9.0	64	8.5	68	8.5	74	8.0	70	7.5	60	7
8-9	7.0	66	6.5	66	6.5	54	5.5	62	6.0	56	8
9-10	5.0	60	4.5	58	4.5	54	4.5	56	4.5	52	9
10-11	5.5	54	5.0	54	4.0	54	4.5	52	5.0	52	10
11-12	6.0	54	5.0	52	4.5	48	4.5	52	5.0	50	11
12-13	5.5	50	5.0	52	5.0	50	4.5	50	5.0	50	12
13-14	5.5	50	5.0	52	5.0	52	5.0	52	5.0	50	13
14-15	6.0	48	5.5	50	5.5	56	5.5	52	5.5	50	14
15-16	6.5	46	6.5	48	7.0	52	6.5	48	7.0	48	15
16-17	9.5	42	9.0	40	9.0	36	9.5	42	8.5	44	16
17-18	7.0	38	8.0	36	8.0	42	8.5	36	7.5	42	17
18-19	4.5	44	5.0	46	5.5	50	6.0	44	6.0	46	18
19-20	3.5	46	4.0	52	4.5	54	4.5	50	5.5	48	19
20-21	2.5	46	3.5	48	3.5	52	3.5	48	4.5	48	20
21-22	2.5	46	3.0	48	3.5	48	3.5	48	4.0	46	21
22-23	2.0	44	3.0	48	3.0	52	2.5	48	3.0	50	22
23-24	2.0	46	2.5	48	2.0	46	2.0	46	2.0	50	23
	100.0		100.0		100.0		100.0		100.0		

a. Source: Reference (36) and nine urbanized area studies.
b. % in a.m. peak direction.

Table A-24. Hourly distribution of total travel on collectors: urbanized area population, 750,000- 2,000,000.

HOUR	DISTRIBUTION & ORIENTATION BY SUBREGION						HOUR
	CBD		Central City		Suburb		
	All Orientations		All Orientations		All Orientations		
	% ADT	DIR SPLT[b]	% ADT	DIR SPLT[b]	% ADT	DIR SPLT[b]	
24-1	1.5	46	2.0	46	1.5	52	24
1-2	1.0	46	1.0	48	0.5	50	1
2-3	0.5	52	0.5	50	0.5	46	2
3-4	0.5	54	0.5	50	0.0	50	3
4-5	1.0	60	1.0	54	0.5	69	4
5-6	2.5	64	1.5	58	1.0	70	5
6-7	4.5	68	4.0	62	3.5	72	6
7-8	10.5	62	8.5	64	8.0	68	7
8-9	7.5	60	6.0	62	7.0	56	6
9-10	3.5	58	4.5	56	4.5	52	9
10-11	5.5	56	4.5	52	4.5	52	10
11-12	6.0	54	4.5	50	5.0	54	11
12-13	5.0	54	5.0	50	5.0	50	12
13-14	5.0	52	4.5	50	5.0	54	13
14-15	5.5	54	5.0	46	5.5	54	14
15-16	6.5	50	6.5	44	6.5	43	15
16-17	9.0	40	10.5	36	9.5	38	16
17-18	7:5	34	9.5	34	9.0	40	17
18-19	4.5	43	5.0	42	6.0	50	13
19-20	3.0	48	4.5	50	5.0	43	19
20-21	2.5	46	3.0	44	4.0	50	20
21-22	2.0	46	3.0	44	3.0	50	21
22-23	1.5	46	2.5	46	3.0	52	22
23-24	1.5	44	2.5	46	2.0	50	23
	100.0		100.0		100.0		

a. Source: Reference (36) and nine urbanized area studies.
b. Percent in a.m. peak direction.

CHAPTER TEN
DIRECTIONAL DISTRIBUTION PROCEDURES

GENERAL

A type of traffic data that is essential to project planning and design but is not provided by system-level traffic assignments is the directional distribution of traffic during the peak hour. It is generally accepted that in urban areas, future change in directional distribution must be expected and should be accounted for in project planning. In particular, whether the peak hour traffic volume by direction is balanced or is unbalanced will have substantial effect on the adequacy of alternative highway designs.

There are two basic types of procedures that may be applied to forecasting directional distribution on highway links. One is used by the Maryland State Highway Administration for peak hour traffic directional distribution forecasts. It involves modifying base year data to reflect future conditions. The other procedure was developed for short-cut "sketch planning" hourly traffic directional distribution estimates and is documented in NCHRP Report 187 (88). It is not known to be used for project planning and design directional distribution forecasts, but may have potential for use. A procedure to apply directional distribution to the adjustment of intersection link volumes is described as the final section of this chapter. Finally, a peak hour traffic assignment may be utilized as discussed in Chapter 9. This method will not be elaborated on because directional distributions are obtained directly from the peak hour assignment.

PROCEDURE USING MODIFICATION OF BASE YEAR DATA

A procedure is used to forecast future peak hour traffic directional distribution using modifications to base year data. Two alternative bases are defined for this modification. The first requires substantially more input data than the other, but is easier to interpret. This more data-intensive modification is based on the comparison of base year and future year home-based work trips in a production-attraction format. This enables analysis of the likely change in home-to-work travel in the AM peak hour and work-to-home travel in the PM peak hour by direction on the facility under analysis. The second, less data-intensive modification is based on the comparison of base year and forecast year land-uses and/or total work trip productions and attractions in the traffic-shed of the facility under analysis. This technique is also intended to permit conclusions to be reached regarding the likely change in home-to-work travel in the AM peak hour by direction and work-to-home travel in the PM peak hour by direction on the facility under analysis.

The procedure is applicable to nearly any urban facility. Use of the procedure on a new facility is somewhat difficult, however, as the base year directional distribution and work travel comparisons must be conducted on base year facilities in the travel corridor(s) from which the new facility will draw traffic.

The procedure is only appropriate for facilities that are dominated by work travel during the peak hour. The use of work travel as the basis for peak hour directional distribution modification results in this limitation. The more data-intensive approach has quite extensive input data requirements, including full trip tables, thereby restricting its use to situations where sufficient data are available. The less-data intensive approach requires fewer traffic volumes and either trip end summaries or land-use estimates.

Basis for Development

This undocumented procedure was developed for use by the Maryland State Highway Administration in conducting planning and design studies. The practical basis for the procedure rests on the fact that changes in work travel patterns on most urban facilities will define the changes in peak hour traffic volume directional distribution.

Input Data Requirements

For the data-intensive approach, the following base year and future year data are required:

- System-level traffic assignments within study area.
- Highway network with identified minimum time paths.
- Home-based work trip tables arranged in production-attraction format.

The less data-intensive approach requires the following base year and future year data:

- Traffic estimates on facility under analysis.
- Zonal home-based work trip ends arranged in production-attraction format for study area.

or

- Residential and employment-related land-uses stratified by zones in the traffic shed.

Directions for Use

Data Intensive Procedure

The following are step-by-step directions for the procedure using the more data-intensive approach to directional distribution modification.

Step 1--Obtain Estimate of Base Year Directional Distribution of Peak Hour Traffic. For an existing facility an estimate need only be made for the facility itself. For a new facility, estimates should be developed for each facility in the corridor(s) from which the new facility will draw trips.

Step 2--Determine the Directional Distribution of Home-to-Work Travel During the Peak Traffic Hour in the Base and Future Years. This would be accomplished by assigning base and future year home-based work trip tables in a production-attraction format to the minimum time paths identified for their respective system-level traffic assignments. The work trips assigned by direction for the base and future years would represent the relative proportion of work travel by direction during the AM peak hour. When reversed by direction, it would represent the relative proportion of work travel by direction during the PM peak hour.

Step 3--Establish the Reasonableness of Base Year Estimated Peak Hour Traffic Directional Distribution Given the Base Year Work Travel Directional Distribution. This step is used as a reasonableness check. Generally, if the peak hour traffic directional distribution is within 10 percent of the work travel directional distribution, it can be considered reasonable.

Step 4--Forecast Future Year Directional Distribution By Factoring Base Year Directional Distribution. This step can be accomplished in two ways. One way is by judgmentally estimating the difference between the base and future year work trip directional distributions and then adjusting the base year total peak hour traffic distribution by a proportional amount. The other way is to factor the base year total traffic directional distribution as follows:

Table A-19. Hourly distribution of total travel on expressways/freeways: urbanized area population, 250,000-750,000.

HOUR	DISTRIBUTION & ORIENTATION BY SUBREGION										HOUR
	CBD		Central City				Suburb				
	All Orientations		Radial		X-Town		Radial		X-Town		
	% ADT	DIR SPLT[b]	% ADT	DIR SPLT[b]	% ADT	DIR SPLT[b]	% ADT	DIR SPLT[b]	% ADT	DIR SPLT[b]	
24-1	1.5	46	2.0	46	1.5	44	2.0	44	2.0	50	24
1-2	1.0	50	1.0	50	1.0	46	1.5	48	1.5	48	1
2-3	1.0	50	1.0	50	0.5	42	1.5	54	0.5	44	2
3-4	1.0	54	1.0	54	0.5	50	1.5	52	0.5	48	3
4-5	1.0	56	1.0	56	1.0	60	2.0	58	0.5	52	4
5-6	3.0	66	2.0	66	2.0	60	2.5	56	1.0	64	5
6-7	5.5	62	4.5	62	5.0	64	4.5	60	5.5	64	6
7-8	7.5	64	6.0	64	8.0	62	5.5	68	10.0	56	7
8-9	6.0	64	5.0	64	6.5	60	5.0	60	6.0	64	8
9-10	5.0	60	5.0	60	5.0	56	5.5	60	4.5	54	9
10-11	5.0	56	5.0	56	4.5	54	5.5	50	4.0	52	10
11-12	4.5	54	5.0	54	4.5	52	5.5	52	4.0	50	11
12-13	4.5	54	5.0	54	5.0	52	5.5	52	4.0	50	12
13-14	4.5	56	5.5	56	5.0	52	5.5	50	4.0	50	13
14-15	5.5	52	6.5	52	6.0	52	6.0	50	4.5	54	14
15-16	7.0	50	7.5	50	7.0	48	6.5	50	7.5	50	15
16-17	8.5	46	8.5	46	8.5	44	7.0	46	10.0	46	16
17-18	7.5	44	7.5	44	7.5	42	6.5	44	9.0	42	17
18-19	5.0	52	5.0	52	5.5	48	4.5	42	5.5	48	18
19-20	4.5	54	4.0	54	4.0	50	4.0	52	4.5	48	19
20-21	3.5	52	3.5	52	3.5	46	3.5	52	3.5	50	20
21-22	3.0	48	3.5	48	3.5	44	3.0	50	3.0	50	21
22-23	2.5	50	3.0	50	7.5	46	3.0	48	2.5	50	22
23-24	2.0	48	2.0	48	2.0	42	2.5	48	2.0	50	23
	100.0		100.0		100.0		100.0		100.0		

a. Source: Reference (36) and nine urbanized area studies.
b. % in a.m. peak direction.

Table A-20. Hourly distribution of total travel on arterials: urbanized area population, 250,000-750,000.

HOUR	DISTRIBUTION & ORIENTATION BY SUBREGION										HOUR
	CBD		Central City				Suburb				
	All Orientations		Radial		X-Town		Radial		X-Town		
	% ADT	DIR SPLT[b]	% ADT	DIR SPLT[b]	% ADT	DIR SPLT[b]	% ADT	DIR SPLT[b]	% ADT	DIR SPLT[b]	
24-1	1.0	50	1.5	40	1.5	40	1.5	32	1.5	50	24
1-2	1.0	50	0.5	44	0.5	44	1.0	34	0.5	56	1
2-3	0.5	50	0.5	42	0.5	48	1.0	34	0.0	50	2
3-4	0.5	52	0.5	48	0.5	42	0.5	44	0.5	52	3
4-5	0.5	54	0.5	56	0.5	54	1.0	52	1.0	64	4
5-6	2.0	58	2.0	54	1.0	64	2.5	70	2.0	72	5
6-7	5.0	60	5.0	68	4.5	68	6.0	72	6.0	82	6
7-8	7.0	64	7.0	70	6.5	74	5.5	68	6.5	68	7
8-9	6.5	64	5.5	64	5.5	54	4.5	60	4.5	60	8
9-10	5.0	58	4.5	58	4.5	54	5.0	56	4.0	58	9
10-11	5.5	54	5.0	52	4.5	54	5.0	54	4.0	54	10
11-12	5.5	52	5.0	52	5.0	48	5.0	50	4.5	54	11
12-13	5.5	52	5.0	50	5.5	50	5.0	50	5.0	48	12
13-14	5.5	52	5.0	50	5.5	52	5.5	52	5.0	50	13
14-15	6.0	52	6.0	52	6.0	56	6.0	54	6.0	52	14
15-16	8.0	50	7.5	42	7.0	52	6.5	46	7.0	44	15
16-17	9.0	44	8.0	38	8.5	36	$\boxed{8.5}$	42	8.0	36	16
17-18	6.5	42	8.0	38	7.5	42	7.5	38	8.5	36	17
18-19	4.5	50	6.0	48	6.0	50	6.0	48	6.5	48	18
19-20	4.0	52	5.0	50	5.5	54	4.5	50	5.5	54	19
20-21	3.5	48	4.0	44	4.5	52	4.0	46	4.5	50	20
21-22	3.0	46	3.5	42	4.0	48	3.5	46	4.0	38	21
22-23	2.5	50	2.5	46	3.0	52	2.5	46	3.0	30	22
23-24	2.0	52	2.0	42	2.0	46	2.0	46	2.0	32	23
	100.0		100.0		100.0		100.0		100.0		

a. Source: Reference (36) and nine urbanized area studies.
b. % in a.m. peak direction.

Table A-21. Hourly distribution of total travel on collectors: urbanized area population, 250,000-750,000.

HOUR	DISTRIBUTION & ORIENTATION BY SUBREGION						HOUR
	CBD		Central City		Suburb		
	All Orientations		All Orientations		All Orientations		
	% ADT	DIR SPLT b	% ADT	DIR SPLT b	% ADT	DIR SPLT b	
24-1	1.0	44	1.5	44	1.0	46	24
1-2	0.5	38	0.5	50	0.5	50	1
2-3	0.5	33	0.5	46	0.5	76	2
3-4	0.5	50	0.5	58	0.5	70	3
4-5	0.5	62	0.5	74	0.5	86	4
5-6	1.5	72	1.5	80	2.0	89	5
6-7	5.5	68	4.5	76	5.5	84	6
7-8	8.5	66	6.5	66	6.5	74	7
8-9	6.0	58	5.0	64	4.5	56	8
9-10	5.5	54	4.5	66	4.0	60	9
10-11	5.5	50	4.5	62	4.5	52	10
11-12	6.5	48	5.0	58	5.0	52	11
12-13	6.0	48	5.5	56	5.5	46	12
13-14	6.5	56	5.5	58	5.5	52	13
14-15	7.5	56	6.0	58	6.0	54	14
15-16	8.0	50	7.5	56	7.5	40	15
16-17	7.5	38	8.5	52	8.0	34	16
17-18	5.5	40	7.5	50	8.0	32	17
18-19	3.5	48	6.0	54	6.5	46	18
19-20	4.0	48	5.5	56	5.5	50	19
20-21	3.5	56	4.5	56	4.5	50	20
21-22	2.0	62	4.0	58	3.5	44	21
22-23	2.0	56	2.5	58	2.5	46	22
23-24	2.0	50	2.0	52	2.0	48	23
	100.0		100.0		100.0		

a. Source: Reference (36) and nine urbanized area studies.
b. % in a.m. peak direction.

Table A-22. Hourly distribution of total travel on expressways/freeways: urbanized area population, 750,000-2,000,000.

HOUR	DISTRIBUTION & ORIENTATION BY SUBREGION										HOUR
	CBD		Central City				Suburb				
	All Orientations		Radial		X-Town		Radial		X-Town		
	% ADT	DIR SPLT b	% ADT	DIR SPLT b	% ADT	DIR SPLT b	% ADT	DIR SPLT b	% ADT	DIR SPLT b	
24-1	1.5	46	1.5	46	1.5	44	1.5	44	2.0	50	24
1-2	1.0	50	1.0	50	1.0	46	1.0	44	1.5	48	1
2-3	0.5	52	0.5	52	0.5	42	0.5	46	0.5	44	2
3-4	0.5	54	0.5	54	0.5	50	0.5	46	0.5	48	3
4-5	1.5	56	0.5	56	1.0	60	1.0	50	0.5	52	4
5-6	3.5	58	1.5	58	2.0	60	2.0	60	1.0	64	5
6-7	6.0	54	5.5	54	5.0	64	5.5	72	5.5	64	6
7-8	8.5	58	9.0	58	8.0	62	8.5	76	10.0	56	7
8-9	5.5	54	7.0	54	6.5	60	6.0	68	6.0	54	8
9-10	3.5	50	5.0	50	5.0	56	4.5	54	4.5	54	9
10-11	3.5	46	4.5	46	4.5	54	4.5	52	4.0	52	10
11-12	4.0	46	4.5	46	4.5	52	4.5	50	4.0	50	11
12-13	4.5	40	4.5	40	5.0	52	4.5	48	4.0	50	12
13-14	4.0	44	5.5	44	5.0	52	4.5	52	4.0	50	13
14-15	5.5	46	5.5	46	6.0	52	5.0	50	4.5	54	14
15-16	7.5	40	7.0	40	7.0	48	7.0	54	7.5	50	15
16-17	9.5	34	8.5	34	8.5	44	9.0	46	10.0	46	16
17-18	7.0	36	7.5	36	7.5	42	8.0	36	9.0	42	17
18-19	5.0	44	5.5	44	5.5	48	5.5	38	5.5	48	18
19-20	4.5	48	4.0	48	4.0	50	4.5	46	4.5	48	19
20-21	4.0	50	3.0	50	3.5	46	3.5	50	3.5	50	20
21-22	3.5	48	3.0	48	3.5	44	3.0	46	3.0	50	21
22-23	3.0	48	3.0	48	2.5	46	3.0	44	2.5	50	22
23-24	2.5	48	2.0	48	2.0	42	2.5	48	2.0	50	23
	100.0		100.0		100.0		100.0		100.0		

a. Source: Reference (36) and nine urbanized area studies.
b. % in a.m. peak direction.

$$DD_F = DD_B * (WT_F/WT_B) \quad \text{(A-48)}$$

where:

DD_F = future year traffic directional distribution;

DD_B = base year traffic directional distribution;

WT_F = future year work trip directional distribution; and

WT_B = base year work trip directional distribution.

If possible, consideration should be given to whether future work travel will constitute the same proportion of total peak hour travels in the base year, and whether the future peak hour direction split of non-work travel will be the same as in the base year. If not, additional judgmental manual adjustments should be performed.

Less Data-Intensive Procedure

The following are step-by-step directions for the procedure using the less data-intensive approach to directional distribution modification.

Step 1--Obtain Estimate of Base Year Directional Distribution of Peak Hour Traffic. This step is the same as described above for the more data-intensive approach.

Step 2--Compare the Base Year and Future Year Distribution of Home-Based Work Productions (or Residential Land Uses) and Home-Based Work Attractions (or Employment-Related Land Uses) Within the Study Area. The intent of this step is to establish whether the basic pattern of home-to-work travel is changing. For example, a concentration of productions at one end of a corridor and attractions at the other end should indicate an imbalance in directional distribution. If in the future this pattern would remain essentially the same, the base year traffic directional distribution could be assumed to be unchanged. Conversely, if attractions were expected to be interspersed with the productions (i.e., more uniform land-use), there would be a basis for assuming that the future traffic directional distribution imbalance would be reduced. Select link analysis (Chapter 4) can often be used to help identify traffic patterns within the study area. It can also be used to help split trips into through and local travel movements.

Step 3--Forecast Future Year Directional Distribution Based on Comparisons Between Base and Future Year Data. This step is best performed judgmentally by using the results from Step 2 to estimate changes in trip patterns. The base year directional distribution (Step 1) will either be the same in the future or be manually adjusted to reflect these trip pattern changes. This adjustment should be performed separately for different trip components (e.g., through trips, local trips) and then combined into an aggregated directional distribution.

Example Problem

The following is an example using both approaches to directional distribution forecasts. The example is a four-lane arterial in an urban area of over 2,000,000 in population. The facility is within a suburb, but should be considered a central city location given the type and density of its adjacent development. The arterial as shown in Figure A-78 has commercial development adjacent development. The arterial as shown in Figure A-78 has commercial development immediately adjacent to it, including a regional level shopping center. Some office development is also situated along the arterial, but such development is particularly concentrated at the east end. At the east end are two major river crossings to the area's CBD.

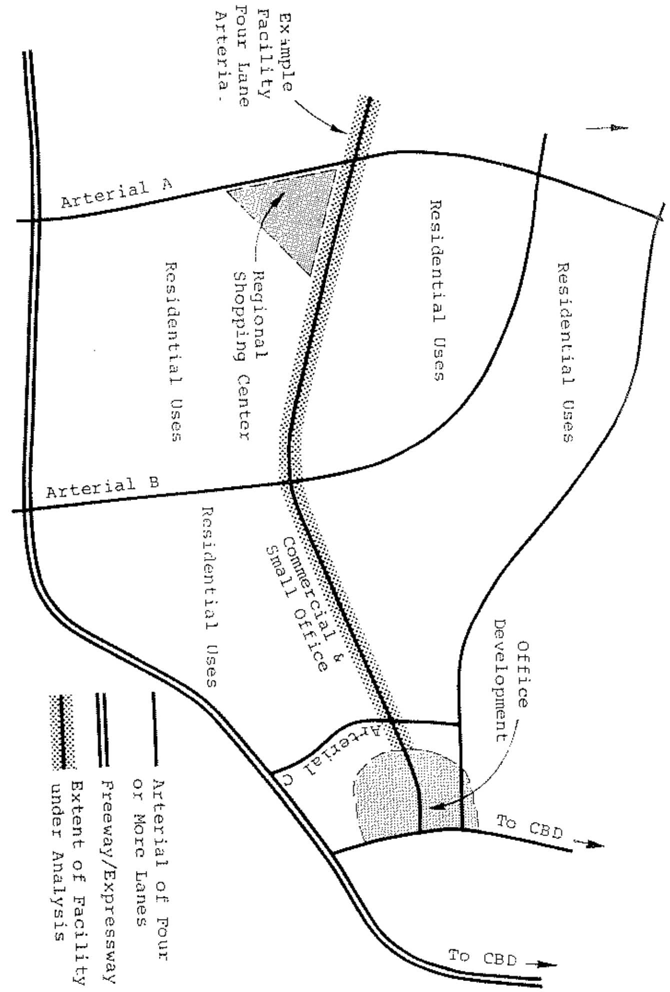


Figure A-78. Existing land uses adjacent to example facility.

The existing office development site is forecasted to increase only slightly, as is office development in the CBD. However, other office development is forecasted to increase significantly throughout the remainder of the corridor, as shown in Figure A-79. These new office developments would each be as large as the existing concentration of office development at the east end of the corridor. These sites would not, however, be comparable in size to the office development in the CBD area.

Data-intensive Approach

The following are the same step-by-step directions for the more data-intensive approach applied to this example:

Step 1--Obtain Estimate of Base Year Directional Distribution. To the west of Arterial C, base year directional distribution of traffic along the facility in the AM peak hour varies from 80 percent-20 percent to 85 percent-15 percent along its entire length. Within the concentration of office development to the east of Arterial C the AM peak hour directional distribution is between 65 percent-35 percent and 60 percent-40 percent.

In the PM peak hour the directional distribution varies from 70 percent-30 percent to 75 percent-25 percent to the west of Arterial C and about 50 percent-50 percent within the office development. These were obtained from base year traffic counts.

Step 2--Determine the Directional Distribution of Home-to-Work Travel in the Base and Future Years. For the base year, the directional distribution of produced-attracted work trips was determined to be about 90 percent-10 percent along the entire arterial. For the future year the directional distribution of produced-attracted work trips along the arterial was determined to be about 80 percent-20 percent between Arterials A and B and 70 percent-30 percent between Arterials B and C.

This distribution was established by assigning base and future year home-based work trip tables in a production-attraction format to the minimum time paths identified for their respective system level traffic assignments. Zonal tree analyses were performed (see Chapter 4) to identify these paths.

Step 3--Establish the Reasonableness of Base Year Estimated Peak Hour Traffic Directional Distribution Given the Base Year Work Travel Directional Distribution. In this step, the base year work travel directional distribution of 90 percent-10 percent is compared to the AM total traffic distribution of between 80 percent-20 percent and 85 percent-15 percent and the PM total traffic distribution of between 70 percent-30 percent and 75 percent-25 percent. It is concluded that the base year data is reasonable as it is generally within a 10 percent difference.

Step 4--Forecast Future Year Directional Distribution By Factoring Base Year Directional Distribution. The work traffic directional distribution indicated that the total traffic directional distribution between Arterials A and B should be factored down by a ratio of about 80 percent/90 percent or 0.90, and between Arterials B and C by a ratio of about 70 percent/90 percent or about 0.80.

Thus, the forecasted AM total traffic directional distributions should be projected to be about 70 percent to 80 percent in the AM peak direction between Arterials A and B, and 60 percent to 70 percent between Arterials B and C. The forecasted PM total traffic directional distribution should be projected to be about 70 percent to 75 percent in the peak direction between Arterials A and B and 60 percent to 65 percent in the peak direction between Arterials B and C.

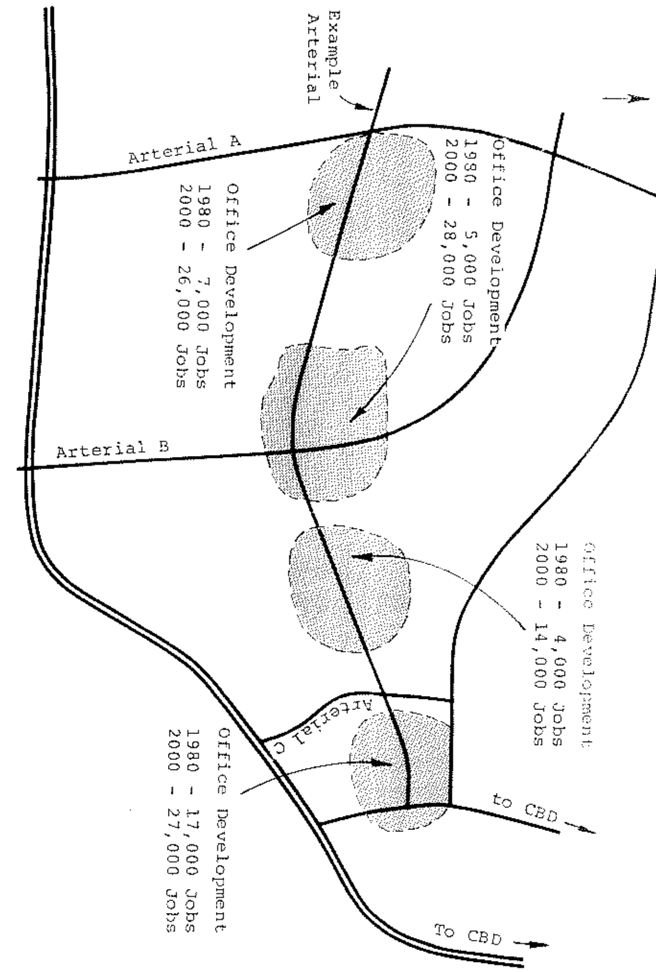


Figure A-79. Forecasted land uses adjacent to example facility.

Less Data-intensive Approach

This example problem could also be approached through the less data-intensive procedure. Under this procedure, the following steps would be used:

Step 1--Obtain Estimate of Base Year Directional Distribution of Peak Hour Traffic. As discussed above the directional split in the AM peak direction is about 80 percent to 85 percent and in the PM peak direction is 70 percent to 75 percent.

Step 2--Compare the Base Year and Future Year Distribution of Home-based Work Productions (or Residential Land-Uses) and Home-based Work Attractions (or Employment-Related Land-Uses) Within the Study Area. This comparison is summarized in Figure A-80. The principal change in the future is employment will be spread throughout the study area. As a result, traffic that has one trip end within the study area will probably be more evenly distributed by direction during the peak hour. Traffic passing through the corridor to the CBD, however, can be expected to continue to be oriented as in the base year. Select link analyses (see Chapter 4) indicate that through traffic on the example facility can be expected to be about 50 percent of total traffic in subarea A and only about 33 percent of total traffic in subarea D.

Step 3--Forecast Future Year Directional Distribution Based on Comparisons Between Base and Future Year Data. The future year directional distribution of trips must be estimated separately for through traffic and for internal traffic originating and/or terminating in the study area.

For the portion of the facility in Area A:

Peak Direction Percentage = % of Through Traffic x Through direction % + % of Internal Traffic x Internal Direction %

$$= .50 \times 0.95 + 0.50 \times 0.60$$

$$= 0.75 = 75\% \qquad \text{(A-49)}$$

For the portion of the facility in Area B:

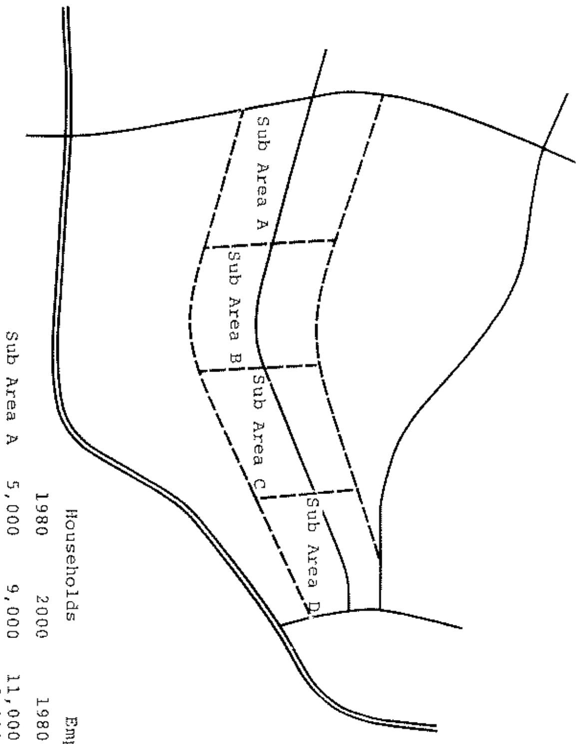
Peak Direction Percentage $= 0.33 \times 0.95 + 0.67 \times 0.50 = 0.65 = 65\%$

Thus, the directional distribution in the peak hours along this arterial using this approach would be forecasted to be 65 percent to 75 percent in the peak direction. This result is very similar to the 60 percent to 80 percent range in the AM peak hour and 60 percent to 75 percent range in the PM peak hour obtained with the more data-intensive procedure.

PROCEDURE USING ANTICIPATED FUTURE CONDITIONS

This procedure forecasts the directional distribution of peak hour traffic on a facility based on its anticipated future characteristics that are known to influence peak hour directional distribution. The procedure may involve the use of statistical analyses such as cross-classification tables and regression equations. Specifically, it can use a cross-classification table of peak hour directional distributions stratified by the facility characteristics established to have the greatest influence. A regression equation with peak-hour peak directional-directional distributions as the dependent variables and facility characteristics as the independent variables may be developed instead.

The advantage of this statistical approach is that it clearly identifies and quantifies the facility characteristics that have been assumed to influence the peak hour directional distribution. The principal disadvantage of this statistical approach, like that used for design hour volume, is its data requirements. A very large peak hour counting program may be necessary for its proper



	Households 1980	Households 2000	Employment 1980	Employment 2000
Sub Area A	5,000	9,000	11,000	32,000
Sub Area B	6,000	11,000	6,000	30,000
Sub Area C	3,000	7,000	4,000	14,000
Sub Area D	3,000	4,000	17,000	27,000

Figure A-80. Forecast residential and employment changes adjacent to example facility.

development and maintenance. For example, a cross-classification table with five directional distributions stratified by five different characteristics requires sufficient data for the calculation of 25 average directional distributions. Special counting programs will probably be necessary to satisfy this data requirement because data will be required for each of the various facility classifications. Care must be taken that this approach is not applied blindly without judgment. This is important in this case because the large data requirements of this approach may dictate that certain of the factors which may marginally influence directional distribution can not be included in a model. Similarly, certain average directional distributions may end up being based on very limited actual traffic data.

The alternative to a statistical approach is what will be called the judgmental approach. It requires the person responsible for the peak hour directional distribution forecast to be aware of the factors that influence directional distribution and their degree of influence. This knowledge would be obtained from a review of existing directional distributions. A peak hour counting program by direction is also required under this approach, but it may not need to be as extensive. For example, if a directional distribution forecast is required for facility with certain characteristics, and if no base year count data were available for such facilities, special counts could be taken on specific facilities that have the appropriate characteristics. The disadvantage of this approach is that the forecast is so totally dependent on the judgment of the person responsible.

This procedure applied either in a statistical or judgmental approach is applicable to any typical urban facility. It is particularly useful for analyzing new facilities or existing facilities for which it is necessary to assume that the base year, peak hour directional distribution will change by the future year. The primary assumption used in the procedure is that selected highway facility design, location, and use characteristics can explain much of the variation in highway facility peak hour directional distribution.

Basis for Development

This procedure is based on materials developed as part of NCHRP Report 187 (88). It therefore represents a quick-response sketch-planning tool for use in producing peak hour directional traffic forecasts. It uses factors such as facility location and orientation and the size of urban area population to estimate these distributions. The factors and resulting directional distributions are reproduced in Tables A-13 through A-24 in the addendum to Chapter 9.

Input Data Requirements

The data required to apply this procedure are the following:

- Future year forecasted peak hour traffic (two-way total).
- Estimated future year facility characteristics (e.g., type, location, orientation to CBD, adjacent land-uses).
- Base year directional distributions on facilities with similar characteristics to those of future facility.

The base year directional distribution data should be derived if possible from actual ground counts or estimated, if necessary, from data from other years. The future year peak hour traffic should be taken from the results of the refinement and detailing procedures in Chapters 4 through 7 as modified. The time-of-day procedures are documented in Chapter 9. The future year facility characteristics should be obtained from design plans or land-use projections.

Directions for Use

The following are step-by-step directions for developing and applying this directional distribution forecasting procedure.

Step 1--Identify the Highway Facility Characteristics Which Influence Directional Distribution and the Degree of Influence of Each Characteristic

The primary emphasis in this step is to determine which highway facility characterictics will influence the future directional distribution. Once this is accomplished, the next task is to quantify the degree of influence of each characteristic such that subsequent adjustments can be made. Facility type has generally been determined to correlate with peak hour directional distribution. The typical stratifications used for arterial facility type are freeways/expressways, major arterials, and minor arterials.

Facility location within the urban area also influences peak hour directional distribution. The typical stratifications used for urban facility location include central business district (CBD), central city, and suburban.

A third influential characteristic is facility orientation with respect to the CBD. The typical stratifications employed are radial and crosstown. These stratifications only apply to facilities located outside of the CBD.

A fourth facility characteristic that has been considered as correlating with the peak hour directional distribution is adjacent land-use. The key considerations with regard to land-use are land use type (e.g., employment, residential), intensity of use (e.g., number of dwelling units per square mile in zone), and location of land-use (e.g., concentrated in one location or spread throughout the study area).

Step 2--Select a Peak Hour Directional Distribution Based on the Anticipated Characteristics of the Facility

For the statistical approach this step requires the development of regression equations or cross-classification tables. For the judgmental approach it requires an examination of the peak directional distribution of existing facilities with characteristics similar to those of the facility under analysis. Special counts may be required.

Step 3--Multiply the Future Estimated Peak Hour Directional Distribution by the Future Year Peak Hour Total Traffic

This step involves the use of a simple equation, as follows:

$$DD_{estimate} * PHT_{future} = DPHT_{future} \qquad \text{(A-50)}$$

where:

$DD_{estimate}$ = estimated future year directional distribution (expressed as percent);

PHT_{future} = future peak hour traffic (total both directions); and

$DPHT_{future}$ = directional future peak hour traffic.

The future PHT value is an input to the procedure, while the directional distribution (DD) is obtained from Steps 1 and 2.

PROCEDURE TO ADJUST INTERSECTION DIRECTIONAL LINK VOLUMES

The results of directional and hourly distribution of traffic should be given special attention when applied to turning movement analyses. The results of link analyses may not balance when considering the volumes into and out of an intersection or node. A procedure is presented here to balance directional link volumes at each approach to an intersection.

Each intersection approach (link) has an inbound and outbound movement to be considered (except for one-way links). For a four-way intersection, shown in Figure A-81, the eight movements are labeled by compass position (N, E, S, W) and the directional movements in relation to the intersection are labeled inbound or outbound (I, O). The hours of the day are represented by the subscript i, where i may range from 1 to 24, depending on the analysis used. Table A-25 depicts this situation. For example, the outbound traffic volume on the east approach between 7 AM and 8 AM , is designated by EO_8. The total volume for all hours is denoted by the subscript T (e.g., NI_T, EO_T) and represents the sum of the traffic volumes across all hours (i.e., from 1 to 24 hours). As an example, the total inbound traffic for the north approach (NI_T) over a 24-hour period would equal the following:

$$NI_T = NI_1 + NI_2 + NI_3 + \dots\dots\dots NI_{23} + NI_{24} \qquad \text{(A-51)}$$

A link's inbound and outbound traffic are then combined to determine the link's two-way volume. For example, the link total on the west approach (W_T) is calculated as follows:

$$W_T = WI_T + WO_T \qquad \text{(A-52)}$$

The difference between inbound and outbound traffic (IO_i) is:

$$IO_i = NI_i + EI_i + SI_i + WI_i - NO_i - EO_i - SO_i - WO_i \qquad \text{(A-53)}$$

Each hour's total inbound volume (I_i) is the sum of the inbound volume of the four links during that hour. Therefore, $I_i = NI_i + EI_i + SI_i + WI_i$. Finally, one-half of the sum of the totals on the four links is equal to the total intersection inbound traffic (TI) or outbound traffic (TO):

$$TI = TO = (NT + ET + ST + WT)/2 \qquad \text{(A-54)}$$

With the above basic terminology, the following procedure can be applied to balance the directional volumes at an intersection.

Basis for Development

The basis for this procedure is that the total directional link traffic heading inbound to an intersection must equal the total traffic heading outbound from that intersection. A straightforward computation is used to adjust the inbound and outbound traffic flows, keeping constant the relative distribution of each directional volume. Assume that the inbound volume on one approach represents 30 percent of the total inbound for the entire intersection. Then the percentage (30 percent) would remain the same for that approach throughout the calculation process, even though the actual magnitude of that inbound volume might change.

A similar logic is used to adjust the outbound volumes and to adjust volumes across several hours of data. In the latter step, the hourly percentage initially used (e.g., peak hour equals 10 percent of 24 hour) would remain constant. Again, the actual hourly volume may change, but not its distribution. In this manner, the matrix of volumes depicted in Table A-25 would be adjusted across the rows (inbound and outbound) and along the columns (hours) in a systematic fashion.

This procedure first adjusts the inbound volume totals. These inbound adjustments are

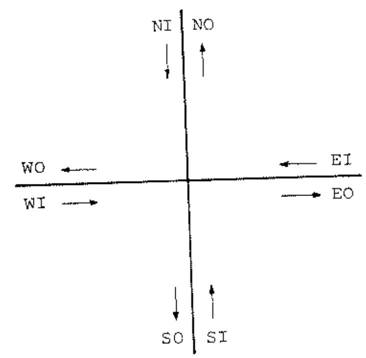


Figure A-81. Intersection link volumes.

Table A-25. Intersection hourly direction link volumes.

Time Period	NI_i	NO_i	EI_i	EO_i	SI_i	SO_i	WI_i	WO_i	IO_i
1	NI_1	NO_1	EI_1	EO_1	SI_1	SO_1	WI_1	WO_1	IO_1
2	NI_2	NO_2	EI_2	EO_2	SI_2	SO_2	WI_2	WO_2	IO_2
3	NI_3	NO_3	EI_3	EO_3	SI_3	SO_3	WI_3	WO_3	IO_3
.	.	.	.	.	.	.	.	.	.
.	.	.	.	.	.	.	.	.	.
.	.	.	.	.	.	.	.	.	.
.	.	.	.	.	.	.	.	.	.
i	NI_i	NO_i	EI_i	EO_i	SI_i	SO_i	WI_i	WO_i	IO_i
Total	NI_T	NO_T	EI_T	EO_T	SI_T	SO_T	WI_T	WO_T	IO_T
2-way Total	N_T		E_T		S_T		W_T		

apportioned to each hour, followed by the outbound adjustments. As a result, the outbound volumes are constrained to match the inbound volumes, rather than vice versa. Identical calculations could be performed by adjusting the outbound volumes first. The differences in these approaches are usually negligible for volumes within each hour, and certainly so for the total volumes.

Input Data Requirements

As shown in Figure A-81 and Table A-25, the input data required are: hourly directional link volumes on each intersection approach (from 1 to 24 hours). These volumes are obtained from applying the link directional distribution procedures described previously in this chapter and the time-of-day procedures presented in Chapter 9. The number of required directional volumes is equal to twice the number of intersection approaches.

Directions for Use

A six-step computational procedure is described below for any number of hours. An example of a 3-hour analysis follows. The results of this procedure applied to a 24-hour scenario is described in the case study in Chapter 16.

Step 1—Check Volume Totals

The purpose of this step is to make sure that mathematical errors were not made in the calculation or display of the initially assumed hourly directional volumes. In a 24-hour scenario, the columns (I_i and O_i) in Table A-25 should be summed to produce the I_T and O_T values for each approach (e.g., north approach: NI_T and NO_T). These values should then be compared with the 24-hour volumes initially assumed or forecasted on the link. The I_T and O_T values can be summed for each approach (e.g., north approach: $NI_T + NO_T = N_T$) for comparison with the actual or forecasted two-way 24-hour volume totals. If these values are close to each other (i.e., plus or minus 5 percent), the time-of-day (Chapter 9) or directional distribution (Chapter 10) link calculations should be rechecked for errors.

Step 2--Calculate the Difference Between the Inbound and Outbound Movements

This step is first performed for the total directional volumes. The following equation is used:

$$IO_T = NI_T + EI_T + SI_T + WI_T - NO_T - EO_T - SO_T - WO_T \quad \text{(A-55)}$$

where IO_T equals difference between total inbound and total outbound trips.

Next perform the same calculation for each of the hourly volumes. For example, in hour 2:

$$IO_2 = NI_2 + EI_2 + SI_2 + WI_2 - NO_2 - EO_2 - SO_2 - WO_2 \quad \text{(A-56)}$$

The value of IO_i (or IO_T) will be positive if the inbound trips exceed the outbound trips, and negative if outbound trips exceed inbound trips.

Several possibilities can occur at this point:

- If IO_T equals zero, and if the IO_i for each hour equal zero, the distribution is balanced and the procedure is finished.
- If IO_T equals zero, but the IO_i for one or more hours do not equal zero, proceed to Step 5 of the procedure.
- If IO_T does not equal zero, proceed to Step 3.

Step 3—Adjust the Total Inbound Trips Among Approaches

Determine the number of trips by which each of the four inbound total trips must be increased or decreased. This is done proportionally based on the movement's proportion of total inbound trips. If the IO_T is greater than zero, the inbound trips must be reduced by one-half of this difference. If IO_T is less than zero, the inbound trips must be increased by one-half of this difference. For example, the change in the west link's total inbound trips (C_{WI}) would be:

$$C_{WI} = -(IO_T/2)\,(WI_T)/(NI_T + EI_T + SI_T + WI_T) \quad \text{(A-57)}$$

The change in the north link's inbound trips (C_{NI})

$$C_{NI} = -(IO_T/2)\,(NI_T)/(NI_T + EI_T + SI_T + WI_T) \quad \text{(A-58)}$$

The value of C_{WI} or C_{NI} may be positive or negative, using the opposite sign from IO_T.

The adjusted * total volumes then equal the following:

$$I_T{}^* = I_T + C_I \quad \text{(A-59)}$$

or, for the north approach:

$$NI_T{}^* = NI_T + C_{NI} \quad \text{(A-60)}$$

where C_{NI} may be positive or negative.

These calculations are performed for all approaches to produce $EI_T{}^*$, $SI_T{}^*$, and $WI_T{}^*$.

Step 4--Distribute the Total Inbound Volume Change Among the Hourly Inbound Volumes

Distribute the change in total inbound trips for each approach over each of the hours according to the same distribution initially applied to develop the hourly volumes. Add or subtract these trips from each hourly volume according to the results of Step 2 to find the adjusted (*) hourly inbound volumes. For example, for hour 3, the following computations are performed for the north approach:

$$NI_3{}^* = NI_T{}^*\,(NI_3/NI_T) \quad \text{(A-61)}$$

or, in more general format:

$$NI_i{}^* = NI_T{}^*\,(NI_i/NI_T) \quad \text{(A-62)}$$

Where:

i	=	hour;
$NI_T{}^*$	=	adjusted total inbound volume from Step 3; and
NI_i, NI_T	=	original (unadjusted) volumes.

The results of this step are a set of adjusted inbound volumes (i.e., $NI_i{}^*$, $EI_i{}^*$, $SI_i{}^*$, $WI_i{}^*$ for all hours i) on all approaches for all hours.

Step 5—Calculate Adjusted Outbound Movements for Each Hour

First calculate the total inbound volume (I_i) for each hour and for the total as follows:

$$I_i{}^{(*)} = NI_i{}^{(*)} + EI_i{}^{(*)} + SI_i{}^{(*)} + WI_i{}^{(*)} \quad \text{(A-63)}$$

where the (*) indicates that the approach inbound volumes may be either the adjusted values obtained from Step 4 or the original volumes if the analysis has moved directly from Step 2 (i.e., $IO_T = 0$ but $IO_i \neq 0$).

Next, distribute the $I_i(*)$ totals among the four outbound movements according to each outbound movement's proportion of the total original outbound traffic. This is calculated by the following, assuming the north approach is an example:

$$NO_i^* = NO_i \times I_i/(NO_i + EO_i + SO_i + WO_i) \qquad \text{(A-64)}$$

where NO_i^* is the adjusted outbound volume on the north approach, and NO_i, EO_i, SO_i, and WO_i are the original outbound volumes. This calculation is repeated for the other approaches. At this point the total inbound and outbound volumes are identical within each hour and for the totals.

Step 6--Make Final Checks

As a final check for reasonableness, the adjusted outbound volumes (NO_i^*, EO_i^*, SO_i^*, and WO_i^*) should be summed for all hours. The following equation is used, again assuming the north approach:

$$NO_T^* = \Sigma\ NO_i^* \text{ for all hours } i \qquad \text{(A-65)}$$

where NO_T^* is the adjusted total peak period outbound volume for the north approach.

These volumes should be added to produce a total adjusted outbound volume:

$$O_T^* = NO_T^* + EO_T^* + SO_T^* + WO_T^* \qquad \text{(A-66)}$$

The O_i^* should equal the I_i^* value calculated in Step 5. Otherwise, an error has been made. If possible, these adjusted totals should then be compared with the actual or forecasted outbound volumes, as in Step 1. This is most readily performed if the volumes have been computed across each of 24 hours, such that the totals can be compared with actual or forecasted directional ADT values.

The outbound volume totals should fall within 5 percent of the actual or forecasted value. Otherwise, an error has probably occurred and the calculations should be rechecked. It is possible at this point to factor the adjusted outbound volumes up or down to better match the actual or forecasted values. However, the analyst would then need to readjust the inbound volumes using a procedure identical, but reversed, to Steps 3 and 4. Usually this effort does not significantly improve the results.

Example Problem

Directional link volume data have been estimated for 3 hours during the PM peak period. A peak period (3-hr) two-way link forecast is also available. These data are given in Table A-26, referring to Figure A-81 for nomenclature. The task is to adjust the link volumes to create a balanced set of volumes to be used in turning movement analyses.

The following steps are used:

Step 1

The approach inbound and outbound volumes are summed for all hours and compared with the peak period forecast. For the north approach, the following calculations occur:

NI_T	=	1,000 + 1,200 + 1,250	= 3,450
NO_T	=	700 + 900 + 1,100	= 2,700
N_T	=	3,450 + 2,700	= 6,150
Peak Period Forecast			= 6,300

The 6,150 estimated total compares favorably with the 6,300 forecast (within 2 percent). The other totals are also within tolerable limits.

Step 2

The differences between the inbound and outbound volumes are calculated, as shown in Table A-26.

IO_T = 3,450 + 4,800 + 4,950 + 2,550 - 2,700 - 3,300 - 3,400 - 6,150 = +200
IO_3 = 1,000 + 1,500 + 1,700 + 700 - 700 - 1,000 - 900 - 2,000 = +300
IO_4 = 1,200 + 1,600 + 1,650 + 900 - 900 - 1,200 - 1,200 - 2,050 = 0
IO_5 = 1,250 + 1,700 + 1,600 + 950 - 1,100 - 1,100 - 1,300 - 2,100 = -100

IO_T does not equal zero. IO_3 and IO_5 also do not equal zero. Therefore, proceed to Step 3.

Step 3

Because IO_T is greater than zero (i.e., +200), the inbound trips must be reduced by one-half this amount (200/2 = 100) for each approach:

Given:
$IO_T/2$ = +200/2 = +100
$NI_T + EI_T + SI_T + WI_T$ = 3,450 + 4,800 + 4,950 + 2,550 = 15,750
Then:
C_{NI} = -(3,450/15,750)(+100) = - 22
C_{EI} = -(4,800/15,750)(+100) = - 30
C_{SI} = -(4,950/15,750)(+100) = - 32
C_{WI} = -(2,550/15,750)(+100) = - 16
-100 checks

Therefore,
NI_T^* = 3,450 - 22 = 3,428
ET_T^* = 4,800 - 30 = 4,770
SI_T^* = 4,950 - 32 = 4,918
WI_T^* = 2,550 - 16 = 2,534

Step 4

The adjusted inbound volumes are now apportioned to each hour. For example, on the north approach:

NI_3^*	= NI_T^* (NI_3/NI_T) = 3,428 (1,000/3,450)	=	994
Similarly, NI_4^*	= 3,428 (1,200/3,450)	=	1,192
NI_5^*	= 3428 (1,250/3,450)	=	1,242
			3,428 check

The results of these calculations and those on the other approaches are given in Table A-27.

Table A-26. Initial link volumes and forecasts.

Start Time	NI_i	NO_i	EI_i	EO_i	SI_i	SO_i	WI_i	WO_i	IO_i
3 PM	1,000	700	1,500	1,000	1,700	900	700	2,000	+ 300
4 PM	1,200	900	1,600	1,200	1,650	1,200	900	2,050	0
5 PM	1,250	1,100	1,700	1,100	1,600	1,300	950	2,100	- 100
3HR TOTAL	3,450	2,700	4,800	3,300	4,950	3,400	2,550	6,150	+ 200
2-way Total	6,150		8,100		8,350		8,700		
Peak Period Forecast (2-way)	6,300		8,150		8,400		9,000		

Table A-27. Balanced link volumes.

Start Time	NI_i*	NO_i*	EI_i*	EO_i*	SI_i*	SO_i*	WI_i*	WO_i*	I_i*
3 PM	994	741	1,491	1,059	1,689	953	696	2,117	4,870
4 PM	1,192	894	1,590	1,192	1,639	1,192	894	2,037	5,315
5 PM	1,242	1,073	1,689	1,073	1,590	1,269	944	2,050	5,465
Total	3,428	2,708	4,770	3,324	4,918	3,414	2,534	6,204	15,650
2-way Total	6,136		8,094		8,332		8,738		
Peak Period Forecast (2-way)	6,300		8,150		8,400		9,000		
Comparison	OK		OK		OK		OK		

Step 5

To calculate the adjusted outbound volumes, first calculate the inbound totals for each hour across all approaches.

$I_3* = 994 + 1{,}491 + 1{,}689 + 696 = 4{,}870$

$I_4* = 1{,}192 + 1{,}590 + 1{,}639 + 894 = 5{,}315$

$I_5* = 1{,}242 + 1{,}689 + 1{,}590 + 944 = \underline{5{,}465}$

$\text{Total} = 15{,}650 = I_T*$

$I_T* = 3{,}428 + 4{,}770 + 4{,}918 + 2{,}534 = 15{,}650$ check

Next, compute the adjusted outbound volumes. For the 3 PM hour, the following computations are made:

Given:

$NO_3 + EO_3 + SO_3 + WO_3 = 700 + 1{,}000 + 900 + 2{,}000 = 4{,}600$

$I_3* = 4{,}870$

Then:

$NO_3* = (700/4{,}600)\ 4{,}870 = 741$

$EO_3* = (1{,}000/4{,}600)\ 4{,}870 = 1{,}059$

$SO_3* = (900/4{,}600)\ 4{,}870 = 953$

$WO_3* = (2{,}000/4{,}600)\ 4{,}870 = \underline{2{,}117}$

4,870 check

Similar computations are performed for hours 4 and 5, with the results given in Table A-27.

Step 6

As a final check, the total adjusted outbound volumes are computed as follows:

$NO_T* = 741 + 8{,}941 + 1{,}073 = 2{,}708$

$EO_T* = 1{,}059 + 1{,}192 + 1{,}073 = 3{,}324$

$SO_T* = 953 + 1{,}192 + 1{,}269 = 3{,}414$

$WO_T* = 2{,}117 + 2{,}037 + 2{,}050 = \underline{6{,}204}$

$O_T* = 15{,}650$ equals the I_T* from Step 5

The outbound totals cannot be directly compared with the two-way forecasted values; however, the sum of the adjusted inbound plus outbound volumes can be compared. For instance,

$NI_T* + NO_T* = 3{,}428 + 2{,}708 = 6{,}136$

which still compares favorably with the forecasted value of 6,300. The other approach totals are also reasonable. Therefore, the intersection volumes are balanced for all hours of study.

CHAPTER ELEVEN
VEHICLE CLASSIFICATION PROCEDURES

GENERAL

A critical type of traffic data needed for highway project planning and design is vehicle classification data. These data typically include various stratifications of light, medium, and heavy-duty vehicles occurring on a facility during specified hours of an average weekday. Vehicle classification data are necessary to perform capacity analyses, pavement design, and environmental analyses.

The typical procedure used to forecast vehicle classification on a facility is to assume that the base year classification of the facility will not change. The base year vehicle classification may be determined through direct measurement, or estimated from data available on facilities that have similar characteristics to the facility under analysis. If a future facility does not exist in the base year, measurement of base year vehicle classification on the facility obviously cannot be made. However, base year measurements can be made on adjacent facilities from which the new facility is expected to draw traffic.

The major weakness of this procedure is that it neglects future land-use changes in the facility study area. These changes may affect the future vehicle classification. In an attempt to abate this possible problem, a revised procedure will be described in this chapter adding one step to the typically used procedure. This step includes an adjustment factor to account for the effects of forecasted land-use changes. Emphasis is placed on those land-uses known to influence, or to be correlated with, truck trip generation. The revised procedure is applicable to any urban facility. However, special considerations will be required if a new, or significantly upgraded, facility is examined.

Long term vehicle classification trends may also be important. For instance, for several years there has been a relative increase in the percentage of 5-axle semi-trailers compared with 4-axle semis. The inclusion of such statewide or localized trends in the forecasting process will improve the estimates of future year vehicle classifications.

BASIS FOR DEVELOPMENT

The basis of the procedure is that vehicle classification on a facility is only likely to change in the future if the adjacent land-uses change such that a substantially different number of truck trips is generated. The procedure was based on discussions with various public agencies and from synthesis from various documents (16, 20, 27, 40, 42, 48, 86).

INPUT DATA REQUIREMENTS

The required data inputs are the following:

- Base year and future year land-uses.
- Base year vehicle classification counts.

The land-use data should concentrate on changes in those uses, such as retail, industrial, or manufacturing, which are most likely to generate truck traffic. Residential land-use data should be obtained for comparison purposes. Future year land-use forecasts will be of assistance. The base year classification counts should be obtained if possible on the facility under analysis; otherwise, counts on similar adjacent facilities may be substituted. If counts in the base year are not available, counts from other years may be adjusted as necessary to reflect base year considerations.

DIRECTIONS FOR USE

The following are step-by-step directions for the forecasting of vehicle classification of a facility.

Step 1—Select Base Year Vehicle Classification

A suitable base year vehicle classification estimate should be selected from the available input data, as discussed above.

Step 2--Compare Base Year and Future Land-Uses

The purpose of this step is to determine whether the land-use changes between the base year and future year are significant enough to produce a change in the vehicle classification. The relative proportion of land-uses that generate truck traffic (e.g., retail, industrial, and manufacturing) should be compared to land-uses that generate automobile traffic (e.g., housing units). If possible, the land-uses should be analyzed separately for different zones along the facility, such that land-use trends can be established.

For more detailed truck analyses, long term vehicle classification trends available at the state or local level can be extrapolated to the futue year. The results of this trend analysis should then be compared for reasonableness with the land-use changes forecasted to occur.

Step 3—Estimate the Future Year Vehicle Classification

This step may be judgmentally performed by manually adjusting the base year vehicle classification to account for changing land-use trends. For instance, if industrial land-uses are expected to increase substantially, the analyst may decide to increase the facility truck percentages. The amount of the change would be based on the analyst's judgment and knowledge of vehicle classifications in similar heavy industrial areas.

A more systematic approach is to calculate a new vehicle classification using actual or percentage changes in relative land-use intensities. Typical values used for comparison are number of employees, square footage of development, and population. The following example uses a technique that estimates trip generation in the base year and future year for selected trip purposes and modes. The trips generated are then compared to determine a change in auto and truck utilization, resulting in a revision of the base year vehicle classification to represent future year conditions. Subsequent adjustments to account for long term vehicle classification trends could be made for more detailed studies.

EXAMPLE PROBLEM

The following is an example of the application of this procedure to estimate an average weekday truck percentage. The facility under consideration is a two-lane arterial at the fringe of an urban area. It presently has an AWDT of 8,000 vehicles per weekday. It is surrounded principally by low-to-medium density residential land-uses and is not within the influence of any major travel generator. Traffic is expected to increase to 15,000 vehicles per weekday within 20 years, and as a result, the facility is being considered for widening to four lanes. The expected increase in AWDT is a result of anticipated land-use changes in the facility corridor. Largely the change is expected to be a uniform increase in the residential land-uses throughout the corridor. The only exception is that a major industrial park having an employment total of 1,000 persons is expected to be located along the arterial and in about the middle of the facility segment. The industrial park will be designed such that access is provided principally to and from the arterial. From current counts on the facility, the base year truck percentage of total weekday traffic is measured to be 4 percent.

The recommended procedure for vehicle classification would be applied as follows:

1. Compare base year and future year land-uses to establish whether the vehicle classification will change. These comparisons were discussed above in the introduction to this example. The addition of the industrial park is expected to increase the percentage of truck trips on the facility. The magnitude of this increase will be computed in Step 2.

2. Estimate the future truck percentage based on land-use change. The analyst determines that this task is best performed by splitting the new trips associated with the industrial park from the forecast AWDT of 15,000.

a. Estimated industrial park total weekday trip generation.

<u>Auto trips</u> (Auto occupancy - 1.2; no transit)

Home-based work purpose	=	1,200 trips
All other purposes	=	<u>1,500</u> trips
Total auto trips	=	2,700 trips
<u>Truck Trips</u>		
Total truck trips	=	600 trips
<u>Total Vehicle Trips</u>	=	3,300 trips

These estimates are based on standard trip generation rates by purpose and mode.

b. Estimate distribution of industrial park trips on facility. Since the park is located in the middle of the facility, it will be assumed that the distribution is 50%-50%, or 0.50 * 3300 = 1650 average industrial park trips on the facility. This includes 0.5 * 600 = 300 truck trips.

c. Split industrial park trips from total AWDT.

In each direction,

Total AWDT	=	15,000
Industrial Park Trips	=	<u>1,650</u>
Other Trips	=	13,350

d. Estimate revised truck percentage assuming that the base year percentage of truck traffic will hold for all but the industrial park trips.

<u>Truck Trips on Facility</u>

From Industrial Park = 300 truck trips

For Other Trips = 13,350 * 0.04 = 534 trips

Therefore:

$$\text{Revised truck percentage} = \frac{(13{,}350 * .04 + 300)}{15{,}000} = \frac{534 + 300}{15{,}000} = 0.06$$

= 6 percent trucks

CHAPTER TWELVE
SPEED, DELAY, AND QUEUE LENGTH PROCEDURES

GENERAL

Traffic data essential to highway project planning and design include speed, delay, and queuing data. These data are necessary to perform project planning studies, user cost analyses, and environmental studies.

This chapter presents procedures for estimating speeds, delay, and queuing on grade separated facilities (i.e., freeways) and on surface arterials. Separate procedures are developed for under-capacity and over-capacity conditions. In each of these situations the traffic flow characteristics are different.

Speed can be defined in a number of ways: average speed, average running speed, operating speed, and design speed (<u>91</u>). Average speed is the commonly used speed in project planning and is defined as the total distance traversed by a vehicle divided by the total time required, including all traffic delays. Average running speed is the average speed of a vehicle only while it is in motion. Where there are no delays causing a vehicle to stop, these speeds are identical. Operating speed is defined as the highest overall speed at which a vehicle can travel under favorable weather conditions and under prevailing traffic conditions without exceeding a safe speed as determined by the design speed. Design speed is defined as the speed upon which the safe operation of vehicles is dependent and is related to the highway's curvature, superelevation, and sight distance (<u>38</u>).

These speeds are considered to be related in the following way (<u>91</u>):

$$ARS = OS - (DS/10\ (1 - V/C) \qquad \text{(A-67)}$$

where:

ARS = average running speed (or average speed if no stops);
OS = operating speed;
DS = design speed;
V = volume; and
C = highway capacity (level of service E).

Care must be taken that each speed is expressed in the same units (e.g., mi/hr; km/hr).

Over-capacity conditions involve vehicle demand on a facility exceeding its capacity, resulting in a build-up of a queue of vehicles. The queue of vehicles will exist and increase as long as demand exceeds capacity. Thus, if the cause of the demand-capacity imbalance was minor or temporary, such as the sudden braking and stopping of a vehicle for an animal in the roadway or a merging platoon of vehicles, the queue may be very small and dissipate quickly. However, if the queue is a regular occurrence at a bottleneck, such as a lane drop on a freeway, the queue will continue until the off-peak hours are reached where traffic demands are less than bottleneck capacity. Typically, speeds in over-capacity conditions will average less than 30 miles per hour on grade separated facilities and 15 miles per hour on surface arterials.

On grade-separated facilities queues do not develop in under-capacity situations. On surface arterials, however, some queuing occurs at traffic signals under all conditions. This queuing is directly related to intersection delay, which is a component of average speed. A queue is typically defined either in terms of the number of vehicles in a backup or in terms of a standard distance measure (e.g., feet, meter, mile). Delay is expressed in unit terms (e.g., minutes/vehicle) or in overall terms (e.g., vehicle-hours). These elements will be examined separately in this chapter.

UNDER-CAPACITY CONDITIONS

Under-capacity conditions are typified by uninterrupted traffic flow on grade-separated facilities and along mid-block sections of surface arterials. There are various degrees of uninterrupted, or continuous, flow (38). Different roadways provide different types of marginal, or side, frictions. For instance, a well-designed freeway provides minimal friction, whereas a surface arterial may have many side streets and driveways that can disrupt traffic flow. As a result, different speed relationships are developed for various highway configurations.

On surface arterials traffic flow in under-capacity conditions is affected by such factors as speed limits, mid-block frictions and the operation of traffic signals. These factors contribute to a situation that is more complex to analyze than under-capacity conditions on grade separated facilities. Therefore, separate procedures are described.

Speed Procedure for Grade-Separated Facilities

The forecast of speed on a grade-separated facility is based on the design speed and forecasted volume-to-capacity ratio on the facility. Typical relationships between average running speed, design speed, and volume-to-capacity ratio have been established for use in this analysis. These relationships can be used to forecast average running speed under any situation except for over-capacity or bottleneck conditions. Procedures to be applied under such conditions will be described later in this chapter.

Basis for Development

The basis of this approach is that the volume-to-capacity ratio on a grade separated facility is theoretically known to influence average running speed. Also, observations of freeway operations have indicated that the volume-to-capacity ratio explains nearly all of the variation in average running speed on most freeways and expressways.

The relationships used in this procedure are based on revised highway capacity procedures presented in TRB Circular No. 212 (45). Similar relationships using operating speed rather than average running speed are presented in the 1965 Highway Capacity Manual (38).

Input Data Requirements

The only input data requirement necessary to estimate grade-separated facility average running speed is the design speed of the facility and its volume-to-capacity ratio. The design speed is available from functional design plans. The volume-to-capacity ratio should be calculated using forecasted traffic volumes that have been refined or detailed using procedures developed in Chapters 4 through 7. The capacity should be the one used in the computer forecast or modified in subsequent analyses. Both the volume and capacity values should be for a one-hour duration during the peak or off-peak period as required by the environmental or planning analysis.

Directions for Use

The following is a two-step procedure to forecast average running speed on a grade separated facility:

Step 1: Apply design speed and volume-to-capacity ratio relationships to estimate average running speed. Typical relationships between speed and volume-to-capacity ratio can be described conveniently and accurately by a series of curves or with tabulated data. These relationships are summarized in Figure A-82 and Table A-28 (45).

Equations may also be used to forecast speed based on the volume-to-capacity ratio. Three different forms of such equations have been used in the traffic assignment step of system-level travel forecasting procedures (124). The three equations are as follows:

$$\text{BPR Equation } S = So/(1 + a(V/Cp)^4) \qquad (A\text{-}68)$$

$$\text{Smock Equation } S = So/e^{(V/Cp - 1)} \qquad (A\text{-}69)$$

$$\text{Schneider Equation } S = So/2^{(V/Cp - 1)} \qquad (A\text{-}70)$$

where:

S = Forecasted speed
So = Speed at practical capacity (Level of Service C)
a = Constant
V = Volume
C_p = Practical capacity
e = Exponential function

These equations should not be used for forecasting average running speed for under-capacity conditions for project planning, because their forecasting accuracy is limited. Given that the fitting of any equation to the curves in Figure A-82 will involve some inaccuracy in predicting speed from the volume-to-capacity ratio, and because the speed is only affected by the volume-to-capacity ratio over a very narrow range, it is recommended that equations not be developed and used for speed forecasting in project planning. Rather, the curves shown in Figure A-82 and tabulated in Table A-28 should be used in a "look-up" format if necessary.

Step 2: Convert average running speed to operating speed, if necessary. Once the average running speed is determined (Step 1), the operating speed can be calculated using Eq. A-67. The average running speed, design speed, and V/C ratio are inputs to compute the operating speed.

Example Problem

The following is an example of speed determination for an under-capacity freeway. A new six-lane roadway is proposed to be built with geometrics designed for 60 mph. During the peak hour the highway is anticipated to operate at a 0.6 volume-to-capacity ratio.

To determine the anticipated operating speed, Figure A-82 is first used to determine the average running speed. Enter the graph on the horizontal scale at the anticipated V/C ratio of 0.6 and move vertically to the family of curves for the 60-mph design speed (Step 1). Continue to move vertically to the middle curve which represents a six-lane facility. From the point where the vertical line meets this curve, move horizontally to the left to read the average running speed of 47 mph. As an alternative method, Table A-28 could be used by entering the column labeled for 60-mph Design Speed Six-Lane and moving down to the row for a V/C Ratio of 0.6, yielding the 47-mph average running speed. In Step 2, Eq. A-67 is modified to solve for operating speed, as follows:

$$OS = ARS + (DS/10(1 - V/C))$$
$$OS = 47 + (60/10(1 - 0.6))$$
$$OS = 49.4 \text{ mph}$$

Table A-28. Average running speeds on freeways/expressways.

V/C Ratio	AVERAGE RUNNING SPEED (mph) 70-mph Design Speed 8 Lane	70-mph 6 Lane	70-mph 4 Lane	60-mph Design Speed 8 Lane	60-mph 6 Lane	60-mph 4 Lane	50-mph Design Speed All Lanes
.35	54	54	53	51	50	50	46
.40	54	54	53	51	50	50	46
.45	54	54	53	50	50	49	45
.50	54	54	53	49	49	48	45
.55	54	54	53	48	48	47	44
.60	54	54	52	47	47	46	43
.65	53	53	51	46	46	45	42
.70	53	53	51	45	45	45	40
.75	52	52	50	44	44	44	38
.80	51	51	49	42	42	42	36
.85	49	49	48	40	40	40	34
.90	47	47	46	38	38	38	32
.95	43	43	43	35	35	35	30
.95	43	43	43	35	35	35	30
1.00	32	32	32	30	30	30	27

Source: Ref. 45

Speed and Delay Procedure for Surface Arterials

The calculation of speed on surface arterials must include the delay at traffic signals. As a result, there are two components of the arterial speed forecasting procedure. One component provides a forecast of "mid-block" speed, or the average running speeds between traffic signals. The other component provides a forecast of delay at traffic signals. The average speed is computed by combining the mid-block speed forecast and the intersection delay forecast.

This procedure may be applied for any under-capacity conditions. Over-capacity conditions require different techniques for forecasting intersection delay. These procedures are discussed later in this chapter.

Basis for Development

The procedure uses material recommended in A Manual on User Benefit Analysis of Highway and Bus-Transit Improvements published by AASHTO (90), and based on procedures developed in NCHRP Report 133 (91). The technique for mid-block speed forecasts assumes speed to be related to the volume-to-capacity ratios of the intersections along the arterial. Figure A-83 permits computation of average running speed (90), and Figure A-84 allows average speed, which includes intersection delay time, to be estimated (38).

Intersection delay forecasts are based on Webster's delay estimates for signalized intersections at fixed time traffic signals (120). These forecasts must be altered to reflect actuated or coordinated traffic signal operation. The forecast of intersection delay is provided in two components: delay due to stopping delay due to idling. Figures A-85 and A-86 enable calculation of stopping delay and idling delay, respectively (90). Total intersection delay is the sum of these values.

Input Data Requirements

The following data are required to apply this procedure:

- Signal cycle length (c): The time period required for one complete sequence of signal phases.
- Approach volume (V): The approach volume expressed in vehicles per hour.
- Approach flow rate (q): The approach volume expressed in vehicles per second.
- Green time (g): The amount of effective green time for an approach.
- Green-to-cycle time ratio (g/c): The ratio of effective green time of the signal to the cycle length of the signal.
- Saturation flow (s): Saturation flow is the approach volume in vehicles per hour of green at maximum capacity (i.e., level-of-service E). This is equivalent to the mid-block link capacity for uninterrupted flow conditions. In the absence of detailed capacity calculations the saturation flow may be assumed to be 1,700 to 1,800 vehicles per hour per approach lane.
- Capacity (C): Capacity is maximum approach capacity (i.e., at level of service E) and is equal to the saturation flow multiplied by the green-to-cycle time ratio = sg/c.
- Degree of saturation (x): The ratio of the volume of traffic approaching the intersection to the capacity of the intersection. The degree of saturation represents the volume-to-capacity ratio of the intersection approach. However, since the approach capacity is constrained by the available green time, the degree of saturation will be less than the volume-to-capacity ratio in the mid-block, or uninterrupted flow segment. The degree of saturation can be calculated as $x = Vc/gS$.

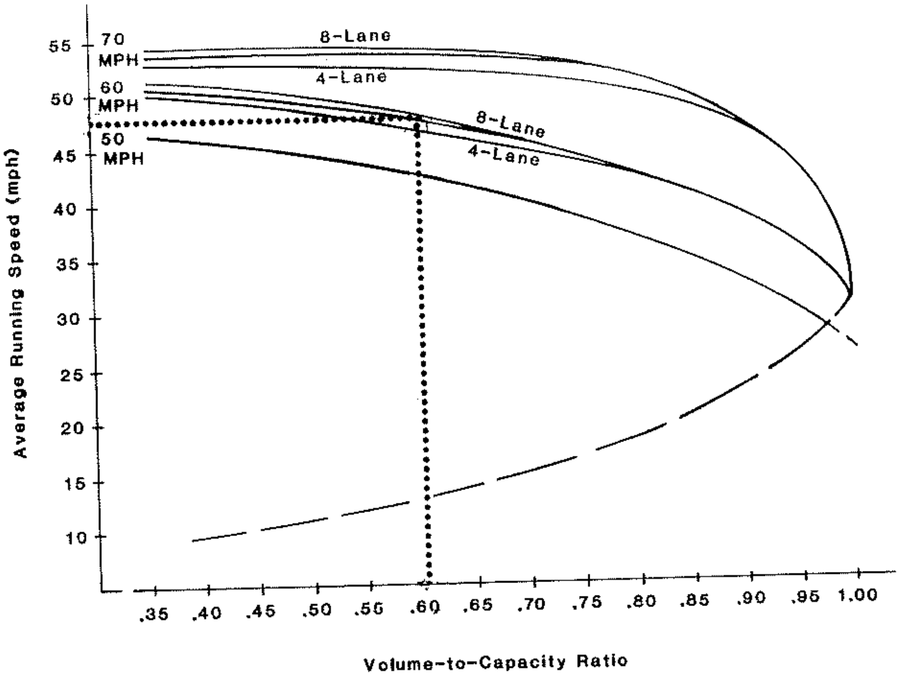
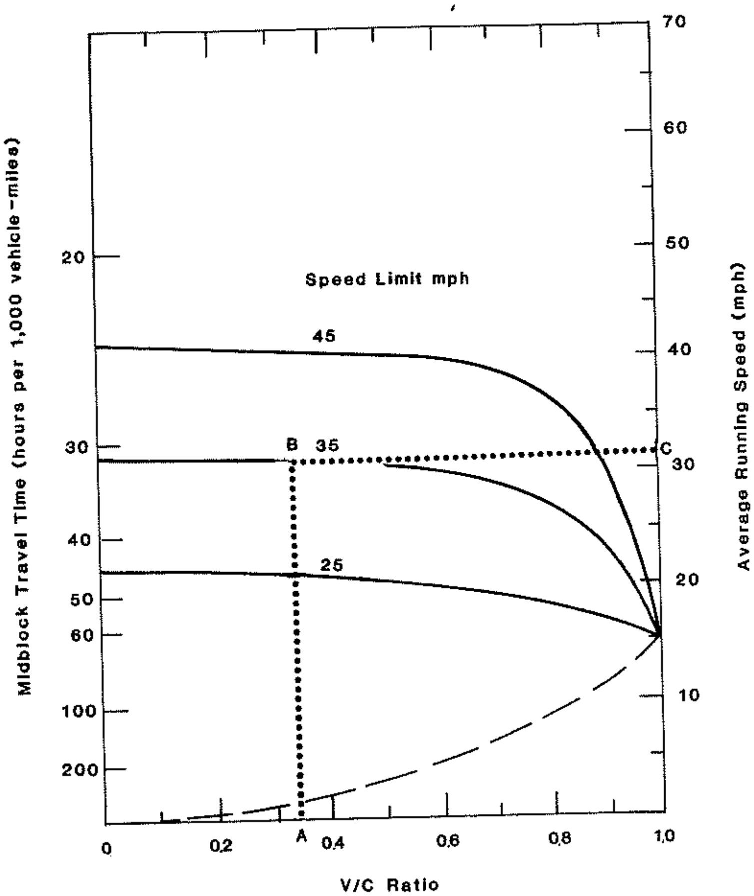


Figure A-82. Average running speeds on freeways.



Source : Ref. 90

Figure A-83. Average mid-block running speeds on surface arterials.

Directions For Use

A six-step set of directions is used in this procedure, as follows:

Step 1: Determine the mid-block average running speed. Use Figure A-83 to estimate average running speed relative to the forecasted volume-to-capacity ratio of the facility. This analysis should be performed for each section of the facility between signalized intersections. Special considerations must be applied when signal spacing is such that traffic at a signal may be affected by operations at an upstream or downstream signal.

If specific knowledge of intersection traffic operations is not available, Figure A-84 can be used to estimate average speed. In such cases, the procedure would now be complete.

Step 2: Calculate intersection delay on each of the facility approaches. Two techniques can be used, either:

a. Use Figures A-85 and A-86 to calculate stopping delay per 1,000 vehicles and idling delay per 1,000 vehicles; or

b. Use Webster's equation (Eq. A-71) to determine average delay per vehicle.

Figure A-85 is used as follows to calculate stopping delay at an intersection approach:

1. Begin with left-hand figure.
2. Enter the bottom of the figure at the appropriate degree of saturation.
3. Move vertically up to the appropriate green-to-cycle time ratio curve.
4. Move horizontally to the right across the left-hand figure to the appropriate approach speed line in the right-hand figure.
5. Move vertically down to the calculated stopping delay in hours per 1,000 vehicles.

Figure A-86 is used as follows to calculate idling delay at an intersection approach:

1. Begin with lower figure.
2. Enter the left side of the lower figure with the approach capacity.
3. Move horizontally to the right to the appropriate degree of saturation curve.
4. Move vertically up through the lower figure to the appropriate green-to-cycle time ratio curve in the upper figure.
5. Move horizontally to the right to the first scale on the right-hand side of the figure--the average delay per vehicle scale--to obtain an uncorrected estimate of average delay.
6. Obtain correction for this average delay estimate (the cycle length correction factor) from the small figure in the upper left hand corner of the upper figure.
   a. Enter bottom of small figure with appropriate cycle length.
   b. Move vertically up to the appropriate green-to-cycle time ratio line.
   c. Move horizontally to the right to the scale giving the average delay correction.
7. Add the average delay correction to the uncorrected average delay estimated in step 5 to obtain forecast average idling delay.
8. Move horizontally to the right to the right-most scale to convert the corrected average delay per vehicle forecast to idling delay hours per 1,000 vehicles.

Using technique (b), average delay per vehicle may be estimated based on Webster's equation (120):

$$d_j = E + F - G \tag{A-71}$$

where:

d_j = average intersection delay per vehicle on approach j (seconds);

E = $c(1 - g/c)^2/2(1 - (g/c)x)$;

F = $x^2/2q(1 - x)$; and

G = $0.65(c/q^2)^{0.33}(x^{(2 + 5g/c)})$. Component G usually equals approximately 10 percent of component F.

This equation is composed of three additive components, E, F, and G. Component "E" represents the delay that would result if traffic arrived in a uniform manner, with each vehicle being equally spaced over time. Component "F" represents delay that results from traffic arriving randomly. Component "G" is a correction factor to permit accurate total delay estimates. Webster's equation is only valid for under-capacity conditions, such that V/C or x is less than 0.975.

To reflect the use of traffic actuated signals or coordinated traffic signals, the total delay equation may be adjusted. The adjustment reflects the objectives of each of these traffic signalization strategies. The traffic actuated signal strategy is based on an adjustable traffic signal cycle length, that can be changed as a result of monitored volumes at each intersection approach. The traffic actuated signal cycle consists of an initial interval of minimum cycle length and green phase for each approach, and a maximum extension interval that may be added in whole or in part to an approach's green phase and the total cycle length as necessary to clear a queue. The traffic actuated control strategy thus reduces delay by distributing total available green time according to the magnitude of approach volumes, and by terminating each green phase as soon as vehicle queues at approaches are dissipated. As a result, traffic signal actuation reduces delay because vehicles will likely wait a shorter time for a green phase, and will probably not have to wait through more than one red phase.

This potential can be accounted for by permitting two cycle lengths in the Webster delay estimation equation (112). One cycle length is the average or minimum length and is used in computations for the first component of the delay equation. This component accounts for uniform or average vehicle arrivals or volumes. The second cycle length used represents the extended or maximum cycle length and is used in the computations for the second component of the delay equation. This component accounts for more random and possibly large approach volumes. The cycle length that would be used in the delay equation for fixed time traffic signals would lie between the minimum and maximum cycle lengths used for actuated traffic signals. It is estimated that modification of the delay equation to account for traffic actuation of signals would result in delay reductions of about 25 to 40 percent over a range of volume-to-capacity ratios at the intersection from 0.50 to 0.85 (112).

The objective of coordinated operation of traffic signals is to increase the proportion of vehicles arriving during the green phases and to decrease the proportion of vehicles arriving during the red phases. Delay is accordingly reduced under this coordination strategy as a smaller proportion of vehicles needs to stop and wait through a red phase.

To account for coordinated traffic signal operation, the first component of the delay Eq. A-71 is modified. For fixed time signals this component assumes that vehicle arrival over the entire signal cycle is uniform for both green and red phases. The modification to the delay equation component is based on a forecast of the potential for the coordination of traffic signals to result in higher vehicle arrival rates under the green phase of the cycle compared to the red phase (112). The first component "E" of the Webster delay Eq. A-71 should be modified as follows under the coordinated operation of traffic signals (92):

$$\text{Component "E"} = Vr\,(c)(1 - g/c)^2/2q\left[1 + \left[Vr/(s - Vg)\right]\right] \tag{A-72}$$

where:

E, c, q, and s are as previously defined and

V_r = vehicle arrival rate under red phase (veh/sec)

V_g = vehicle arrival rate under green phase (veh/sec)

The other components "F" and "G" remain the same.

Step 3: Calculate total intersection delay on each of the facility approaches. For technique (a), total intersection delay (D) on an approach j is determined by:

$$D_j = (d_{ji} + d_{js})\, P_S\, V_j/1{,}000 \quad \text{(A-73)}$$

where:

D_j = total delay on approach j (hours);

d_{ji} = total idling delay on approach j per 1,000 vehicles (hours) (Fig. A-86);

d_{js} = total stopping delay on approach j per 1,000 vehicles (hours) (Fig. 85);

P_S = proportion of stops (from Fig. A-85 or calculated from Eqs. A-80 or A-81); and

V_j = approach volume (vph).

For technique (b) using Webster's formulation, total intersection delay for all vehicles equals the following:

$$D_j = d_j\, V_j/3600 \quad \text{(A-74)}$$

where:

D_j = total delay on approach j (hours);

d_j = total delay per vehicle on approach j (seconds); and

V_j = approach volume (vehicles per hour).

For either technique (a) or (b), sum the delay values (D_j) for each approach to obtain total intersection delay.

Step 4: Obtain vehicle-miles and vehicle-hours of travel for each section of the facility. Multiply the section length (L_j) by the volume (V_j) to calculate vehicle-miles of travel (VHT). Obtain vehicle-hours of travel (VHT) by dividing the vehicle-miles of travel (VMT_j) by the average running speed (ARS_j). Therefore:

$$VMT_j = V_j\, L_j \quad \text{(A-75)}$$

$$VHT_j = VMT_j/ARS_j \quad \text{(A-76)}$$

This VHT_j value does not yet include intersection delay.

Step 5: Calculate total facility vehicle-miles and vehicle-hours of travel. Sum the vehicle-miles of travel (VMT) over each roadway section j. Calculate total vehicle-hours of travel (VHT) by summing the vehicle-hours of travel on each segment (Step 4) plus the total intersection delay (Step 3).

$$VMT = \sum_j VMT_j \quad \text{(A-77)}$$

$$VHT = \sum_j (VHT_j + D_j) \quad \text{(A-78)}$$

Step 6: Calculate average speed on entire facility. This is easily obtained by dividing total vehicle-miles of travel by total vehicle-hours of travel (Step 5).

$$AS = VMT/VHT \quad \text{(A-79)}$$

An example of this procedure is presented later in this chapter.

Queue Length Calculation Procedures for Surface Arterials

Surface arterials will experience queues at their signalized intersections during under-capacity conditions. Procedures to calculate the average length of these queues are derived from the same basis as the intersection delay estimation procedures previously discussed for arterials. The queue length procedure involves estimation of the average percentage of vehicles which must stop during a typical signal cycle.

This procedure is applicable to arterials that have traffic volumes operating at less than the arterial capacity. The basic procedure is designed for use with pre-timed, uncoordinated traffic signals. It therefore assumes that cycle length and phases are fixed and that vehicle arrival rates do not differ between red and green phases.

Basis for Development

The procedure is described in A Manual on User Benefit Analysis of Highway and Bus-Transit Improvements, published by AASHTO (90) and in Signal Operations Analysis Package (SOAP), published by USDOT/FHWA (112). The procedure assumes that the average queue will consist of those vehicles that arrive at the signal during the red phase of the signal cycle plus those vehicles that arrive at the beginning of the green phase and must stop and join the queue until it discharges.

Input Data Requirements

The required input data are the following:

- Approach flow rate (q): The approach volume expressed in vehicles per second.
- Degree of saturation (x) for each approach (see previous procedure for description).
- Cycle length (c).
- Green-to-cycle time ratio (g/c): The ratio of effective green time to total cycle time at the intersection approach.

The volume and capacity data are obtained directly from the computer forecasts or from the results of the refinement and detailing procedures presented in Chapters 4 through 7. The green-to-cycle time ratio is estimated from base year signal operations or from conditions at intersections with similar characteristics. For coordinated signal systems, the analyst must also have knowledge of the specific phasing involved, such that for each approach the vehicle arrival rate can be estimated during the red and green phases.

Directions for Use

A two-step set of directions is used in this procedure, as follows:

Step 1: Estimate the average proportion of vehicles stopping during a signal cycle. For fixed-time signals, use the following equation:

$$P_S = (1 - g/c)/(1 - gx/c) \quad \text{(A-80)}$$

where g/c and x are the input data and P_S = proportion of vehicle stopping during signal cycle.

For coordinated traffic signals a different equation is applied to establish E. The differences between coordinated and fixed time signals are based on the quality of the coordination achieved. As a result, these differences are measured between the vehicle arrival rate during the red phase,

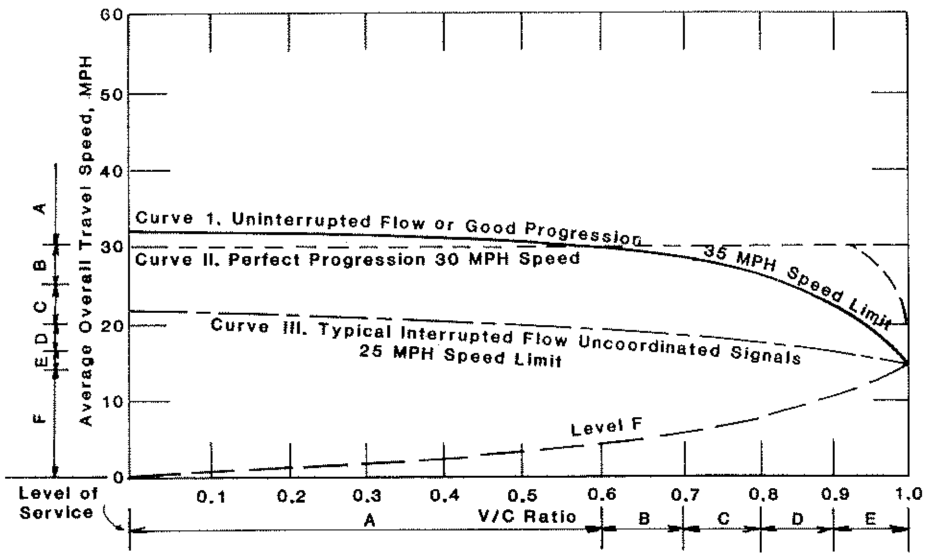


Figure A-84. Average speeds on surface arterials.

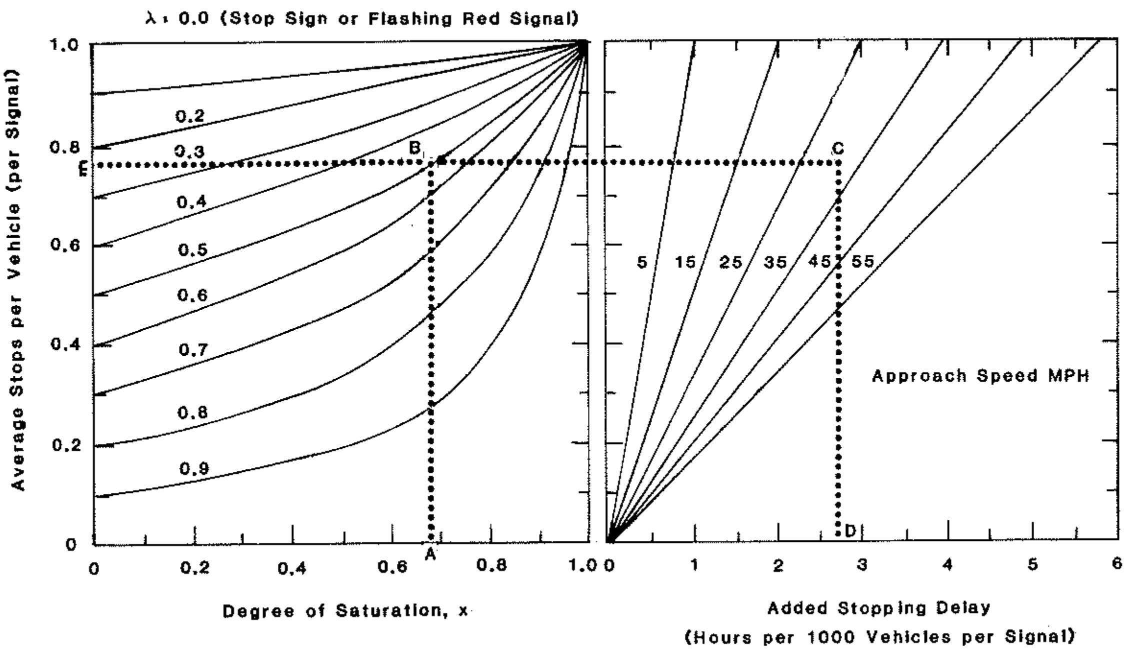


Figure A-85. Nomograph of intersection stopping delay.

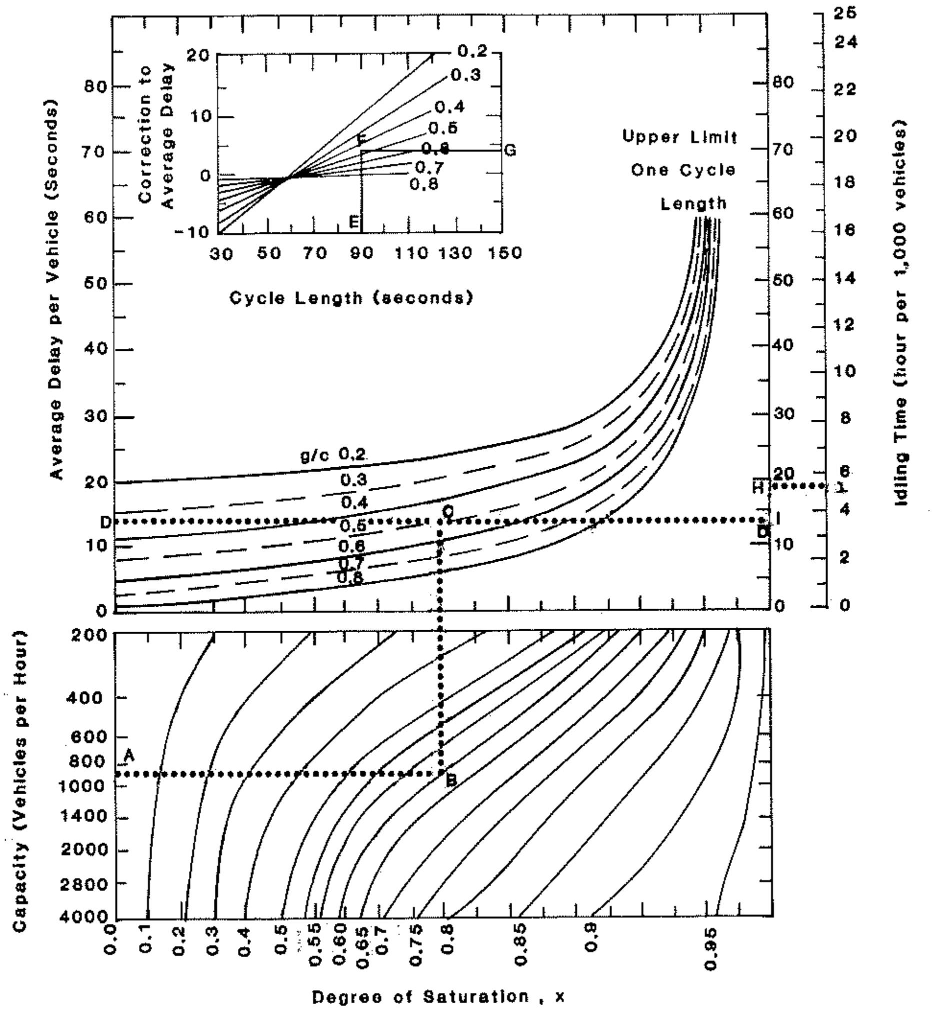


Figure A-86. Nomograph of intersection idling delay.

V_r, and the vehicle arrival rate during the green phase, V_g. Traffic signal coordination seeks to maximize V_g and minimize V_r. Under the fixed time, uncoordinated signal arrival rates are considered to be equal. The equation for proportion stopping at coordinated traffic signals is as follows (112):

$$P_s = (RV_r/cV_s) / \left[1 - x \left[1 - (RV_r/cV_s)\right]\right] \qquad \text{(A-81)}$$

where:

P_s, c, and x are previously defined;
V_r = vehicle arrival rate during red phase (vehicles per second);
R = length of red phase (seconds); and
V_s = vehicle arrival rate during signal cycle (vehicles per second).

The average stops per vehicle can also be estimated from Figure A-85, given the degree of saturation (x) and the g/c ratio.

Step 2: Calculate the average queue length. Use the following equation:

$$Q = P_s * q * c \qquad \text{(A-82)}$$

where:

Q = average queue length per cycle (number of vehicles);
q = approach flow rate (vehicles per second);
P_s = proportion of vehicles stopping (Step 1); and
c = cycle length in seconds.

There is no known procedure to account for differences between forecasted queue lengths for fixed time, uncoordinated or traffic actuated signals. It is generally accepted, however, that the proportion of vehicles required to stop at traffic actuated signals will be higher (112). Such signals terminate green phases after queue discharge and do not have any further green phase during which subsequent vehicles arriving at the approach can proceed through without stopping. This situation, however, should not be interpreted as resulting in longer queues because, as noted earlier, average traffic actuated signal cycle lengths are shorter than for fixed time signals and the timing of phases is keyed to minimizing queue lengths.

Example Problem

The following example illustrates the method for estimating speed and queue length for an under-capacity arterial. An arterial segment with a 35-mph speed limit has a mid-block V/C ratio of 0.33. The signal downstream has a cycle length of 90 seconds with an effective green time of 45 seconds. The saturation flow(s) is estimated to be 1,800 vphg. Determine the total average running speed, the average speed and the average queue length for a demand of 600 vph.

Step 1. Since the V/C ratio and speed limit are known, determine the mid-block average running speed from Figure A-83. Entering the horizontal axis with the V/C ratio of 0.33 (point A) moving vertically to the 35-mph speed limit curve (point B) and horizontally to the right, the average mid-block running speed (ARS) is determined to be 31 mph (point C).

Step 2.

a. Use Figures A-85 and A-86. Determine stopped delay per 1,000 vehicles. First determine the degree of saturation (x) at the intersection.

$x = Vc/gs = 600(90)/45(1800) = 0.66$

Calculate the green-to-cycle time ratio:

$g/c = 45/90 = 0.50$

For stopping delay, enter the horizontal axis of the left graph in Figure A-85 with the 0.66 degree of saturation (point A). Move vertically to the green-to-cycle curve of 0.5 (point B), then move horizontally to the right into the second graph to the point approximating a 31-mph line (point C). Finally, move vertically down to determine an added stopping delay of 2.7 hours per 1,000 vehicles per signal (point D).

For idling delay, first calculate the approach capacity:

$C = (g/c)s = 0.5(1,800) = 900$ vph

Enter the vertical axis of the lower graph in Figure A-86 with the 900-vph capacity (point A). Move horizontally to the right to the 0.66 degree of saturation curve (point B). Then move vertically into the upper graph to the green-to-cycle ratio of 0.5 curve (point C) and finally move horizontally to the left or right to determine an unadjusted average idling delay per vehicle of 14 seconds (Point E). Determine the correction from the inset graph. Enter with the 90-second cycle on the horizontal axis (point E) and move vertically to the 0.5 green-to-cycle ratio curve (Point F). Move horizontally to the right to obtain a correction of 4-seconds (point G). Add the 4-second correction to the unadjusted average idling delay per vehicle to yield an 18-second total average idling delay per vehicle (point H). Finally, move horizontally from H to the right to find the total idling time (point I) equal to 5 hours per 1,000 vehicles.

b. Or, use Webster's Eq. A-71 to calculate average delay per vehicle:

$$d_j = E + F - G \qquad \text{(A-71)}$$

where:

$E = 90\,(1 - 0.5)^2/2\,(1 - (0.5)(0.66)) = 16.79$
$F = (0.66)^2/2\,(600/3600)\,(1 - 0.66) = 3.84$
$G = 0.65\,(90/(600/3600)^2)^{0.33}(0.66)^{(2 + 5\,(0.5))} = 1.44$

Then $d_j = 16.79 + 3.84 - 1.44 = 19.2$ seconds per vehicle.

Step 3.

a. Calculate total delay using Figures A-85 and A-86 and Eq. A-73:

$D_j = (d_{ji} + d_{js})\, P_s\, V/1,000$
$D_j = (5 + 2.7) \times 0.75 \times 600/1,000$
$D_j = 3.28$ vehicle hours.

b. Calculate total delay using Webster's Eq. A-74:

$D_j = D_j\,(v/3,600)$
$D_j = 19.2 \times 600/3,600$
$D_j = 3.20$ vehicle-hours

Use 3.20 veh-hrs for the remainder of the example.

Step 4.

a. Calculate vehicle-miles of travel for the section:

VMT = L x V = 0.75 mi x 600 veh = 450 veh-mi

b. Calculate the vehicle hours of travel for the section:

VHT = VMT/ARS = 450/31 = 14.5 veh-hr

Step 5.

a. Calculate total vehicle miles of travel. Because there is only one section, the total vehicle miles (VMT) is equal to the vehicle-miles of travel in the section:

$VMT = VMT_j = 450$ veh-mi

b. Calculate total vehicle hours of travel. Since there is only one section,

$VHT = D_j + VHT_j = 3.2 + 14.5$

$VHT = 17.7$ veh-hrs

Step 6.

a. Calculate average speed (AS) by dividing total VMT by total VHT:

$AS = VMT/VHT = 450/17.7 = 25.4$ mph

Notice that the average speed (AS) is less than the average running speed (ARS) because of the addition of the intersection delay.

b. Determine the average queue length using the queuing procedure:

Step 1. Estimate the proportion of vehicles stopping at least once. Since the signal is not in a progression system use Eq. A-80:

$P_s = (1 - 0.5)/(1 - 0.5(0.66)) = 0.75$

Similarly, from Figure A-85 (point E), $P_s = 0.75$. This serves as a check.

Step 2. Calculate average queue length (Q):

$Q = 0.75 * 600/3,600 * 90$

$Q = 11.25$ vehicles

OVER-CAPACITY CONDITIONS

Over-capacity conditions and the queuing caused by such conditions may result in increased delay and travel costs to the facility user compared to the worst under-capacity conditions. As a result, it is unlikely that over-capacity conditions should be anticipated to occur regularly in the future even under a "no-build" alternative. It is more likely that potential facility users will divert to other facilities, will change their time periods of travel, or will even change their modes of travel.

Before procedures for calculating the effects of facility over-capacity conditions are applied, facility traffic projections should first be reviewed, as discussed in Chapter 3 (91). This review should include a very basic check of the reasonableness of the overall traffic demand forecast. It should also check whether the demand is consistent with the forecasted changes in land-uses in the facility corridor, and whether the forecasted land-use changes themselves are reasonable. Also, the projected traffic demand should be checked against programmed capacity changes.

Following this basic review of reasonableness, a review of vehicle diversion potential should be considered. First, route diversion to alternate facilities should be considered. Second, the potential for temporal demand shifts, such as extension of the peak period on the facility should be considered. And third, the potential for diversion to other modes, especially public transit, should be reviewed in light of the inadequate traffic conditions forecasted in the corridor. After this review is complete, if it is still anticipated that over-capacity conditions will occur, the following procedures should be applied to estimate speeds, delay, and queuing.

Procedure for Grade-Separated Facilities

The determination of speed, delay, and queuing data on grade-separated facilities for over-capacity conditions can be accomplished using a straightforward worksheet procedure. It can be applied to any freeway segment. The procedure provides a somewhat simplified view of the freeway operations, as it does not specifically account for the effects of on- and off-ramps along the freeway.

Basis for Development

The procedure is based on the shock-wave method of queuing analysis as described in NCHRP Report 133 (91). This approach assumes that the queuing effects of a bottleneck will move upstream similar to "shock waves" in compressible fluids. If necessary, the peak traffic hour is extended by the duration of time required to dissipate the queue. Additional theory is provided in the original document (91).

Input Data Requirements

The following data are required:

- Identification of bottleneck location.
- Directional volumes (peak hour and immedimately following off-peak hour).
- Time duration of peak volume.
- Capacity (bottleneck and upstream facility segments).
- Design speed of facility.

The hourly volume data can be derived from computer traffic forecasts before or after applying the refinement or detailing procedures in Chapters 4 through 7. The techniques in Chapters 9 and 10 will provide a basis for developing diurnal and directional volume distributions. Base year conditions on similar facilities should be examined to estimate the duration of the peak demand. The bottleneck and upstream facility capacities and design speed should be calculated directly from functional design plans, because these values must be carefully determined.

Directions for Use

To apply the procedure the worksheet in Figure A-87 may be used along with the following step-by-step directions (91):

Step 1: Identify bottleneck freeway section and immediate upstream section.

Step 2: Fill out heading of worksheet.

Step 3: Enter input bottleneck demand volume. This is conducted for peak and off-peak periods (Item 1).

Step 4: Enter duration of peak demand (Item 2). Note that it is assumed there is no limit to the duration of off-peak demand.

Step 5: Enter the capacity values. The upstream section capacity is placed in Item 3 and the capacity of the bottleneck section in Item 4.

Step 6: Calculate the volume-to-capacity ratios (Item 5). Item 5.1 should be the ratio for the upstream section that will be unaffected by the queue. It is equal to the upstream volume divided by the upstream capacity. Item 5.2 will be the ratio for the queue. It is equal to the bottleneck

Project No. Example Upstream section identification AB
Year 1982 Bottleneck section identification BC
Time of Day PM Peak

		Peak	Off Peak
1.	Demand volume for bottleneck	4,100 veh/hr	3,200 veh/hr
2.	Time duration of demand volume	1 hrs	
3.	Capacity of upstream section	5,700 veh/hr	
4.	Capacity of bottleneck	3,800 veh/hr	
5.	V/C ratios		
	5.1 Unaffected upstream subsection	0.72	
	5.2 Queued upstream subsection	0.66	
	5.3 Bottleneck section	1.0	
6.	Rate of queueing (a positive value indicates an increasing queue)	300 veh/hr	600 veh/hr
7.	Speed of vehicles through each section		
	7.1 Upstream unaffected section	53 mi/hr	
	7.2 Upstream queue subsection	15 mi/hr	
	7.3 Bottleneck section	32 mi/hr	
8.	Density of vehicles using each section		
	8.1 In upstream unaffected section	77.4 veh/mi	
	8.2 In upstream queue subsection	153.3 veh/mi	
	8.3 In bottleneck section	118.8 veh/mi	
9.	Change in density in going from the upstream unaffected section to the queue section	175.9 veh/mi	
10.	Average length of queue	0.85 mi	
11.	Time required during off-peak to dissipate queue, in hours		0.5 hrs
12.	Average running speed over entire freeway segment	27 mph	

Figure A-87. Queuing and speed calculations for grade-separated facilities in over-capacity conditions.

capacity divided by the upstream section capacity. Item 5.3 is the ratio for the bottleneck and is usually equal to 1.0. Values greater than 1.0 are not used.

Step 7: Enter the rate of queuing (Item 6). For the peak time, it is equal to the peak demand volume for the bottleneck (Item 1) minus the capacity of the bottleneck (Item 4). For the off-peak, use the off-peak time demand volume (Item 1) minus the bottleneck section capacity (Item 4).

Step 8: Estimate average running speeds in each freeway segment. These are based on the V/C ratios from Item 5. For the unaffected upstream section (Item 7.1) and the bottleneck (Item 7.3), Table A-28 or the solid lines in Figure A-82 of Chapter 10 should be used. For the upstream queue section, the dashed line at the bottom of Figure A-82 is recommended for use.

Step 9: Estimate density of vehicles in each freeway segment. Enter in Item 8.

Item 8.1 - For upstream section not affected by queue, density equals upstream demand (Item 1) divided by its speed (Item 7.1)

Item 8.2 - For upstream section in queue, density equals bottleneck capacity (Item 4) divided by upstream queue speed (Item 7.2)

Item 8.3 - For bottleneck section, density equals bottleneck capacity (Item 4) divided by bottleneck queue speed (Item 7.3)

Step 10: Enter the change in density going from the upstream unaffected section to the upstream queue section (Item 8.2 minus Item 8.1). Enter in Item 9.

Step 11: Calculate the average length of queue. The queue is assumed to be at its maximum length at the end of the peak period. At this time the level of demand decreases. The maximum length of the queue is equal to the peak rate of queuing (vehicles per hour—Item 6) multiplied by the time duration of peak volume (hours-Item 2) divided by the change in density at the queue build-up point (vehicles per mile-Item 9). The average queue length is equal to the maximum queue length divided by two and is entered in Item 10.

Step 12: Enter the time required to dissipate the queue in the off-peak period (Item 11). This is equal to peak rate of queuing (Item 6) divided by off-peak rate of queuing (Item 6) multiplied by duration of peak traffic demand (Item 2).

Step 13: Calculate the average running speeds for the entire freeway segment. Enter this value in Item 12 as follows:

$$ARS = \frac{L}{\frac{L_v - L_q}{AS_v} + \frac{L_q}{AS_q} + \frac{L_b}{AS_b}} \qquad \text{(A-83)}$$

where:

ARS = average running speed of entire freeway segment (mph);
ARS_v = average running speed of unaffected upstream segment (Item 7.1);
ARS_q = average running speed of queue upstream segment (Item 7.2);
ARS_b = average running speed of bottleneck segment (Item 7.3);
L_v = length of total upstream segment (miles);
L_q = length of queue upstream segment (miles);
L_b = length of bottleneck (miles); and
L = total length of freeway under analysis (miles) (equal to $L_v + L_b$).

Example Problem

The following example illustrates the methodology used to determine freeway speed and queuing on grade-separated facilities for over-capacity conditions. The situation is depicted in Figure A-88.

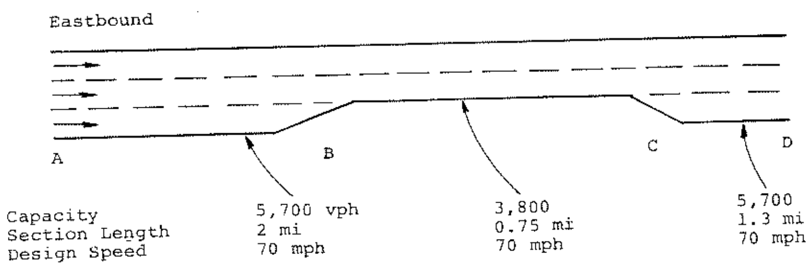


Figure A-88. Example problem characteristics.

Prior to the PM peak the eastbound demand on section AD is 2,200 vph. Demand increases to 4,100 vph during the peak hour and decreases to 3,200 vph following it. Determine the average length of the queue and the average running speed over the entire freeway segment.

Refer to Figure A-87.

Step 1. Identify the bottleneck section BC.

Step 2. Complete the worksheet heading as shown.

Step 3. Enter the peak demand volume of 4,100 vph and the post peak demand volume of 3,200 vph on line 1.

Step 4. Enter the peak demand duration of one hour on line 2.

Step 5. Enter the upstream and bottleneck capacities of 5,700 vph and 3,800 vph respectively on lines 3 and 4.

Step 6. Calculate the V/C ratios.

Unaffected upstream subsection

V/C = 4,100/5,700 = 0.72 Enter on line 5.1

Queued upstream subsection

V/C = 3,800/5,700 = 0.66 Enter on line 5.2

Bottleneck section

V/C = 1.00 Enter on line 5.3

Step 7. Calculate queuing rates.

For peak:

queuing rate = peak demand volume - bottleneck capacity
= 4,100 - 3,800 = 300 Enter on line 6

For off-peak:

queuing rate = off-peak demand volume - bottleneck capacity
= 3,200 - 3,800 = -600 Enter on line 6

Step 8. Estimate average running speeds.

For the upstream unaffected section use Table A-28. Enter the 70-mph design speed with six-lanes column and move down to V/C ratio of 0.70. By inspection of the table the speed is determined to be 53 mph.

For the upstream queue subsection use Figure A-82. Enter the horizontal axis with the V/C ratio of 0.66, move vertically to the dashed line, and move horizontally to the left to determine a speed of 15 mph.

For the bottleneck section use Table A-28. Enter the 70-mph design speed with four-lanes column and move down to a V/C ratio of 1.0 to determine the speed as 32 mph.

Step 9. Calculate densities.

For the upstream unaffected section

density (K) = demand volume/speed = 4,100/53 = 77.4 Enter on line 8.1

For the upstream queue subsection

K = bottleneck capacity/upstream queue speed = 3,800/15 = 253.3 Enter on line 8.2

For the bottleneck section

K = bottleneck capacity/bottleneck queue speed 3,800/32 = 118.8 Enter on line 8.3

Step 10. Calculate the change in density.

Change in density = (upstream queue subsection density) - (unaffected upstream density)
= 253.3 - 77.4 = 175.9. Enter on Line 9. (A-84)

Step 11. Calculate average queue length.

$$\text{average queue length} = \frac{\text{peak queuing rate} \times \text{demand volume time duration}}{2 \times \text{change in density}} \quad \text{(A-85)}$$

= (300 vph x 1 hr) / (2 x 175.9) = 0.85

Step 12. Calculate queue dissipation time.

$$\text{queue dissipation time} = \frac{\text{peak queuing rate} \times \text{demand volume time duration}}{\text{off-peak queuing rate}} \quad \text{(A-86)}$$

= (300 vph x 1 hr) / 600 vph = 0.5 hr

Step 13. Calculate average peak demand period running speed.

$$\text{average running speed} = \frac{2.75}{2 - 0.85/53 + 0.85/15 + 0.75/32}$$

= 27.0 mph

Procedure for Surface Arterials

The procedure for calculating arterial speeds, delays, and queues for over-capacity conditions includes a mid-block speed forecast and an intersection delay forecast. The mid-block speed forecast would use the same mid-block speed forecast procedures described earlier for under-capacity conditions. The V/C ratio for the over-capacity intersection approach would always be equal to 1.0.

Intersection delay forecasting techniques would be different because they must include the effect of queuing. The delay calculated in the procedure includes the delay due to insufficient capacity, plus the delay due to the signal cycling from green to red and back.

This procedure is applicable to all arterials at signalized intersections that operate over their capacity. The procedure assumes that demand will be over-capacity during a known time period and then drop to a uniform level that is under-capacity for the next time period. The procedure, however, could be modified for different assumptions.

Basis for Development

Delay at an over-capacity intersection approach will have two components. One component will result from the excess vehicles that will build up in a queue as long as the approach traffic demand exceeds capacity. The queue will decline when demand drops to less than capacity and delay will decline at a rate defined by how much the demand is less than capacity. The second delay component will result from the cycling of the traffic signal between red and green phases. In this method, the duration of the peak and off-peak periods is unchanged.

The procedure for calculating intersection delay under circumstances of queuing consists of the deterministic queuing model recommended in NCHRP Report 133 (91). The model must be specified in terms of the time at which over-capacity operations begin, the time at which such operations end, and the time at which the resulting queue ultimately discharges.

Input Data Requirements

This procedure requires the following input data for each intersection approach:

- Intersection directional approach volumes (peak hour and immediately following off-peak hour).
- Approach capacity.
- Time duration of peak volume.
- Signal cycle length during peak period.
- Effective green time of signal during peak period.
- Number of approach lanes.

The hourly volume data can be derived from computer traffic forecasts before or after applying the refinement or detailing procedures in Chapters 4 through 7. The techniques in Chapters 7 and 10 will provide a basis for developing diurnal and directional volume distributions. The duration of peak demand and the traffic signal operations data should be estimated by examining base year conditions on similar facilities. The approach capacity should be estimated from the number of lanes, the lane configuration, and the traffic signal operations.

Directions for Use

Step-by-step directions, based on the worksheet in Figure A-89 (91), are as follows:

Step 1: Enter the headings on worksheet.

Step 2: Enter the demand volumes. For peak volumes use Item 1 (demand should be greater than capacity). For off-peak volumes use Item 2 (demand should be less than capacity).

Step 3: Enter the approach capacity. Item 3.

Step 4: Enter time duration of peak volumes. Item 4.

Step 5: Enter signal cycle length. Item 5.

Step 6: Enter effective green time for over-capacity intersection approach. Item 6.

Step 7: Enter averaging running speed of vehicles into intersection queue. Estimate average running speed using V/C ratio of previous upstream intersection and curves in Figures A-83.

Step 8: Enter the number of lanes at the over-capacity intersection approach.

Step 9: Calculate the rate of arrival of vehicles into intersection approach queue. This rate will be slightly greater than the peak demand volume because, as the queue builds, it moves upstream and accordingly increases the vehicle arrival rate in the queue.

Step 9.1: First estimate the queue density. This value can be assumed to be 240 veh/mi/lane or a 22 ft spacing in the queue if other data are unavailable. Enter this value into Item 9.1.

Step 9.2: Calculate the rate of vehicle arrival as follows:

$$\text{Rate of vehicle arrival} = A \times (1 + B/(C - A)) \qquad \text{(A-87)}$$

where:

A = peak volume (Item 1);
B = peak volume (Item 1) - capacity (Item 3); and
C = number of lanes (Item 2) x approach speed (Item 7) x density (Item 9.1).

Step 10: Enter the duration of interruption of the signal (Item 10). This equals the total cycle length (Item 5) less the effective green time (Item 6). This value should be expressed in hours.

Step 11: Calculate the queue lengths. Four elements to this step are involved, as follows:

Step 11.1: Calculate the maximum queue length in terms of vehicles. The following equation is used:

$$\text{Maximum queue length (vehicles)} = D \times E \qquad \text{(A-88)}$$

where:

D = duration of peak demand (Item 4); and
E = peak queue arrival rate (Item 9) - capacity (Item 3).

Step 11.2: Calculate maximum queue length in terms of distance. The following equation is used:

$$\text{Maximum queue length (distance)} = F/G \qquad \text{(A-89)}$$

where:

F = maximum queue length (vehicles); and
G = density (Item 9.1) x number of lanes (Item 8).

Step 11.3: Calculate the adjusted maximum queue length to reflect the interruption of the signal. The following equation is used:

$$\text{Adjusted Maximum Queue Length} = K + L \qquad \text{(A-90)}$$

Project No. Example Intersection Identification Street A and Road A

Year 1985 Time 7-9 AM Approach Identification EB

1.	Demand volume for peak	1,850	veh/hr.
2.	Demand volume for off-peak	1,200	veh/hr.
3.	Capacity of intersection approach	1,800	veh/hr.
4.	Time duration of peak	2	hrs.
5.	Cycle length of signal	120	sec.
6.	Effective green time	60	sec.
7.	Speed of vehicles on the approach to the intersection during the peak	30	mi/hr.
8.	Number of lanes of the approach	2	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 Arrival Rate	1,857	veh/hr
10.	Duration of interruption by signal	0.017	hrs.
11.	Queue length		
	11.1 Maximum queue length (vehicles)	114	vehicles
	11.2 Maximum queue length (distance)	0.23	mi
	11.3 Adjusted maximum queue length	145	vehicles
	11.4 Average adjusted queue length	73	vehicles
12.	Queue discharge time	0.167	hr.
13.	Average delay per vehicle	146	sec/veh

Figure A-89. Queuing and delay calculations for surface arterials in over-capacity conditions.

where:

K = maximum queue length (vehicles) (Item 11.1); and

L = capacity (Item 3) x length of signal interruption (Item 10).

Step 11.4: Calculate the average adjusted queue length. This value equals one-half of the adjusted maximum queue length (Item 11.3) and is expressed in terms of vehicles. An adjustment similar to that used in Step 11.2 could be used to express this value on a distance basis.

Step 12: Compute the queue discharge time. This value is determined from the following equation:

$$\text{Queue Discharge Time} = (H \times I) / J \qquad \text{(A-91)}$$

where:

H = length of peak demand (Item 4);

I = peak volume (Item 1) - capacity (Item 3); and

J = capacity (Item 3) - off-peak volume (Item 2).

Step 13: Calculate the average delay per vehicle. This value is determined as follows:

$$\text{Delay per vehicle} = \text{average adjusted queue length/capacity} = (\text{item } 11.4)/(\text{item } 3) \qquad \text{(A-92)}$$

This value should be expressed in seconds per vehicle.

Example Problem

The following is an example of estimating queue length and delay for an intersection that is over-capacity. The eastbound approach (intersection capacity of 1,800 vph for two lanes) to the intersection of Street A and Road B is expected to have an hourly demand of 1,850 vph during the hours of 7 AM to 9 AM in 1985. After this period the demand is expected to drop to 1,200 vph. Determine the average adjusted length and average vehicle delay if the signal cycle length is 120 seconds and the effective green time is 60 seconds. Assume a mid-block V/C ratio of 0.51 upstream of the intersection and speed limit of 35-mph.

Step 1. Enter headings as appropriate. (See Fig. A-99).

Step 2. Enter the demand volume during the peak on line 1. Enter the off-peak demand volume on line 2.

Step 3. Enter the intersection capacity on line 3.

Step 4. Enter the duration of the peak volume on line 4.

Step 5. Enter the signal cycle length on line 5.

Step 6. Enter the effective green time for the approach on line 6.

Step 7. Determine the average running speed from Figure A-83 with a V/C of 0.51 and a speed limit of 35 mph. Enter on line 7.

Step 8. Enter the number of lanes on line 8.

Step 9. Calculate the rate of vehicle arrival with a density assumption of 240 veh/mi/lane (line 9.1).

1850 (1 + ((1850 - 1800)/((2 x 30 x 24) - 1850))) = Rate of Vehicle Arrival = 1,857 veh/hr

Step 10. Calculate the duration of interruption of the signal.

Duration of interruption = cycle length - effective green time

= 120 sec - 60 sec = 60 sec

= 60/3600 = 0.017 hr Enter on line 10.

Step 11. Calculate the queue lengths.

First calculate the maximum queue length:

11.1 Maximum queue length (vehicles) = 2 (1,857 - 1,800) = 114 vehicles.

11.2 Maximum queue length (distance) = 114/240 x 2 = 0.23 mi.

11.3 Adjusted maximum queue length = 114 + (1800 * 0.017) = 145 vehicles.

11.4 Average adjusted queue length = 145/2 = 73 vehicles

Converted to distance, this equates to: 73/240 x 2 = 0.15 mi.

Step 12. Calculate queue discharge time.

Queue Discharge Time = 2(1850 - 1800)/(1800 - 1200) = 0.167 hr

Step 13. Calculate average delay per vehicle.

Delay per vehicle = 73/1800 = 0.041 hr = 146 sec/veh

Since 146 seconds is greater than the signal cycle length of 120 seconds, enter 146 on line 13.

CHAPTER THIRTEEN
DESIGN OF HIGHWAY PAVEMENTS

GENERAL

The recommended procedure to develop the data necessary for highway pavement design is described in the AASHTO Interim Guide for Design of Pavement Structures (5). The AASHTO procedure converts traffic data (ADT and vehicle classification) to 18-Kip equivalent single-axle loadings (ESAL). The ESAL data is the one essential traffic data input used to determine the structure depths for subbase, base, and surface layers for flexible pavement and slab depth and slab connections for rigid pavements.

The procedure to determine ESAL may be applied to a new, upgraded, or existing facility. The procedure is applicable in any terrain or under any environmental conditions.

INPUT DATA REQUIREMENTS

The procedure to determine 18-kip equivalent single-axle loadings requires selected traffic data items and several assumptions. Some of these assumptions are not related to traffic data. First, a structural number (SN) must be assumed for flexible pavement design and a depth (D) must be assumed for rigid pavement design. These assumptions are used as "initial" values for the structural number or depth to be estimated by the pavement design procedures. AASHTO suggests that a value of 3.0 be assumed for flexible pavements and a value of 8.0 be assumed for rigid pavements. If the discrepancy between these initial values and final values determined by the procedure is too large, the process must be repeated assuming a new structural number or depth.

A second required assumption is the terminal Pavement Serviceability Index (Pt). This index reflects the ability of a pavement to serve high-speed, high-volume automobile and truck traffic. The terminal pavement serviceability index thus represents the lowest index that will be tolerated at the end of the design period (usually 20 years). An index of 2.5 is suggested by AASHTO as a guide for design of major highways, while a value of 2.0 is suggested for highways with lesser traffic volumes.

Other assumptions necessary for pavement design include a soil support value and regional factor for flexible pavements, a concrete working stress factor, a modulus of elasticity, and a subgrade reaction factor for rigid pavements. The regional factor accounts for regional environmental and climatic conditions. The soil support value and subgrade reaction factor account for soil conditions. The working stress and modulus of elasticity factors account for characteristics of the concrete used in the rigid pavement.

The procedure for determining ESAL and pavement design also requires the following traffic data:

- Directional average daily traffic volumes (ADTs) and lane-use distributions for the base year and the 20-year design period. When lane-use distributions are not available, the AASHTO Guide suggests the following values:

Number of Lanes in Both Directions	Percent of Traffic in Design Lane
2	100
4	80-100
6	60- 80

- 18-kip truck axle loading characteristics for single and tandem axle loadings. These base year values should be obtained from a loadometer station that represents the anticipated traffic usage of the design facility.
- Vehicle classification (i.e., percent trucks) of the traffic flow based upon a 24-hour traffic volume. These data should be obtained for the base year and the 20-year design period.

If directional traffic volumes are not available, directional distributions are usually made by assigning 50 percent of the traffic in each direction. Most states also assign 100 percent of the directional traffic to the design lane, thereby ignoring lane distribution of traffic. This conservative approach helps ensure that the pavement structure can withstand the total traffic volume given a lane closure due to unforeseen circumstances.

DIRECTIONS FOR USE

The following steps are required to produce the ESAL values for use in highway design:

Step 1: Calculate the average annual ADT for a 20-year design period in both directions.

ADT (avg) = (ADT (base year) + ADT (20-year))/2 (A-93)

Step 2: Calculate the design lane ADT.

ADT (design) = ADT (avg.) x Directional Split (proportion) x lane use factor (proportion) (A-94)

Step 3: Calculate the design lane truck volumes.

Truck (Daily) = ADT (design) x Percent Trucks (base year) (A-95)

Truck (20-yr) = ADT (trucks) x 365 days/yr x 20 yr (A-96)

Step 4: Determine the ESAL rate. This rate is the number of 18-kip equivalent single axle loadings per truck assumed to be made on the facility under analysis. The ESAL rate is simply multiplied by the number of trucks on the facility to result in the number of 18-kip equivalent single axle loads to be used in pavement design (see step 5).

The ESAL rate is a combination of two factors. One factor is the 18-kip axle equivalency factor, or the number of 18-kip single axle equivalent loads per type and weight classification of axle load. The equivalence factors from the AASHTO manual are given in Tables A-29 through A-34. Different factors are used for different types of pavement, and different values of Pt, SN, and D.

The second factor is the number of truck axle loads by type and weight classification expected on the facility. This information may be obtained by using statewide data, data from a particular loadometer station on a similar facility, or data extrapolated from loadometer sites to project sites. The FHWA maintains information on a statewide and loadometer station basis.

The computation of ESAL data is accomplished in a straightforward manner. Table A-35 summarizes the equivalence factors and number of axle loads for each classification of axle load type and weight from a typical loadometer station. The data represent a sample of 3,146 trucks weighed. The equivalence factors were obtained from Table A-31 (flexible pavement, Pt = 2.5, SN = 3). By obtaining the product of equivalence factors and number of axle loads for each axle load group, a number of equivalent 18-kip single axle loads (1826.8) is obtained for the 3,146 trucks weighed (see Table A-35). The ESAL rate for the station equals the number of loads divided by the number of trucks weighted and multiplied by 1,000, or (1826.8/3,146) * 1,000 = 580.7.

Figure A-90 provides an example of statewide ESAL data for flexible and rigid pavement design. Figure A-91 provides an example of station-specific ESAL data and rates for tractor semi-trailor combinations. Similar data are available for other truck classifications, such as single-unit trucks, semi-trailer trailer, and truck and trailer. Figure A-92 provides a summary of these data for this station. The data shown in these figures are also referred to as FHWA loadometer data.

Rarely would the facility being studied have the same vehicle classification as a loadometer station. The analyst must therefore extrapolate data from loadometer stations to project sites. One technique is to begin with station-specific loadometer data, such as that shown in Table A-35, and then modify the proportion of truck types (e.g., single, tandem) and axle load groups to match the vehicle classification on the specific facility. In so doing, the quantity of axles in each axle load group would change, as would the total number of trucks counted. Multiplying the modified number of axles by the previous equivalence factors and dividing by the number of actual trucks observed on the facility would provide the analyst with a modified ESAL rate.

Vehicle counts typically classify trucks according to number of axles and truck type (e.g., single unit, combination) rather than by axle load groups. As a result, several agencies have established average equivalency factors for each truck classification. These factors are based upon typical weight distributions for each type of truck. Once the average equivalency factors are estimated, they can be applied to any vehicle mix to produce an ESAL rate and total ESAL for a specific facility. For example, the following analysis might be performed for one classification of truck:

Number of single-unit, 2-axle dual-tire trucks (obtained from vehicle classification count)	=	150 ADT
Typical weight for this truck type (obtained from loadometer station data)	=	20,000 lb.
Average equivalency factor (assumes flexible pavement, P_t = 2.5 an SN = 3.0; Table A-31)	=	1.49
Equivalent 18-kip single axles (ESAL) for this truck classification	=	150 * 2-axle/truck * 1.49 = 447

Similar computations could be performed for each truck classification. The individual ESAL values should then be summed and divided by the total number of trucks to produce the desired ESAL rate.

These techniques can be used effectively to account for expected future changes in vehicle mix. For instance, if long term trends show an increase in the number of multiple-axle trucks, the vehicle classification counts can be adjusted (see Chapter 11) such that a more accurate ESAL rate is obtained for design purposes.

Step 5: Calculate daily ESAL and 20-year ESAL.

ESAL (daily) = Truck (Daily) x ESAL (rate) (A-97)

ESAL (20 yr) = Truck (20 yr) x ESAL (rate) (A-98)

These two values, ESAL (daily) and ESAL (20 yr), are then used in Figure A-93 or Figure A-94 for flexible pavements, or Figure A-95 for rigid pavements to determine the final structural number (SN) or depth of slab (D) for flexible and rigid pavements respectively. If the calculated final values of SN or D are significantly different (i.e., greater than 10 percent) from the initial assumed value (see input data), the procedure should be repeated using a new assumed SN or D.

Table A-29. Traffic equivalency factors, flexible pavement, single axles, $P_t = 2.0$.

Axle Load		Structural Number, SN					
Kips	kN	1	2	3	4	5	6
2	8.9	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
4	17.8	0.002	0.003	0.002	0.002	0.002	0.002
6	26.7	0.01	0.01	0.01	0.01	0.01	0.01
8	35.6	0.03	0.04	0.04	0.03	0.03	0.03
10	44.5	0.08	0.08	0.09	0.08	0.08	0.08
12	53.4	0.16	0.18	0.19	0.18	0.17	0.17
14	62.3	0.32	0.34	0.35	0.35	0.34	0.33
16	71.2	0.59	0.60	0.61	0.61	0.60	0.60
18	80.1	1.00	1.00	1.00	1.00	1.00	1.00
20	89.1	1.61	1.59	1.56	1.55	1.57	1.60
22	97.9	2.49	2.44	2.35	2.31	2.35	2.41
24	106.8	3.71	3.62	3.43	3.33	3.40	3.51
26	115.7	5.36	5.21	4.88	4.68	4.77	4.96
28	124.6	7.54	7.31	6.78	6.42	6.52	6.83
30	133.4	10.38	10.03	9.24	8.65	8.73	9.17
32	142.3	14.00	13.51	12.37	11.46	11.48	12.07
34	151.2	18.55	17.87	16.30	14.97	14.87	15.63
36	160.1	24.20	23.30	21.16	19.28	19.02	19.93
38	169.0	31.14	29.95	27.12	24.55	24.03	25.10
40	177.9	39.57	38.02	34.34	30.92	30.04	31.25

Table A-30. Traffic equivalency factors, flexible pavement, tandem axles, $P_t = 2.0$.

Axle Load		Structural Number, SN					
Kips	kN	1	2	3	4	5	6
10	44.5	0.01	0.01	0.01	0.01	0.01	0.01
12	53.4	0.01	0.02	0.02	0.01	0.01	0.01
14	62.3	0.02	0.03	0.03	0.03	0.02	0.02
16	71.2	0.04	0.05	0.05	0.05	0.04	0.04
18	80.1	0.07	0.08	0.08	0.08	0.07	0.07
20	89.0	0.10	0.12	0.12	0.12	0.11	0.10
22	97.9	0.16	0.17	0.18	0.17	0.16	0.16
24	106.8	0.23	0.24	0.26	0.25	0.24	0.23
26	115.7	0.32	0.34	0.36	0.35	0.34	0.33
28	124.6	0.45	0.46	0.49	0.48	0.47	0.46
30	133.4	0.61	0.62	0.65	0.64	0.63	0.62
32	142.3	0.81	0.82	0.84	0.84	0.83	0.82
34	151.2	1.06	1.07	1.08	1.08	1.08	1.07
36	160.1	1.38	1.38	1.38	1.38	1.38	1.38
38	169.0	1.76	1.75	1.73	1.72	1.73	1.74
40	177.9	2.22	2.19	2.15	2.13	2.16	2.18
42	186.8	2.77	2.73	2.64	2.62	2.66	2.70
44	195.7	3.42	3.36	3.23	3.18	3.24	3.31
46	204.6	4.20	4.11	3.92	3.83	3.91	4.02
48	213.5	5.10	4.98	4.72	4.58	4.68	4.83

Table A-31. Traffic equivalency factors, flexible pavement, single axles, $P_t = 2.5$.

Axle Load		Structural Number, SN					
Kips	kN	1	2	3	4	5	6
2	8.9	0.0004	0.0004	0.0003	0.0002	0.0002	0.0002
4	17.8	0.003	0.004	0.004	0.003	0.003	0.002
6	26.7	0.01	0.02	0.02	0.01	0.01	0.01
8	35.6	0.03	0.05	0.05	0.04	0.03	0.03
10	44.5	0.08	0.10	0.12	0.10	0.09	0.08
12	53.4	0.17	0.20	0.23	0.21	0.19	0.18
14	62.3	0.33	0.36	0.40	0.39	0.36	0.34
16	71.2	0.59	0.61	0.65	0.65	0.62	0.61
18	80.1	1.00	1.00	1.00	1.00	1.00	1.00
20	89.0	1.61	1.57	1.49	1.47	1.51	1.55
22	97.9	2.48	2.38	2.17	2.09	2.18	2.30
24	106.8	3.69	3.49	3.09	2.89	3.03	3.27
26	115.7	5.33	4.99	4.31	3.91	4.09	4.48
28	124.6	7.49	6.98	5.90	5.21	5.39	5.98
30	133.4	10.31	9.55	7.94	6.83	6.97	7.79
32	142.3	13.90	12.82	10.52	8.85	8.88	9.95
34	151.2	18.41	16.94	13.74	11.34	11.18	12.51
36	160.1	24.02	22.04	17.73	14.38	13.93	15.50
38	169.0	30.90	28.30	22.61	18.06	17.20	18.98
40	177.9	39.26	35.89	28.51	22.50	21.08	23.04

Table A-32. Traffic equivalency factors, flexible pavement, tandem axles, $P_t = 2.5$.

Axle Load		Structural Number, SN					
Kips	kN	1	2	3	4	5	6
10	44.5	0.01	0.01	0.01	0.01	0.01	0.01
12	53.4	0.02	0.02	0.02	0.02	0.01	0.01
14	62.3	0.03	0.04	0.04	0.03	0.03	0.02
16	71.2	0.04	0.07	0.07	0.06	0.05	0.04
18	80.1	0.07	0.10	0.11	0.09	0.08	0.07
20	89.0	0.11	0.14	0.16	0.14	0.12	0.11
22	97.9	0.16	0.20	0.23	0.21	0.18	0.17
24	106.8	0.23	0.27	0.31	0.29	0.26	0.24
26	115.7	0.33	0.37	0.42	0.40	0.36	0.34
28	124.6	0.45	0.49	0.55	0.53	0.50	0.47
30	133.4	0.61	0.65	0.70	0.70	0.66	0.63
32	142.3	0.81	0.84	0.89	0.89	0.86	0.83
34	151.2	1.06	1.08	1.11	1.11	1.09	1.08
36	160.1	1.38	1.38	1.38	1.38	1.38	1.38
38	169.0	1.75	1.73	1.69	1.68	1.70	1.73
40	177.9	2.21	2.16	2.06	2.03	2.08	2.14
42	186.8	2.76	2.67	2.49	2.43	2.51	2.61
44	195.7	3.41	3.27	2.99	2.88	3.00	3.16
46	204.6	4.18	3.98	3.58	3.40	3.55	3.79
48	213.5	5.08	4.80	4.25	3.98	4.17	4.49

Table A-33. Traffic equivalency factors, rigid pavement, single axles, $P_t = 2.5$.

Axle Load		D - Slab Thickness - inches						
Kips	kN	6	7	8	9	10	11	12
2	8.9	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
4	17.8	0.003	0.002	0.002	0.002	0.002	0.002	0.002
6	26.7	0.01	0.01	0.01	0.01	0.01	0.01	0.01
8	35.6	0.04	0.04	0.03	0.03	0.03	0.03	0.03
10	44.5	0.10	0.09	0.08	0.08	0.08	0.08	0.08
12	53.4	0.20	0.19	0.18	0.18	0.18	0.17	0.17
14	62.3	0.38	0.36	0.35	0.34	0.34	0.34	0.34
16	71.2	0.63	0.62	0.61	0.60	0.60	0.60	0.60
18	80.1	1.00	1.00	1.00	1.00	1.00	1.00	1.00
20	89.0	1.51	1.52	1.55	1.57	1.58	1.58	1.59
22	97.9	2.21	2.20	2.28	2.34	2.38	2.40	2.41
24	106.8	3.16	3.10	3.23	3.36	3.45	3.50	3.53
26	115.7	4.41	4.26	4.42	4.67	4.85	4.95	5.01
28	124.6	6.05	5.76	5.92	6.29	6.61	6.81	6.92
30	133.4	8.16	7.67	7.79	8.28	8.79	9.14	9.34
32	142.3	10.81	10.06	10.10	10.70	11.43	11.99	12.35
34	151.2	14.12	13.04	12.34	13.62	14.59	15.43	16.01
36	160.1	18.20	16.69	16.41	17.12	18.33	19.52	20.39
38	169.0	23.15	21.14	20.61	21.31	22.74	24.31	25.58
40	177.9	29.11	26.49	25.65	26.29	27.91	29.90	31.64

Table A-34. Traffic equivalency factors, flexible pavement, tandem axles, $P_t = 2.5$.

Axle Load		D - Slab Thickness - inches						
Kips	kN	6	7	8	9	10	11	12
10	44.5	0.01	0.01	0.01	0.01	0.01	0.01	0.01
12	53.4	0.03	0.03	0.03	0.03	0.03	0.03	0.03
14	62.3	0.06	0.05	0.05	0.05	0.05	0.05	0.05
16	71.2	0.10	0.09	0.08	0.08	0.08	0.08	0.08
18	80.1	0.16	0.14	0.14	0.13	0.13	0.13	0.13
20	89.0	0.23	0.22	0.21	0.21	0.20	0.20	0.20
22	97.9	0.34	0.32	0.31	0.31	0.30	0.30	0.30
24	106.8	0.48	0.46	0.45	0.44	0.44	0.44	0.44
26	115.7	0.64	0.64	0.63	0.62	0.62	0.62	0.62
28	124.6	0.85	0.85	0.85	0.85	0.85	0.85	0.85
30	133.4	1.11	1.12	1.13	1.14	1.14	1.14	1.14
32	142.3	1.43	1.44	1.47	1.49	1.50	1.51	1.51
34	151.2	1.82	1.82	1.87	1.92	1.95	1.96	1.97
36	160.1	2.29	2.27	2.35	2.43	2.48	2.51	2.52
38	169.0	2.85	2.80	2.91	3.04	3.12	3.16	3.18
40	177.9	3.52	3.42	3.55	3.74	3.87	3.94	3.98
42	186.8	4.32	4.16	4.30	4.55	4.74	4.86	4.91
44	195.7	5.26	5.01	5.16	5.48	5.75	5.92	6.01
46	204.6	6.36	6.01	6.14	6.53	6.90	7.14	7.28
48	213.5	7.64	7.16	7.27	7.73	8.21	8.55	8.75

SUMMARY OF 18-KIP RATES AND EQUIVALENTS FOR FLEXIBLE PAVEMENT DESIGN (P = 2.5 AND SN = 5)

STATE OF ILLINOIS INTERSTATE RURAL FUNC. CLASS NUMBER OF STATIONS = 6	NUMBER OF TRUCKS WEIGHED 1981	RATIO OF TRUCKS WEIGHED 1981/79	NUMBER OF TRUCKS COUNTED 1981	% DISTR OF TRUCKS COUNTED 1981	RATIO OF TRUCKS COUNTED 1981/79	18-K EQV PER 1000 TRUCKS WEIGHED 1981	RATIO OF 18-K EQV PER 1000 WEIGHED 1981/79	18-K EQV FOR ALL TRUCKS COUNTED 1981	% DISTR 18-K EQV ALL TRKS COUNTED 1981	RATIO OF 18-K EQV ALL TRKS COUNTED 1981/79
SINGLE UNIT TRUCKS										
PANEL AND PICKUP	175	1.07	8,401	26.19	1.20	1.9	0.25	16.1	0.11	0.30
OTHER 2 AXLE 4 TIRE	77	2.75	693	2.16	0.53	54.0	0.27	37.5	0.24	0.14
2 AXLE 6 TIRE	112	0.83	2,515	7.84	1.06	204.2	1.60	513.6	3.30	1.70
3 AXLE OR MORE	41	0.82	565	1.76	0.84	300.0	0.85	169.5	1.09	0.72
ALL SINGLE UNIT TRKS	405	1.08	12,174	37.96	1.07	60.1	0.80	736.7	4.74	0.87
TRACTOR SEMI TRLRS										
3 AXLE	32	0.74	313	0.98	0.82	469.7	1.71	147.0	0.94	1.41
4 AXLE	123	0.71	1,319	4.11	0.93	456.5	0.87	602.2	3.86	0.82
5 AXLE OR MORE	1,325	0.84	17,532	54.66	1.05	739.8	1.00	12,970.0	83.24	1.05
ALL TRAC. SEMI TRLRS	1,480	0.83	19,164	59.75	1.04	715.8	1.00	13,719.2	88.04	1.04
ALL COMBINATIONS	1,556	0.85	19,899	62.04	1.04	745.6	1.03	14,845.1	95.26	1.08
TOTAL TRUCKS	1,961	0.89	32,073	100.00	1.05	$\boxed{485.5}$ ESAL RATE	1.01	15,581.8	100.00	1.07

SUMMARY OF 18-KIP RATES AND EQUIVALENTS FOR RIGID PAVEMENT DESIGN (P = 2.5 AND D = 9 IN.)

STATE OF ILLINOIS INTERSTATE RURAL FUNC. CLASS NUMBER OF STATIONS = 6	NUMBER OF TRUCKS WEIGHED 1981	RATIO OF TRUCKS WEIGHED 1981/79	NUMBER OF TRUCKS COUNTED 1981	% DISTR OF TRUCKS COUNTED 1981	RATIO OF TRUCKS COUNTED 1981/79	18-K EQV PER 1000 TRUCKS WEIGHED 1981	RATIO OF 18-K EQV PER 1000 WEIGHED 1981/79	18-K EQV FOR ALL TRUCKS COUNTED 1981	% DISTR 18-K EQV ALL TRKS COUNTED 1981	RATIO OF 18-K EQV ALL TRKS COUNTED 1981/79
SINGLE UNIT TRUCKS										
PANEL AND PICKUP	175	1.07	8,401	26.19	1.20	1.9	0.25	16.1	0.08	0.31
OTHER 2 AXLE 4 TIRE	77	2.75	693	2.16	0.53	51.5	0.27	35.7	0.15	0.14
2 AXLE 6 TIRE	112	0.83	2,515	7.84	1.06	215.5	1.76	542.0	2.30	1.87
3 AXLE OR MORE	41	0.82	565	1.76	0.84	381.6	0.79	215.6	0.91	0.67
ALL SINGLE UNIT TRKS	405	1.08	12,174	37.96	1.07	66.0	0.82	809.4	3.44	0.89
TRACTOR SEMI TRLRS										
3 AXLE	32	0.74	313	0.98	0.82	446.5	1.73	139.8	0.59	1.43
4 AXLE	123	0.71	1,319	4.11	0.93	497.2	0.86	655.8	2.78	0.81
5 AXLE OR MORE	1,325	0.84	17,532	54.66	1.05	1,187.6	1.00	20,820.2	88.28	1.06
ALL TRAC. SEMI TRLRS	1,480	0.83	19,164	59.75	1.04	1,127.8	1.01	21,615.8	91.65	1.05
ALL COMBINATIONS	1,556	0.85	19,899	62.04	1.04	1,143.9	1.02	22,773.9	96.56	1.07
TOTAL TRUCKS	1,961	0.89	32,073	100.00	1.05	$\boxed{734.8}$ ESAL RATE	1.01	23,583.3	100.00	1.07

Figure A-90. Example of statewide ESAL.

Table A-35. Determination of ESAL from loadometer station data.

Axle Load Groups, lb	Representative Axle Load, lb	Equiv. Factor 1/	No. of Axles 2/	Equiv. 18-kip Single Axles 3/
Single Axles				
Under 3,000	2,000	0.0003	512	0.2
3,000- 6,999	5,000	0.012	536	6.4
7,000- 7,999	7,500	0.0425	239	10.2
8,000-11,999	10,000	0.12	1,453	174.4
12,000-15,999	14,000	0.40	279	111.6
16,000-18,000	17,000	0.825	106	87.5
18,001-20,000	19,000	1.245	43	53.5
20,001-21,999	21,000	1.83	4	7.3
22,000-23,999	23,000	2.63	3	7.9
24,000 and over	-	-	0	-
				459.0
Tandem Axles				
Under 6,000	4,000	0.01	9	-
6,000-11,999	9,000	0.008	337	2.7
12,000-17,999	15,000	0.055	396	21.8
18,000-23,999	21,000	0.195	457	89.1
24,000-29,999	27,000	0.485	815	395.3
30,000-32,000	31,000	0.795	342	271.9
32,001-33,999	33,000	1.00	243	243.0
34,000-35,999	35,000	1.245	173	215.4
36,000-37,999	37,000	1.535	71	109.0
38,000-39,999	39,000	1.875	9	16.9
40,000-41,999	41,000	2.275	0	-
42,000-43,999	43,000	2.74	1	2.7
44,000 and over	-	-	0	-
				1,367.8

Total ESAL = 459.0 + 1367.8 = 1826.8

ESAL Rate = (1826.8/13146) * 1000 = 580.7

1/ Pt = 2.5 and SN = 3.0 (obtained from Tables A-31 and A-34 with interpolation).

2/ Total number of trucks represented by this axle load data is 3,146 trucks.

3/ As noted earlier in text, ESAL or Equiv. 18-kip Single Axles = Equiv. Factor x Number of Axles.

STATE OF
STATION 083
FUNC CLASS 11

PART 2 OF 5

TABLE W-4

STATE OF
STATION 083
FUNC CLASS 11

AXLE LOADS IN POUNDS AND EIGHTEEN KIP AXLE EQUIVALENCY ITEMS	18 KIP AXLE EQUIVALENCY FACTOR: RIGID PAVEMENT P=2.5, D=9"	18 KIP AXLE EQUIVALENCY FACTOR: FLEXIBLE PAVEMENT P=2.5, SN=5	TRACTOR SEMI-TRAILER COMBINATIONS: 3 AXLE 1981	3 AXLE 1979	4 AXLE 1981	4 AXLE 1979	5 AXLE OR MORE 1981	5 AXLE OR MORE 1979	TRACTOR SEMI-TRAILER COMBINATIONS PROBABLE NO. 1981	1979
SINGLE AXLES										
			0	0	0	0	0	0	0	0
UNDER 3,000	0.0002	0.0002	3	0	1	1	12	9	268	221
3,000 - 6,999	0.0050	0.0050	3	5	6	2	10	8	275	287
7,000 - 7,999	0.0260	0.0320	3	2	25	14	163	119	3265	2963
8,000 - 11,999	0.0820	0.0870	0	0	2	4	4	8	92	262
12,000 - 15,999	0.3410	0.3600	0	1	6	1	0	0	53	34
16,000 - 18,000	0.7830	0.7960	0	1	0	0	1	0	18	13
18,001 - 18,500	1.0650	1.0600	0	0	0	0	1	0	18	0
18,501 - 20,000	1.3360	1.3070	0	0	0	0	0	0	0	0
20,001 - 21,999	1.9260	1.8260	0	0	0	0	0	0	0	0
22,000 - 23,999	2.8180	2.5830	0	0	0	0	0	0	0	0
24,000 - 25,999	3.9760	3.5330	0	0	0	0	0	0	0	0
26,000 - 29,999	6.2890	5.3890	0	0	0	0	0	0	0	0
30,000 OR OVER	11.3950	9.4320	0	0	0	0	0	0	0	0
TOTAL SINGLE AXLES WEIGHED			9	9	40	22	191	144		
TOTAL SINGLE AXLES COUNTED			114	120	354	468	3521	3192	3989	3780
TANDEM AXLE GROUPS										
UNDER 6,000	0.0100	0.0100	0	0	0	0	0	0	0	0
6,000 - 11,999	0.0100	0.0100	0	0	8	6	64	74	1250	1767
12,000 - 17,999	0.0620	0.0440	0	0	6	3	48	42	938	995
18,000 - 23,999	0.2530	0.1480	0	0	6	2	55	29	1067	686
24,000 - 29,999	0.7290	0.4260	0	0	0	0	98	62	1807	1374
30,000 - 32,000	1.3050	0.7530	0	0	0	0	74	52	1364	1153
32,001 - 32,500	1.5420	0.8850	0	0	0	0	6	1	111	22
32,501 - 33,999	1.7510	1.0020	0	0	0	0	3	2	55	44
34,000 - 35,999	2.1650	1.2300	0	0	0	0	1	1	18	22
36,000 - 37,999	2.7210	1.5330	0	0	0	0	0	0	0	0
38,000 - 39,999	3.3730	1.8850	0	0	0	0	0	0	0	0
40,000 - 41,999	4.1290	2.2890	0	0	0	0	0	0	0	0
42,000 - 43,999	4.9970	2.7490	0	0	0	0	0	0	0	0
44,000 - 45,999	5.9870	3.2690	0	0	0	0	0	0	0	0
46,000 - 49,999	7.7250	4.1700	0	0	0	0	0	0	0	0
50,000 OR OVER	10.1600	5.1000	0	0	0	0	0	0	0	0
TOTAL TANDEM AXLES WEIGHED			0	0	20	11	349	263		
TOTAL TANDEM AXLES COUNTED			0	0	177	234	6432	5829	6610	6063
UNDER 3,000			0	0	0	0	10	7	184	156
3,000 - 6,999			3	0	21	17	170	186	3357	4485
7,000 - 7,999			3	5	7	4	40	33	837	883
8,000 - 11,999			3	2	43	18	300	201	5949	4865
12,000 - 15,999			0	0	3	4	302	202	5594	4562
16,000 - 16,250			0	0	2	0	17	14	331	310
16,251 - 17,999			0	1	4	1	43	26	828	610
18,000 - 18,500			0	1	0	0	4	1	74	35
18,501 - 19,999			0	0	0	0	3	0	55	0
20,000 - 21,999			0	0	0	0	0	0	0	0
22,000 - 23,999			0	0	0	0	0	0	0	0
24,000 - 25,999			0	0	0	0	0	0	0	0
26,000 - 29,999			0	0	0	0	0	0	0	0
30,000 OR OVER			0	0	0	0	0	0	0	0
TOTAL AXLES WEIGHED			9	9	80	44	885	670		
TOTAL AXLES COUNTED			114	120	708	936	16387	14850	17209	15906
TOTAL VEHICLES COUNTED			38	40	177	234	3281	2970	3496	3244
18 KIP AXLE EQUIVALENTS										
RIGID PAVEMENT, P=2.5, D=9"										
18 K EQV FOR ALL TRUCKS WEIGHED			0.3	2.1	5.6	4.1	215.7	143.7	229.6	149.9
18 K EQV PER 1000 TRUCKS WEIGHED			113.0	714.0	478.0	373.1	1234.1	1072.3	1183.3	1017.0
18 K EQV FOR ALL TRUCKS COUNTED			4.3	28.6	84.6	87.3	4045.0	3184.7	4137.9	3300.6
PERCENT DISTRIBUTION OF 18 K EQV			0.10	0.79	1.94	2.41	92.72	87.77	94.76	90.97
FLEXIBLE PAVEMENT, P=2.5, SN=5										
18 K EQV FOR ALL TRUCKS WEIGHED			0.4	2.2	5.1	4.0	136.3	90.1	145.8	96.3
18 K EQV PER 1000 TRUCKS WEIGHED			124.0	730.0	455.0	364.6	765.6	672.4	742.6	650.5
18 K EQV FOR ALL TRUCKS COUNTED			4.7	29.2	80.5	85.3	2511.9	1997.0	2597.1	2111.5
PERCENT DISTRIBUTION OF 18 K EQV			0.17	1.20	2.88	3.50	65.84	81.84	92.89	86.54

Figure A-91. Example of station-specific ESAL data for tractor semi-trailer combinations.

STATE OF
STATION 08?
FUNC CLASS 11

PART 5 OF 5

TABLE W-4

STATE OF
STATION 083
FUNC CLASS 11

AXLE LOADS IN POUNDS AND EIGHTEEN KIP AXLE EQUIVALENCY ITEMS	18 KIP AXLE EQUIVALENCY FACTOR RIGID PAVEMENT P=2.5, D=9"	18 KIP AXLE EQUIVALENCY FACTOR FLEXIBLE PAVEMENT P=2.5, SN=5	TOTAL ALL COMBINATIONS PROBABLE NO. 1981	TOTAL ALL COMBINATIONS PROBABLE NO. 1979	TOTAL ALL TRUCKS AND COMBINATIONS PROBABLE NO. 1981	TOTAL ALL TRUCKS AND COMBINATIONS PROBABLE NO. 1979	PERCENT HEAVIER THAN LOW WEIGHT INTERVAL 1981	PERCENT HEAVIER THAN LOW WEIGHT INTERVAL 1979	AXLES PER 1000 TRUCKS AND COMBINATIONS 1981	AXLES PER 1000 TRUCKS AND COMBINATIONS 1979	RATIO 1981 1979
SINGLE AXLES											
UNDER 3,000	0.0002	0.0002	0	4	7227	6698	100.00	100.00	833.56	801.39	1.040
3,000 - 6,999	0.0050	0.0050	297	315	2479	2512	49.28	52.71	285.93	300.55	0.951
7,000 - 7,999	0.0260	0.0320	289	307	515	492	31.88	34.98	59.40	58.87	1.009
8,000 - 11,999	0.0820	0.0870	3338	3090	3643	3587	28.26	31.51	420.18	429.17	0.979
12,000 - 15,999	0.3410	0.3600	162	516	248	755	2.70	6.18	28.60	90.33	0.317
16,000 - 18,000	0.7830	0.7960	100	88	100	108	0.95	0.85	11.53	12.92	0.893
18,001 - 18,500	1.0650	1.0600	18	13	18	13	0.25	0.09	2.08	1.56	1.335
18,501 - 20,000	1.3360	1.3070	18	0	18	0	0.13	0.0	2.08	0.0	0.0
20,001 - 21,999	1.9260	1.8260	0	0	0	0	0.0	0.0	0.0	0.0	0.0
22,000 - 23,999	2.8180	2.5830	0	0	0	0	0.0	0.0	0.0	0.0	0.0
24,000 - 25,999	3.9760	3.5330	0	0	0	0	0.0	0.0	0.0	0.0	0.0
26,000 - 29,999	6.2890	5.3890	0	0	0	0	0.0	0.0	0.0	0.0	0.0
30,000 OR OVER	11.3950	9.4320	0	0	0	0	0.0	0.0	0.0	0.0	0.0
TOTAL SINGLE AXLES WEIGHED											
TOTAL SINGLE AXLES COUNTED			4222	4333	14248	14165	100.00	100.00	0.0	0.0	0.0
TANDEM AXLE GROUPS											
UNDER 6,000	0.0100	0.0100	0	2	0	2	100.00	100.00	0.0	0.24	0.0
6,000 - 11,999	0.0100	0.0100	1250	1767	1281	1804	100.00	99.97	147.75	215.84	0.685
12,000 - 17,999	0.0620	0.0440	994	997	994	1017	81.13	70.71	114.65	121.68	0.942
18,000 - 23,999	0.2530	0.1480	1067	696	1067	706	66.49	54.22	123.07	84.47	1.457
24,000 - 29,999	0.7290	0.4260	1807	1376	1869	1376	50.78	42.77	215.57	164.63	1.309
30,000 - 32,000	1.3050	0.7530	1364	1153	1395	1173	23.25	20.45	160.90	140.34	1.146
32,001 - 32,500	1.5420	0.8850	111	22	111	22	2.71	1.43	12.80	2.63	4.864
32,501 - 33,999	1.7510	1.0020	55	44	55	44	1.08	1.07	6.34	5.26	1.205
34,000 - 35,999	2.1650	1.2300	18	22	18	22	0.27	0.36	2.08	2.63	0.789
36,000 - 37,999	2.7210	1.5330	0	0	0	0	0.0	0.0	0.0	0.0	0.0
38,000 - 39,999	3.3730	1.8850	0	0	0	0	0.0	0.0	0.0	0.0	0.0
40,000 - 41,999	4.1290	2.2890	0	0	0	0	0.0	0.0	0.0	0.0	0.0
42,000 - 43,999	4.9970	2.7490	0	0	0	0	0.0	0.0	0.0	0.0	0.0
44,000 - 45,999	5.9870	3.2690	0	0	0	0	0.0	0.0	0.0	0.0	0.0
46,000 - 49,999	7.7250	4.1700	0	0	0	0	0.0	0.0	0.0	0.0	0.0
50,000 OR OVER	10.1600	5.1000	0	0	0	0	0.0	0.0	0.0	0.0	0.0
TOTAL TANDEM AXLES WEIGHED											
TOTAL TANDEM AXLES COUNTED			6666	6069	6790	6166	100.00	100.00	0.0	0.0	0.0
ALL AXLES											
UNDER 3,000			184	164	7411	6858	100.00	100.00	854.79	820.53	1.042
3,000 - 6,999			3456	4579	5700	6854	73.37	74.12	657.44	820.05	0.802
7,000 - 7,999			865	905	1091	1129	52.89	48.25	125.84	135.08	0.932
8,000 - 11,999			6050	4994	6355	5510	48.57	43.59	732.99	659.25	1.112
12,000 - 15,999			5664	4818	5905	5096	26.13	23.20	681.08	609.72	1.117
16,000 - 16,250			343	328	374	348	4.51	3.57	43.14	41.64	1.036
16,251 - 17,999			863	648	863	668	3.56	2.65	99.54	79.92	1.245
18,000 - 18,500			74	35	74	35	0.46	0.13	8.54	4.19	2.038
18,501 - 19,999			55	0	55	0	0.20	0.0	6.34	0.0	0.0
20,000 - 21,999			0	0	0	0	0.0	0.0	0.0	0.0	0.0
22,000 - 23,999			0	0	0	0	0.0	0.0	0.0	0.0	0.0
24,000 - 25,999			0	0	0	0	0.0	0.0	0.0	0.0	0.0
26,000 - 29,999			0	0	0	0	0.0	0.0	0.0	0.0	0.0
30,000 OR OVER			0	0	0	0	0.0	0.0	0.0	0.0	0.0
TOTAL AXLES WEIGHED											
TOTAL AXLES COUNTED			17554	16471	27828	26498	100.00	100.00	0.0	0.0	0.0
TOTAL VEHICLES COUNTED			3573	3369	8670	8358	0.0	0.0	0.0	0.0	0.0

18 KIP AXLE EQUIVALENTS

RIGID PAVEMENT, P=2.5, D=9"				
18 K EQV FOR ALL TRUCKS WEIGHED	241.2	159.2	245.3	163.9
18 K EQV PER 1000 TRUCKS WEIGHED	1177.5	1021.4	$\boxed{503.6}$	434.2
18 K EQV FOR ALL TRUCKS COUNTED	4208.5	3442.0	4366.9	3628.3
PERCENT DISTRIBUTION OF 18 K EQV	96.37	94.87	100.00	100.00
FLEXIBLE PAVEMENT, P=2.5, SN=5				
18 K EQV FOR ALL TRUCKS WEIGHED	157.7	105.7	160.8	109.8
18 K EQV PER 1000 TRUCKS WEIGHED	746.8	670.2	$\boxed{322.4}$	292.0
18 K EQV FOR ALL TRUCKS COUNTED	2669.0	2258.5	2796.0	2440.0
PERCENT DISTRIBUTION OF 18 K EQV	95.46	92.56	100.00	100.00

ESAL RATE 1981

Figure A-92. Example summary of station-specific ESAL data.

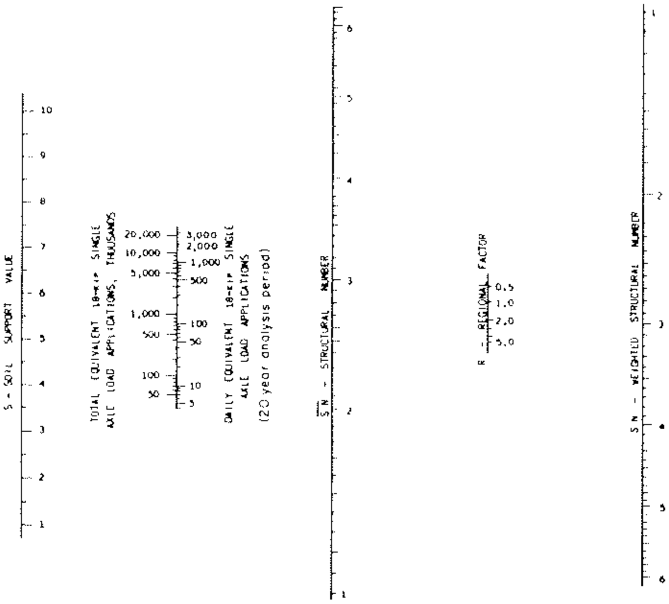


Figure A-93. Design chart for flexible pavements, $P_t = 2.0$.

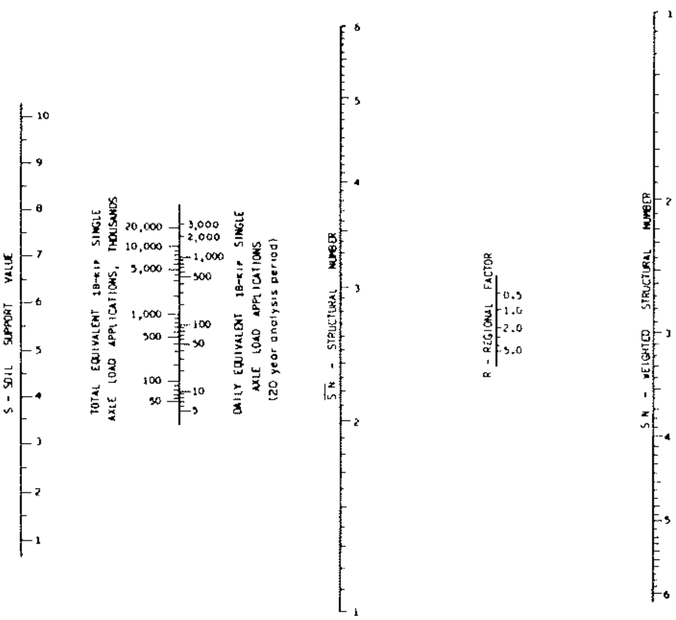


Figure A-94. Design chart for flexible pavements, $P_t = 2.5$.

EXAMPLE PROBLEM

The following example illustrates these computations when the ESAL rate is derived from a loadometer station or statewide average.

Input Data and Assumptions:

Highway Type	=	main rural road (Non-Interstate)
Pavement Type	=	flexible (P_+ = 2.5 and SN = 5)
Base Year ADT	=	4,000 vehicles per day
20 yr ADT	=	6,000 vehicles per day
Directional Distribution	=	60 percent/40 percent
Base Year Percent Trucks	=	20 percent
18-kip ESAL rate	=	485.5 per 1,000 trucks obtained from Figure A-90 an example FHWA loadometer table for rural interstate highways.

Step 1.	ADT (avg)	=	(4,000 + 6,000)/2 = 5,000 vehicles per day
Step 2.	ADT (design)	=	5,000 x .60 x 1.00 = 3,000 vehicles per day
Step 3.	Truck (Daily)	=	3,000 x .20 = 600 trucks per day
	Truck (20 yr)	=	600 x 365 x 20 = 4,380,000 trucks in 20 yrs
Step 4.	ESAL (rate)	=	485.5 per 1,000 trucks (see input data)
Step 5.	ESAL (daily)	=	600 x (485.5/1,000) = 291.3
	ESAL (20 yr)	=	4,380,000 x (485.5/1,000) = 2,126,490

If the ESAL rate is extrapolated from loadometer station data to site-specific conditions, the value used in Step 4 would be different. However, the remainder of the methodology would remain the same.

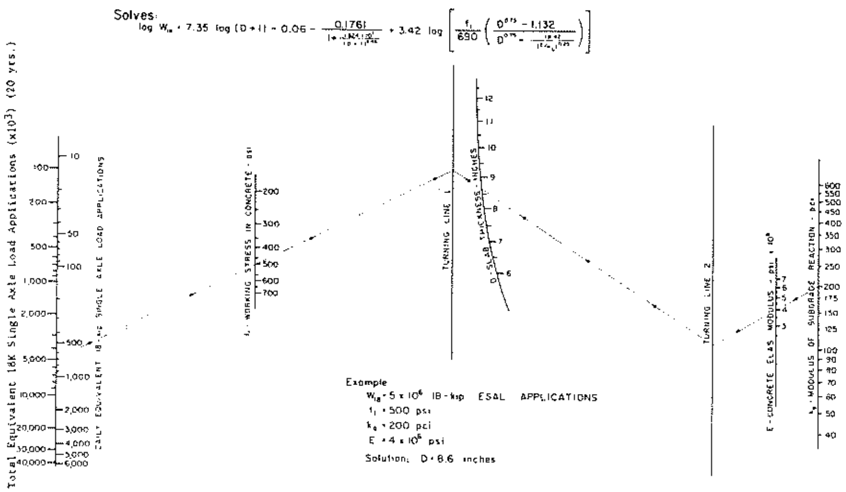


Figure A-95. Design chart for rigid pavements, P_t = 2.5.

CHAPTER FOURTEEN
CASE STUDY: USE OF REFINEMENT PROCEDURES FOR UPGRADING OF A LIMITED ACCESS HIGHWAY

INTRODUCTION

With the current emphasis on Transportation System Management, many studies are being conducted with the intent to improve the capacity of existing roadways rather than to build a completely new facility on a new right-of-way. This case study examines a four-lane highway that is being considered for upgrading to a six-lane limited access facility. The analysis concentrates on techniques for refining computerized travel demand forecasts, and developing design hour volume and directional distribution data. Capacity and level-of-service analyses are included as a check of the results.

Route A, shown in Figure A-96, is a major radial highway located in a metropolitan area with a population of over 2 million. The highway connects the CBD with outlying suburbs and provides access to recreational areas. Land use in the corridor is primarily residential, but commercial uses are expected to develop in the corridor's extreme eastern and western portions over the next 20 years. The link-node network for the corridor is shown in Figure A-96, along with the available base year ADT counts. Two existing two-lane roadways, Route N to the north and Route S to the south, provide competitive travel facilities to Route A. The only proposed transportation improvement in the corridor is the upgrading of Route A.

The computer-generated travel assignments for the base year and the future year are shown in Figures A-97 and A-98 respectively. These data are the raw assignments and must be refined for design purposes. The remainder of this chapter reviews the process to refine these assignments.

SUMMARY OF SCENARIO STEPS

The following steps were performed in the analysis:

Step 1: Prepare data base.
Step 2: Select screenlines and check screenline assignments.
Step 3: Perform calculations.
Step 4: Conduct final assignment checks.
Step 5: Determine future year peak hour directional volumes.
Step 6: Perform capacity calculations.

SCENARIO DETAILS

The following sections describe in detail the steps performed in analyzing the travel demand forecasts for the Route A corridor. Examples are given of all pertinent calculations.

Step 1—Prepare Data Base

The five steps involved in preparing the data base for application of the traffic forecast refinement procedure include the following:

1. Define study area boundaries.
2. Define base year and future year.

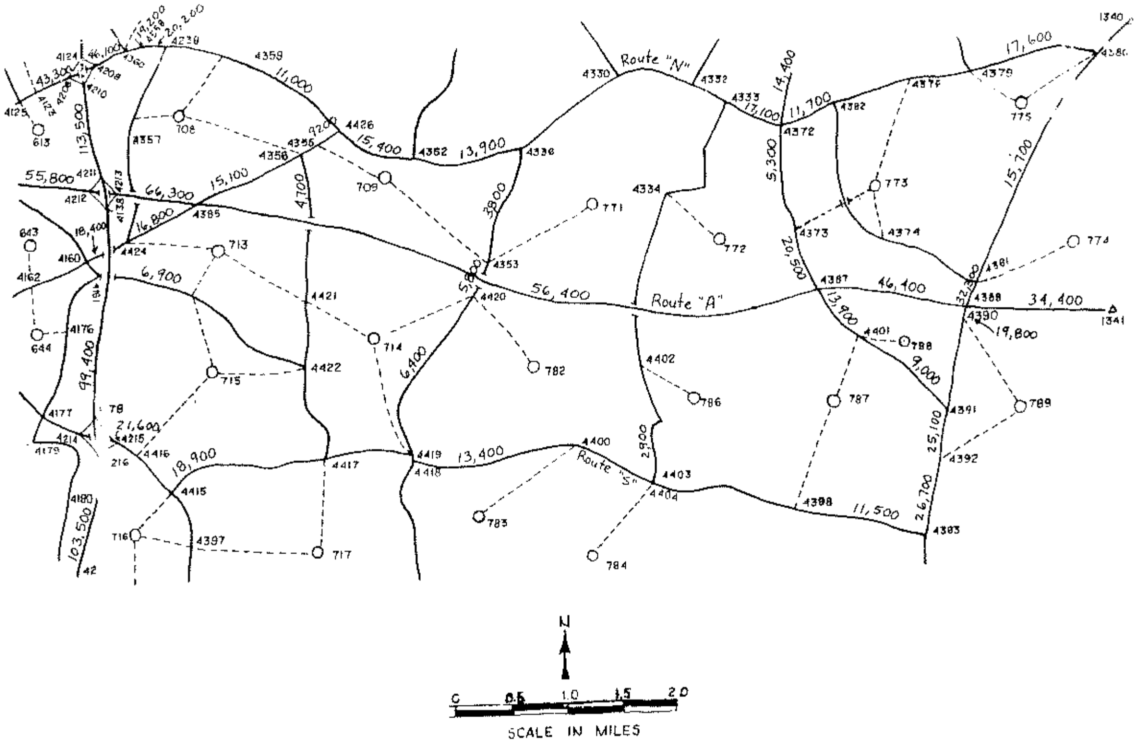


Figure A-96. Case study network and base year counts.

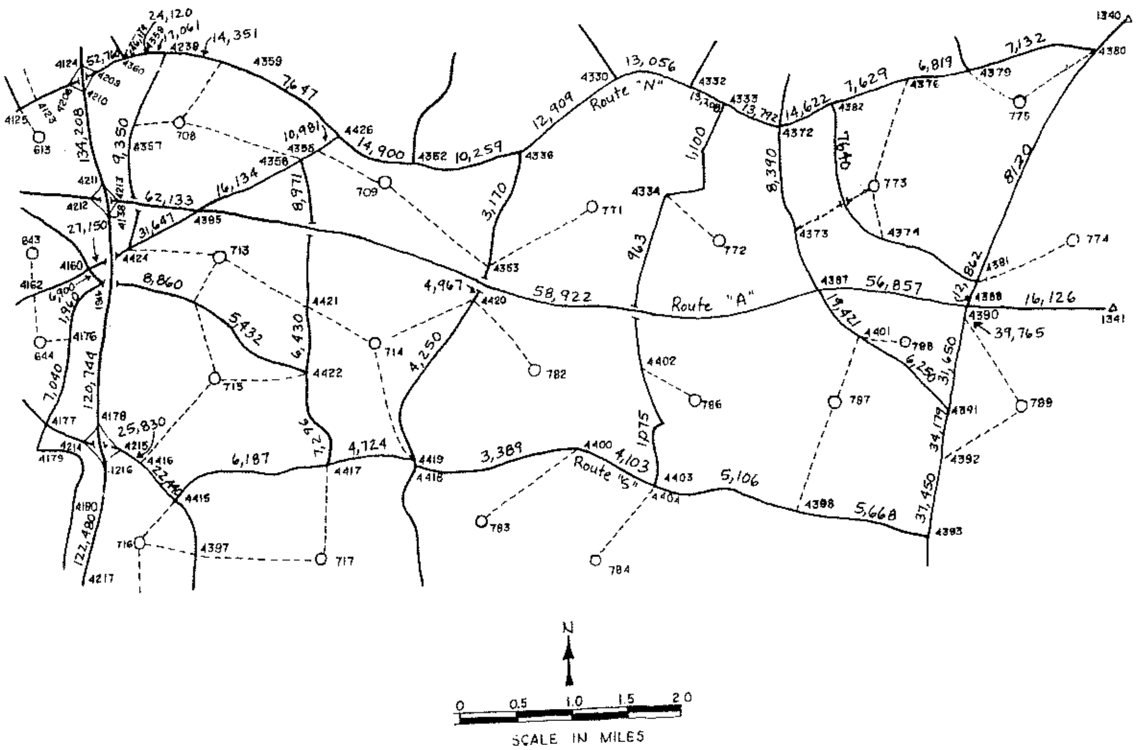


Figure A-97. Base year traffic assignment.

3. Identify link and/or node characteristics.
4. Record base year traffic counts.
5. Record base and future year traffic assignments.

These steps were previously performed; the results are shown in Figures A-96, A-97, and A-98. Figure A-96 indicates the study area and base year ADT counts, and Figures A-97 and A-98 show the base year and future year computer traffic assignments respectively. The preparation of this data base was contingent on the base year traffic assignment passing the five reasonableness checks outlined in Chapter 3, Preliminary Checks of Computerized Traffic Volume Forecasts.

Step 2—Select Screenlines and Check Screenline Assignments

The screenlines selected for the refinement process are shown in Figure A-99. Screenlines A-A, B-B, and C-C provided three opportunities to balance east/west traffic assignments on Route A and its two competitive facilities N and S. North-south travel to and from Route A would be balanced using screenlines D-D, E-E, F-F, and G-G. This selection of screenlines permitted refined volumes to be determined for each north-south or east-west route in the corridor.

Selecting the locations of screenlines A-A and D-D required the use of judgment. For instance, screenline A-A would have been more useful if placed east of Node 4387 on Route A because this would have offered the opportunity to refine the traffic volume on another link of Route A. However, such a screenline would have had a diagonal roadway crossing, a situation which should be avoided. Therefore, A-A-s selected location was the most reasonable available. Conversely, screenline D-D did not avoid a diagonal roadway. In this case there was no alternative screenline location for D-D, and therefore the need to have a screenline in this portion of the corridor overrode the desire to avoid a diagonal roadway crossing.

The screenlines were subjected to two checks in order to assess their adequacy and reliability. Each screenline's length and link density were determined as shown in Table A-36, and then plotted on Figure A-100 (Fig. A-7 from Chapter 4) to determine their adequacy. All screenlines proved to be acceptable, although screenlines E-E and F-F were at the border of the acceptable range. Therefore, the results of the refinement procedure should be reviewed carefully for those two screenlines. Each screenline was also checked for the adequacy of the base year traffic assignment as compared to the base year counts. Table A-37 shows the calculations involved; the data points were plotted on Figure A-101 (Fig. A-9 from Chapter 4). Again, all of the screenlines were within the acceptable range.

Step 3—Perform Calculations

The traffic assignment refinement calculations for screenlines A-A through G-G are shown in Figures A-102 through A-108 respectively. These straightforward calculations follow the procedures detailed in Chapter 4. There is one item to note regarding screenlines A-A, B-B, and C-C. Route A's capacity increased significantly (50%) from the base year. Because of this large capacity increase, the future year traffic assignment on Route A was not subject to the first adjustment by the ratio and difference methods (see Chapter 4, Step 3-2). Therefore, these adjustments were left blank on the calculation sheets for each of the screenlines.

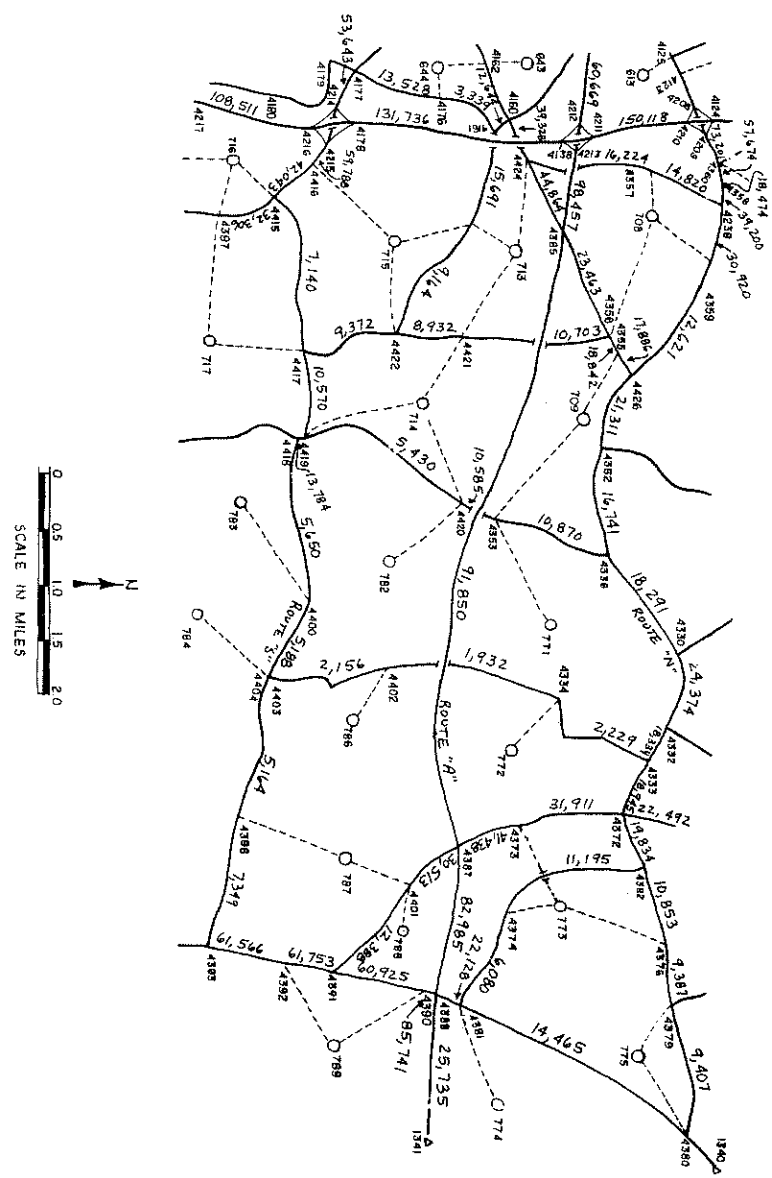


Figure A-98. Future year traffic forecast.

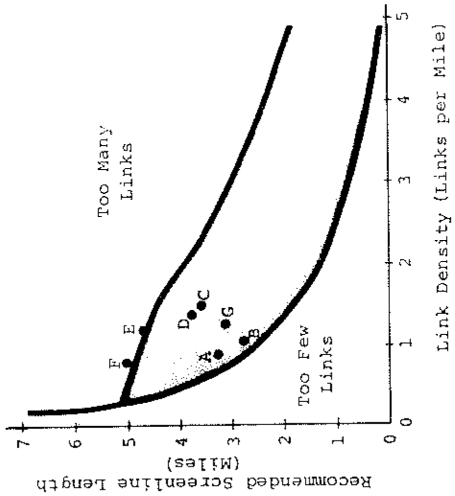


Figure A-100. Check of screenline lengths.

Table A-36. Recommended screenline length.

Screenline	Links Crossed	Screenline Length	Link Density
A-A	3	3.4	0.9
B-B	3	2.6	1.1
C-C	5	3.4	1.5
D-D	5	3.6	1.4
E-E	5	4.8	1.0
F-F	3	5.2	0.6
G-G	4	3.0	1.3

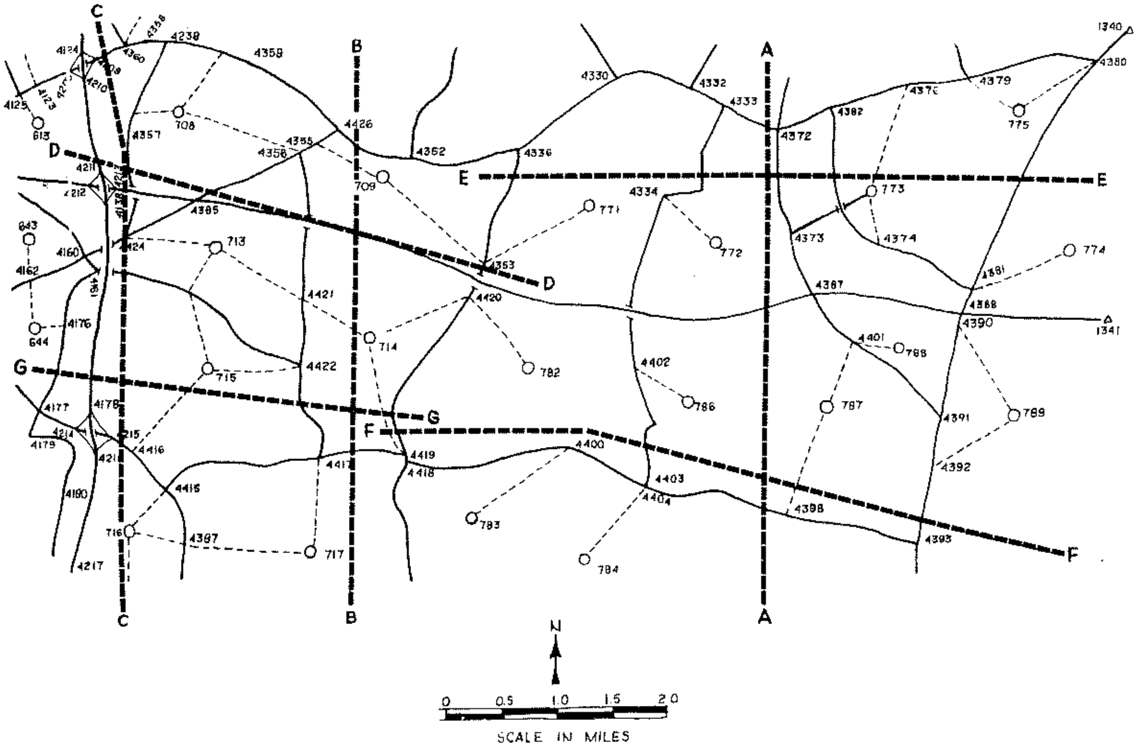


Figure A-99. Screenlines selected for refinement process.

Study Area Case Study

Screenline A-A

(1) Facility (Nodes)	(2) COUNT	(3) % TCOUNT	(4) A_b	(5) A_f	(6) Adjustment RATIO	(7) Adjustment DIFFERENCE	(8) RA_f	(9) C_b	(10) C_f	(11) % TC_f	(12) $\frac{RA_f}{C_f}$	(13) Adjustment CAPACITY	(14) Adjustment BASE COUNT	(15) FA_f	(16) $\frac{FA_f}{C_f}$	(17) $\frac{COUNT}{C_b}$
4333-4372	17,100	0.201	13,792	18,945	23,490	22,250	22,870	20,400	20,400	.152	1.12	11,900	9,650	21,550	1.06	0.84
4385-4387	56,400	0.664	58,922	91,850	—	—	91,850	65,000	97,500	.726	0.94	56,860	31,870	88,730	0.91	0.87
4403-4398	11,500	0.135	5,106	5,164	11,630	11,558	11,600	16,400	16,400	.122	0.71	9550	6,480	16,030	0.98	0.70
TOTALS	85,000 TCOUNT		77,820 TA_b	115,959 TA_f			126,320 TRA_f	101,800 TC_b	134,300 TC_f		0.94 $\frac{TRA_f}{TC_f}$	See Fig. A-14	0.62 FCAP; 0.38 FCOUNT	126,310 TFA_f	0.94 $\frac{TFA_f}{TC_f}$	0.83 $\frac{TCOUNT}{TC_b}$

Figure A-102. Refinement calculations for screenline A-A.

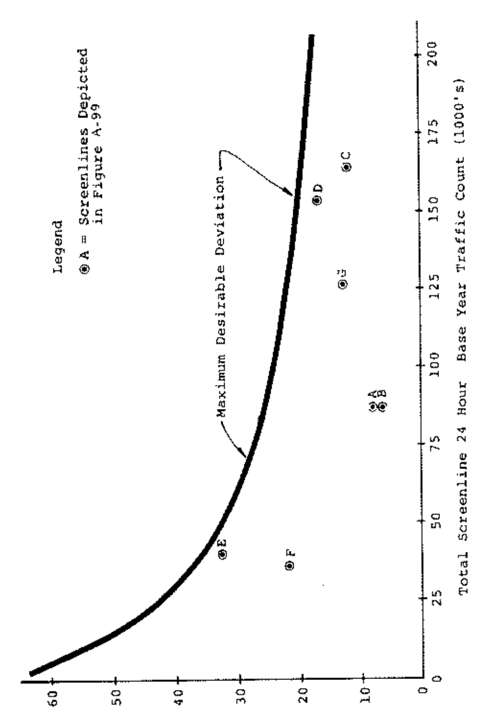


Figure A-101. Check of base year counts and assignments.

Table A-37. Comparison of screenline base year counts and assignments.

Screenline	Base Year Assignment	Base Year Count	Deviation	Percent Deviation	Exceed Allowable Deviation?
A-A	77,800	85,000	- 7,200	- 8.5	No
B-B	78,500	85,200	- 6,700	- 7.9	No
C-C	176,700	159,300	17,400	10.9	No
D-D	173,600	147,200	26,400	17.9	No
E-E	28,400	42,400	- 14,000	- 33.0	No
F-F	44,100	36,000	8,100	22.5	No
G-G	138,400	121,800	16,600	13.6	No

Study Area Case Study

Screenline B-B

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
Facility (Nodes)	COUNT	% TCOUNT	A_b	A_f	Adjustment RATIO	Adjustment DIFFERENCE	RA_f	C_b	C_f	% TC_f	$\frac{RA_f}{C_f}$	Adjustment CAPACITY	Adjustment BASE COUNT	FA_f	$\frac{FA_f}{C_f}$	$\frac{COUNT}{C_b}$
4426 - 4352	15,400	.181	19,900	21,311	22,026	21,811	21,918	20,400	20,400	.143	1.07	12,170	8,670	20,840	1.02	0.76
4385 - 4387	56,400	.662	58,922	91,850	—	—	91,850	65,000	97,500	.684	0.94	58,230	31,700	89,930	0.92	0.87
4417 - 4419	13,400	.157	4,724	10,570	29,980	19,250	19,250	24,600	24,600	.173	0.78	14,730	7,520	22,250	0.90	0.55
TOTALS	85,200 TCOUNT		78,546 TA_b	123,731 TA_f			133,018 TRA_f	110,000 TC_b	142,500 TC_f		0.93 $\frac{TRA_f}{TC_f}$	See Fig. A-14	0.64 FCAP; 0.36 FCOUNT	133,020 TFA_f	0.93 $\frac{TFA_f}{TC_f}$	0.77 $\frac{TCOUNT}{TC_b}$

Figure A-103. Refinement calculations for screenline B-B.

Study Area Case Study

Screenline C-C

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
Facility (Nodes)	COUNT	% TCOUNT	A_b	A_f	Adjustment RATIO	Adjustment DIFFERENCE	RA_f	C_b	C_f	% TC_f	$\frac{RA_f}{C_f}$	Adjustment CAPACITY	Adjustment BASE COUNT	FA_f	$\frac{FA_f}{C_f}$	$\frac{COUNT}{C_b}$
4209 - 4360	46,100	.289	52,760	73,201	63,960	66,540	65,250	66,700	66,700	.259	0.98	46,790	22,380	69,170	1.04	0.69
4213 - 4385	66,300	.416	62,133	98,457	—	—	98,460	65,000	97,500	.377	1.01	68,110	32,210	100,320	1.03	1.02
4160 - 4424	18,400	.116	27,150	39,328	26,650	30,578	28,610	32,700	32,700	.127	0.87	22,940	8,980	31,920	0.98	0.56
4161 - 4423	6,900	.043	8,860	15,691	12,220	13,730	12,975	16,400	16,400	.063	0.79	11,380	3,330	14,710	0.90	0.42
4215 - 4416	21,600	.136	25,830	59,788	50,000	55,560	52,780	45,000	45,000	.174	1.17	31,430	10,530	41,960	0.93	0.48
TOTALS	159,300 TCOUNT		176,733 TA_b	286,465 TA_f			258,075 TRA_f	225,800 TC_b	258,300 TC_f		1.00 $\frac{TRA_f}{TC_f}$	See Fig. A-14	0.70 FCAP; 0.30 FCOUNT	258,080 TFA_f	1.00 $\frac{TFA_f}{TC_f}$	0.71 $\frac{TCOUNT}{TC_b}$

Figure A-104. Refinement calculations for screenline C-C.

Study Area Case Study

Screenline D-D

(1) Facility (Nodes)	(2) COUNT	(3) % TCOUNT	(4) A_b	(5) A_f	(6) Adjustment: RATIO	(7) Adjustment: DIFFERENCE	(8) RA_f	(9) C_b	(10) C_f	(11) % TC_f	(12) $\frac{RA_f}{C_f}$	(13) Adjustment: CAPACITY	(14) Adjustment: BASE COUNT	(15) FA_f	(16) $\frac{FA_f}{C_f}$	(17) $\frac{COUNT}{C_b}$
4210-4211	113,500	.771	134,208	150,118	126,955	129,411	128,200	130,200	130,200	.710	0.98	90,860	42,290	133,150	1.02	0.87
4357-4424	8,100	.055	9,350	16,224	14,055	14,974	14,510	12,000	12,000	.066	1.20	8,450	3,020	11,470	0.96	0.68
4356-4385	15,100	.103	16,134	23,463	21,959	22,430	22,190	22,000	22,000	.120	1.01	15,360	5,650	21,010	0.96	0.69
4356-4421	4,700	.032	8971	10,703	5,607	6,432	6,020	7,000	7,000	.038	0.86	4,860	1,760	6,600	0.95	0.67
4353-4420	5,800	.039	4961	10,585	12,375	11,424	11,900	12,000	12,000	.066	0.99	8,450	2,140	10,590	0.88	0.48
TOTALS	147,200 TCOUNT		173,624 TA_b	211,093 TA_f			182,820 TRA_f	183,200 TC_b	183,200 TC_f		1.00 $\frac{TRA_f}{TC_f}$	See Fig. A-14	0.70 FCAP; 0.30 FCOUNT	182,820 TFA_f	1.00 $\frac{TFA_f}{TC_f}$	0.80 $\frac{TCOUNT}{TC_b}$

Figure A-105. Refinement calculations for screenline D-D.

Study Area Case Study

Screenline E-E

(1) Facility (Nodes)	(2) COUNT	(3) % TCOUNT	(4) A_b	(5) A_f	(6) Adjustment: RATIO	(7) Adjustment: DIFFERENCE	(8) RA_f	(9) C_b	(10) C_f	(11) % TC_f	(12) $\frac{RA_f}{C_f}$	(13) Adjustment: CAPACITY	(14) Adjustment: BASE COUNT	(15) FA_f	(16) $\frac{FA_f}{C_f}$	(17) $\frac{COUNT}{C_b}$
4336-4353	3,800	.090	3,170	10,870	13,030	11,500	12,270	12,000	12,000	.104	1.02	4,700	3,760	8,460	0.71	0.32
4333-4334	4,100	.097	1,100	2,229	8,310	5,230	6,770	7,200	7,200	.062	0.94	2,800	4,050	6,850	0.95	0.57
4372-4373	5,300	.125	8,390	31,911	20,160	28,821	24,490	27,200	27,200	.237	0.90	10,720	5,220	15,940	0.59	0.19
4382-4374	13,520	.319	7,640	11,195	19,811	17,080	18,450	16,500	16,500	.144	1.12	6,510	13,310	19,820	1.20	0.82
4380-4381	15,675	.369	8,120	14,465	27,920	22,020	24,970	52,000	52,000	.453	0.48	20,480	15,400	35,880	0.69	0.30
TOTALS	42,395 TCOUNT		28,420 TA_b	70,670 TA_f			86,950 TRA_f	114,900 TC_b	114,900 TC_f		0.76 $\frac{TRA_f}{TC_f}$	See Fig. A-14	0.52 FCAP; 0.48 FCOUNT	86,950 TFA_f	0.76 $\frac{TFA_f}{TC_f}$	0.37 $\frac{TCOUNT}{TC_b}$

Figure A-106. Refinement calculations for screenline E-E.

Study Area Case Study

Screenline F-F

(1) Facility (Nodes)	(2) COUNT	(3) % TCOUNT	(4) A_b	(5) A_f	(6) Adjustment RATIO	(7) Adjustment DIFFERENCE	(8) RA_f	(9) C_b	(10) C_f	(11) % TC_f	(12) $\frac{RA_f}{C_f}$	(13) Adjustment CAPACITY	(14) Adjustment BASE COUNT	(15) FA_f	(16) $\frac{FA_f}{C_f}$	(17) $\frac{COUNT}{C_b}$
4420-4419	6,400	.178	4,250	5,430	8,180	7,580	7,880	12,000	12,000	.169	0.66	5,890	4,500	10,390	0.87	0.53
4402-4403	2,900	.081	1,075	2,156	5,820	3,980	4,900	7,200	7,200	.101	0.68	3,520	2,050	5,570	0.77	0.40
4392-4393	26,700	.741	37,450	61,566	43,890	50,820	47,360	52,000	52,000	.730	0.91	25,460	18,720	44,180	0.85	0.51
TOTALS	36,000 TCOUNT		44,097 TA_b	69,152 TA_f			60,140 TRA_f	71,200 TC_b	71,200 TC_f		0.84 $\frac{TRA_f}{TC_f}$	See Fig. A-14	0.58 FCAP; 0.42 FCOUNT	60,140 TFA_f	0.84 $\frac{TFA_f}{TC_f}$	0.51 $\frac{TCOUNT}{TC_b}$

Figure A-107. Refinement calculations for screenline F-F.

Study Area Case Study

Screenline G-G

(1) Facility (Nodes)	(2) COUNT	(3) % TCOUNT	(4) A_b	(5) A_f	(6) Adjustment RATIO	(7) Adjustment DIFFERENCE	(8) RA_f	(9) C_b	(10) C_f	(11) % TC_f	(12) $\frac{RA_f}{C_f}$	(13) Adjustment CAPACITY	(14) Adjustment BASE COUNT	(15) FA_f	(16) $\frac{FA_f}{C_f}$	(17) $\frac{COUNT}{C_b}$
4176-4177	9,100	.075	7,040	13,528	17,486	15,588	16,540	21,600	21,600	.127	0.77	10,600	4,530	15,130	0.70	0.42
4138-4178	99,400	.816	120,744	131,736	108,449	110,392	109,420	130,000	130,000	.762	0.84	63,610	49,320	112,930	0.87	0.76
4422-4417	6,900	.057	6,330	9,372	10,216	9,942	10,080	7,000	7,000	.041	1.44	3,420	3,450	6,870	0.98	0.99
4420-4419	6,400	.052	4,250	5,430	8,177	7,580	7,880	12,000	12,000	.070	0.65	5,840	3,140	8,980	0.75	0.53
TOTALS	121,800 TCOUNT		138,364 TA_b	160,066 TA_f			143,920 TRA_f	170,600 TC_b	170,600 TC_f		0.84 $\frac{TRA_f}{TC_f}$	See Fig. A-14	0.58 FCAP; 0.42 FCOUNT	143,910 TFA_f	0.84 $\frac{TFA_f}{TC_f}$	0.71 $\frac{TCOUNT}{TC_b}$

Figure A-108. Refinement calculations for screenline G-G.

Step 4—Conduct Final Assignment Checks

The refined future year traffic assignments were checked and then plotted as shown on Figure A-109. The first check consisted of first calculating the unrefined future year V/C ratios (A_f/C_f) for each screenline link. These ratios were then compared with the range of the final refined V/C ratios (FA_f/C_f) obtained from column 16 of the calculation form. These values are given in Table A-38. For each screenline. This check indicated that the range of V/C ratios for each screenline had been narrowed by the refinement process. However, there were a few extreme changes in the future year V/C ratios created by the refinement process. Therefore, the refined future year V/C ratios were also compared with the base year V/C ratios from column 17, as depicted in Table A-38. This analysis shows that in most cases the refined future year V/C ratios were reasonable when compared to the base year V/C's. In some situations the refined FA_f/C_f ratios were approximately the average of the unrefined A_f/C_f and base year $COUNT/C_b$ ratios. These values seemed reasonable when compared to the V/C's on adjacent links.

Links 4382-4374 and 4380-4381 in screenline E-E exhibited unusually high FA_f/C_f ratios relative to A_f/C_f and $COUNT/C_b$. Before making any manual adjustments, however, an additional check of the relative importance of these links was required. This check is described at the end of this step.

The second check consisted of examining links that were included in more than one screenline, to determine whether the refined traffic assignments on the links were consistent between screenlines. This situation occurred for two links in the network. The first was link 4385-4387 of Route A, which was common to screenlines A-A and B-B, with refined assignments of 88,730 and 89,930 respectively. Because these refined assignments were close, the higher value was conservatively selected as the final value. If the values had shown greater variation, an average value may have been appropriate. If an extreme variation between values existed, judgment would have been required to determine which value was most representative with regards to traffic patterns. After selecting one value as most appropriate, the refinement calculations would have been repeated for the screenline with the unacceptable assignment. When repeating the refinement calculations, the link common to both screenlines would have its assignment fixed at the acceptable value and the remainder of the total screenline assignment would be adjusted among the other links. Judgment must be used to determine in which of the above categories the variation in assignment would fall.

The other link common to two screenlines was link 4420-4419, which runs north-south in the central portion of the study area. It was common to screenlines F-F and G-G, with assignments of 10,390 and 8,980 respectively. In this instance the 10,390 assignment was selected as most appropriate because it was the higher value and it best formed a future year traffic pattern which was representative of the base year.

A final check was applied to establish the relative importance of each link. In this case study, the two primary links in question were 4382-4374 and 4380-4381 in screenline E-E, as discussed above. By examining %COUNT and $\%TC_f$, and the ratio of FA_f/TFA_f in Figure A-106, the following relationships were developed:

Link	% TCOUNT	% TC_f	FA_f/TFA_f
4382-4374	0.32	0.14	0.23
4380-4381	0.37	0.45	0.41
Total	0.69	0.59	0.64

It is seen that each link carried roughly the same relative percentage of base year (% TCOUNT) and future year (FA_f/TFA_f) volumes. Similarly, the future year volumes on link 4380-4381 appeared to be in scale with the relative future year capacity (% TC_f) on that link. Conversely, link 4382-4374 carried a higher relative future volume than it had relative capacity. As a result, it would be reasonable to try to divert some of the link 4382-4374 traffic to other links.

The most reasonable alternative route was link 4372-4273, which actually has a higher capacity and which represents a more major through facility than did link 4382-4374. The analyst decided that link 4382-4374 would likely operate at but not over-capacity. Therefore, its FA_f/C_f ratio was reduced to 1.00 and the excess volume was added to link 4372-4373, as follows:

For Link 4382-4374:

(FA_f/C_f) revised x C_f = (FA_f) revised

1.00 x 16,500 = 16,500

Excess = FA_f -(FA_f) revised

= 19,820 -16,500 = 3,320

For Link 4372-4373:

(FA_f) revised = FA_f + Excess

= 15,940 + 3,320= 19,260

Then: (FA_f/C_f) revised = 19,260/27,200 = 0.71
which is reasonable given
the A_f/C_f and $COUNT/C_b$ values

This revision is shown on Figure A-109. Similar manual adjustments could be performed as necessary to other links. For the purpose of this case study, no further refinements were required.

Step 5—Determine Future Year Peak Hour Directional Values

From reviewing base year traffic counts, Route A was found to have a peak hour volume equal to 10 percent of the roadway's ADT. Roadways with a large growth in ADT usually experience a decrease in this percentage, but the 10 percent value was used initially while keeping this thought in mind.

The directional distributions of peak hour traffic on Route A were calculated for the base year and are shown in Figure A-110. The less data-intensive approach to directional distribution modification (see Chapter 10) was applied to determine whether or not to update the directional splits. The base year and future year households and employment (i.e., proxies for productions and attractions) within the corridor were tabulated and compared as shown in Table A-39 for each zone within the study area. After a review of these values it was decided to alter the directional distribution on Route A by 5 percent because of the significant employment increases forecasted to occur in the eastern portion of the corridor (i.e., zones 787 and 789).

Step 6—Perform Capacity Calculations

Peak hour directional volumes on Route A were calculated based on the peak hour percentage and change in directional distribution discussed in the preceding section. These volumes are shown in Figure A-111 for the AM and PM peak hours. A capacity analysis for the six-lane design was performed using the TRB Circular 212 procedure (45) and the assumptions listed in Figure A-111. Sections 1-2 and 2-3 of Route A were found to operate at level-of-service F and were therefore unacceptable.

Table A-38. Check of volume/capacity ratios.

Screenline	Future Year				Base Year	
	Facility Nodes	Unrefined A_f/C_f	Refined[1/] FA_f/C_f	How does FA_f/C_f Compare?	$COUNT/C_b$[2/]	How does FA_f/C_f Compare?
A-A	4333-4372	0.93	1.06	OK	Not Needed	
	4385-4387	0.94	0.91	OK	Not Needed	
	4403-4398	0.32	0.98	Check Base Year	0.70	Closer
B-B	4426-4352	1.05	1.02	OK	Not Needed	
	4385-4387	0.94	0.92	OK	Not Needed	
	4417-4419	0.43	0.92	Check Base Year	0.55	Closer
C-C	4209-4360	1.10	1.04	OK	Not Needed	
	4213-4385	1.01	1.03	OK	Not Needed	
	4160-4424	1.20	0.98	OK	Not Needed	
	4161-4423	0.96	0.90	OK	Not Needed	
	4215-4416	1.32	0.93	Check Base Year	0.48	Average of Base and Future Year
D-D	4210-4211	1.15	1.02	OK	Not Needed	
	4357-4424	1.35	0.96	Check Base Year	0.68	Average of Base and Future Year
	4356-4385	1.07	0.96	OK	Not Needed	
	4356-4421	1.53	0.95	Check Base Year	0.67	Average of Base and Future Year
	4353-4420	0.88	0.88	OK	Not Needed	
E-E	4336-4353	0.91	0.71	Check Base Year	0.32	Average of Base and Future Year
	4333-4334	0.31	1.95	Check Base Year	0.57	High, but Low Volume
	4372-4373	1.17	0.59	Check Base Year	0.19	Average of Base and Future Year
	4382-4374	0.68	1.20	Check Base Year	0.82	Still High
	4380-4381	0.28	0.69	Check Base Year	0.30	Still High
F-F	4420-4419	0.45	0.87	Check Base Year	0.53	High, but Low Volume
	4402-4403	0.30	0.77	Check Base Year	0.40	High, but Low Volume
	4392-4393	1.18	0.85	Check Base Year	0.51	Average of Base and Future Year
G-G	4176-4177	0.63	0.70	OK	Not Needed	
	4138-4178	1.01	0.87	OK	Not Needed	
	4422-4417	1.34	1.98	Check Base Year	0.99	Closer
	4420-4419	0.45	0.75	Check Base	0.53	Closer; Low Volume

1/ FA_f/C_f is found in column 16 of calculation forms.

2/ $COUNT/C_b$ found in column 17 of calculation forms.

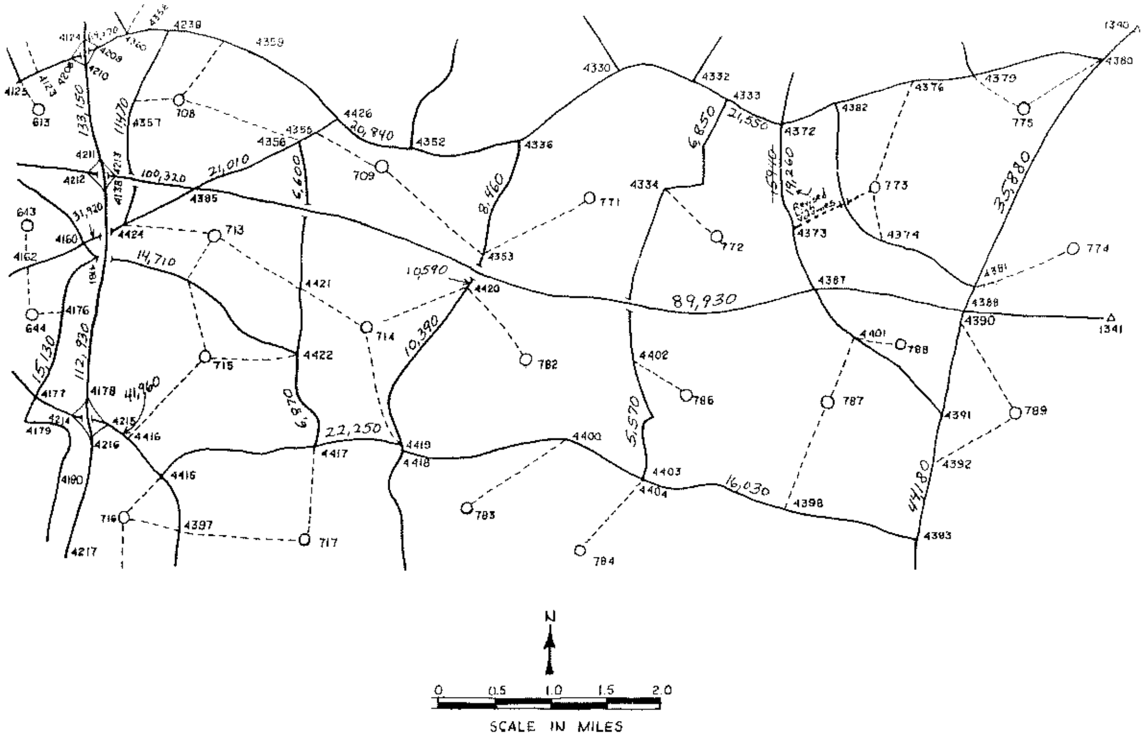


Figure A-109. Refined future year assignments.

Table A-39. Socioeconomic data.

Zone	Base Yr. Households	Future Yr. Households	Base Yr. Employment	Future Yr. Employment	Future Yr. - Base Yr. Households	Future Yr. - Base Yr. Employment
708	1,677	1,677	3,749	4,249	10	500
709	134	211	107	112	77	5
713	427	653	64	94	226	30
714	400	526	122	122	126	0
715	409	489	54	54	80	0
771	36	37	66	66	1	0
772	69	69	11	11	0	0
773	3,159	3,159	1,175	1,175	0	0
774	95	100	8	8	5	0
782	33	249	36	36	216	0
786	29	29	18	18	0	0
787	399	657	278	1,749	258	1,471
788	51	52	36	36	1	0
789	29	64	101	1,683	35	1,582
Total	6,947	7,982	5,825	9,413	+ 1,035	+ 3,588

Because of the large growth in traffic on Route A it was decided to lower the peak hour percentage of ADT from 10 percent to 9 percent to represent the probable temporal shift (or spreading) of traffic over the peak period. It was also decided to increase the width in section 1-2 from 3 to 4 lanes. All other assumptions were maintained. The volumes and level of service for each section under the new design conditions are shown in Figure A-112. For these conditions an acceptable level of service was attainable in each section. However, it should be noted that weaving volumes and distances must be considered before finalizing the lane configurations for these sections. Such analyses would require additional refinement of interchange turning volumes.

TIME REQUIREMENTS

The performance of this case study required approximately 22 person-hours. This effort was divided by step as follows:

	Person Hours
Step 1: Prepare data base	1
Step 2: Select screenlines and check screenline assignments	3
Step 3: Perform calculations	12
Step 4: Conduct final assignment checks	2
Step 5: Determine future year peak hour directional volumes	2
Step 6: Perform capacity calculations	2
Total	22

The time for Steps 3 and 4 together would increase or decrease by about 1 hour for each screenline added or deleted from the analysis. For this case study, the peak hour directional volumes and capacity calculations (Steps 5 and 6) were only derived for the facility under analysis, Route A. Therefore, a proportional amount of time would be required to produce similar data for other facilities in the network.

Overall, this case study indicates that the refinement procedures can be applied in a timely manner for small-to-medium sized networks. Once the analyst has become familiar with the techniques, the calculation and checking times can be reduced even further.

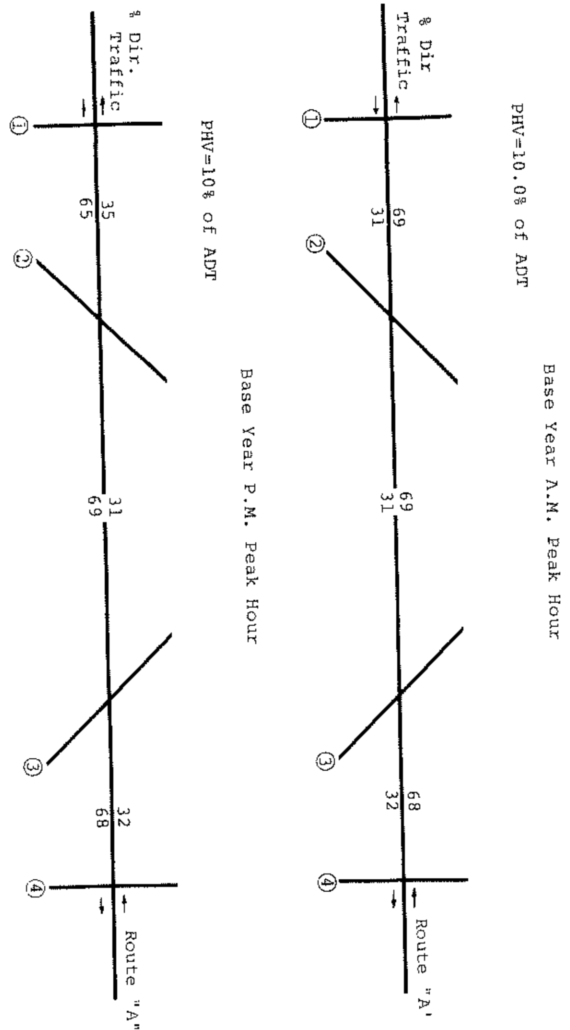


Figure A-110. Base year peak hour directional distribution.

CHAPTER FIFTEEN
CASE STUDY: USE OF WINDOWING PROCEDURES FOR EVALUATING AN ARTERIAL IMPROVEMENT

INTRODUCTION

This case study demonstrates the manual windowing technique (Chapter 6) for analyzing improvements to an arterial intersection. The systems-level zone and transportation network for the area under study is shown in Figure A-113. This area is a portion of a suburb located outside of a large metropolitan area (population over 750,000). Freeway M and arterial Q are parallel facilities that are radially oriented with respect to the CBD. Arterial C provides an important circumferential connection between arterial Q and freeway M. Arterial Q is a six-lane facility with turning lanes at all major intersections. Arterial C is presently a four-lane arterial operating at capacity and it is planned to upgrade it to six lanes. A systems-level future year traffic forecast was available for the area, from which peak hour volumes for the intersection of Q and C were derived, as shown in Figure A-114. The peak hour percentages and directional distributions were assumed to closely replicate base year conditions. A subsequent capacity analysis (Fig. A-115) using the systems-level assignment indicated that the upgraded intersection, as proposed, would not operate at an adequate level of service and that a grade-separated interchange would most likely be required. The capacity analysis was performed using the Transportation Research Circular 212 procedures (45) for illustrative purposes. Similar computations could be made using other accepted procedures.

The systems-level highway network did not include two alternative travel routes located in the area of the intersection. The existence of these routes may influence the operation of the intersection and therefore an analysis needs to be performed to determine the effects.

SUMMARY OF SCENARIO STEPS

The following steps were performed in the analysis:

Step 1: Define study area.
Step 2: Define revised network and zone system.
Step 3: Define trip table for revised network.
Step 4: Assign trips to revised network.
Step 5: Refine trip assignment.
Step 6: Determine peak hour volumes and turning movements.
Step 7: Perform capacity analysis.

SCENARIO DETAIL

The following sections describe the steps performed in analyzing the intersection at arterials Q and C.

Step 1--Define Study Area

The two alternative routes affecting the intersection of Q and C are located in zones 93 and 88. These roadways also affect travel among the zones to the east and west; therefore, the six

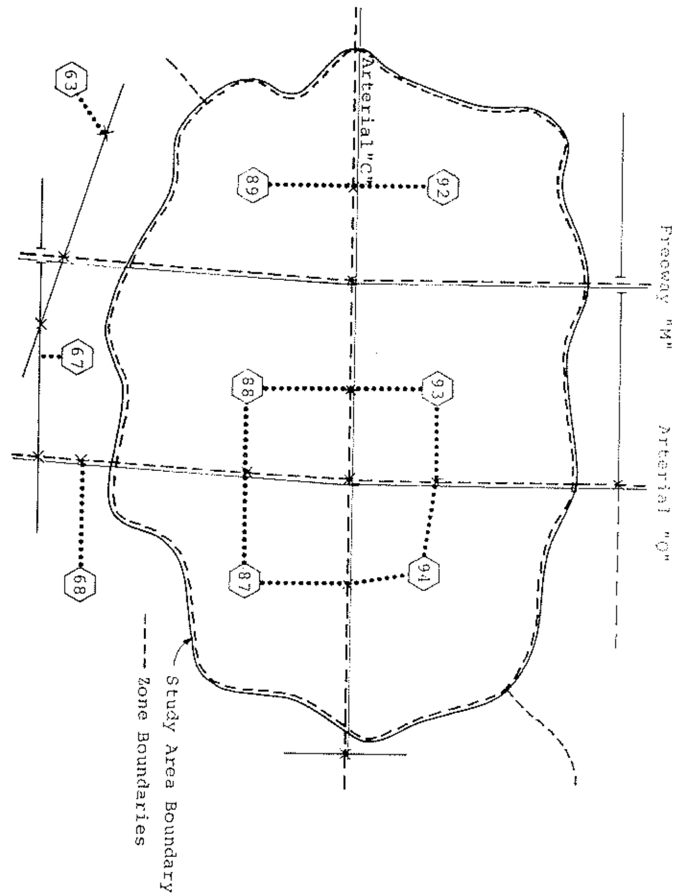
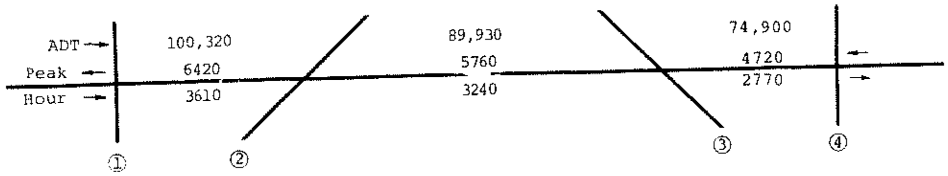
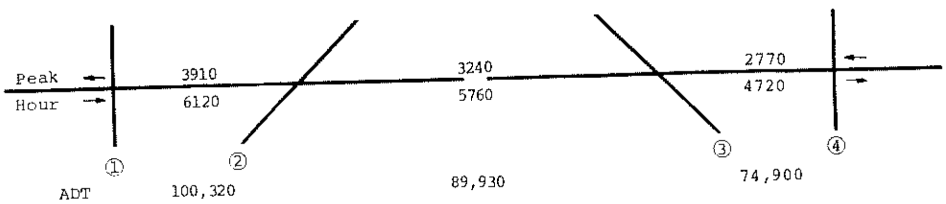


Figure A-113. Systems-level network and zones.

Future Year A.M. Peak Hour



Future Year P.M. Peak Hour



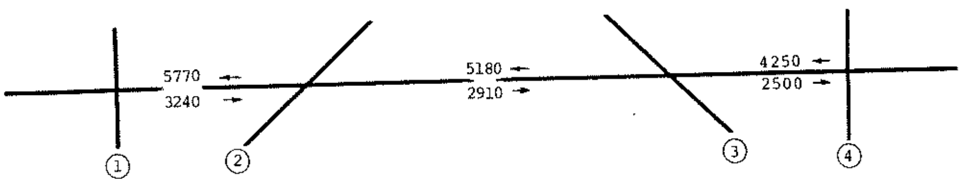
Assumptions : PHV = 10% of ADT
5% shift in directional distribution
3-12′ traffic lanes and 10′ shoulders both directions
5% trucks and level terrain
60 mph AHS
PHF = 0.95

Capacity Analysis from TRB Circular 212 (45)

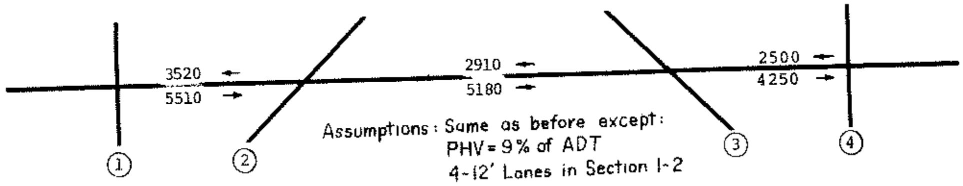
Section	Volume	SV	W	Q	MSV	LOS
1-2	6420	6900	1.0	0.95	7260	F
2-3	5760	6030	1.0	0.95	6350	F
3-4	4720	4970	1.0	0.95	5230	D

Figure A-111. Future year peak hour volumes.

Future Year P.M. Peak Hour



Future Year P.M. Peak Hour



Assumptions : Same as before except:
PHV = 9% of ADT
4-12′ Lanes in Section 1-2

Section	Volume	SV	W	Q	MSV	LOS
1-2	5770	6200	1.0	0.95	6530	D
2-3	5180	5430	1.0	0.95	5720	D
3-4	4250	4470	1.0	0.95	4700	D

Figure A-112. Revised future year peak hour volumes.

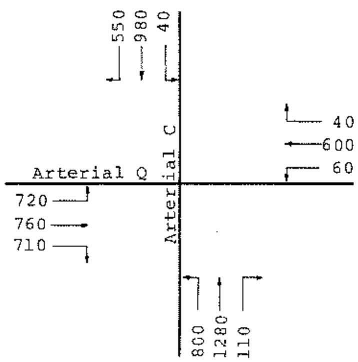


Figure A-114. Peak hour intersection volumes at arterials Q and C using systems-level data.

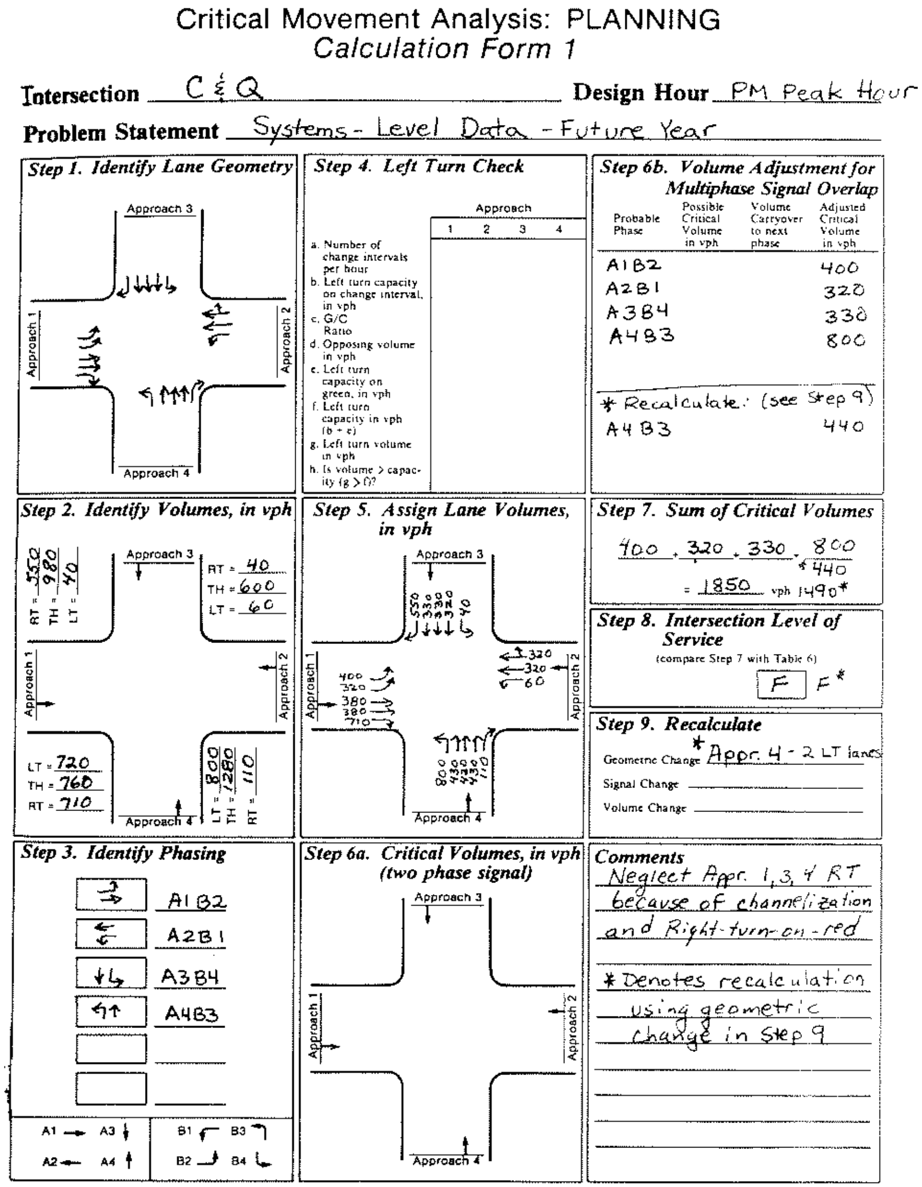


Figure A-115. Intersection capacity analysis at arterials Q and C using systems-level data.

zones outlined in Figure A-113 were selected to form the study area. It is important to note that the boundary of the study area coincides with the boundaries of these zones and that there are six roadways which cross the boundary.

Step 2—Define Revised Network and Zone System

By the problem definition, two alternative routes have been identified as possibly having impacts on the subject intersection. These routes are included in the revised network as shown in Figure A-116. The inclusion of these routes into the network is expected to significantly impact the traffic movements at the Q and C intersection. The zones have been renumbered for convenience and the new numbers as shown in Figure A-116 will be used hereafter.

With the new transportation network, several zone changes became apparent to the analyst. Nondirectional subzoning was selected for the analysis because of the symmetry of the study area and the wide diversity of trip paths. Zone 5 was divided into two new zones labeled 5a and 5b. Zone 5b now had access to the new route in the highway network. However, zone 5a was determined not to have access to the new route because of geographical constraints, resulting in all zone 5a traffic entering the system on arterial C. Zone 3 was subdivided into three new zones, 3a, 3b, and 3c. Zone 3a was a shopping center accessible only by arterial C. Zones 3b and 3c represented different accessibility to the transportation system, with zone 3c connecting to the new route and zone 3b having direct access with the two arterials. New external zone centroids (7 through 12) were selected in accordance with the six links that cross the study area boundary. The remaining zones within the study were unchanged because their zone connectors adequately represented traffic loading onto the new highway network.

Step 3—Define Trip Table for Revised Network

Within the windowing process, the creation of a revised trip table was accomplished by four substeps. These substeps were:

a. Identify zonal interchanges.
b. Allocate total trips to subzones.
c. Allocate total trips to external zones.
d. Calculate zonal trip interchanges.

A production-attraction formatted trip table and select link analyses data were available from the systems-level assignment for use in identifying zonal interchanges (a). The trip table provided the total number of productions and attractions for each zone and the system-level trip interchanges. Similar information could also have been obtained from a trip table constructed in origin-destination format. The select link analyses provided information as to how internal-external trips were distributed within the study area.

Using land-use data for zones 3 and 5, percentages were developed to distribute the total productions and total attractions among the new subzones (b). In the case of zone 3, 20 percent of the total trip productions were allocated to both zones 3a and 3b while 60 percent were allocated to zone 3c. The total trip attractions were distributed in different proportions, with zone 3a receiving 35 percent, zone 3b receiving 45 percent, and zone 3c receiving 20 percent. Similarly, zone 5's productions were allocated 20 percent and 80 percent and its attractions 60 percent and 40 percent between zones 5a and 5b respectively. Table A-40 indicates the distribution of productions and attractions for zones 3 and 5.

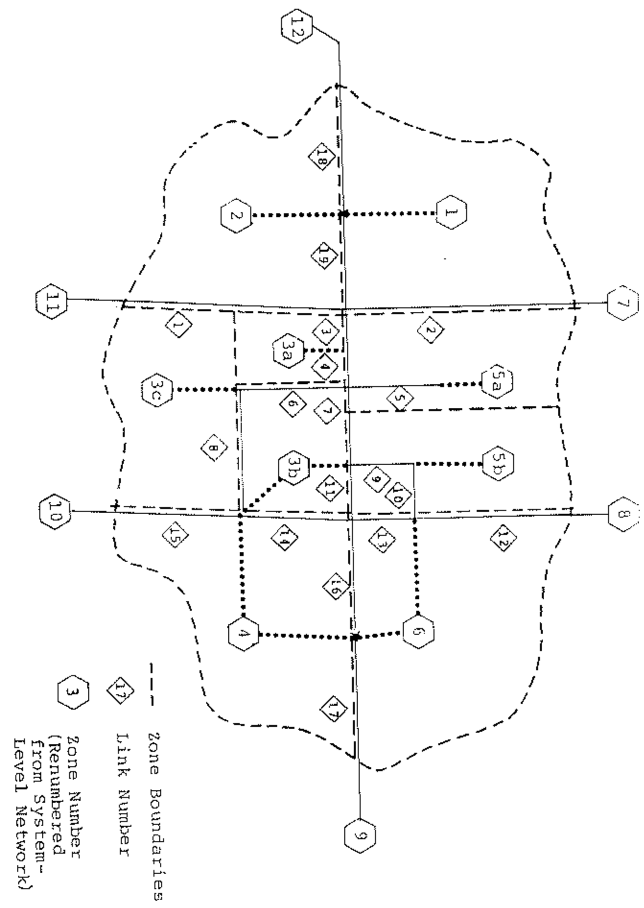


Figure A-116. Revised network and zone system.

Total trips were allocated to external zones 7 through 12 according to the systems-level traffic assignments on the highway links that crossed the study area boundary (c). No new boundary crossings were added. These highway links became the zone connectors for the new external zones. The resulting external zone trip allocation is shown in Table A-41.

Constructing the new trip table consisted of calculating zonal trip interchanges for internal-internal(II) trips, internal-external (IE) trips, and external-external (EE) trips (d). Each trip type (II, IE, EE) had its own trip table, combined at the end into one table for the study area.

The original systems-level trip table of II trips for zones within the study area is shown in Table A-42. This trip table was revised according to the distributions defined in Table A-40. For example, in the original trip table, zone 3 had 174 trip productions oriented to zone 1. These 174 productions were distributed to zones 3a, 3b, and 3c according to the proportions in Table A-40 for productions from zone 3. This distribution of productions is shown by the following equations:

$$T_{3a-1} = T_{3-1} * S_{3a} = 174 * 0.2 = 35$$
$$T_{3b-1} = T_{3-1} * S_{3b} = 174 * 0.2 = 35$$
$$T_{3c-1} = T_{3-1} * S_{3c} = 174 * 0.6 = 76$$

where:

T_{3a-1} = the trips between zone 3a and zone 1
S_{3a-1} = the proportion of trips to be allocated to zone 3a

Similarly, for the trips between zone 3 and zone 5, the same procedure was used except that the appropriate subzone proportions were applied for the subzones in zones 3 and 5. As an example, trips from zone 3c to zone 5b were determined as follows:

$$T_{3c-5b} = T_{3-5} * S_{3c} * S_{5b} = 137 * 0.6 * 0.4 = 33$$

In this case, because of the production-attraction trip table format, the value for S_{3c} was the proportion of productions assigned to zone 3c, and S_{5b} was the proportion of attractions assigned to zone 5b. It should be noted that for cases in which production zones and attraction zones were not subzoned, the value in the trip table cell was not revised. The completed II trip table is shown in Table A-43.

The II trips were now subtracted from total zonal trips to determine the total number of IE trips. These IE trips was then allocated to the various external zones. Allocation of IE trips to the external zones was based on data from the select link analysis, patterns identified through the systems-level trip table, and judgmentally. Zone 5 provides an example of this process. The total productions from zone 5 total 9,568, of which 1,913 were suballocated to zone 5a and 7,655 to zone 5b. From the II trip table (Table A-43) it is seen that 1,325 of zone 5b's total productions were II trips. Subtracting the 1,325 II trip productions from the 7,655 total productions leaves 6,330 IE trips produced from zone 5b. Using the select link analysis, trips from original zone 5 which had been assigned to links serving external zones were determined. These values are given in Table A-44. These trips were subsequently split between zones 5a and 5b according to the same percentages used for II trip productions. The resulting IE trip table is shown in Table A-45.

Because of the availability of select link analysis, construction of the EE trip table was simplified. Systems-level zones that were located outside of the study area were allocated to one of the new study area external zones without the need to manually construct "spheres of influence," as discussed in Chapter 6. From the select link analysis, trips were able to be categorized into the possible external-external zone pairs and then summed. The resulting EE trip table is shown in Table A-46. The three individual trip tables were now combined into a single trip table for the

Table A-40. Distribution of total trip productions and attractions for Zones 3 and 5.

Zone	Productions (Distributed)	Attractions (Distributed)	Productions (Number)	Attractions (Number)
3a	0.2	0.35	1,335	2,371
3b	0.2	0.45	1,335	3,054
3c	0.6	0.7	4,065	1,356
Total	1.0	1.0	6,775	6,781
5a	0.2	0.6	1,913	5,741
5b	0.8	0.4	7,655	3,826
Total	1.0	1.0	9,568	9,567

Table A-41. External zone productions and attractions.

Zone	7	8	9	10	11	12
Productions	37,031	19,548	5,067	25,110	38,524	2,914
Attractions	36,531	19,048	7,067	24,610	38,025	2,914

Table A-42. Systems-level II trip table.

Zone	1	2	3	4	5	6
1	-	358	174	210	463	373
2	358	-	126	168	278	263
3	174	126	-	300	137	321
4	210	168	300	-	158	195
5	463	278	137	258	-	520
6	373	263	221	195	520	-

Table A-43. Revised II trip table.

Zone	1	2	3a	3b	3c	4	5a	5b	6
1	-	358	60	78	36	210	278	185	373
2	358	-	44	57	25	168	167	111	263
3a	35	25	-	-	-	60	16	11	64
3b	35	25	-	-	-	60	16	11	64
3c	104	76	-	-	-	180	48	33	193
4	210	168	105	135	60	-	155	103	195
5a	93	56	10	12	5	52	-	-	104
5b	370	222	39	50	22	206	-	-	416
6	373	263	112	144	64	195	312	208	-

Table A-44. Select link data for zone 5.

	External Zone					
	7	8	9	10	11	12
Trips	1,568	940	1,046	1,571	2,366	420

Table A-45. IE trip table.

Zone	7	8	9	10	11	12
1	2,000	703	250	720	1,067	1,235
2	3,353	-	390	497	-	317
3a	183	120	65	340	426	10
3b	183	120	65	340	426	10
3c	548	360	196	1,021	1,279	29
4	910	-	542	1,391	1,733	49
5a	313	188	209	314	473	84
5b	1,255	752	837	1,257	1,893	336
6	-	831	-	1,608	1,944	355

Table A-46. EE trip table.

Zone	7	8	9	10	11	12
7	-	-	900	3,821	23,382	183
8	-	-	522	12,482	3,841	229
9	400	22	-	339	1,099	52
10	3,821	12,482	839	-	457	22
11	23,382	3,841	1,599	457	-	3
12	183	229	52	22	3	-

entire study area and checked for reasonableness by comparing total zone productions and attractions against the systems-level trip table.

Step 4—Assign Trips to Revised Network

The tabular method of trip assignment, documented in Chapter 6, was selected for this study because of its ease in ordering data. A matrix such as the one shown in Figure A-117 was constructed for each zone (15 total), and trips were assigned to each link along minimum paths as an all-or-nothing assignment process. The assignment of trips from zone 5b is demonstrated in Figure A-117. For example, the trips from zone 5b to zone 7 would follow along links 9, 7, 4, 3, and 2. Because 1,255 trips travel between 5b and 7, the value 1,255 is entered in the column for each link. After all trip interchanges have been assigned, the link volumes are totalled at the bottom of each column. After a matrix has been completed for each zone, the total link assignment was determined by summing the link assignments from each of the matrices. The link 1 assignment of trips for zone 1 is added to the link assignment of trips for zone 2, and so on until all of the zones have been summed. This process is repeated for each link in the network to obtain the total assignment. A similar process can be used to obtain intersection turning movements. The resulting assignment in the area of the subject intersection is shown in Figure A-118. Note that this assignment is still shown in production-attraction format.

Step 5—Refine Trip Assignment

A review was performed to check the reasonableness of the link assignments within the study area. If necessary, a screenline refinement should be performed, as explained in Chapters 4 and 6. In this case, the screenline refinement was not considered necessary by the analyst because the volumes in the vicinity of the subject intersection Q and C were reasonable.

Step 6—Determine Peak Hour Volumes and Turning Movements

The production-attraction assignment was transformed to an origin-destination assignment for the development of peak hour volumes and turning movements. First, the directional link assignments (in production-attraction format) were summed to produce 2-way, 24-hour totals. This resulted in the assignment shown in Figure A-119 for the approaches to the intersection of arterials C and Q. Second, the peak hour volume percentage of ADT and the directional distribution at the intersection were expected to remain the same as in the base year. On the basis of these assumptions, the peak hour directional volumes were derived for the intersection and are shown in Figure A-120. The intersection directional volumes were then balanced using the procedure described in Chapter 10, with the results shown in Figure A-121.

Turning movements were calculated using the procedure described in Chapter 7. The initial inputs to the turning movement analysis were estimated turning percentages based on the production-attraction assignment (Fig. A-118) for the intersection. The turning movement derivation is shown in Figure A-122.

Step 7—Perform Capacity Analysis

Another capacity analysis, using the Transportation Research Circular 212 procedures (45) for illustrative purposes, was performed for the intersection using the volumes derived through the windowing technique. With the revised turning movement volumes the proposed upgraded intersection was found to operate at level-of-service E during the peak hour, as shown in Figure A-123. The analysis using the more detailed windowed volumes therefore indicated that the proposed intersection design may be able to operate at an acceptable level of service without the need to construct an expensive interchange. Further, at-grade design modifications should be explored by the analyst at this point in order to possibly improve the intersection level of service even more.

TIME REQUIREMENTS

The windowing and related analyses were accomplished by professional traffic analysts. The application of the procedures from the user's manual to this case study required approximately 40 person-hours, itemized by step as follows:

		Person Hours
Step 1:	Define study area	1
Step 2:	Define revised network and zone system	2
Step 3:	Define trip table for revised network	12
Step 4:	Assign trips to revised network	16
Step 5:	Refine trip assignment	2
Step 6:	Determine peak hour volumes and turning movements	5
Step 7:	Perform capacity analysis	2
	Total	40

The greatest effort was involved in developing the trip table (Step 3) and assigning the trips (Step 4) given the fairly large zone system. On the other hand, the study area was quite readily identified in this case. Similar studies, with smaller networks, may require additional time for Steps 1 and 2, while the computation time for Steps 3 and 4 would be reduced.

Origin Zone	#Trips	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	Destination Zone
		Link Numbers																			
5b	370			370	370			370		370										370	1
	222			222	222			222		222										222	2
	39				39			39		39											3a
	50									50											3b
	22						22	22		22											3c
	206										206			206	206						4
	—																				5a
	—																				5b
	416										416										6
	1255		1255	1255	1255			1255		1255											7
	752										752		752								8
	837									837		837					837	837			9
	1257										1257			1257	1257	1257					10
	1893	1893		1893	1893			1893		1893											11
	336			336	336			336		336									336	336	12
	Total	1893	1255	4076	4115	—	22	4137	—	5024	2631	837	752	1463	1463	1257	837	837	336	928	Total

Figure A-117. Tabular assignment process for origin zone 5b.

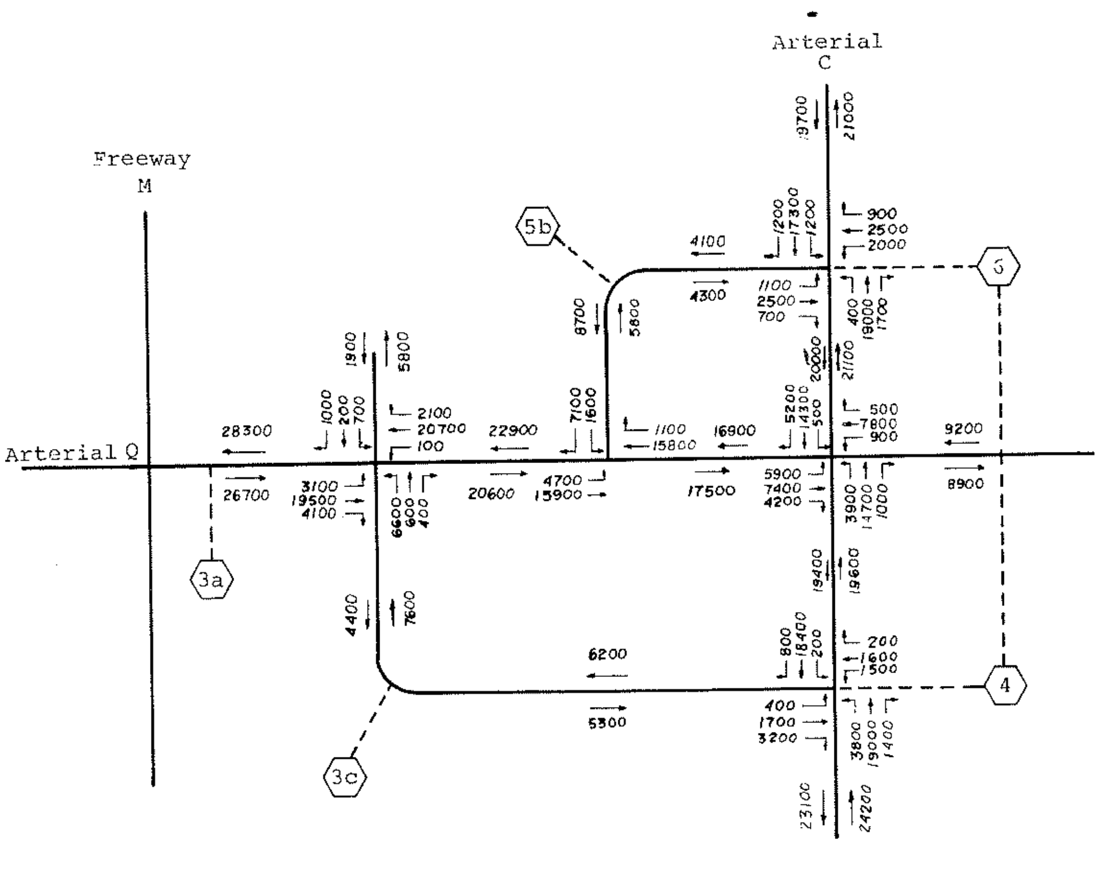


Figure A-118. Manual traffic assignment in production-attraction format.

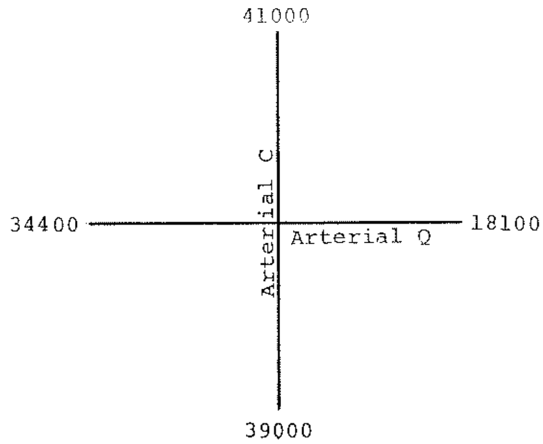


Figure A-119. Intersection link volumes in origin-destination format.

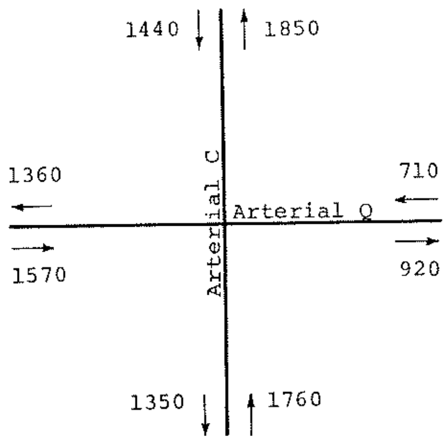


Figure A-120. Initial peak hour directional traffic assignment.

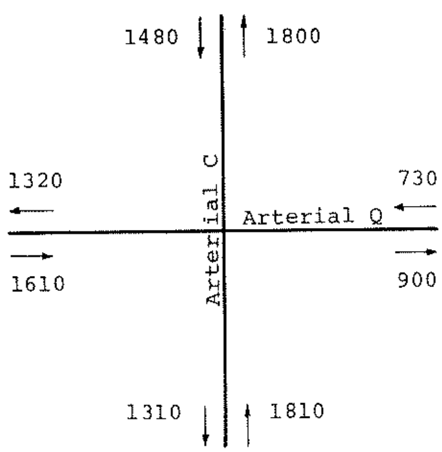
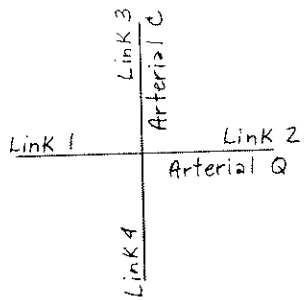


Figure A-121. Balanced peak hour directional traffic assignment.

Link	O_{if} Inflows	D_{jf} Outflows
1	1570	1360
2	710	920
3	1440	1850
4	1760	1350
	5480	5480



Initial Matrix of Percentages from analyzing production-attraction turns

Inflows	Outflows				Total
	—	0.42	0.34	0.24	1.00
	0.85	—	0.05	0.10	1.00
	0.25	0.03	—	0.72	1.00
	0.20	0.05	0.75	—	1.00

Step 1B Equivalent to first row Iteration

O_{if} \ D_{jf}^*	1315	791	1889	1485
1570	—	660	533	377
710	603	—	36	71
1440	360	43	—	1037
1760	352	88	1320	—

D_{jf}^*	D_{jf}	Δ %	$\frac{D_{jf}}{D_{jf}^*}$
1315	1360	-3.3	1.034
791	921	-14.1	1.165
1889	1850	+2.1	0.979
1485	1350	10.0	0.909
5480	5480 ✓		

Step 3 First Column Iteration

O_{if}^* \ D_{jf}	1360	920	1850	1350
1633	—	768	522	343
723	624	—	35	64
1365	372	50	—	943
1759	364	102	1293	—

O_{if}^*	O_{if}	Δ %	$\frac{O_{if}}{O_{if}^*}$
1633	1570	+4.0	0.961
723	710	+1.8	0.982
1365	1440	- 5.2	1.055
1759	1760	- 0.1	1.000
5480	5480 ✓		

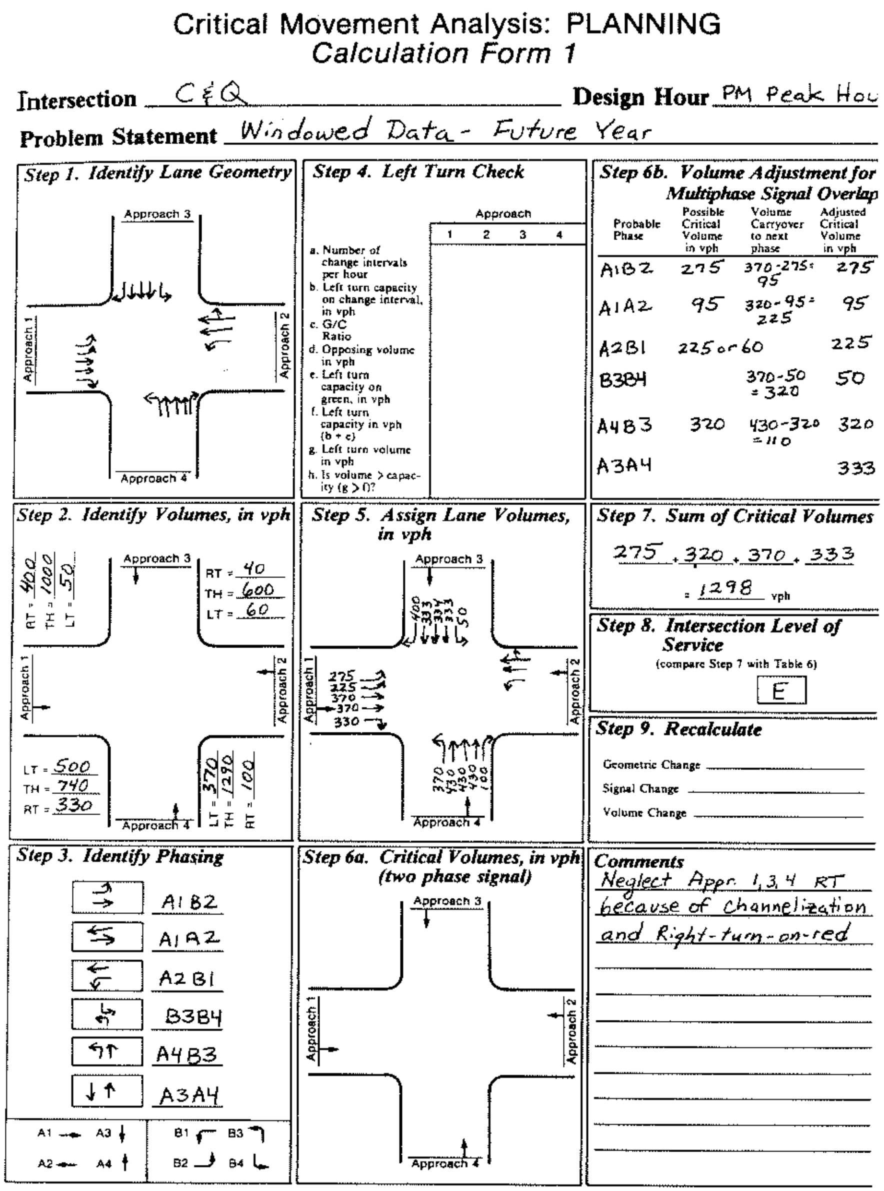
Step 4 Second Row Iteration

O_{if} \ D_{jf}^*	1370	893	1828	1389
1570	—	738	502	330
710	612	—	34	64
1440	392	53	—	995
1760	366	102	1292	—

D_{jf}^*	D_{jf}	Δ %
1370	1360	+0.7
893	920	-2.9
1828	1850	-1.2
1389	1350	+2.9

Acceptable Closure

Figure A-122. Derivation of intersection turning movements.



Critical Movement Analysis: PLANNING
Calculation Form 1

Intersection C & Q **Design Hour** PM Peak Hou

Problem Statement Windowed Data - Future Year

Step 1. Identify Lane Geometry

Approach 3, Approach 1, Approach 2, Approach 4

Step 4. Left Turn Check

	Approach 1	2	3	4
a. Number of change intervals per hour				
b. Left turn capacity on change interval, in vph				
c. G/C Ratio				
d. Opposing volume in vph				
e. Left turn capacity on green, in vph				
f. Left turn capacity in vph (b + e)				
g. Left turn volume in vph				
h. Is volume > capacity (g > f)?				

Step 6b. Volume Adjustment for Multiphase Signal Overlap

Probable Phase	Possible Critical Volume in vph	Volume Carryover to next phase	Adjusted Critical Volume in vph
A1B2	275	370-275 = 95	275
A1A2	95	320-95 = 225	95
A2B1	225 or 60		225
B3B4		370-50 = 320	50
A4B3	320	430-320 = 110	320
A3A4			333

Step 2. Identify Volumes, in vph

Approach 1: LT = 50, TH = 1000, RT = 400

Approach 3: RT = 40, TH = 600, LT = 60

Approach 4 (left): LT = 500, TH = 740, RT = 330

Approach 2: LT = 370, TH = 1290, RT = 100

Step 5. Assign Lane Volumes, in vph

Approach 3: 400, 333, 333, 333, 50

Approach 1: 275, 225, 370, 370, 330

Approach 4: 370, 430, 430, 430, 100

Approach 2

Step 7. Sum of Critical Volumes

275 + 320 + 370 + 333

= 1298 vph

Step 8. Intersection Level of Service

(compare Step 7 with Table 6)

E

Step 9. Recalculate

Geometric Change

Signal Change

Volume Change

Step 3. Identify Phasing

A1 B2

A1 A2

A2 B1

B3B4

A4B3

A3A4

A1, A2, A3, A4, B1, B2, B3, B4

Step 6a. Critical Volumes, in vph (two phase signal)

Approach 3, Approach 1, Approach 2, Approach 4

Comments

Neglect Appr. 1,3,4 RT because of Channelization and Right-turn-on-red

Figure A-123. Intersection capacity analysis at arterials Q and C using windowed data.

INTRODUCTION

In many instances, isolated locations have transportation problems to be analyzed in detail with a limited amount of information available regarding the future. The available data are usually used to develop a broader amount of detailed information by applying accepted techniques and pertinent data. The purpose of this scenario is to demonstrate these techniques in a detailed study. Specifically, the objective is to demonstrate techniques for:

- Determining design hourly volumes for capacity analyses.
- Determining traffic data for environmental analyses.
- Determining traffic data for pavement design.

The problem involves the design of an intersection or interchange at the junction of two major arterials. The arterials are located in a major metropolitan area with a population in excess of 2.5 million people. The junction of the arterials is in a suburb located north of the central city. Arterial G is a major north-south radial roadway that carries traffic into and out of the CBD. Arterial R is an important east-west circumferential facility that carries cross-county and local traffic.

Land use surrounding the intersection is predominantly residential. Commercial property occupies the northeast quadrant. Because of the school's classification as a "sensitive receptor," any studies of transportation improvements must include the generation of traffic data suitable for input to air and noise forecasting models.

A planned intersection design for the junction is shown in Figure A-124. There is concern over whether or not the design will be capable of an adequate level of service in the year 2005. Figures A-125, A-126, and A-127 show the respective base year (1980) ADT, AM peak hour, and PM peak hour traffic at the intersection. A year 2005 computer forecast for the area yielded the nondirectional volumes shown in Figure A-128. A high number of future turns are expected at the intersection because of traffic destined to a major new employment site north of the intersection. The large effect of the location of the new employment site on the traffic patterns is indicated by comparing the difference in volumes between the north and south approaches of the intersection for the year 2005 (Fig. 128) ADT and for the base year 1980 (Fig. 125).

A distribution of existing vehicle classification by hour of the day is included in Table A-47. This vehicle mix is not expected to change in the future due to any changes in land use. For computing the daily and equivalent single axle loadings, an 18-kip ESAL rate of 508.2 per 1,000 trucks was assumed, based on data from similar facilities in the region.

SUMMARY OF SCENARIO STEPS

The following steps were performed in the analysis:

Step 1: Develop hourly directional volumes.

Step 2: Determine turning movements for the peak hours.

Step 3: Perform capacity analyses.

Step 4: Determine traffic data for environmental analysis.

Step 5: Determine traffic data for pavement design.

Table A-47. Vehicle classification.

Auto	Trucks Light	Trucks Medium	Trucks Heavy	Time
				AM
90.5	2.3	5.0	2.2	12- 1
85.2	4.0	6.6	4.2	1- 2
79.2	5.3	10.1	5.3	2- 3
81.8	2.7	13.6	1.9	3- 4
89.0	3.0	6.0	2.0	4- 5
90.1	2.5	5.6	1.8	5- 6
94.1	2.4	2.5	1.0	6- 7
92.9	3.0	3.0	1.1	7- 8
89.3	3.2	5.5	2.0	8- 9
86.9	3.4	6.9	2.8	9-10
87.3	3.0	6.9	2.8	10-11
88.8	2.7	6.1	2.4	11-12
				PM
89.4	2.9	5.4	2.3	12- 1
87.3	3.1	6.7	2.9	1- 2
85.6	3.4	7.9	3.1	2- 3
85.4	3.2	8.0	3.3	3- 4
89.5	3.6	4.9	2.0	4- 5
92.4	3.4	3.1	1.1	5- 6
93.4	2.9	2.6	1.1	6- 7
93.2	2.8	2.8	1.2	7- 8
93.7	2.7	2.6	1.0	8- 9
94.7	2.4	2.1	0.8	9-10
94.7	2.3	1.9	1.0	10-11
94.0	2.2	2.2	1.6	11-12

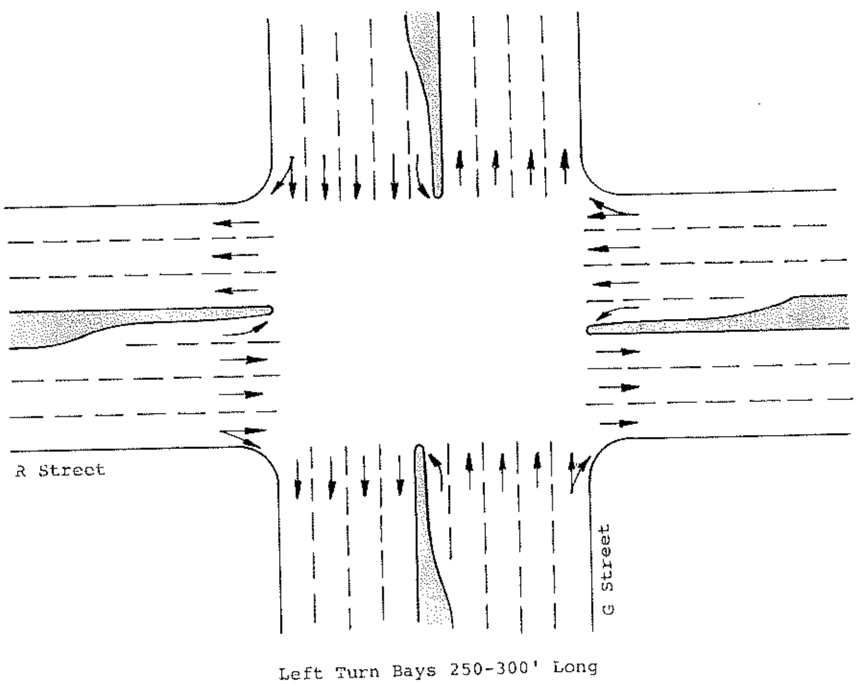


Figure A-124. Planned intersection design.

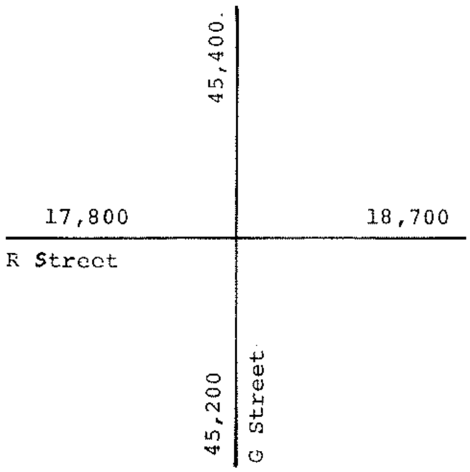


Figure A-125. Base year ADT.

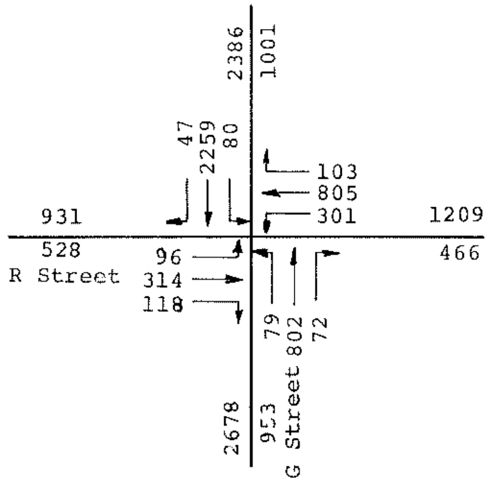
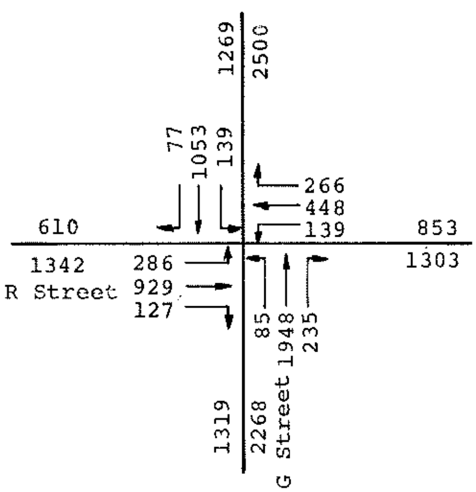


Figure A-126. Base year AM peak hour volumes.



ıre A-127. Base year PM peak hour volumes.

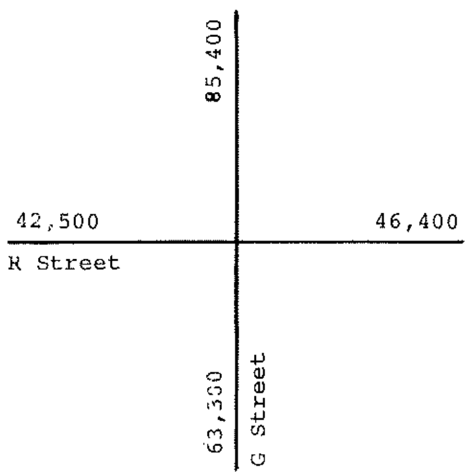


Figure A-128. Future year forecasted ADT.

SCENARIO DETAIL

The following sections describe in detail the steps performed in analyzing the intersection design at the junction of arterials G and R.

Step 1 — Develop Hourly Directional Volumes

The travel pattern on the two arterials is dominated by work trips. This fact is substantiated by the location of the commuter rail station to the north of the intersection. For this example the most desirable method of estimating hourly volume (including peak hour traffic) and directional volumes would be through statistical approaches (Chapters 9 and 10). Because a cross-classification table based on count data from the local area is not available, the hourly and directional distributions of traffic from NCHRP Report 187 (88) were used. The application of these tables demonstrates how similar tables developed from local data would be applied.

Four characteristics which define the distribution to be applied are summarized as follows:

a. Urbanized area population.
b. Facility type.
c. Urban subregion.
d. Facility orientation.

For the characteristics of this example, it was appropriate to use Table A-23 from Chapter 10 to obtain the directional distributions. Arterial G uses the Suburb/Radial column and arterial R uses the Suburb/X-Town column in Table A-23. By applying the percent of ADT and the directional split percentages to the link's ADT, the directional hourly volumes of Table A-48 were obtained.

A check was performed to ensure that the traffic volume entering the intersection equalled the traffic volume exiting the intersection. The check revealed that the inbound traffic volume had to be reduced by 337 vehicles and the outbound traffic volume had to be increased by the same amount. The difference between inbound and outbound traffic during each of the day's 24 hours indicated that specific hourly volumes also needed balancing. The method described in Chapter 10 was then applied to obtain the volumes in Table A-49.

Step 2 — Determine Turning Movements

Because of the new employment sites the turning movements were expected to change substantially at the intersection. A high percentage of east-west traffic was anticipated to turn northerly towards the employment center. Therefore, a judgmental approach was applied to estimate initial future year turning percentages rather than use the base year counts. The iterative directional volume method from Chapter 8 was applied to determine turning movements. The calculations are shown in Figure A-129 and the results are diagrammed in Figure A-130. The iterative derivation of turning movements for the PM peak hour required two steps to reach an acceptable closure, although a third iteration was performed in this case to provide more accurate results. Similar calculations could be performed for the AM peak hour.

Step 3: Perform Capacity Analyses

After turning volumes were determined, a capacity analysis of the proposed intersection design was performed using procedures in the TRB Circular 212 critical movement analysis (45). The analysis indicated that the proposed intersection configuration would operate at level-of-service F for the proposed design. As a result, design alternatives such as the addition of lanes or the construction of a grade-separated interchange were identified. Similar capacity analyses could then be performed for each design alternative. Although these calculations were not conducted for this example, detailed intersection turning movements can be developed with the user's manual procedures to provide sufficient information to perform needed capacity analyses.

Step 4 — Determine Traffic Data for Environmental Analysis

The specific traffic data required for environmental analyses vary with the model used, as shown previously in Table A-2 (Chapter 2). However, several of these data are common to several models.

Air Quality Traffic Data

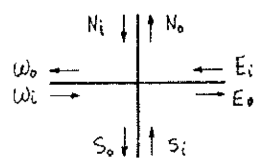
In most cases the air quality models require vehicle volumes by class for the peak hour and the peak consecutive 8-hours which produce the highest emissions. In most cases, the highest traffic volume 8-hour period also produces the maximum level of emissions. Using this assumption, the peak consecutive 8-hours for this case study were determined by analyzing the hourly total combined volumes on the four intersection approaches (see Table A-49). The eight hours selected were 11 AM to 7 PM. The link volumes for these hours, taken from Table A-49, were then multiplied by the vehicle classification percentages from Table A-47 to determine the hourly volumes by type of vehicle, as depicted in Table A-50. Subsequent stratifications of heavy trucks into gasoline and diesel may be required for some models. An estimate of motorcycle classification may also be necessary. In both cases, base year data can frequently be used to make these more detailed stratifications.

Volume-to-capacity ratios were determined for each hour and used with Figure A-83 from Chapter 12 to estimate average running speed, as shown in Table A-50. Because the location under analysis is an intersection, the air quality models require estimates of idle delay time, stops, queue lengths, and traffic signal timing. These data can also be obtained from the procedures presented in Chapter 12. An example of the calculation for estimating delay and queuing on the westbound approach of this intersection is shown in Figure A-131. These calculations were based on over-capacity conditions prevailing on that approach. Signal timings were estimated from base year conditions at intersections with characteristics similar to the intersection under analysis.

Energy Consumption Traffic Data

The FHWA "Energy Factor Handbook" (102) describes various traffic data needs for estimating energy consumption. In most cases, only 24-hour and peak hour traffic volumes are needed, obtainable from Table A-49. The vehicle classification percentages given in Tables A-47 and A-50 are directly applicable to the energy methodology, including autos, medium and heavy trucks. Some more detailed analyses in the Handbook require the split between diesel and gasoline heavy trucks; these data can be derived from counts on similar existing facilities or by using statewide or national factors. The speed, delay, and queuing data calculated using Figures A-83 and A-131 can also be used directly.

Table A-48. Hourly directional volumes.



Time	NI_i	NO_i	SI_i	SO_i	EI_i	EO_i	WI_i	WO_i	IO_i
12-1 Am	564	717	532	418	306	390	357	281	-47
1-2	342	512	380	253	195	269	246	179	-50
2-3	188	239	177	139	102	130	119	94	-16
3-4	214	214	158	158	116	116	106	106	0
4-5	248	179	133	184	125	107	97	115	+18
5-6	1127	581	430	836	278	186	170	255	+147
6-7	3381	1315	975	2507	1039	585	536	952	+572
7-8	4782	2050	1519	3545	2088	392	1275	1913	+764
8-9	2912	1785	1323	2159	1559	1225	1122	1428	+319
9-10	2152	1691	1253	1595	1086	1002	918	995	+126
10-11	1998	1845	1367	1481	1206	1114	1020	1105	+46
11-12	1998	1845	1367	1481	1160	1160	1062	1062	+39
12-1 pm	1921	1921	1424	1424	1160	1160	1062	1062	0
1-2	2220	2050	1519	1646	1160	1160	1062	1062	+43
2-3	2442	2255	1671	1810	1276	1276	1169	1169	+48
3-4	2664	2887	2140	1975	1559	1689	1547	1428	-69
4-5	3407	4706	3488	2526	1736	2209	2023	1590	-377
5-6	2613	4646	3444	1937	1462	2018	1849	1339	-572
6-7	2255	2869	2127	1671	1281	1503	1377	1173	-176
7-8	1922	1922	1424	1424	1225	1327	1216	1122	-8
8-9	1435	1554	1152	1063	1002	1086	995	918	-37
9-10	1435	1554	1152	1063	854	1007	918	782	-41
10-11	1025	1110	823	760	696	696	637	637	-22
11-12	786	922	685	582	464	464	425	425	-33
Total	44,031	41,369	30,663	32,637	23,135	23,265	21,308	21,192	+674
2-Way Total	85,400		63,300		46,400		42,500		
Forcasted ADT	85,400		63,300		46,400		42,500		
Difference	0		0		0		0		
Comparison	OK		OK		OK		OK		

	Link	O_{if}	D_{jf}
Inflows (O_{if}) and Outflows (D_{jf}) 4–5 PM (see Table 48)	1 (North)	3395	4533
	2 (East)	1731	2128
	3 (South)	3480	2433
	4 (West)	2019	1531
		10,625	10,625

Matrix of Percentages	D_1	D_2	D_3	D_4	Total
O_1	—	0.2	0.5	0.3	1.0 (100%)
O_2	0.5	—	0.3	0.2	1.0 (100%)
O_3	0.8	0.1	—	0.1	1.0 (100%)
O_4	0.3	0.6	0.1	—	1.0 (100%)

Step 1B Equal to First Row Iteration

O_{if} \ D_{jf}^*	4256	2238	2419	1712
3395	—	679	1698	1018
1731	866	—	519	346
3480	2784	348	—	348
2019	606	1211	202	—
	4256	1027	2419	1712

D_{jf}^*	D_{jf}	Δ%	$\frac{D_{jf}}{D_{jf}^*}$
4256	4533	−6.1	1.065
2238	2128	+5.0	0.951
2419	2433	−0.6	1.006
1712	1531	+11.8	0.894
10,625	10,625 ✓		

Step 3 First Column Iteration

O_{if}^* \ D_{jf}	4533	2128	2433	1531
3264	—	646	1708	910
1754	923	—	522	309
3608	2965	331	—	312
1999	645	1151	203	—

O_{if}^*	O_{if}	Δ%	$\frac{O_{if}}{O_{if}^*}$
3264	3395	−3.9	1.040
1754	1731	+1.3	0.987
3608	3480	+3.7	0.965
1999	2019	−1.0	1.010
10,625	10,625 ✓		

Step 4 Second Row Iteration

O_{if} \ D_{jf}^*	4422	2155	2496	1552
3395	—	672	1776	947
1731	911	—	515	305
3480	2860	320	—	300
2019	651	1163	205	—

D_{jf}^*	D_{jf}	Δ%
4422	4533	−2.4
2155	2128	+1.3
2496	2433	+2.6
1552	1531	+1.4
10,625	10,625 ✓	

Figure A-129. Turning movement computations for PM peak hour.

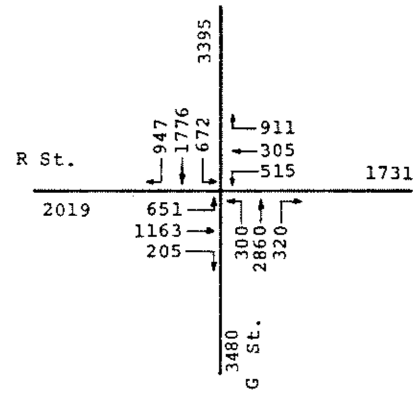


Figure A-130. Future year estimated turning improvements for PM peak hour.

Project No. Example Intersection Identification G&R
Year 2005 Time 4-5 PM Approach Identification WEST

1.	Demand volume for peak	2019	veh/hr. } From Table 48
2.	Demand volume for off-peak	1845	veh/hr.
3.	Capacity of intersection approach	1800	veh/hr.
4.	Time duration of peak	2	hrs.
5.	Cycle length of signal	150	sec.
6.	Effective green time	30	sec.
7.	Speed of vehicles on the approach to the intersection during the peak	15	mi/hr.
8.	Number of lanes of the approach	4	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	Arrival Rate	2050	veh/hr
10.	Duration of interruption by signal	120	sec.
11.	Queue length		
11.1	Maximum queue length (vehicles)	500	vehicles
11.2	Maximum queue length (distance)	0.52	mi
11.3	Adjusted maximum queue length	580	vehicles
11.4	Average adjusted queue length	280	vehicles
12.	Queue discharge time	-9.7	hr. (queue continues into next hour)
13.	Average delay per vehicle	0.16 hr = 560	sec/veh

Figure A-131. Intersection delay and queuing computations.

Noise Quality Traffic Data

The noise quality models require three basic inputs--automobile volumes, truck volumes (medium and heavy), and operating speeds. The automobile volumes are normally the lesser of the design hour volume or the maximum volume that can be handled under level-of-service C conditions. Because of its high volumes, this intersection was expected to operate at level-of-service C or better conditions only during a small portion of each day. Therefore, the reasonable automobile and truck volumes selected for the analysis should be the average of the three highest volume hours. If different design alternatives were analyzed, the volume inputs to the noise quality model would be varied based on the effect of the design on traffic operations and speed.

In order to better replicate the conditions due to the influence of a traffic signal and interrupted flow, the average running speeds determined from Figure A-83 and shown in Table A-50 should be substituted for operating speeds in the noise models. Consideration may also be given to increasing the heavy truck noise factors (or increasing truck volumes) to account for frequent accelerating conditions.

Step 5: Determine Traffic Data for Pavement Design

The key traffic data required for pavement design are 24-hour volumes (ADT) classified by total and truck traffic. An equivalent single-axle loading rate must also be determined. The desired product is the total 18-kip equivalent single-axle loads for the 20-year design period (1985 through 2005).

The equivalent single-axle loading calculations for the west intersection approach are shown here. The base year ADT is 17,800 vehicles that was forecasted to increase to 42,500 over 20 years. The daily directional distribution was determined from Table A-49 to be approximately 50%/50% and the daily percentage of trucks was determined to be approximately 10 percent using the hourly volumes and vehicle classifications from Tables A-47 and A-49 respectively. The assumed ESAL rate (508.2) was from a loadometer station on a similar major arterial constructed using a flexible pavement with a P_t of 2.5 and a SN of 3. Sixty percent of the traffic is assumed to be in the design lane. The following calculations show the derivation of the daily and 20 year ESAL.

$$\text{ADT (avg)} = \frac{17{,}800 + 42{,}500}{2} = 30{,}150$$

$$\text{ADT (design)} = 30{,}150 * 0.50 * 0.60 = 9{,}045$$

$$\text{ADT (trucks)} = 9{,}045 * 0.10 = 905$$

$$\text{Trucks (20 years)} = 905 \times 365 \times 20 = 6{,}606{,}500$$

$$\text{ESAL rate} = 508.2 \text{ per } 1{,}000 \text{ trucks}$$

$$\text{ESAL (daily)} = 905 \times \frac{508.2}{1{,}000} = 460 \text{ kips}$$

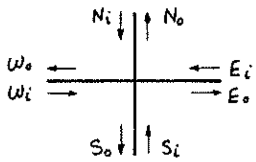
$$\text{ESAL (20 years)} = 6{,}606{,}500 \times \frac{508.2}{1{,}000} = 3{,}357{,}400 = 3{,}357 \text{ kips}$$

These ESAL data could now be used to calculate required pavement thicknesses, as presented in the AASHTO guide (5).

TIME REQUIREMENTS

This case study required approximately 16 person-hours to be performed. These hours do not

Table A-49. Balanced hourly directional volumes.



Time	NI_i^*	NO_i^*	SI_i^*	SO_i^*	EI_i^*	EO_i^*	WI_i^*	WO_i^*	I_i^*
12-1 Am	562	696	531	406	305	379	356	273	1754
1-2	341	489	379	242	194	257	245	171	1159
2-3	187	232	176	135	102	126	119	91	584
3-4	213	213	157	157	116	116	106	106	592
4-5	247	184	132	189	125	110	97	118	601
5-6	1124	625	428	899	277	200	169	274	1998
6-7	3376	1452	972	2769	1037	646	534	1052	5919
7-8	4771	2220	1511	3839	2083	1507	1272	2071	9637
8-9	2903	1865	1317	2256	1555	1280	1118	1492	6893
9-10	2146	1726	1249	1628	1083	1023	915	1016	5393
10-11	1992	1855	1362	1489	1203	1120	1017	1110	5574
11-12	1991	1852	1362	1487	1157	1164	1059	1066	5569
12-1 Pm	1914	1914	1419	1419	1157	1157	1059	1059	5549
1-2	2213	2059	1514	1653	1157	1165	1059	1066	5943
2-3	2435	2265	1666	1818	1272	1281	1165	1174	6538
3-4	2656	2854	2135	1952	1554	1670	1543	1412	7888
4-5	3395	4533	3480	2433	1731	2128	2019	1531	10,625
5-6	2604	4367	3438	1821	1457	1897	1845	1259	9344
6-7	2249	2792	2123	1626	1277	1463	1373	1141	7022
7-8	1918	1914	1421	1418	1221	1322	1212	1118	5772
8-9	1432	1538	1150	1052	999	1075	992	908	4573
9-10	1432	1536	1150	1050	851	989	915	773	4348
10-11	1022	1099	821	753	694	689	635	631	3172
11-12	783	907	683	572	463	456	424	418	2353
Total	43,906	41,187	30,576	33,063	23,070	23,220	21,248	21,330	118,800

	N	S	E	W
2-Way Total	85,093	63,639	46,290	42,578
Forcasted ADT	85,400	63,300	46,400	42,500
Difference	-307 (0.4%)	+ 339 (0.5%)	-110 (0.2%)	+78 (0.2%)
Comparison	OK	OK	OK	OK

Table A-50. Peak hourly vehicle classifications for each link.

NORTH LINK

Time	Inbound Auto	Light	Med	Heavy	Avg. Running Speed
11A-12P	1768	54	121	48	31
12-1P	1711	56	103	44	31
1-2P	1932	69	148	64	30
2-3P	2084	83	192	75	30
3-4P	2268	85	212	88	29
4-5P	3039	122	166	68	28
5-6P	2406	89	81	28	29
6-7P	2101	65	58	25	30
Outbound					
11A-12P	1644	50	113	45	31
12-1P	1711	56	103	44	31
1-2P	1798	64	138	60	30
2-3P	1939	77	179	70	30
3-4P	2437	91	228	94	29
4-5P	4057	163	222	91	25
5-6P	4035	148	135	48	26
6-7P	2608	81	73	31	29

EAST LINK

Time	Inbound Auto	Light	Med	Heavy	Avg. Running Speed
11A-12P	1027	31	71	28	32
12-1P	1034	34	62	27	32
1-2P	1010	36	78	34	31
2-3P	1089	43	100	39	31
3-4P	1327	50	124	51	30
4-5P	1549	62	85	35	31
5-6P	1346	50	45	16	31
6-7P	1193	37	33	14	31
Outbound					
11A-12P	1034	31	71	28	32
12-1P	1034	34	62	27	32
1-2P	1017	36	78	34	31
2-3P	1097	44	101	39	31
3-4P	1426	53	134	55	30
4-5P	1905	77	104	43	29
5-6P	1753	64	59	21	29
6-7P	1366	42	38	16	30

SOUTH LINK

Time	Inbound Auto	Light	Med	Heavy	Avg. Running Speed
11A-12P	1209	37	83	33	32
12-1P	1269	40	77	33	32
1-2P	1322	47	101	44	31
2-3P	1426	57	132	52	31
3-4P	1823	68	170	70	30
4-5P	3115	125	171	70	28
5-6P	3177	117	107	38	28
6-7P	1983	62	55	23	30
Outbound					
11A-12P	1320	40	91	36	32
12-1P	1269	41	76	33	32
1-2P	1443	51	111	48	31
2-3P	1556	62	144	56	31
3-4P	1667	62	156	64	30
4-5P	2178	88	119	48	30
5-6P	1683	62	56	20	31
6-7P	1519	47	42	18	31

WEST LINK

Time	Inbound Auto	Light	Med	Heavy	Avg. Running Speed
11A-12P	940	29	65	25	32
12-1P	947	31	57	24	32
1-2P	925	33	71	31	31
2-3P	997	40	92	36	31
3-4P	1318	49	123	51	30
4-5P	1807	73	99	40	29
5-6P	1705	63	57	20	29
6-7P	1282	40	36	15	30
Outbound					
11A-12P	952	31	58	25	32
12-1P	947	31	57	24	32
1-2P	931	33	71	31	31
2-3P	1005	40	93	36	31
3-4P	1206	45	113	47	30
4-5P	1370	55	75	31	30
5-6P	1163	43	39	14	31
6-7P	1066	33	30	13	31

include time to format the traffic data for specific environmental models; however, this effort would be minmal.

This time is divided by steps, as follows:

	Person-hours
Step 1: Develop hourly directional volumes	8
Step 2: Determine turning movements	1
Step 3: Perform capacity analysis	2
Step 4: Determine traffic data for environmental analysis	2
Step 5: Determine traffic data for pavement design	3
Total	16

The largest single effort is to develop the hourly directional volumes and to balance the inbound and outbound intersection movements (Step 1). The remaining steps require minimal time. However, if additional design alternatives are to be analyzed, the time requirements for Steps 3, 4, and 5 would increase roughly by a factor equal to the number of alternatives. Therefore, for three alternatives, the capacity analysis may require 3 x 2 hr = 6 hr.

In summary, these manual procedures can be isolated highway design options in a cost-efficient manner, such that sufficient data are provided for evaluation, environmental analyses, and for pavement design.