NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP Report 387

Planning Techniques to Estimate Speeds and Service Volumes for Planning Applications

Transportation Research Board
National Research Council

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Planning Techniques to Estimate Speeds and Service Volumes for Planning Applications

RICHARD DOWLING
Dowling Associates
Oakland, CA

with

WAYNE KITTELSON
JOHN ZEGEER
Kittelson Associates

ALEXANDER SKABARDONIS
Deakin, Harvey, Skabardonis

Barton Aschman Associates

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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FOREWORD

By Staff
Transportation Research Board

This report recommends methods for transportation planners to estimate speeds and service volumes. Methods are presented for long-range transportation planning and other planning applications such as traffic impact analyses and major investment studies.

ISTEA and the Clean Air Act Amendments of 1990 require transportation planners to monitor and assess the performance of the transportation system. Planners must be able to measure and predict level of service (LOS) and speed at facility, corridor, and area-wide levels for both the short term (5 to 10 years) and the long term (10 to 25 years). The results of these analyses determine the region's or state's eligibility and priority for current and future federal transportation funds. The large size of the networks that must be analyzed by planners and the necessity of forecasting conditions 20 to 25 years in the future require that any planning technique used to forecast speed and LOS be computationally and data efficient.

Planning agencies perform many different types of planning studies, which vary widely in the data that are available, the confidence in that data, the form of the results of the study, and the precision and accuracy required in those results. Typical studies include long-range transportation plans, transportation improvement programs, air quality conformity studies, major investment studies, intermodal studies, congestion management studies, growth management studies, site and project impact studies, and the highway performance monitoring system.

Estimation of the traffic speed is a critical part of most planning studies. Many planning studies also require the LOS to be estimated. Service volumes are an effective method of estimating LOS because, given constant geometric and control characteristics for a facility, different levels of traffic demand can be quickly analyzed.

Under NCHRP Project 3-55(2), Dowling Associates surveyed planning agencies to determine their needs and capabilities, reviewed existing methods of estimating speeds and service volumes, and developed improved methods. Recommendations are presented for long-range transportation planning and sketch planning (for which very little data are available and computational efficiency is very important) and for other types of planning studies.

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CHAPTER 1

SUMMARY

The Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 and the Clean Air Act Amendments (CAAA) of 1990 have made transportation planning analyses more relevant to national policy issues and concerns. Current techniques for estimating speed and service volume are inadequate for meeting present transportation planning needs. The purpose of this research was to develop the most appropriate techniques for estimating speed and service volume for use throughout a broad range of planning applications.

1.1 RESEARCH PLAN

The research plan was divided into two phases. In Phase 1, the deficiencies of current planning techniques for estimating speed and service volume were assessed and recommendations were developed for improved techniques. The improved techniques were developed and tested in Phase 2 of the research. The following tasks were completed in Phase 1:

1. A review of the available literature;
2. A survey of planning organizations' experience with current techniques;
3. An assessment of existing techniques;
4. Identification of data needs and possible data sources; and
5. Preparation of an interim report.

Phase 2 tasks consisted of the following:

1. Development of alternative techniques for applications that lack satisfactory techniques;
2. Evaluation of the alternative techniques; and
3. Preparation of the final report.

This report presents the results of this research.

1.2 FINDINGS

The research evaluated existing speed estimation and level of service estimation techniques. These techniques were evaluated in terms of data requirements, ease of use, accuracy, consistency with the *Highway Capacity Manual* (HCM), and range of application.

1.2.1 Current Speed Estimation Techniques

The research concentrated on two types of existing speed estimation techniques: the Bureau of Public Roads (BPR) technique and those presented in the HCM. The BPR technique and similar volume/capacity (v/c) ratio–based speed-flow curve techniques are very useful for long-range regional travel forecasting because they require very little data and can be quickly processed by computers. These techniques, however, have the following major deficiencies:

- The BPR curve and other v/c ratio curves are not sensitive to the impact of signal spacing, timing, and coordination, which can significantly influence the level of service for interrupted flow road facilities. The BPR-estimated speeds were found to diverge significantly from those obtained from traffic system simulation programs under different signal timing conditions.
- The v/c ratios computed using the default capacity look-up tables associated with the BPR curve are not reliable for determining facility level of service. Tests with the validation dataset found that the technique produced the correct level of service only 8 percent of the time for urban arterials.
- The BPR speed-flow curve was fitted to uncongested data contained in the 1965 HCM. The BPR curve speed estimates drop too fast as demand approaches capacity. The speed-flow curve needs to be updated with 1994 HCM data.
- The BPR technique uses look-up tables for free-flow speed and capacity that may represent national averages but that rarely reflect local conditions. Inaccuracies in these default speeds and capacities are a major source of error in the application of the BPR technique. The use of facility-specific values of free-flow speed and capacity cuts the error of the BPR technique in half.
- The BPR speed-flow curve does not accurately model speed when demand exceeds capacity. Comparison with simulation model results found that the BPR curve underestimates the impact of queuing on the mean speed of traffic.

The standard BPR technique performs best on freeways, estimating the mean facility speed with a mean error of 10 percent or less. The error, however, increases to 40 percent to 50 percent of the true mean speed for all other facility types. Much of this error is associated with the default capacities contained in the look-up table for this technique or with the lack of signal timing variables.

The 1994 HCM provides three techniques for uninterrupted flow facilities: freeways, multilane highways, and two-lane rural roads. The manual provides one technique for interrupted flow facilities: urban and suburban arterials. These techniques, although comprehensive, have the following major deficiencies:

- The HCM techniques require extensive data for reliable results. (Florida, however, has produced a set of default parameters that greatly simplify the application of HCM techniques.)
- The HCM techniques are complex and difficult to apply without specialized software.
- There is no planning technique in the HCM for analyzing the overall level of service of a freeway containing more than just basic sections.
- The HCM techniques can estimate mean facility speeds generally within 10 percent of the correct mean speed for most facilities. The techniques, however, do not perform well on urban arterials, usually underestimating speeds by 19 percent, with a root mean square (RMS) error of 26 percent.
- The HCM techniques are limited to volumes that are less than or equal to capacity.

1.2.2 Current Level of Service and Service Volume Estimation Techniques

The research concentrated on four existing level of service and service volume estimation techniques: v/c ratio, techniques in the HCM, the Florida Generalized Service Volume Tables, and the Florida Table Generating Software. The latter two techniques are based on HCM techniques.

The v/c ratios were found to be a reliable proxy for the primary level of service criteria for all facilities, except for urban interrupted flow facilities. In this case, the primary level of service measure, speed, is not directly related to the v/c ratio.

The Florida Generalized Service Volume Tables are easy to use, requiring little data (usually the facility type, the area type in which the road facility is located, and the number of lanes). The tables, however, only predicted the correct level of service between 17 percent and 54 percent of the time, depending on the facility type. These tables were most accurate for urban freeways and least accurate for rural freeways and urban arterials.

The Florida Table Generating Software (FREETAB, RMULTAB, R2LNTAB, and ARTTAB) allows the user to enter data that are specific to the facility being analyzed. Consequently, the software was found to be two to four times more accurate than the generalized tables. Software-generated tables predicted the correct level of service between 33 percent and 62 percent of the time, depending on the type of facility.

With all existing methods, it is difficult to accurately predict the level of service for urban interrupted flow facilities. Even the best methods (the ARTPLAN implementation of Chapter 11 of the HCM and Florida's ARTTAB) predicted the correct arterial level of service less than 33 percent of the time.

1.3 RECOMMENDATIONS

It is recommended that transportation planning agencies adopt two planning methods for estimating mean traffic speed, level of service, and service volume: the enhanced BPR and advanced ARTPLAN techniques.

The enhanced BPR technique is designed for application in long-range transportation planning studies and for sketch planning analyses. The technique is fast and requires few data. It produces speed, service volume, and level of service estimates superior to those obtained from the current BPR technique.

The enhanced ARTPLAN technique is designed for all other planning applications that focus on a single facility or a few facilities. The technique requires more data, but is sensitive to more facility design/operation issues and produces more accurate estimates of mean facility speed and level of service than does the enhanced BPR technique.

The enhanced BPR technique has the following key features:

- Separate speed-flow curves are used for unsignalized facilities and signalized facilities.
- The parameters of the speed-flow curves have been updated to better fit current speed-flow data. The new curve uses "capacity" rather than "practical capacity."
- The technique uses a new equation fitted to field data for estimating free-flow speed based on the posted speed limit and signal spacing.
- The default look-up tables of free-flow speed and capacity have been replaced with equations that allow users to input facility-specific data or to develop locally customized speed and capacity look-up tables. It is recommended that facility-specific data be used wherever possible.

The enhanced ARTPLAN technique is an extension of the current Florida ARTPLAN spreadsheet for urban arterials, which in turn is an implementation of the urban and suburban arterials method in the HCM. The enhanced technique extends ARTPLAN to other facility types and to demand conditions that exceed capacity. The technique emphasizes the estimation of speed, level of service, and service volumes for specific facilities and is compatible with HCM techniques. The technique has the following key features:

- The technique extends the HCM arterial analysis concept of dividing a facility into segments and intersec-

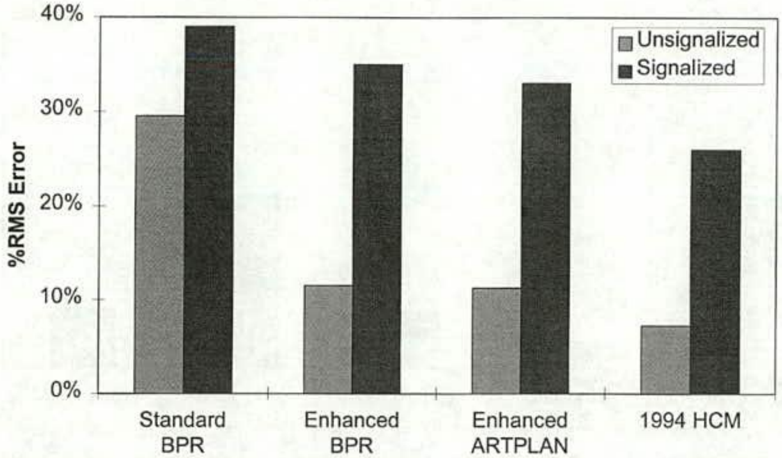

Figure 1-1. Error in estimated speeds.

tions for analysis purposes to all facility types, including freeways.

- The technique splits the peak period into a sequence of hourly time periods for the purpose of analyzing queuing.
- Segment running times are estimated using the free-flow speed, which is computed from the posted speed limit, signal spacing, signal timing, and signal coordination.
- The arterial running timetable in the HCM is replaced with an equation relating running time to posted speed limit.
- Through the use of a queue delay equation, the technique extends the application of the HCM methods to conditions in which demand significantly exceeds capacity.
- The existing arterial level of service criteria in the HCM are extended to other interrupted flow facilities by setting minimum speed criteria for each level of service that are a function of the facility's midblock free-flow speed.

1.4 CONCLUSIONS

The recommended planning techniques for estimating mean facility speed and average facility level of service were found to perform significantly better than existing planning techniques. Both the enhanced BPR and enhanced ARTPLAN techniques performed better than the standard BPR technique at estimating mean facility speed (Figure 1-1). As expected, the techniques in the 1994 HCM, with their more extensive input data requirements, performed better than any of the aforementioned planning techniques.

The enhanced BPR and enhanced ARTPLAN techniques are better than the standard BPR technique at estimating the same level of service for unsignalized facilities as predicted by the HCM method for unsignalized facilities (Figure 1-2).

The enhanced ARTPLAN method proved to be superior to even the 1994 HCM method for signalized facilities for pre-

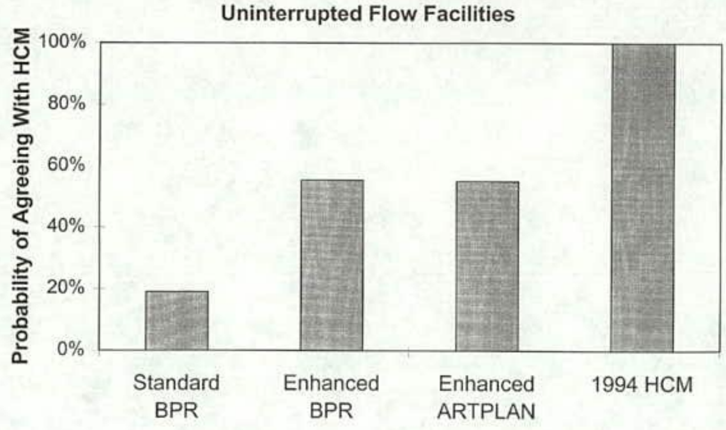

Figure 1-2. Accuracy of level of service estimates for unsignalized facilities.

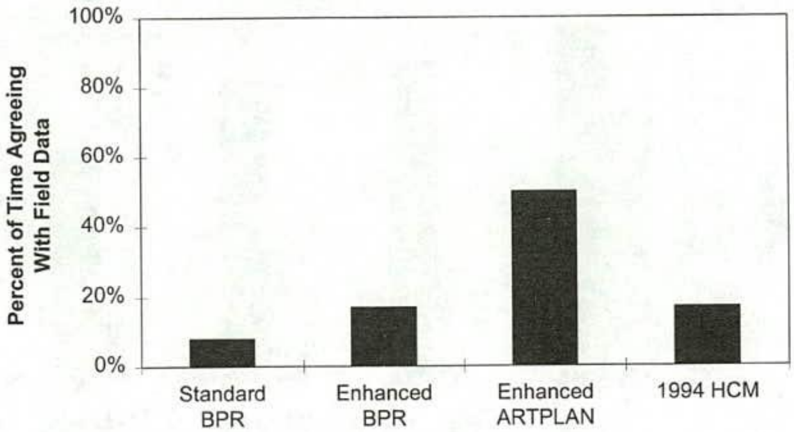

Figure 1-3. Accuracy of level of service estimates for signalized facilities.

dicting the same level of service for signalized facilities as measured in the field (Figure 1-3). The HCM method generally predicted speeds for signalized facilities more precisely (lower RMS error) than did the enhanced ARTPLAN method, but the HCM method underestimated speeds by 19 percent on the average. The HCM method, therefore, was less accurate (more biased) than the enhanced ARTPLAN method. As a result, the HCM method obtained the correct level of service for signalized facilities less frequently than did the enhanced ARTPLAN technique.

1.5 SUGGESTED FUTURE RESEARCH

It is recommended that the ARTPLAN spreadsheet software be revised to incorporate the recommended enhancements to this technique. The revised spreadsheet will make it possible for transportation planners to estimate speed, level of service, and service volume for a wide range of road facilities more accurately than they can with current techniques, with an accuracy approaching that of HCM techniques.

The estimation of speed and level of service on urban arterials is a major weakness of the techniques considered in this research. The RMS error is still between 25 percent and 33 percent of the true mean speed for the best of the existing and recommended speed estimation techniques evaluated. It is hoped that further research on speed estimation techniques for signalized arterials will improve these techniques.

CHAPTER 2

CURRENT PLANNING PRACTICES

2.1 INTRODUCTION

2.1.1 Purpose

This chapter describes current transportation planning practices throughout the United States at the state, regional, and local levels. The focus of the discussion is on the planning techniques used to estimate speed, capacity, and service volumes of road facilities. Although current practice differs from agency to agency, this chapter provides a general understanding of the planning applications for which these agencies are responsible and the planning techniques they use. Through agency profiles, the chapter provides a more detailed look at the planning process and techniques.

2.1.2 Organization

The chapter starts with a general discussion of the background needed to understand the need for estimating speed and level of service of road facilities. This is followed by a description of current practices around the country. The results from the user survey are incorporated into the discussion. Regional and agency differences are noted, and profiles of specific agencies are provided to give an in-depth description of the planning process.

2.2 BACKGROUND

The continuing, comprehensive, cooperative (3C) planning process was established by the Federal Aid Highway Act of 1962. The 3C process includes the development of regional transportation plans (RTPs) and the assurance that roads are consistent with local development plans. Metropolitan planning organizations (MPOs) were designated for each urban area in response to new federal transportation planning requirements resulting from the 1973 Federal Aid Highway Act. At that time, the 1965 *Highway Capacity Manual* (HCM) was in place and the volume/capacity (v/c) ratio was the only criterion used by planners to determine road facility level of service and the adequacy of planned improvements.

Legislation in the early 1990s brought changes to transportation planning practices. The passage of the Clean Air Act Amendments (CAAA) in 1990 and the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 significantly increased the demands placed on planners and the tools they use. As a result, the 1994 HCM updated the 1985 HCM. Some procedures were significantly modified or updated, capacity values were increased, and speed-flow relationships were modified.

Legislation brought forth the need for forecasting and analyzing parameters other than v/c ratios for estimating levels of service. It is no longer sufficient to estimate service volumes and capacity. Because of the need to predict the impact of road facilities on air quality, the need for estimating speed has increased. Not only must transportation planners meet federal requirements, they also must be responsive to state and local laws covering congestion and growth management.

2.2.1 Federal Regulations

CAAA and ISTEA have affected transportation planning at the state and local levels. The requirements of these legislative actions are changing the approaches to transportation planning. In addition, both pieces of legislation have brought about an integration of transportation planning and air quality management (*1*).

2.2.1.1 Clean Air Act Amendments

CAAA led to the joining of transportation planning and air quality management. CAAA requires that transportation plans, programs, and projects contribute toward the improvement of air quality in nonattainment areas. Nonattainment areas are air quality management districts and air pollution control districts that do not meet national ambient air quality standards. Most major metropolitan areas fall into this category.

Transportation plans, programs, and projects must conform to CAAA provisions. The development of criteria and procedures for ensuring such conformity was included in the legislation. Conformity guidance was subsequently issued. CAAA brought about the need to consider air emissions in transportation planning.

Estimates of mobile source emissions are highly dependent on speed estimates. Auto emissions, which vary by pollutant, reach a minimum at a particular speed (Figure 2-1) (*2*). The speed at which the least amount of pollutant is emitted depends on the type of engine and the pollutant. At low speeds, engines do not burn efficiently, and at high speeds, emissions increase again. For an engine with a catalytic converter, the minimum emission rate ranges from about 40 miles per hour (mph) for nitric oxides (NO_x) to about 55 mph for carbon dioxide (CO) and hydrocarbons (HC). Estimates of speed are crucial to the mobile source emissions inventory. Emissions models require the average speed of a trip rather than the link speed.

In November 1993, the Environmental Protection Agency (EPA) published the final rules for implementing CAAA transportation planning conformity requirements. Because emissions calculations are greatly influenced by speed, incorporating speed forecasting in transportation planners' forecasting techniques is required for determining mobile source emissions resulting from regional plans and programs. Regional transportation planning models in ozone nonattainment areas must use "reasonable methods in accordance with good practice to estimate traffic speeds and delays in a manner that is sensitive to the estimated volume of travel on each roadway segment represented in the network model" (*3*). Free-flow speeds used in transportation network demand models require field verification. All RTPs, transportation improvement programs (TIPs), and amendments to them must be analyzed for conformity to the state implementation plan. Although these requirements only apply to ozone nonattainment areas, most urban areas fall into this category.

2.2.1.2 ISTEA

ISTEA furthered the requirements of CAAA. The provisions of ISTEA, which are designed to protect the environment, must be incorporated into the metropolitan and state planning processes. The Federal Highway Administration (FHWA) and the Federal Transit Administration (FTA) jointly published the Metropolitan and Statewide Planning Rule (*4*) in October 1993 to ensure the adequacy of state and metropolitan transportation planning processes.

Before the passage of ISTEA, statewide transportation planning activities occurred without specific planning requirements. ISTEA requires a statewide planning process, the consideration of 23 factors, a long-range plan, a program of projects, and specific project selection procedures. Some of these factors are highlighted in Table 2-1. Each of the 23 factors must be considered and analyzed as part of the statewide transportation planning process.

ISTEA has enhanced MPO responsibilities and provided additional funding to help MPOs handle these responsibilities. The metropolitan planning regulation moves toward common analysis requirements for highway and transit projects. The regulation also requires that transportation plans and programs consider more efficient use of road facilities.

Before the passage of CAAA and ISTEA, transportation planning consisted primarily of predicting future traffic volume to ensure that future road facilities were sufficient to meet demand. Transportation planning, however, has become more than just the development of a capital improvement program. It now needs to include transportation system management and transportation demand management strategies. The metropolitan planning process must account for 15 factors in

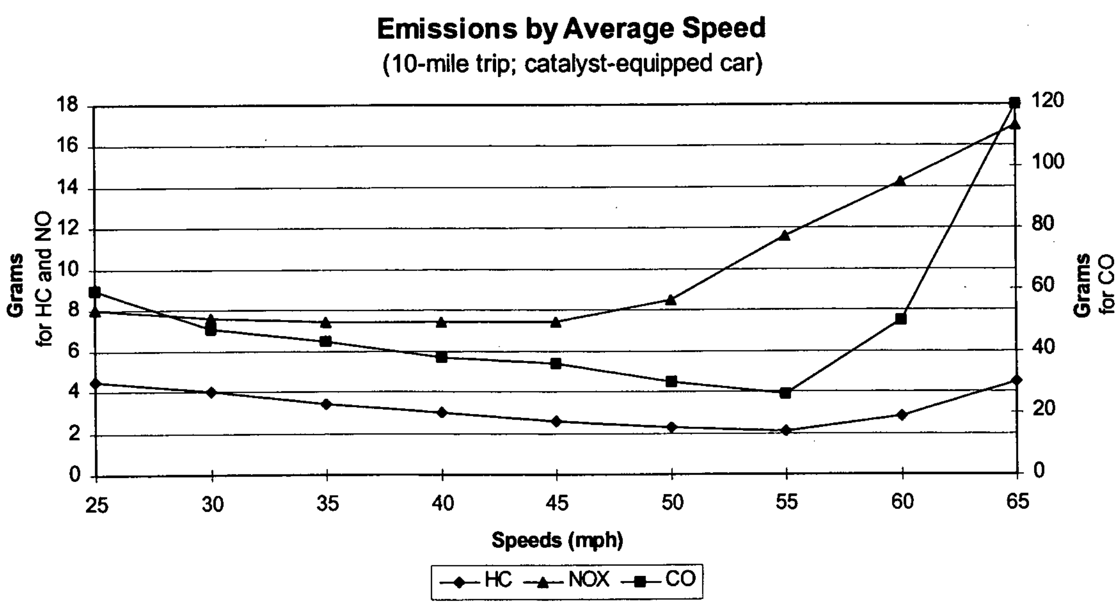

Figure 2-1. Emissions by average speed.

TABLE 2-1 ISTEA statewide planning factors (*5*)

(1) The transportation needs (strategies and other results) identified through the six management systems.
(5) Transportation needs of non-metropolitan area through a process that includes consultation with local elected officials with jurisdiction over transportation.
(10) Transportation system management and investment strategies designed to make the most efficient use of existing transportation facilities (including consideration of all modes).
(12) Methods to reduce traffic congestion and to prevent traffic congestion from developing in areas where it does not yet occur.
(14) The effect of transportation decisions on land use and land development, including the need for consistency between transportation decision making and the provisions of applicable land use and development plans.
(16) The use of innovative mechanisms for financing projects, including value capture pricing, tolls, and congestion pricing.

the preparation of transportation plans and programs. Some of these factors are listed in Table 2-2.

ISTEA established the need for six management systems: bridge, pavement, public transportation, safety, congestion, and intermodal. States were responsible for implementing these management systems with the cooperation of the MPOs. The first three management systems focus more on the management of transportation system assets. The latter three management systems deal more with the performance aspects of the transportation system.

The congestion management system (CMS), intermodal transportation management system, and public transportation management system are strongly encouraged in the transportation planning process. The CMS, however, is required in transportation management areas (TMAs), which are urban areas whose population exceeds 200,000.

CMSs and intermodal transportation management systems require that performance measures other than level of service be identified. Alternative measures can include accessibility, jobs/housing balance, cost, and time.

Major investment studies are now required as part of the move toward intermodalism. To encourage multimodal planning, these studies bring together the previously separate FHWA and FTA planning processes and include a technical analysis process that includes both highway and transit alternatives.

Highway performance monitoring systems (HPMSs) are required in each state for the purpose of monitoring road facility performance and providing data for the monitoring of air quality. A representative set of locations are selected by functional class and county for monitoring traffic volume. The vehicle miles measured at these locations are then factored by total miles of functional class to obtain an estimate of total vehicle miles traveled (VMT).

These regulations apply mostly to state departments of transportation (DOTs) and MPOs. However, they also apply to local agencies that are seeking federal transportation dollars. Typically, MPOs take care of necessary planning requirements so that local agencies can obtain federal money.

2.2.2 State Regulations

State legislatures also have been increasing the role of transportation planners in congestion and growth management. Some states, such as California and Florida, have established statewide growth and congestion management programs (CMPs). Such programs vary from state to state.

2.2.2.1 Congestion Management Program in California

In California, Proposition 111, which was passed in 1990, required that all counties with a population exceeding 50,000 prepare and update biannual CMPs (*7*). These programs must include a procedure for forecasting the impact of land use

TABLE 2-2 ISTEA metropolitan planning factors (*6*)

(1) Preservation of existing transportation facilities and, where practical, ways to meet transportation needs by using existing transportation facilities more efficiently.
(3) The need to relieve congestion and prevent congestion from occurring including: consideration of congestion management strategies and actions and the development of the congestion management system in TMA's.
(4) The likely effect of transportation policy decisions on land use and development and the consistency with the provisions of land use and development plans.
(6) The effect of all transportation projects within the planning area regardless of funding source.
(9) Transportation needs identified through the use of the six management systems by analyzing strategies developed under each system during the development of the transportation plan.

decisions on regional highways and on level of service standards estimated using the method in *Transportation Research Circular 212: Interim Materials on Highway Capacity*, the most recent HCM methods, or an alternative method consistent with HCM methods.

Substantial changes were made to this legislation with the adoption of AB 1963 in September 1994, which included a performance element. Even though there are no specific thresholds to achieve, the performance element identifies performance measures in addition to service standards. A performance measure is defined in the legislation as "an analytical planning tool that is used to quantitatively evaluate transportation improvements and to assist in determining effective implementation actions, considering all modes and strategies" (*8*). In Contra Costa County, for example, the CMP includes such measures as level of service, v/c ratio, speed, delay, duration of congestion, and peak-hour vehicle occupancy.

2.2.2.2 Growth Management in Florida

In response to growth, Florida passed the 1984 State and Regional Planning Act, which required development of the State Comprehensive Plan, state agency function plans, and comprehensive regional policy plans. The following year the 1985 Omnibus Growth Management Act introduced an integrated planning process for state, regional, and local governments. As a result, minimum acceptable level of service standards were adopted for roads throughout the state. Local and state agencies must now forecast future levels of service in accordance with HCM methods (*9*).

2.2.3 Other Regulations

In addition to federal and state legislation, local jurisdictions have their own requirements. Some counties and cities have begun to implement their own growth management programs. For example, in Contra Costa County, California, voters passed the Measure C Growth Management Program, which provides funds through a county sales tax for transportation improvements. To be eligible for funds, local jurisdictions are required to prepare action plans and identify traffic service objectives. Subregional planning models are used to forecast traffic, and the Circular 212 method is used to analyze signalized intersections.

General and specific local plans are another form of growth management to which local transportation planners must be responsive. Local standards and requirements must be met when site impact analyses and environmental impact reports and studies are prepared.

2.3 CURRENT PRACTICES

This section describes current transportation planning practices around the United States as agencies respond to new legal requirements. Planners are required to estimate speed, service volume, and capacity with greater precision and limited resources. Planning applications and estimation techniques are described, and results of the user survey are discussed.

2.3.1 Summary of Survey Results

The user survey was distributed nationwide to determine current capabilities and resources for estimating travel speed and service volume of road facilities for use in various planning applications. Respondents included planners at state DOTs (22 percent), regional agencies (27 percent), local agencies (32 percent), and private firms (19 percent). The 77 responses represent a 38 percent return rate. Geographically, the responses, when grouped by region, represent the country well, ranging from 18 percent from the Central region to 32 percent from the West Coast region.

When asked about planning responsibilities and measures of effectiveness, respondents indicated that site impact studies are by far their most common responsibility and that capacity, followed by service volume, is the most commonly used measure of effectiveness. Other common applications include congestion management studies, RTPs, TIPs, and major investment studies. Capacity and service volume are used as measures of effectiveness about 90 percent and 78 percent of time, respectively. Speed is used as a measure of effectiveness only about 50 percent of the time.

When asked which techniques they use to estimate level of service, respondents indicated that they use HCM methods most frequently. About 34 percent of respondents use HCM methods, compared with 28 percent and 23 percent who use v/c ratios and service volume, respectively. The Florida method and other methods amounted to about 15 percent of the total. Figure 2-2 shows the distribution of techniques used for estimating level of service. Significant differences in methods used did not appear to be the result of facility type (interrupted or uninterrupted) or area type (rural or urban). By application, for RTPs, the v/c ratio is used about 40 percent of the time, compared with HCM methods, which are used about 24 percent of the time. For all other applications, the methods used were consistent with the overall response.

Survey question 6 dealt with the techniques used to estimate road capacity. Overall, the responses were evenly divided between HCM methods and look-up tables. About 41 percent of respondents indicated that they use HCM methods, whereas custom tables and Urban Transportation Planning System (UTPS) default tables combined are used by about 41 percent of respondents. Figure 2-3 summarizes the distribution of techniques used for estimating capacity. The difference in techniques used is more pronounced when viewed by individual applications. Look-up tables are used twice as often as HCM methods for RTPs, whereas HCM methods are used more than twice as often as look-up tables for traffic impact studies.

Techniques used to estimate free-flow travel speed varied from road facility design and posted speed, to UTPS and

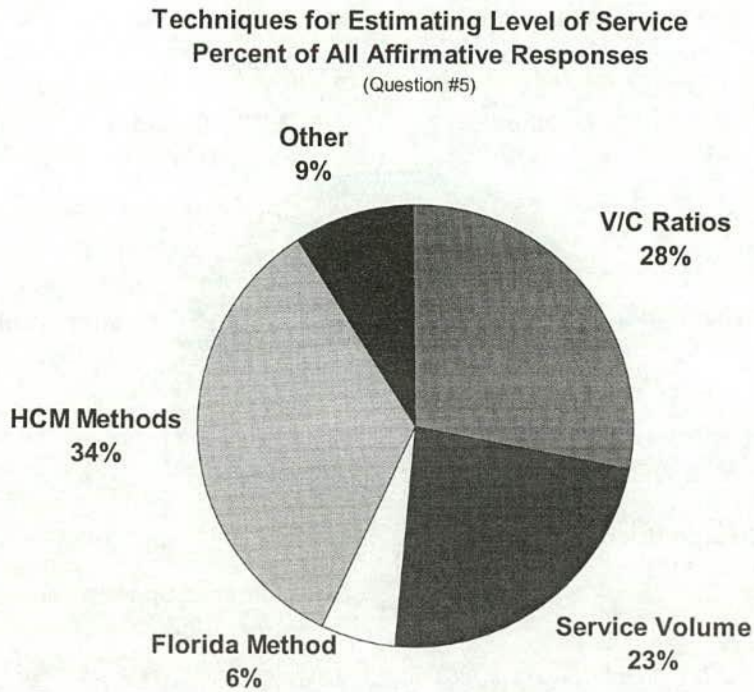

Figure 2-2. Distribution of techniques used for estimating level of service.

custom tables, to HCM methods. The most frequently cited technique was posted speed, which comprises 30 percent of responses. UTPS default and custom look-up tables amounted to about 29 percent of the total. Only 16 percent of respondents indicated that they use HCM methods. Figure 2-4 illustrates the distribution of techniques used for estimating free-flow speed. Look-up tables were cited most often for use with RTPs, which typically involve running a transportation network demand model.

For congested conditions, one-third of respondents indicated that they estimate speed by using HCM methods. Field measurements of congested speed also received a high

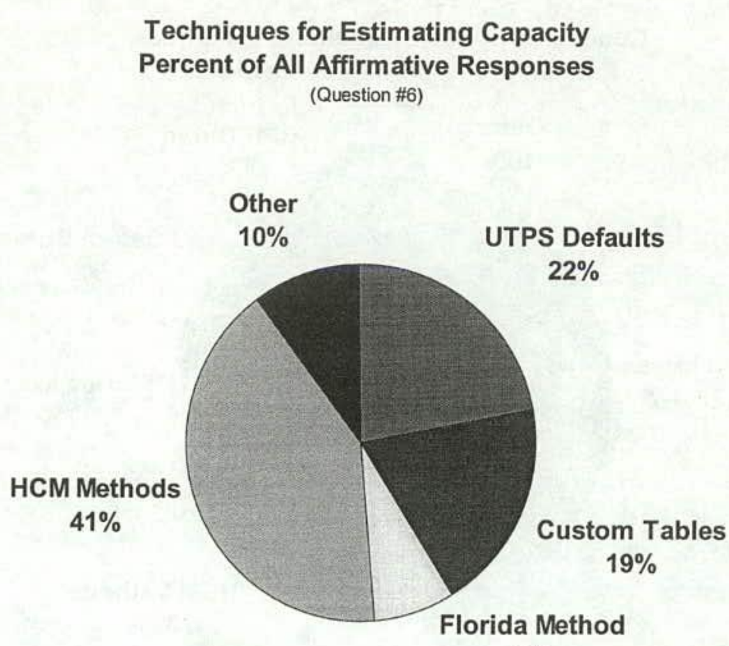

Figure 2-3. Distribution of techniques used for estimating capacity.

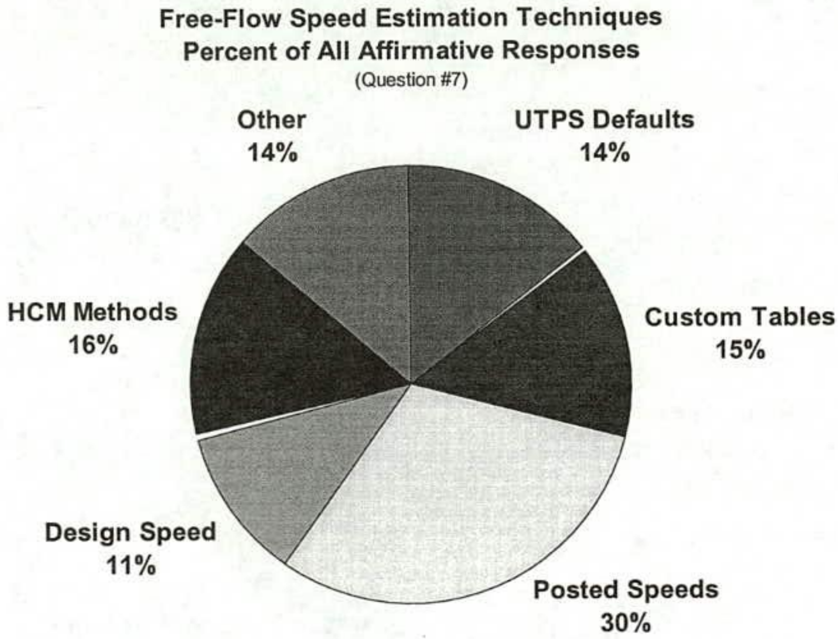

Figure 2-4. Distribution of techniques used to estimate free-flow speed.

response rate, at 28 percent. Each of the other techniques were cited by less than 15 percent of respondents. The distribution of techniques used to estimate speed on congested facilities appears in Figure 2-5. By application, HCM methods are used most often, compared with the Bureau of Public Roads (BPR) or custom curves, except for RTPs, in which the BPR or custom curve is used slightly more often.

The survey also inquired about an agency's ability to collect and provide data. Count data are most readily available to all respondents, and the availability and feasibility of collecting other data items varied by respondent (Figure 2-6). Detailed question-by-question results appear in Appendix B.

2.3.2 Overall Transportation Planning Practices

The transportation planning process differs from state to state, region to region, and city to city. A variety of speed, volume, and level of service estimation techniques are used for

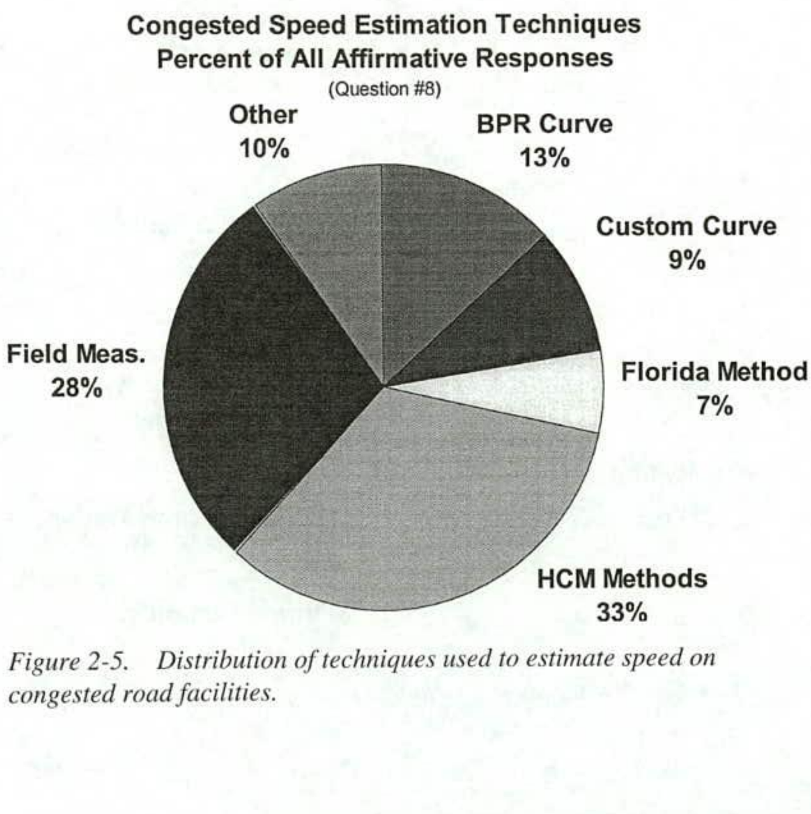

Figure 2-5. Distribution of techniques used to estimate speed on congested road facilities.

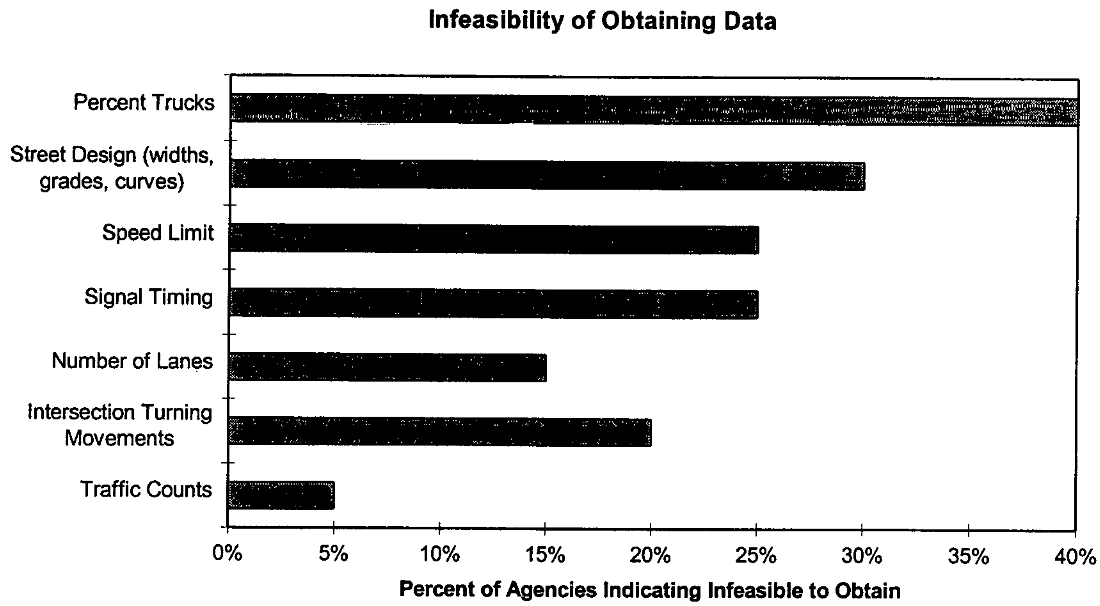

Figure 2-6. Relative infeasibility of obtaining input data.

transportation planning purposes. Each agency has its own approach to planning; however, some practices are used by all.

Planning techniques are used at the earliest stage of planning, when precise data often are not available. Estimates often are based on the average annual daily traffic (AADT) volume and assumptions about traffic, roadway, and control conditions. The initial objective is to determine the number of lanes required to achieve a given level of service. Most of the operational analysis in HCM methods requires more detailed data, although the defaults provided can be used.

Some common transportation planning applications include development of regional transportation plans and transportation improvement programs and conduct of site impact analyses. Long-range plans and TIPs typically are the responsibility of MPOs; however, states also are now required to prepare long-range plans, and local jurisdictions sometimes are responsible for TIPs. All agencies are responsible for site impact analyses. Congestion management, intermodal planning, major investment, and air quality conformity studies (AQCSs) increasingly have become part of transportation planners' responsibilities. The techniques used by planners differ by application and agency.

Many current planning techniques for estimating speed and level of service are not based on nor are consistent with HCM techniques. About one-third of respondents use the HCM method for estimating level of service, whereas about half of respondents use either the v/c ratio or service volume. These planning techniques are not sensitive to many transportation control measures such as improved signal coordination and intelligent transportation system strategies.

As mentioned previously, transportation planning has become more than just the development of capital improvement programs. It must now include strategies to manage transportation systems and transportation demand. To analyze these strategies, better planning tools are needed to estimate speed and service volume.

Some transportation planners estimate capacity for their transportation network demand models by using ad hoc, unverified, and rarely documented assumptions about facility characteristics that vary by facility type and area type. Default UTPS capacity tables and custom tables estimate capacity based on facility type and area type. Some planners estimate link capacity based on cross-section green time per cycle (g/C) and signal capacity. About 40 percent of respondents use HCM techniques, and a similar percentage use either default UTPS tables or custom tables. The capacity estimate used in models also can vary because some models use practical capacity rather than maximum capacity.

Some techniques used to estimate speed on congested road facilities are the standard BPR speed-flow curve, variations of the BPR speed-flow curve, the method in the Florida *Level of Service Manual*, and the HCM method for estimating speed on congested road facilities. Few planners estimate speed; most of them estimate capacity or service volume.

Most transportation planning models incorporate average travel speed that is estimated using variations of the BPR speed-flow curve that relate congested speed to starting free-flow speed and v/c ratio. These BPR curves generally do not match HCM speed-flow curves very well in v/c ratios less than 1. In addition, there is no known validation of the accuracy of these BPR curves for v/c ratios that exceed 1, where queuing occurs.

Survey results indicate that field measurements and HCM techniques are the most common speed estimation techniques. Only about 20 percent of respondents use BPR and custom curves to estimate speed. However, for RTPs, the BPR curve or custom curve is more commonly used.

2.3.3 Agency Differences

2.3.3.1 State Departments of Transportation

Transportation planning at state DOTs generally is conducted to support corridorwide and systemwide studies of state highway facilities. With the passage of ISTEA and CAAA, some state DOTs have become involved in developing transportation plans and programs. State DOTs more commonly estimate capacity and service volume. However, survey results show that speed is estimated for a considerable number of planning applications and that state DOTs prefer to use HCM methods when estimating level of service and speed on congested road facilities.

Florida is one of the few states to methodically estimate service volume by facility type based on HCM techniques. The state, however, estimates speed only for arterials. HCM level of service criteria for low-design-speed urban streets can prevent some streets from operating at a level of service better than D regardless of traffic volume. An evaluation needs to be made to determine whether the defaults used by Florida can be legitimately extended to other states or whether procedures need to be developed for use by localities in determining these defaults.

2.3.3.2 Metropolitan Planning Organizations

MPOs are responsible for preparing long-range transportation plans covering a 20-year period, transportation improvement programs, and unified planning work programs (UPWPs). UPWPs define specific planning activities to be undertaken for all transportation modes and programs. Other MPO activities include development of transit development programs and state implementation plans. MPOs must work with local jurisdictions as well as state DOTs to ensure a cooperative, comprehensive approach to transportation planning. As noted previously, MPOs have been given greater responsibilities with the passage of CAAA and ISTEA. Although estimates of capacity and service volume are the more common measures of effectiveness, estimates of speed are an important part of AQCSs and congestion management studies.

MPOs are the focus for the development of regionwide travel demand projections. Regional travel demand models, which typically are operated by regional agencies, tend to cover multijurisdictional areas. At the regional planning level, therefore, emphasis is placed on the development of capacity, volume, and speed estimates for collector, arterial, and freeway segments. Capacity is estimated using a variety of techniques, depending on the analysis capabilities of the MPO. The Portland, Oregon, metropolitan area, for example, uses a modified BPR curve.

The use of transportation network–based travel demand models by MPOs for RTPs may explain the differences often found in the techniques agencies use for RTPs. The survey indicates that MPOs, unlike all other agency types, are more likely to use v/c ratios instead of HCM methods for estimating level of service. For speed estimation on congested road facilities, MPOs cited the use of the BPR or custom curve more often than average and almost as often as HCM methods, whereas state DOTs tend to use HCM methods and local agencies tend to use field measurements.

MPOs are the least likely of the agency types surveyed to have the resources to collect detailed data on certain items. Except for count data, more than 25 percent of MPOs indicated that it is infeasible for them to collect data on such items as on-street parking, percentage of heavy vehicles, off-peak travel time or speed, design speed, lane widths, grades and curves, signal type, and coordination quality. This is in sharp contrast to state DOTs and local agencies, in which almost all these data items are feasible to obtain.

2.3.3.3 Local Agencies

At the county and city levels, transportation planning needs include a range of applications from systemwide evaluations to arterial- and intersection-specific analyses. It is not surprising, therefore, that a wide range of techniques to estimate speed, volume, and capacity are used by transportation planners. Capacity estimates are most often used by local agencies. Local agencies are the least likely of the agency types surveyed to use speed as a measure of effectiveness. The survey shows that local agencies are most likely to use HCM methods to estimate level of service and to rely on field measurements to estimate speed on congested road facilities.

Estimation techniques used by local agencies vary widely. For example, in Portland, speed is estimated by application of the standard BPR curve, by modifications to the BPR curve, or by techniques from the HCM. Level of service estimates generally are developed either from the Oregon DOT methodology using SIGCAP or, more frequently, from HCM techniques. Most often, HCM techniques are implemented through application of highway capacity software.

Local agencies are the most likely of the agency types surveyed to have the resources to collect data on specific items such as those mentioned previously. The survey indicates that 11 of the 20 data items (see Appendix B) were either available or obtainable by local agencies. The three data items most difficult to obtain by local agencies are directional volume, percentage of heavy vehicles, and off-peak travel time or speed.

2.3.4 Regional Differences

This section highlights differences in planning techniques that may be attributed to or reflect geographic differences. Overall, survey results did not indicate any significant regional differences. The use of level of service and speed-estimation techniques tends to be similar across the country. However, some slight differences were noted.

For the following discussion, the country was divided into four general geographic regions: the West Coast, Mountain, Central, and East Coast regions. The survey indicates the following regional biases for estimating levels of service:

- The Mountain and West Coast regions prefer using v/c ratios and service volume instead of HCM techniques. In the Mountain region, level of service estimation techniques are evenly divided among v/c ratios, service volume, and HCM methods. West Coast respondents indicated that v/c ratios were used most often, followed closely by HCM methods.
- The East Coast and Central regions prefer to use HCM methods. This is most pronounced in the Central region, where almost 50 percent of respondents indicated that they use HCM methods.
- The West Coast region has the highest percentage (13 percent) of respondents who use other methods, whereas the Mountain region has the lowest percentage (3 percent).

For estimating speed on congested road facilities, all regions except the West Coast region prefer to use HCM methods. More than 35 percent of respondents from the East Coast, Mountain, and Central regions indicated that they use HCM methods. In the West Coast region, field measurements are preferred. The West Coast region is the most likely region to use custom curves to estimate speed.

Respondents from the West Coast and East Coast regions are less likely to have the resources to collect data. The Central and Mountain regions tend to have more data available than other regions. Of the 20 data items, all respondents in the Mountain and Central regions found it feasible to obtain 14 and 13 of the items, respectively. In the West Coast and East Coast regions, only one or two items are feasible for all respondents to collect.

2.4 AGENCY PROFILES

This section provides a detailed look at the transportation planning practices of several agencies. The agencies represent state DOTs, MPOs, and local agencies. Each agency profile includes a brief description of the planning applications for which the agency is responsible and methods used to estimate speed, capacity, and service volume.

2.4.1 State Departments of Transportation

2.4.1.1 Oregon Department of Transportation

Planning within ODOT is generally conducted in support of corridorwide and systemwide studies on one or a few state highway facilities. The focus of ODOT analyses, therefore, tends to be more on the operational adequacy of single points and linear systems than on overall networkwide performance characteristics.

ODOT relies almost exclusively on estimation of the v/c ratio as the primary means for measuring the adequacy of a point or facility. The analytic means by which the v/c ratio is estimated at signalized intersections is a computerized critical movement technique developed by ODOT staff and named SIGCAP. SIGCAP approximates the operational analysis procedure described in Circular 212. Threshold v/c values developed by ODOT staff are used as the basis for assigning level of service grades to these intersections.

2.4.1.2 New Hampshire Department of Transportation

NHDOT is responsible for a full range of transportation planning applications, including the state's TIP, CMSs, AQCSs, intermodal planning studies, HPMS reports, and site impact analyses. Speed, service volume, and capacity are used as performance measures.

HCM techniques are used exclusively to estimate service volume and capacity. To estimate free-flow speed and speed on congested road facilities, in addition to using HCM methods, NHDOT uses its own procedures. Posted speed limit is also used to estimate free-flow speed.

NHDOT has compiled detailed procedures for estimating daily vehicle miles traveled (VMT) and average speed for the state's emissions inventory (*10*). NHDOT, which is responsible for providing VMT and speed estimates for the mobile source emissions calculation, used a step-by-step process to develop these estimates.

Service volume and speed data were derived from HPMS samples taken throughout the state on non-local highway segments. VMT was obtained from the HPMS database, which provides estimates of average annual daily traffic (AADT) adjusted to reflect the average summer condition. Daily speed was calculated from HPMS data by adding 65 percent of the speed limit and 35 percent of the peak-hour speed. Peak-hour speed was estimated using the speed limit, the AADT, a *k*-factor, and the hourly capacity of the segment from the HPMS database. The peak-hour v/c ratio was based on the AADT, the *k*-factor, and the hourly capacity. Using the following equations developed from Chapters 7 and 11 of the HCM, peak-hour speed was estimated.

For multilane highways:

$$\text{travel speed} = \text{speed limit if } v/c < 0.65, \text{ and}$$
$$\text{travel speed} = \text{speed limit} - 5 * v/c^4 \text{ if } v/c > 0.65.$$

For urban streets:

$$\text{travel speed} = \text{speed limit} - 20 * v/c.$$

Because HPMS samples do not exist for local roads, VMT and speed were calculated differently. HPMS includes an estimate of the local road VMT, which was allocated throughout the state based on mileage. Speed was assumed to be equal to that of the closest function class.

2.4.1.3 New York State Department of Transportation

NYSDOT documented its procedure for estimating speed for the 1994 Air Quality State Implementation Plan (*11*). For a 1990 base year and several horizon years, NYSDOT estimated speed for six urban and six rural functional classes in four time periods by geographic area. A Lotus spreadsheet was prepared for the speed estimation and as a tool for managing the data.

Peak and off-peak speed data for a base and a future year are collected from MPOs by using network analysis models. Because the data available for the New York City metropolitan area consisted of the 24-hour average speed for three functional classes, specific procedures used for upstate MPOs and the New York City area differed. However, the overall approach for both is similar:

1. Speed for intermediate years was linearly interpolated.
2. Speed-flow curves relating speed to v/c ratio from the HCM were identified for different functional classes. Both the 1985 and the 1965 HCM were used as sources. Table 2-3 illustrates the speed-flow relationships used for the different functional classes.
3. For the model peak-hour speed for upstate roadways, the peak-hour v/c ratio was determined using the speed-flow equations. For the New York City area, the peak-hour v/c ratio was assumed to range from 0.8 to 1.0, depending on the area.
4. Using the peak-hour v/c ratio and the time-of-day distribution for the four time periods, the time period v/c ratio was estimated, and the corresponding speed was calculated using HCM speed-flow relationships. After some adjustments were made to account for differences between model speed and HCM-estimated speed, the final speed was determined.

2.4.2 Regional Agencies

2.4.2.1 San Francisco Metropolitan Transportation Commission

The San Francisco Metropolitan Transportation Commission (MTC) uses a BPR-type curve for both freeways and arterials, but with parameters *a* equal to 0.45 and *b* equal to 4 (*12*). This example illustrates the practical application of a BPR curve with a slightly different shape that is applied universally to all facility types and the practical application of v/c ratios.

This form was selected based on a statistical analysis of field data at 119 freeway locations. The freeway data, however, had no observed speed greater than 56 mph, and many observations appear to have been arbitrarily limited to a maximum of 55 mph. Practical capacity is replaced with maximum capacity in the equation. Free-flow speeds are input, rather than computed as a ratio of the speed at capacity. Maximum capacity and free-flow speeds are obtained from a look-up table by facility type and area type, as shown in Tables 2-4 and 2-5.

TABLE 2-3 NYSDOT speed-flow relationships

Functional Class	Speed Flow Equation	Source
Freeways and expressways in NY metro area	Spd from HCM Figure 3-4	85 HCM, Ch 3.
Upstate urban and rural freeways, urban expressways, and rural principal arterials	$Spd = 30 + (5000*(1-v/c))^{0.333}$ for $0.80 < v/c \leq 1$ $Spd = 60 - 1.46*v/c - 11.46*v/c^2$ for $0 < v/c < 0.80$	85 HCM, Ch 3.
Principal and minor arterials in Manhattan	Spd from HCM Table 11-4, pg. 11-9	85 HCM, Ch 11.
Arterials in urban areas other than Manhattan Collectors and local streets in NY metro area	$Spd = 12.8 + 21*(1-v/c)^{0.65}$ for $0.74 < v/c \leq 1$ $Spd = 12.8 + 12*(1-v/c)^{0.30}$ for $0 < v/c < 0.75$	85 HCM, Ch 11.
Rural minor arterials and major collectors	$Spd = 55 - 25*v/c$	65 HCM, Ch 10.
Urban collectors and local streets outside NY metro area	$Spd = 15 + 24.4*(1-v/c)^{0.48}$ for $0.65 < v/c \leq 1$ $Spd = 17 + 15*(1-v/c)^{0.14}$ for $0 < v/c < 0.65$	65 HCM, Ch 10.
Rural minor collectors and local roads	$Spd = 45 - 25*v/c$	65 HCM, Ch 10.

TABLE 2-4 San Francisco MTC one-way capacity per lane per hour for use with MTC curve (in vph)

Area Type	Freeway	Expressway	Major Arterial	Collector	Ramp	Metered Ramp
Core	1850	1300	850	600	1300	700
CBD	1850	1300	850	600	1300	700
Urban BD	1850	1450	950	650	1400	800
Urban	1850	1450	950	650	1400	800
Suburban	1850	1450	950	800	1400	900
Rural	1850	1450	950	850	1400	900

The area type in which a facility is located was determined based on the density of the traffic analysis zone in which the facility is located (Table 2-6). The equation for computing density is as follows:

$$\textit{Density} = \frac{\textit{Population} + 2.5 \times \textit{Employment}}{\textit{Residential Acres} + \textit{Commercial Acres} + \textit{Industrial Acres}}$$

Freeway free-flow speeds were taken from field data for 119 freeway links in the San Francisco Bay Area. These data, however, are suspect because the highest observed free-flow speed was 56 mph and the most common observed free-flow speed was precisely 55 mph. Ramp free-flow speeds were arbitrarily set about 25 mph lower than freeway speeds.

The interrupted flow facility (arterial, collector, and expressway) free-flow speeds were taken from the 1985 HCM. Expressway free-flow speeds were arbitrarily increased by 10 mph over arterial speeds.

The maximum capacity for freeways was computed using the 1985 HCM, assuming 7 percent trucks and buses plus 3 percent recreational vehicles. Weaving effects on capacity were accounted for by applying the 1965 HCM weaving analysis procedures to a typical freeway segment equal to the mean length, mean number of lanes, and mean flows of the MTC freeway network.

The capacity of expressways, arterials, and collectors was computed by treating all as signalized urban streets. The ideal saturation flow for each facility type was reduced according to factors derived from the HCM, using the assumptions shown in Table 2-7.

2.4.2.2 Portland Metropolitan Area

In Portland, the MPO is responsible for developing region-wide travel demand forecasts. Capacity, speed, and service volume estimates depend on conical volume-delay functions developed at the University of Montreal. These functions are variations of the more widely used BPR curves. The Metropolitan Service District in Portland uses the conical volume-delay function in lieu of the BPR curves because it believes that with this function, the assignment process can achieve equilibrium in fewer iterations. (Because of the nature of the BPR formula, the travel times for links with very low v/c values are equivalent to free-flow time. Hence, the path choice algorithm is no longer volume dependent and reduces locally to an all-or-nothing assignment, which is contrary to the philosophy that every assignment of trips to a network should yield a unique solution.)

Estimates of speed and capacity are updated wherever possible on the basis of field observations. Typical link capacities are initially estimated on the basis of cross-section g/C ratio and signal density. Where necessary and on the basis of field observations, link capacities are adjusted so that they are at least as high as actual measured volumes. Estimates of

TABLE 2-5 San Francisco MTC free-flow speeds for use with MTC curve (in mph)

Area Type	Freeway	Expressway	Major Arterial	Collector	Ramp	Metered Ramp
Core	55	40	25	20	30	25
CBD	55	40	30	25	30	25
Urban BD	60	45	35	30	35	30
Urban	60	45	35	30	35	30
Suburban	65	50	40	35	40	35
Rural	65	55	40	40	40	35

TABLE 2-6 San Francisco MTC area type by density range

Density Range	Area Type
Greater Than 300	Core
100 - 299	Central Business District
55 - 99	Urban Business District
30 - 54	Urban
6 - 29	Suburban
Less Than 6	Rural

arterial free-flow speeds are based on posted speed limits rather than actual speed studies.

2.4.2.3 Southeast Michigan Council of Governments

The Southeast Michigan Council of Governments (SEMCOG) is an MPO covering Detroit and surrounding communities. SEMCOG is responsible for the area's RTPs, TIPs, AQCSs, intermodal planning analyses, and HPMSs. SEMCOG indicates that speed, service volume, and capacity are relevant measures of effectiveness for both RTPs and AQCSs, but does not specify which measures of effectiveness it uses for other planning applications.

SEMCOG provided the NCHRP research team with information on how it obtains some of the data required for applying estimation methods such as those in the HCM. SEMCOG receives the roadway number of lanes from the Michigan State Police Crash Database; functional class of roadways comes from the Michigan Department of Transportation; and area type is determined by SEMCOG.

To estimate level of service, SEMCOG relies on two techniques: the v/c ratio method and HCM method. Both methods are used to estimate level of service for RTPs and AQCSs, and both are equally valid for urban/rural areas and for interrupted/uninterrupted facilities. SEMCOG, however, realizes that there is a drawback to the v/c ratio method in that it does not reflect the impact of left turns, uncontrolled access points, or capacity constraints. The HCM method, although ideal for short roadway segments, does not work well with intersections, weaving, or merges.

SEMCOG uses UTPS default capacity tables to estimate roadway capacity. As is the case with level of service, these capacities are used only for RTPs and AQCSs. These default capacity tables are good for estimating capacity of freeway segments but not as good for estimating capacity of weaving segments, intersections, or roadways with merge/diverge areas. SEMCOG has produced a capacity table as part of its long-range plan to identify current and future roadway capacity deficiencies. The v/c ratio must be at least 0.80 before a roadway segment is considered congested. This capacity table appears in Table 2-8.

SEMCOG has provided a detailed description of how it estimates speed (*13*). Knowledge of actual speeds is critical for SEMCOG's RTPs and AQCSs. SEMCOG began using UTPS default speed tables and speed-flow curves in the early 1990s, but discovered that some of the initial speed calculations were too low. Therefore, SEMCOG developed its own empirical dataset of speeds based on hundreds of time and delay studies conducted in 1992, which were based on roadway functional classification (*fc*), area type (*at*), and time-of-day (*tod*). Based on these tables, SEMCOG then modified the default speed equations with its own equation:

$$\begin{aligned}&\textit{Average Link Speed} = \\ &\quad \textit{Average Queue Speed} * (\textit{Average Queue Length/Length}) \\ &\quad + \textit{Uncongested Speed} \\ &\quad * (1 - (\textit{Average Queue Length/Length}))\end{aligned}$$

where:

$$\textit{Non-Queuing Speed} = 1.24 * \textit{Speed Survey}_{(fc,\,at,\,tod)} / (1 + (v/c)^{11})$$

(If *Uncongested Speed* < 25 mph, *Uncongested Speed* = 25 mph.)

TABLE 2-7 San Francisco MTC capacity assumptions

Facility	Area	Ideal Sat/ Lane	Area Factor	Truck Factor	G/C	Parking Factor	Local Bus Factor	Capacity / Lane
Expressway	CBD	2000	x 0.9	x 0.95	x 0.75	x 1.00	1.00 =	1282
	Non-CBD	2000	1.0	0.95	0.75	1.00	1.00	1425
Arterial	CBD	1800	0.9	0.95	0.65	0.87	0.96	835
	Non-CBD	1800	1.0	0.95	0.65	0.87	0.96	928
	Suburban Rural	1800	1.0	0.95	0.65	0.89	0.98	969
Collector	CBD	1800	0.9	0.95	0.50	0.80	0.96	591
	Non-CBD	1800	1.0	0.95	0.50	0.80	0.96	656
	Suburban	1800	1.0	0.95	0.50	0.90	1.00	770
	Rural	1800	1.0	0.95	0.50	1.00	1.00	855

TABLE 2-8 SEMCOG-estimated per lane capacities

Functional Classification		Vehicles Per Hour (VPH)
Rural	Interstate	2,000
	Freeway (Non-Interstate)	2,000
	Principal Arterial (Non-Freeway)	900
	Minor Arterial	850
	Minor Collector	650
Urban	Interstate	2,000
	Freeway (Non-Interstate)	2,000
	Principal Arterial (Non-Freeway)	825
	Minor Arterial	800
	Minor Collector	625

$$\begin{aligned}
\textit{Average Queue Speed} &= \text{capacity/lane} * 25 \text{ ft/vehicle} \\
\textit{Average Queue Length} &= \textit{Average Queue} * 25 \text{ ft/vehicle} \\
\textit{Average Queue} &= (Q1 + Q2)/2 \\
Q1 &= \text{queue at start of time slice} \\
Q2 &= Q1 + (1 \text{ hr traffic/lane} - 1 \text{ hr capacity/lane}) \\
\textit{Speed Survey}_{(fc,\ at,\ tod)} &= \text{link speeds from 1992 speed survey by function class, area type, and time of day.}
\end{aligned}$$

SEMCOG places a minimum uncongested speed of 25 mph on all its facilities. This is the MPO's method of replicating actual conditions. The link speed tables are provided for AM, PM, and off-peak periods for urban and rural area types.

Because of its limited planning functions, SEMCOG does not gather approximately half the data required for using the more common level of service and capacity estimation techniques. Data that SEMCOG does have, or can easily obtain, follow:

- 24-hour link traffic count data
- Peak-hour link traffic count data
- 15-min peak-hour link traffic count data
- Link volume directional distribution
- Number of midblock lanes
- Through and turning lanes at intersections
- On-street parking
- Percent trucks, buses, and recreational vehicles
- Peak-hour travel time or speed
- Off-peak travel time or speed.

2.4.2.4 Rochester-Olmsted Minnesota Council of Governments

The Rochester-Olmsted Council of Governments (COG) is an MPO responsible for the area's RTPs, transportation improvement program, site impact analyses, and any special corridor or subarea studies that may be needed. Speed, service volume, and capacity are all considered measures of effectiveness by this MPO. However, service volume is not considered for RTPs, and speed is not considered for site impact analyses.

The Rochester-Olmsted COG estimates level of service using two primary methods: the v/c ratio and HCM method. The former is used for RTPs; the latter is used for major investment studies and site impact analyses. The MPO points out that the v/c ratio method is not applied to rural areas. In addition, it performs simulation of its area roadways using CORFLO, which provides a means to estimate level of service.

The simulation program, CORFLO, is even more widely applied when estimating capacity. It is particularly useful for major investment studies. The HCM is applied to these studies as well as to site impact analyses. For RTPs, the MPO has created its own customized capacity tables. The Rochester-Olmsted COG facility database divides roadway facilities into 25 facility code groups. Data for each facility code group include speed, capacity, delay, number of lanes, medians, left turn lanes, and heavy or small volume of opposing traffic. For each facility code group, capacity is calculated by multiplying lane capacity by the number of lanes. Adjustments to this include the following factors:

- Level of Service C
- Metropolitan area
- Number of hours
- Left turns
- Side friction
- Operational friction.

The facility database not only is used to estimate capacity, but also free-flow speed. Other methods the MPO uses to estimate speed include the HCM method and field measurements. Again, the only planning applications that use capacity estimates are RTPs and major investment studies. In addition to not being able to be used to analyze free-flow speed for rural areas, the HCM method has an additional limitation of being applicable only to interrupted facilities. The same holds true for the estimation of speed on congested road facilities. However, in addition to RTPs and major investment studies, site impact analyses can be conducted with the HCM and field measurement methods.

The Rochester-Olmsted COG collects most of the data needed to estimate speed, capacity, and level of service. The following four data items are difficult for the MPO to obtain:

- Percentage of trucks, buses, and recreational vehicles
- Peak-hour travel time or speed
- Off-peak travel time or speed
- Quality of signal coordination.

2.4.2.5 Transportation Agency of Monterey County, California

The Transportation Agency of Monterey County (TAMC) is an MPO in west-central California. This agency is responsible for the county's RTP, TIP, major investment studies, CMSs, and intermodal planning studies. TAMC uses only service volume and capacity as measures of effectiveness. Speed usually is not considered.

TAMC uses two level of service estimation techniques: the v/c ratio method and the Florida level of service method. Both techniques are applied for RTPs, major investment studies, CMPs, growth management programs, intermodal planning studies, and site impact analyses. Although TAMC uses both estimation methods for urban and rural areas, it concentrates the v/c ratio method for uninterrupted facilities and the Florida level of service method for interrupted facilities.

The Florida level of service method, in addition to the HCM method, also is used to estimate capacity in Monterey County. Both methods are applied to the same planning applications, in addition to TIPs, as are the level of service estimation techniques. The HCM method is not used for interrupted facilities, leaving this task to the Florida level of service method.

These planning applications also are evaluated for free-flow speed using posted speed limit and design speed. The former is applied to interrupted facilities, whereas the latter is strictly for uninterrupted facilities. In a similar manner, congested speed is estimated using the Florida level of service method and the HCM method. Again, all of the previously mentioned planning applications are evaluated using these two methods. The HCM method is used for uninterrupted facilities, whereas the Florida level of service method is used for interrupted facilities.

TAMC states that it can easily obtain all but two of the data items required to implement the various estimation techniques for its planning applications. The two exceptions are peak-hour travel time/speed and off-peak travel time/speed.

One of the reasons TAMC is profiled for the state of California is because the agency recently conducted a study on the use of CMPs among the state's 32 congestion management agencies (CMAs). The unofficial results of the survey are as follows.

- Every CMA in California currently is using or developing a travel demand model for land use impact analyses. The majority of these models reflect peak-hour conditions, whereas the remaining models are based on daily travel demand. California CMAs limit their analyses to arterial level of service because the models currently in use are limited in their ability to produce accurate turning movements. Approximately half of the CMAs conduct intersection level of service studies as part of their monitoring programs.
- Approximately one-third of the CMAs conducting arterial level of service studies based on model forecasts use the arterial methodology developed by the Florida DOT. The California Department of Transportation (Caltrans) has certified the use of the FDOT Generalized Tables for interrupted flow facilities, and some Caltrans districts recommend that local agencies and consultants use this estimation method. The Riverside CMA currently is in the process of developing similar generalized tables for the state of California based on the Florida model.

2.4.2.6 Rapid City, South Dakota, MPO

In Rapid City, South Dakota, the approach to estimating level of service combines travel time studies with HCM methods. Rather than calculate level of service using the HCM method, with extensive data collection and intersection counts to calculate the average travel speed, speed is collected directly and then compared with the level of service criteria in the HCM.

To determine the level of service of major roadways in the Rapid City MPO, floating car travel time runs were conducted during the PM peak hour on 15 roadways during a two-week period. The data were compiled in a spreadsheet. Arterial classes were identified for each segment analyzed, and the average speed was compared with the speed thresholds from the HCM.

In addition to calculating level of service, travel time is used for traffic modeling calibration and for comparisons of congested roadways throughout the state, as part of the CMS. This approach was used to prioritize the need for improvements based on roadway operations.

2.4.2.7 Ada Planning Association, Boise, Idaho

The Ada Planning Association, in Boise, Idaho, is responsible for TIPs, AQCSs, and site impact analyses. All three measures of effectiveness are used: speed, v/c ratio, and level of service.

For level of service analyses, v/c ratios and service volume are used almost exclusively for all applications, except that the HCM method is used for some site impact analyses. Customized speed and capacity tables are used to estimate free-flow speed and roadway capacity. Roadways are divided into 19 facility types and 8 area types. Capacity ranges from 1,800 vehicles per hour (vph) per lane on high occupancy vehicle (HOV) lanes to 400 vph per lane on substandard minor arterials and collectors in special constrained areas. Free-flow speed that appears to be based on posted speed limit ranges from 65 mph on rural interstates to 15 mph on local streets in the central business district and urban areas. Depending on the number of lanes, the base capacity per lane and base free-flow speed are adjusted for two-way facilities.

For speed on congested roadways, the Ada Planning Association uses the default BPR speed-flow curves for all applications.

2.4.3 Local Agencies

2.4.3.1 City of Garland, Texas

The Garland Transportation Department is primarily responsible for two planning functions: TIPs and site impact analyses. Capacity and service volume are used as measures of effectiveness for site impact analyses, but only service volume is measured for TIPs. Because Garland is a suburb of Dallas, no rural or uninterrupted facilities are considered in estimating level of service.

The Garland Transportation Department estimates level of service using the North Central Texas (Dallas area) COG table, which shows the average daily traffic volume ranges for various levels of service, or what the department calls "qualities of flow." The volume ranges are based on a combination of v/c ratios and service volume. Several highway classes are identified by function, divided or undivided, and number of lanes. Level of service letter grades have been replaced with more general equivalents of "good flow," "tolerable flow," and "undesirable flow." Level of service E is considered capacity and falls under "undesirable flow."

The North Central Texas COG also provides capacity tables, which are used to estimate roadway capacity in Garland. Capacity basically comes from the UTPS default capacity tables. Tables are provided for both hourly and daily service volumes, by divided and undivided roadways. Service volume corresponds to level of service E, representing capacity.

Service volume depends on function class and area type. The determination of area type in Garland is based on the "demographic intensity" (DI), which is calculated from population density and employment density as shown in the following:

Area Type	*Determination*
Central Business District	$DI \geq 125$
Fringe	$18 \leq DI < 125$
Urban Residential	$7.5 \leq DI < 18$
Suburban Residential	$1.8 \leq DI < 7.5$
Rural	$DI < 1.8$

where:

DI = demographic intensity = $PD + 1.91 * ED$
PD = population density in persons per acre
ED = employment density in employees per acre.

Free-flow speed is estimated using posted speed limit, design speed, or both. The Garland Transportation Department only estimates free-flow speed for TIPs. It does not estimate congested speed on any of its roadways.

The Garland Transportation Department obtains most of the data it needs for completing any of the various estimation techniques, even though the department does not use many of these techniques. There are five data items the city does not maintain on file but should have no problem obtaining if the need arises:

- Percentage of trucks, buses, and recreational vehicles
- Peak-hour travel time or speed
- Off-peak travel time or speed
- Grades and curvature
- Signal spacing.

2.4.3.2 Orlando, Florida

The Orlando Planning and Development Department is responsible for the city's TIP, CMSs, AQCSs, intermodal planning analyses, site impact analyses, and growth management programs. These six planning applications have three measures of effectiveness in common: speed, service volume, and capacity. Because roadway geometry and signal input data are required for measuring these three variables, the department maintains a close working relationship with the Orlando Public Works Department, from which this information is obtained.

The Orlando Planning and Development Department uses two techniques for estimating level of service of interrupted and uninterrupted facilities in the urban area. Because of the city's large size, no rural facilities are considered. The two techniques are the Florida Standard Urban Transportation Modeling Structure (FSUTMS) and the HCM 1994 software.

The six planning applications listed previously also incorporate these two level of service estimation techniques. However, two applications—congestion management and growth management—also require the use of a trip allocation program. This program converts projected urban population and employment growth into vehicle trips. Each traffic zone can then be monitored to ensure that it does not exceed preset maximum trip generations. Incidentally, the Orlando Planning and Development Department does not apply the v/c ratio level of service estimation technique to any of its planning applications.

The department estimates roadway capacity in a manner similar to the way it estimates level of service. However, there are a few differences. Only interrupted, state arterial roadways are analyzed using the HCM 1994 software. For other interrupted facilities, FSUTMS is the only technique used. Although the default FSUTMS capacity tables normally are used, some detailed capacity evaluation tables occasionally may be needed. This usually occurs only when the department conducts CMPs or site impact analyses for urban interrupted facilities.

Free-flow speed also is estimated using HCM 1994 software and FSUTMS. The use of default versus customized speed tables for estimating free-flow speed is the same as for estimating capacity. Design speed and posted speed limit are not used as proxies for estimating free-flow speed, and the HCM method is only applied to interrupted facilities for CMPs, growth management programs, intermodal planning studies, and site impact analyses.

The Orlando Planning and Development Department uses FSUTMS and HCM 1994 software to estimate speed on congested roadways for the six planning applications. Again, the HCM software is not incorporated for uninterrupted facilities, unlike FSUTMS. In addition, certain site impact analyses may require field measurements, which are conducted by the department.

The Orlando Planning and Development Department obtains the following traffic data as part of its regular data collection effort:

- 24-hour link traffic count data
- Peak-hour link traffic count data
- Link volume directional distribution
- Number of midblock lanes
- Signal spacing
- Signal cycle length
- Signal green time.

All other forms of traffic data can be obtained by the department if the need arises.

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CHAPTER 3

CURRENT SPEED ESTIMATION TECHNIQUES

Speed estimation techniques are used in every step of the planning process at the state and regional levels to predict future destination choice, mode choice, and route choice. Speed is used to compute vehicle hours traveled, delay, and air pollutant emissions. These computations are used to evaluate alternatives, develop regional transportation plans (RTPs) plus supporting documents, and establish conformity of these documents with the state implementation plan for achieving federal air quality standards.

Speed estimation techniques are used at the local level to evaluate the impact of new development on the level of service of interrupted flow road facilities. The *Highway Capacity Manual* (HCM) is the single most frequently used source of speed estimation methods used by planners for every purpose except for developing RTPs. The Bureau of Public Roads (BPR) curve and related volume/capacity (v/c) ratio techniques are the most frequently used techniques for the preparation of RTPs because they are simple to use in transportation planning models.

This chapter describes the current techniques used by planning agencies to predict the average travel speed of highways. It begins by defining the meaning of average travel speed. The next three sections describe v/c ratio–based techniques, HCM techniques, and HCM-based techniques for predicting speed. The techniques are described in terms of their typical usage for planning analyses, input data requirements, equations and procedures, accuracy, and user criticisms.

3.1 DEFINITION OF SPEED

This section defines various measuring and averaging techniques for computing vehicle speed.

3.1.1 Objectives of Speed Estimation

The objective of estimating vehicle speed is to compute vehicle hours of travel, delay, vehicular air pollutant emissions, level of service, and costs (monetary and time) of traveling by vehicle. The average trip speed is needed to determine the pollutant emission rate of the motor vehicle. The trip length and amount of delay are necessary for computing vehicle operating costs and the amount of time invested by those who travel by motor vehicle.

3.1.2 Design, Operating, and Running Speed

Design speed is "the maximum safe speed that can be maintained over a specified section of highway when conditions are so favorable that the design features of the highway govern" (*1*). Operating speed is "the highest overall speed at which a driver can travel on a given highway under favorable weather conditions and under prevailing traffic conditions without at any time exceeding the safe speed as determined by the design speed on a section by section basis" (*2*). Operating speed is always equal to or less than design speed.

Average travel speed is the length of a segment of road divided by the average travel time of vehicles traversing that segment. This speed, which includes stopped time, also is the "space mean speed" (*3*). Average running speed is the length of a road segment divided by the average running time of vehicles traversing the segment. Running time excludes all stopped time. Running speed is always equal to or greater than average travel speed. Running speed equals average travel speed if there are no stops.

Free-flow speed is the average travel speed at which a single vehicle traverses a segment of road if no other vehicles are present on that segment (there might be vehicles on the side streets). This speed is defined as the length of the segment divided by the time to traverse the segment. Congested speed is the average travel speed at which vehicles traverse a segment of road, when more than one vehicle is present on the segment. This speed is defined as the length of the segment divided by the average travel time to traverse the segment.

Space mean speed is the average speed of all vehicles on a given segment of road (hence the term "space") at a particular time. Aerial photos and floating cars measure space mean speed. Time mean speed is the average speed of all vehicles passing a given point during a certain period. Loops and radar guns measure time mean speed.

Wardrop (*4*) developed the following equation for calculating time mean speed from space mean speed:

$$\mu_{TMS} = \mu_{SMS} + \frac{\sigma_{SMS}^2}{\mu_{SMS}} \tag{3-1}$$

Recognizing that this equation is a quadratic equation for μ_{SMS}, we can solve as follows for μ_{SMS}:

$$\mu_{SMS} = \frac{\mu_{TMS} + \sqrt{\mu_{TMS}^2 - 4\sigma_{SMS}^2}}{2} \qquad (3\text{-}2)$$

Note that a quadratic equation normally has two solutions. We have rejected the lower value solution, which would only apply at very low space mean speeds ($\mu_{SMS} < \sqrt{\sigma_{SMS}^2}$)(see Figure 3-1).

3.1.3 Computation of Individual Vehicle Speed

The average vehicle speed over the length of a trip is equal to the length of the trip divided by the total travel time. If we did not know the entire length of the trip or the travel time, we could obtain this information if we knew the length of each segment of the trip and the average speed of travel for each segment. The harmonic mean of the speed for each segment would give us the average travel speed over the entire trip.

$$s = \frac{D}{\sum_i \frac{d_i}{s_i}} \qquad (3\text{-}3)$$

where:

s = average travel speed for the trip
s_i = average travel speed on segment, i
D = total length of trip
d_i = length of segment, i.

3.1.4 Computation of Mean Speed

Because speed is a rate (distance divided by time), a simple average of the speeds of vehicles doesn't give the true mean speed of all vehicles. There are two techniques for computing the mean speed of vehicles: harmonic mean and arithmetic mean.

Space mean speed is the harmonic mean of the speeds of many vehicles. The total distance traveled times the number of vehicles, divided by the space mean speed gives total vehicle hours traveled.

$$s_{SMS} = \frac{Nd}{\sum_i \frac{d}{s_i}} = \frac{N}{\sum_i \frac{1}{s_i}} \qquad (3\text{-}4)$$

where:

s_{SMS} = average travel speed (space mean speed) for the segment
s_i = average travel speed for vehicle, i
N = number of vehicles
d = length of the segment.

The time mean speed, which is an arithmetic average of the speeds of many vehicles, is a biased estimator of average speed, because more high-speed vehicles than slow-speed vehicles will pass a given point during a fixed time period. Consequently, high-speed vehicles are given disproportionate weight in the average.

$$s_{TMS} = \frac{\sum_i s_i}{N} \qquad (3\text{-}5)$$

where:

s_{TMS} = time mean speed (arithmetic mean speed) for the segment
s_i = average travel speed for vehicle, i
N = number of vehicles.

If there is no difference in the speeds of the vehicles in the sample, the harmonic and arithmetic means are identical. The greater the variance in speeds between vehicles, the greater the difference between the harmonic and arithmetic mean speeds.

The differences between the two methods of averaging speeds can be as little as 1 mph on an uncongested express-

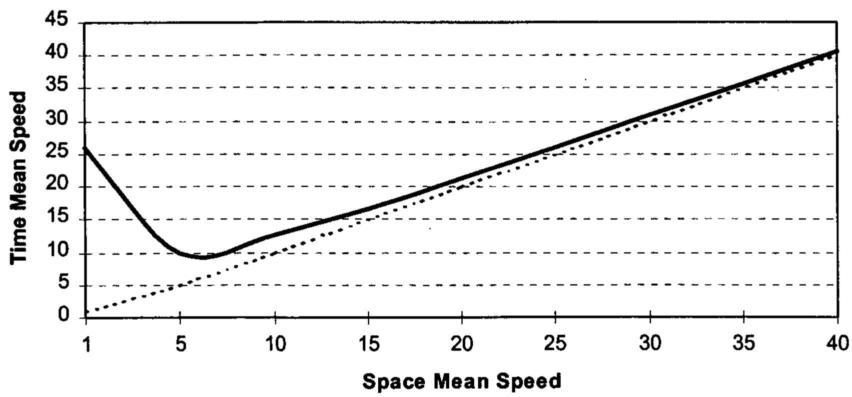

Figure 3-1. Space mean versus time mean speed.

way (*4*) or as great as 7 mph to 10 mph on a signalized arterial or congested freeway (*5*).

3.1.5 Sampling Techniques

Measuring the average speed of traffic on a street segment during a specified time period presents a three-dimensional measurement problem: vehicles, space, and time (see Figure 3-2). The available measuring techniques can measure two of the dimensions thoroughly, but only a small sample of the third dimension.

Loops and radar guns can sample a large number of vehicles for long time periods, but they are restricted to specific spots on the road segment. Aerial photos can sample a large number of vehicles over the entire length of the segment, but they are limited to just a few seconds of the entire desired time period. Floating cars can sample over the entire length of the segment for long time periods, but they are limited to just a few vehicles.

The remainder of this chapter discusses the various techniques for estimating average speed. Our goal in all cases is to estimate the harmonic mean or space mean speed of vehicles on the facility so that the average speed can be multiplied by the length of the facility to obtain total vehicle hours of travel.

3.2 VOLUME/CAPACITY RATIO–BASED METHODS

Volume/capacity ratio–based methods consist of the BPR curve and its variations. These methods predict speed based on three pieces of information: free-flow speed, capacity, and volume.

The standard BPR curve was developed in the late 1960s by BPR (predecessor to the Federal Highway Administration (FHWA)) by fitting a polynomial equation to the freeway speed-flow curves in the 1965 HCM.

Various metropolitan planning organizations (MPOs) have sought to improve and update the original formulation of the BPR curve. This has resulted in numerous variations in the BPR curve throughout the United States.

3.2.1 Typical Usage

The simplicity of v/c ratio curves has facilitated their use in regional travel forecasting models throughout the world. The BPR curve and its variations are the single most frequently used technique for developing RTPs.

The popularity of the BPR curve and its variations is a result of their simplicity. Traffic forecasting models frequently used in RTP analyses must be able to analyze between 5,000 and 10,000 links in each model run. Processing time is reduced by using a simple equation rather than a complex procedure to predict speed. In addition, the simple data requirements of the BPR curve and its variations facilitate data entry for planners.

Regional traffic forecasting models generally require that travel time be a monotonically increasing function of volume. This ensures that the model will be able to find a single user equilibrium solution to the traffic assignment problem. The BPR curve is differentiable, which makes its easier to develop efficient routines for finding the equilibrium solution.

The BPR curve and its variations, however, are rarely used outside the regional transportation modeling environment because their accuracy is inferior to that obtained from more sophisticated speed forecasting techniques.

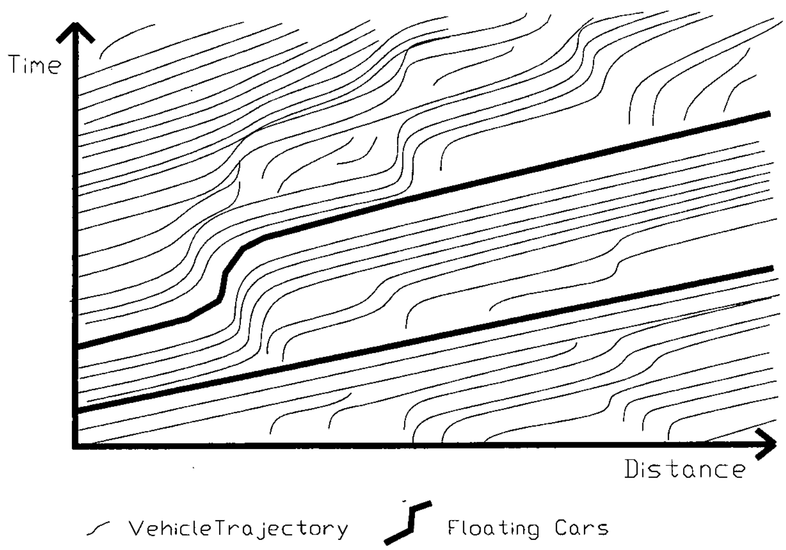

Figure 3-2. The three dimensions of speed averaging.

3.2.2 Input Requirements

The v/c ratio methods require the following as input:

- Free-flow speed
- Capacity
- Volume.

Planners typically use look-up tables based on area type and facility type to assist them in coding free-flow speed and capacity data. These look-up tables allow planners to use simple road maps and aerial photos to code the free-flow speed and capacity information for 5,000 to 10,000 links in a region.

A common error of practitioners is to overlook the fact that "capacity" in the standard BPR curve is actually "practical capacity," which is closer to 80 percent of the actual capacity of the facility. Tables 3-1 and 3-2 are look-up tables for practical capacity and free-flow speed, which were developed by FHWA for use with the BPR curve.

3.2.3 Description

The standard BPR equation follows:

$$s = \frac{s_f}{1 + a(v/c)^b} \qquad (3\text{-}6)$$

where:

s = predicted mean speed
s_f = free-flow speed
v = volume
c = practical capacity
a = 0.15
b = 4.

Practical capacity is defined in this equation as 80 percent of the capacity. Free-flow speed is defined as 1.15 times the speed at the practical capacity.

The parameter, a, determines the ratio of free-flow speed to the speed at capacity. The parameter, b, determines how abruptly the curve drops from the free-flow speed. A high value of b causes speed to be insensitive to v/c until the v/c ratio gets close to 1.0, then the speed drops abruptly (see Figure 3-3).

3.2.4 Variations of the BPR Curve

Many MPOs have been concerned about inaccuracies in the speeds estimated by the standard BPR curve. These MPOs have updated the basic BPR curve based on more recent data in the 1985 HCM or locally collected speed-flow data. The updated BPR curves have a parameters that vary from 0.1 to 1.0 and b (power) parameters that vary from 4 to 11.

One area, Dallas-Forth Worth, uses an exponential equation instead of the standard polynomial form. Others have been concerned about the very low speeds predicted at extremely high v/c ratios. These agencies use an updated version of the BPR curve for v/c ratios less than a certain limit (usually between 1.33 and 2.00) and use a completely different equation for higher v/c ratios. These "split" equations are designed to expedite the rate of closure for the traffic assignment algorithm. Extremely low speeds at high v/c ratios tend to make traffic assignment results fluctuate wildly between iterations.

The following sections describe four of these many adaptations of the basic BPR curve (*8*).

3.2.4.1 San Francisco

The San Francisco Metropolitan Transportation Commission uses a BPR-type curve for both freeways and arterials, but with parameter a equal to 0.45 and b equal to 4. This BPR curve is applied universally to all facility types and v/c ratios.

$$s = \frac{s_f}{1 + 0.45(v/c)^4} \qquad (3\text{-}7)$$

This form was selected based on a statistical analysis of floating car runs made at 119 freeway locations by the California Department of Transportation (Caltrans).

3.2.4.2 Detroit

Detroit uses the standard BPR curve for v/c ratios of 1.85 and less. For higher v/c ratios, the city sets a minimum speed

TABLE 3-1 Practical capacity look-up table for BPR curve (*6*)

One-Way Level of Service "C" Vehicles Per Lane Per Hour (VPH)						
	Freeway	Express-way	2-Way Arterial (Parking)	One-Way Arterial (Parking)	Centroid Connector	2-Way Arterial (No Park)
CBD	1750	800	600	700	10,000	600
Fringe	1750	1000	550	550	10,000	800
Outer CBD	1750	1000	550	650	10,000	800
Rural/ Residential	1750	1100	550	900	10,000	800

TABLE 3-2 Free-flow speed look-up table for BPR curve (7)

Free-Flow Speeds (MPH)						
	Freeway	Expressway	2-Way Arterial (Parking)	One-Way Arterial (Parking)	Centroid Connector	2-Way Arterial (No Park)
CBD	48	37	22	22	10	22
Fringe	48	44	25	29	15	25
Outer CBD	58	37	22	24	15	22
Rural/ Residential	67	47	28	32	15	28

at about one-third the free-flow speed. A separate equation is used for severe congestion. The equations are as follows:

$$s = \frac{s_f}{1 + 0.15(v/c)^4} \tag{3-8}$$

if $v/c \leq 1.85$, else

$$s = \frac{s_f}{2.857} \tag{3-9}$$

3.2.4.3 Phoenix

Phoenix fitted different speed-flow curves for arterials and freeways, and it breaks each curve at a v/c ratio of 1.33. A flatter curve is used for v/c ratios greater than 1.33. The equations are as follows.

For freeways:

$$s = \frac{s_f}{1 + 0.1225(v/c)^8} \tag{3-10}$$

if $v/c \leq 1.33$, else

$$s = s_f * [0.25 + 0.4374(v/c)^{-3}] \tag{3-11}$$

For arterials:

$$s = \frac{s_f}{1 + 0.1513(v/c)^7} \tag{3-12}$$

if $v/c \leq 1.33$, else

$$s = s_f * [0.25 + 0.5184(v/c)^{-3}] \tag{3-13}$$

3.2.4.4 Dallas-Fort Worth

The Dallas-Fort Worth area developed an exponential speed-flow equation with a single parameter, b, which is varied for peak-hour and daily traffic forecasts. Peak-hour model traffic assignments use a higher value of b. The equation is as follows:

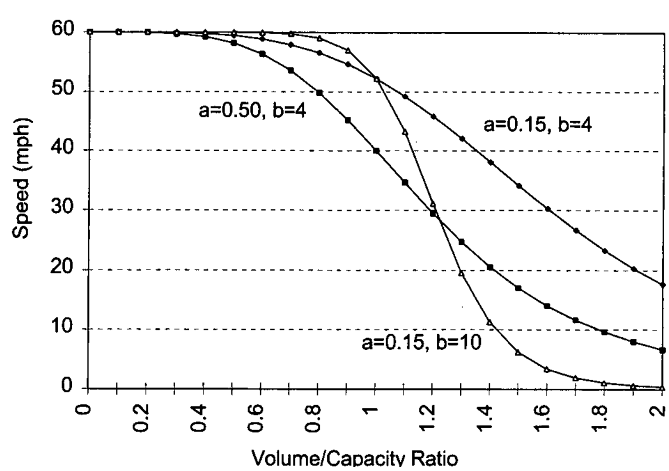

Figure 3-3. Plot of BPR curve and several variations.

$$s = \frac{s_f}{1 + 0.015 \exp(b * v/c)} \quad (3\text{-}14)$$

where $b = 4.0$ to 6.0 depending on the time period covered by the traffic assignment.

3.2.5 Accuracy

Figure 3-4 shows how the standard BPR curve (with a parameter of 0.15 and a power of 4, using practical capacity) compares with 15-min volume and speed data gathered during a 24-hr period on Interstate 8 in San Diego. The data were gathered January 25, 1994, for the westbound direction, upstream of the College Avenue on-ramp. There were a total of 96 data points. Capacity was estimated at 2,300 vehicles per hour (vph) per lane for the purpose of converting measured volumes to v/c ratios. The speed and volume data were obtained from the loop sensors. The loop speed data were not corrected for the difference between time mean speed and space mean speed.

The standard BPR curve fits the freeway data quite well, except for the half dozen data points that reflect congested conditions. These points are plotted in terms of the capacity of the section where the volumes were measured, but the actual capacity has been reduced in this section by a downstream bottleneck (probably caused by an increase in on-ramp volume). The actual "through" capacity of the section has been reduced to the measured flow rate of cars queued in the section.

The apparent dip in speeds at extremely low v/c ratios in Figure 3-4 is an artifact of the single loop speed measurement technique and should be ignored. The conversion of occupancy time into speed for single loop detectors requires an assumption about the length of the vehicle crossing the loop. The extremely low volumes usually occur in the early morning hours, when trucks represent a higher proportion of the vehicle stream. Thus, the use of the same assumed average vehicle length during the 24-hr period results in an apparent dip in speeds in the early morning hours.

Figure 3-5 shows the standard BPR curve plotted against speed data gathered for an urban arterial. The data are 15-min speed and flow data gathered at loop detectors on the approaches to four signalized intersections on Ventura Boulevard. The data were gathered on December 7, 1993, from 7 a.m. to 11 a.m. in the westbound and eastbound directions. A total of 64 data points were obtained. The loop speeds were not corrected for the difference between time mean speed and space mean speed. Capacity was estimated assuming 1,600 vehicles per lane per hour of green. The green time per cycle (g/C) ratio was computed for the fixed time signals from signal timing data. Each data point represents a single one-direction segment of the street between a pair of traffic signals. The v/c ratio for each segment is the approach v/c ratio for the downstream intersection.

The BPR curve plotted in Figure 3-5 is for a 35-mph free-flow speed. A better fit could have been obtained with a slightly lower assumed free-flow speed. However, arterial speeds show a much wider dispersion for a given v/c ratio than freeway speeds. This is because other factors, such as signal timing, have as great an impact on vehicle speeds as the v/c ratio.

These two example data sets do not address the issue of predicting speeds for v/c ratios that exceed 1. Chapter 12 provides more discussion on the accuracy of the technique for these v/c ratios.

3.2.6 User Critique

The planning community generally has recognized the need to update the standard BPR curve in light of the publication of the 1985 and 1994 HCMs. Planners also have recognized that the BPR curve should be calibrated to local conditions when resources permit. The result has been that there are as many variations of the BPR curve as there are urban areas in the United States.

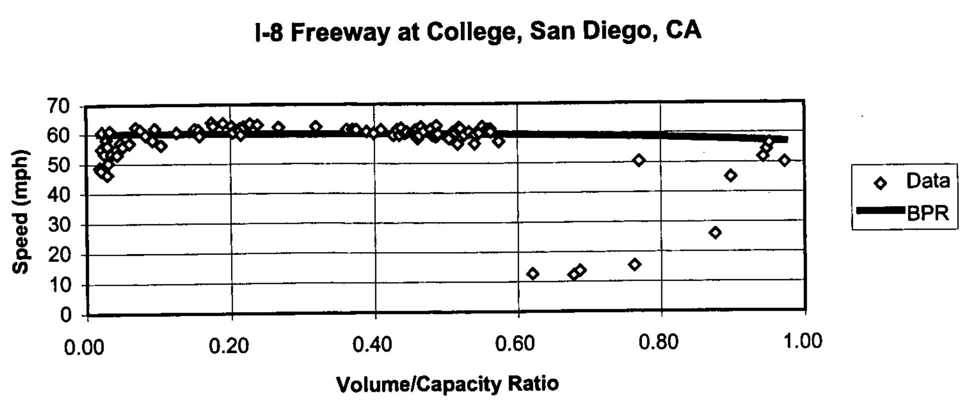

Figure 3-4. Standard BPR curve versus freeway speed data.

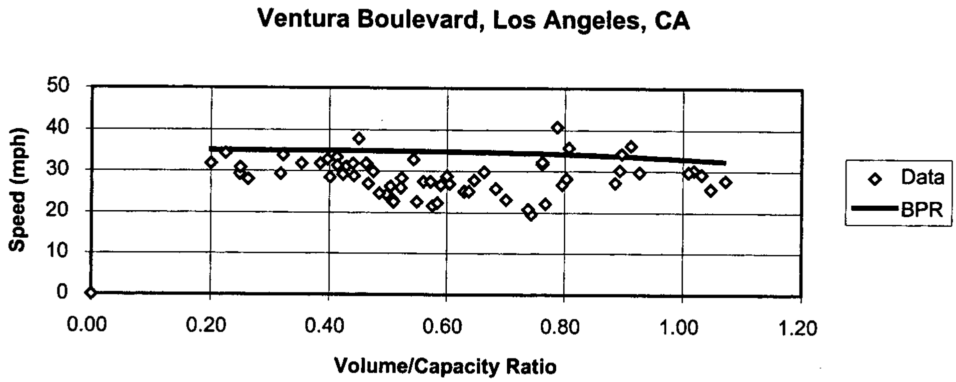

Figure 3-5. Standard BPR curve versus arterial speed data.

Some agencies have been concerned about the low speeds predicted by the standard BPR curve at extremely high v/c ratios. These agencies have adopted split forms of the equation to apply at different v/c ranges. Nevertheless, v/c ratio techniques are extremely popular for use in regional traffic forecasting models because of their simplicity. The BPR curve and its variations require relatively little data and fit the traffic model requirements for a continuous, monotonically increasing function of volume.

Speed forecasting was not a goal of the traffic forecasting process until the passage of ISTEA and CAAA; thus, the inaccuracies of the BPR curve had not been a serious concern to traffic forecasters. Now that models must produce both volume and speed forecasts, modelers are focusing more on the speed forecasting process. Postprocessors such as DTIM (*9*) have been developed to recompute the model speed estimates using more elaborate procedures than the BPR curve.

Any replacement or enhancement to the BPR curve and its variations needs to maintain the simplicity of a single differentiable, monotonically increasing function in order to be useful in traffic forecasting models.

3.3 HIGHWAY CAPACITY MANUAL METHODS

This section describes the planning methods available in the 1994 HCM for estimating congested speeds.

3.3.1 Typical Usage

The HCM is the single most frequently used source of techniques for predicting speeds for all planning purposes except regional traffic forecasting. The complexity of its procedures prohibits their use in regional traffic models.

HCM methods are the user-preferred speed estimation methods for major investment studies, congestion management programs, growth management programs, air quality conformity, highway performance monitoring systems, and site impact studies. State departments of transportation (DOTs) and private consultants show the greatest preference for using these techniques to estimate speeds.

Planners rarely have available to them all the data required by the HCM techniques; therefore, they frequently use HCM-recommended (or locally determined) default values for much of the required input data. Florida DOT has codified and standardized the selection of default values for all planning agencies in the state.

HCM procedures vary by facility type; therefore, the remainder of the discussion is organized by facility type.

3.3.2 Uninterrupted Flow Facilities

The HCM provides procedures for three uninterrupted flow facilities: freeways, multilane highways, and two-lane rural roads. The procedures for freeways and multilane highways are similar, and there is some discussion about merging the two techniques in the next edition of the HCM.

3.3.2.1 Input Requirements

The HCM techniques for uninterrupted flow facilities require the following basic input data for the facility:

- Hourly volume
- Number of lanes
- Free-flow speed.

These data items are the same as the input requirements of the BPR curve. The freeway and multilane highway techniques, however, also require the following data:

- Peak-hour factor
- Lane and shoulder widths
- Percent trucks
- Percent recreational vehicles
- Terrain type
- Predominant driver type.

The two-lane rural road technique requires these data, with the exception of driver population type. This technique, however, does require the directional distribution of traffic on the facility.

The additional data required by the HCM techniques impose an added burden on planners, but it also allows the planner greater flexibility in calibrating the techniques to local conditions. The planning application sections of each chapter of the HCM provide an equation for converting average annual daily traffic to design hour volume (see pages 3-21 and 7-19 of the 1994 HCM). However, no additional guidance is provided for substituting defaults for some of the other data requirements of these techniques. The design hour volume equation is as follows:

$$DHV = AADT \times K \times D \tag{3-15}$$

where:

DHV = design hour volume;
$AADT$ = average annual daily traffic (total of both directions);
K = proportion of two-way design hour volume to AADT (typically 8 percent to 20 percent); and
D = proportion of two-way traffic during design hour that flows in peak direction (typically 52 percent to 80 percent).

Although the design hour volume in some design applications has been the 30th highest hour volume of the year, this is not necessarily the case for planning applications. The design hour for planning purposes could be the 100th highest hour.

3.3.2.2 Description

Although some readers may find that the following descriptions duplicate the material contained in the 1994 HCM, these descriptions are provided from the planning application perspective and show how the techniques would be applied in a planning situation to estimate speed.

3.3.2.2.1 Freeways (Chapters 3, 4, 5, and 6 of the HCM)

The HCM provides separate procedures for computing the average speed of traffic on basic sections, weaving sections, and ramp merge/diverge sections of freeways. Chapter 6 provides a procedure for analyzing a freeway composed of these three section types, but provides no guidance on how the results might be combined to obtain an overall average speed (or level of service) for the entire freeway.

The computation of speeds for weaving sections (Chapter 4) is not feasible for planning applications because it requires knowledge of the lane striping (lane adds and drops) in the weaving section. The methodology is limited to weaving sections under 2,500 ft in length.

The analytical procedure for ramps (Chapter 5) produces speed estimates for only the two right-most lanes of the freeway. The two regression equations for predicting the speed of traffic on the freeway apply only to the immediate vicinity of an on-ramp or off-ramp (1,500 ft upstream of an off-ramp and 1,500 ft downstream of an on-ramp) (see Table 5-4 of the HCM). The equations are applicable only for "stable flow regimes" (level of service better than E and speeds greater than 42 mph).

These equations for ramp merge/diverge areas are not practical for planning purposes because (*a*) they cover a small portion of the freeway (the 1,500-ft ramp influence area) at each ramp merge and diverge area, (*b*) they apply only to speeds greater than 42 mph, and (*c*) the equations do not predict speeds for vehicles outside the right-most two lanes of the freeway.

The average speed on a basic freeway section is computed in two steps.

Step 1: Convert Predicted Hourly Volume to Ideal Volume

The final report of NCHRP Project 3-45 recommends that adjustment factors for width and population be dropped from the following equation.

$$V_{ideal} = \frac{V_{predicted}}{PHF \times f_{width} \times f_{heavyvehicles} \times f_{population}} \tag{3-16}$$

(Equation 3-4 of the HCM)

where:

V_{ideal} = ideal flow rate used to look up speed in Figure 3-2 of the HCM
$V_{predicted}$ = predicted volume (vph)
PHF = peak-hour factor to convert hourly flow rate to equivalent hourly rate for peak 15 min
f_{width} = lane and shoulder width adjustment factors (Table 3-2 of the HCM)
$f_{population}$ = driver aggressiveness adjustment factor (Table 3-7 of the HCM)
= 1.00 for weekday commuter type facility
= 0.75 to 0.99 for recreational and other type of facility
$f_{heavyvehicles}$ = adjustment factor for effect of heavy vehicles, computed using Equation 3-5 of the HCM as follows:

$$f_{heavyvehicles} = \frac{1}{1 + P_{trucks}(E_{trucks} - 1) + P_{rv's}(E_{rv's} - 1)} \tag{3-17}$$

(HCM Equation 3-5)

where:

P_{trucks} = percentage of trucks in traffic stream

TABLE 3-3 Heavy vehicle equivalence factors (Table 3-3 of the HCM)

Adjustment Factor	Type of Terrain		
	Level	Rolling	Mountainous
E_T for trucks and buses	1.5	3.0	6.0
E_R for recreational vehicles	1.2	2.0	4.0

$P_{rv's}$ = percentage of recreational vehicles in traffic stream
E_{trucks} = truck equivalence factor obtained from Table 3-3 of the HCM (Table 3-3 in this report)
$E_{rv's}$ = recreational vehicle equivalence factor from Table 3-3 of the HCM.

Step 2: Look Up Speed

Once the ideal flow rate has been computed, the speed can be obtained by looking up the speed for the given ideal flow rate in Figure 3-2 of the HCM (Figure 3-6). The planner must know the free-flow speed. (NCHRP Project 3-45 provides a recommended procedure for computing free-flow speed, given the lane width, shoulder width, number of lanes, and number of interchanges per mile.) Separate charts are provided for facilities with four and six or more lanes. None of these charts can be used for v/c ratios greater than 1.

According to these figures, volume has no effect on speed until the volume approaches 1,300 passenger car equivalents per hour per lane (about 56 percent of the ideal capacity of a facility with six or more lanes). The free-flow speed is as important as the v/c ratio in determining the speed.

The procedure for basic freeway sections is useful for planning applications because of its wider range of applications; however, it is still limited to v/c ratios less than 1.

3.3.2.2.2 Multilane Highways (Chapter 7 of the HCM)

The procedure for determining the congested speed for multilane highways is similar to that for freeways. The predicted volume must be converted to an ideal flow rate using Equation 7-3 of the HCM.

Two adjustment factors are applied to arrive at the ideal flow rate: the peak-hour factor and the heavy vehicle factor. (The lane width and driver population factors used to adjust the volumes for freeways are used to estimate the free-flow speed for multilane highways.) The computation of the heavy vehicle factor for multilane highways is the same as that for freeways.

Chapter 7 of the HCM provides a procedure for computing the actual free-flow speed, given the median type, lane width, lateral clearance, and number of access points per mile (see Equation 7-1 and Tables 7-2, 7-3, 7-4, and 7-5 of the HCM). Unfortunately, the user must also provide the ideal free-flow speed before these adjustments can be applied to arrive at the actual free-flow speed. (The final report of NCHRP Project 3-45 recommends a similar free-flow speed computation procedure for freeways.)

$$FFS = FFS_I - F_M - F_{LW} - F_{LC} - F_A \qquad (3\text{-}18)$$

where:

FFS = computed free-flow speed (mph),
FFS_I = ideal free-flow speed (mph),
F_M = adjustment factor for median type,
F_{LW} = lane width adjustment,
F_{LC} = lateral clearance adjustment, and
F_A = access points density adjustment.

Page 7-10 of the HCM cites nonreferenced recent research that found that ideal free-flow speed is 5 mph to 7 mph higher than the posted speed limit.

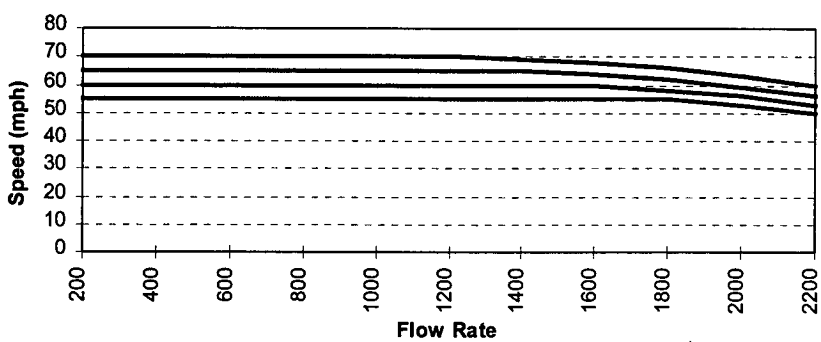

Figure 3-6. Speed-flow curves for four-lane freeway (Figure 3-2 of the HCM).

Once the user has computed the ideal flow rate and the actual free-flow speed, Figure 7-3 of the HCM can be entered to obtain the congested speed. This figure is almost identical to the freeway speed-flow curves for a four-lane freeway, with the exception that the multilane highway figure allows for lower free-flow speeds, down to 45 mph. This figure also does not provide for v/c ratios greater than 1.

3.3.2.2.3 Rural Two-Lane Roads (Chapter 8 of the HCM)

Figure 8-1 of the HCM provides a speed-flow curve that applies to ideal conditions (design speed 60 mph or more, lane widths 12 ft or more, shoulders 6 ft wide or more, passing allowed everywhere, only passenger cars, 50/50 directional split, and level terrain). The user must convert the predicted volume to the equivalent hourly flow rate for the peak 15 min in terms of passenger car equivalents.

The user then divides the equivalent hourly flow rate (in pcu) by the directional distribution adjustment factor (F_d), width adjustment factor (F_w), and heavy vehicle adjustment factor (F_{hv}) to obtain the ideal flow rate for entering Figure 8-1 (Figure 3-7) (see Equation 8-1 of the HCM for details).

Unfortunately, this speed estimation method only applies to facilities with design speeds of 60 mph and volumes less than capacity. No adjustment process is provided for estimating average speeds for facilities with lower design speeds.

3.3.3 Interrupted Flow Facility Techniques

Chapter 11 of the HCM provides one procedure for interrupted flow facilities.

3.3.3.1 Input Requirements

The HCM technique for interrupted flow facilities requires the following basic input data:

- Hourly volume
- Number of lanes
- Free-flow speed.

These data are the same as those required by the BPR curve technique. The HCM technique, however, requires the following additional data:

- Arterial class,
- Density of signals per mile,
- Peak-hour factor,
- Percentage of turning traffic from exclusive lanes,
- Medians,
- Exclusive turn lanes,
- Green time per cycle,
- Cycle length,
- Quality of signal progression, and
- Signal controller type.

The additional data required by the HCM technique impose an added burden on planners unless defaults are used, but these data also give the planner greater flexibility in calibrating the techniques to local conditions.

3.3.3.2 Description

The planning applications section of the HCM describes the steps for estimating average speed for planning purposes. The average travel speed for signalized facilities is computed in six steps:

1. Convert Daily Traffic to Peak Hour
2. Convert Two-Way Peak-hour Volume to Peak Direction Volume
3. Subtract Turning Volumes Made from Exclusive Lanes
4. Compute Arterial Running Time
5. Compute Intersection Approach Total Delay
6. Compute Arterial Average Travel Speed.

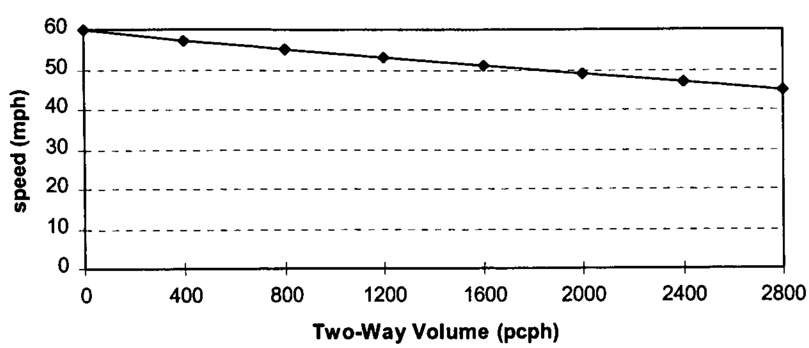

Figure 3-7. Speed-flow curve for two-lane rural roads (Figure 8-1 of the HCM).

Steps 1 through 3: Compute Through Volume

The first three steps consist of converting two-way average daily traffic into a single direction peak-hour volume.

Step 4: Compute Running Time

The running time can be obtained from Table 11-4 of the HCM or the following equations, which have been fitted by the study team to the table:

$$Running\ Time/Mile = \frac{3600}{s_f - A * \exp(B * dist)} \quad (3\text{-}19)$$

$$A = 18 + \frac{s_f - 25}{2.22} \quad (3\text{-}20)$$

$$B = \frac{s_f - 25}{5} - 9 \quad (3\text{-}21)$$

where:

s_f = free-flow speed (mph)
$dist$ = average distance between signals (mi).

Step 5: Compute Intersection Approach Delay (D)

To calculate the arterial or section speed, the individual intersection approach delays are needed. The intersection approach total delay is calculated by applying the following equations:

$$D = 1.3 * (d_u * DF + d_i) \quad (3\text{-}22)$$

$$d_u = (0.38) * C * \frac{[1 - (g/C)]^2}{[1 - (g/C) * \min(X, 1.0)]} \quad (3\text{-}23)$$

$$d_i = 173 * X^2 * \left\{(X - 1) + \sqrt{(X - 1)^2 + m * (X/c)}\right\} \quad (3\text{-}24)$$

where:

d = approach stopped delay, in sec/veh
d_u = approach uniform delay, in sec/veh
d_i = approach incremental delay, in sec/veh
DF = delay adjustment factor (look up in Table 3-4 in this report)
C = cycle length, in sec
g = effective green time for the lane group, in sec
g/C = green ratio for the subject lane group
X = v/c ratio for the subject lane group
= maximum of v/c, 1.00
v = volume per hour
c = capacity for the through lane group
= (capacity per hour of green per hour) * (number of lanes) * (g/C)
m = a calibration term (look up in Table 4-4 of the HCM)
D = approach total delay.

Step 6: Compute Average Travel Speed

The estimated average travel speed in miles per hour is computed as follows:

$$Speed = \frac{[3600 * Length]}{[(RunningTimePerMile) * (Length) + D]} \quad (3\text{-}25)$$

where:

Speed = average travel speed, in mph
Length = length of link, in mi
Running Time per Mile = running time, in sec per mi
D = intersection approach delay for through movements, in sec.

3.3.4 Accuracy

The accuracy of the HCM procedure for predicting average speeds for interrupted flow facilities is evaluated in Chapter 12. Figure 3-8 and Figure 3-9 compare the 1994

TABLE 3-4 Delay adjustment factor (*df*) and incremental delay calibration term (*m*) (*10*)

		Quality of Progression					
	g/C	Very Poor	Unfavorable	Random Arrivals	Favorable	Highly Favorable	Exceptionally Good
Delay Adjustment Factor (DF)	0.20	1.167	1.007	1.000	1.000	0.833	0.750
	0.30	1.286	1.063	1.000	0.986	0.714	0.571
	0.40	1.445	1.136	1.000	0.895	0.555	0.333
	0.50	1.667	1.240	1.000	0.767	0.333	0.000
	0.60	2.001	1.395	1.000	0.576	0.000	0.000
	0.70	2.556	1.653	1.000	0.256	0.000	0.000
Delay Calibration Term (m)		8	12	16	12	8	4

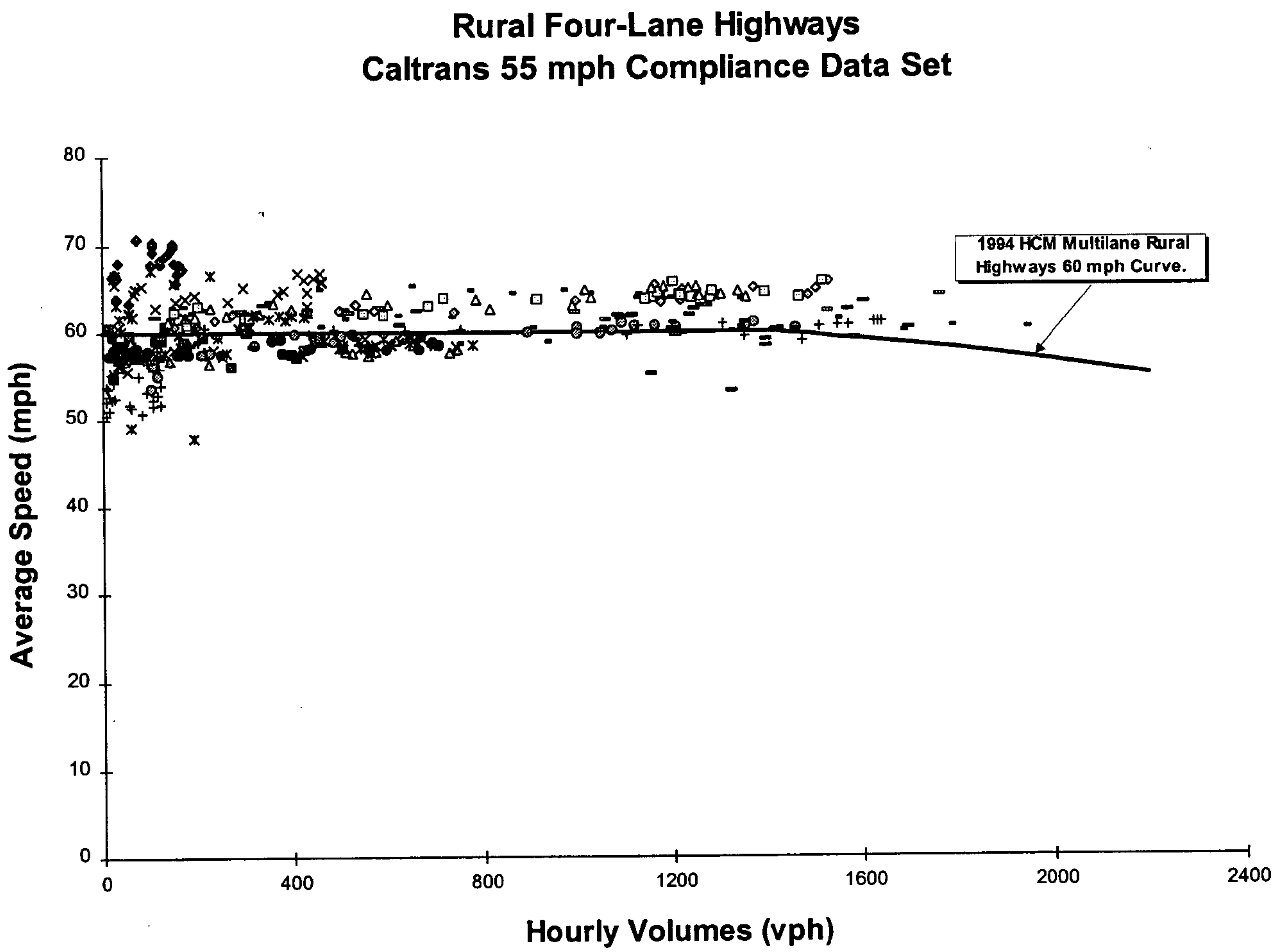

Figure 3-8. 1994 HCM multilane curve versus four-lane rural highway data.

HCM multilane and two-lane rural road speed-flow curves with rural state highway data taken from Caltrans 55 mph speed compliance surveys. The variation in speeds at a given volume is often greater than the variation of the mean speeds for the entire range of volume data in these figures.

Figure 3-10 presents a preliminary comparison of the HCM method with field data gathered for a section of Ventura Boulevard in the city of Los Angeles that is 8.2 mi (13 km) in length. The figure compares the mean speed in each direction with the length of the facility for each of 4 hr. The thick line labeled "Float" in the figure shows the mean speeds obtained from floating car measurements.

This section of Ventura Boulevard is a major divided arterial with left turn pockets at intersections and a two-way left turn lane between intersections. Ventura Boulevard widens from four to six lanes east of Reseda Boulevard. Ventura Boulevard carries between 30,000 and 45,000 vehicle trips per day. The speed limit is 35 mph (58 km/hr).

The HCM predictions of mean travel speed were on the average consistently 4.5 mph (7.2 km/hr) lower than the floating car speed measurements. An adjustment to the HCM predictions that is recommended by Courage et al. (discussed in Chapter 5) compensates for the HCM procedure's tendency to underpredict speeds.

3.3.5 User Critique

Planners' criticisms of HCM techniques generally have focused on the data and procedural requirements. The data requirements can be quite extensive, but defaults can be used for most of the data items. The procedural requirements are not particularly complex, and software can be used to facilitate adherence to these requirements. Florida, in particular, has developed defaults and software that greatly facilitate the use of HCM techniques for planning purposes.

Others have noted that HCM techniques tend to underpredict speeds. West Coast planning agencies tend to rely more on field measurements of speeds than on speeds obtained from

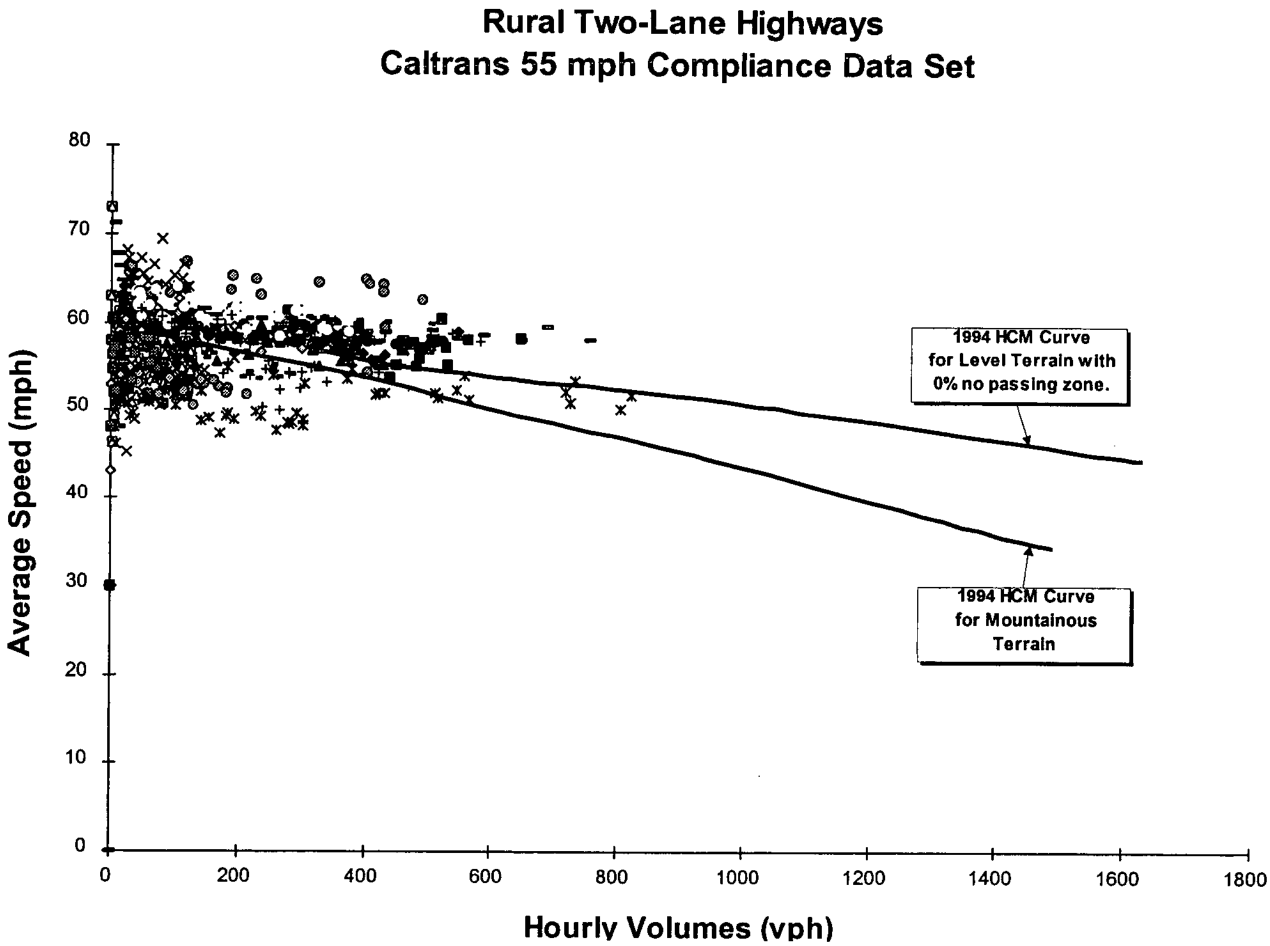

Figure 3-9. 1994 HCM two-lane rural road curve versus speed data.

the HCM method, perhaps because they have greater resources with which to measure speeds in the field. In the case of California, agencies have strong financial incentives to make speed estimates as accurate as possible. Local agencies in the state lose some of their state gas tax revenues if they are unable to demonstrate that speeds exceed a minimum standard.

3.4 OTHER METHODS

Other methods cited by planning agencies for predicting speeds are various traffic operations models such as TRANSYT-7F and NETSIM. These are not realistic tools, however, for any but the most detailed planning analyses.

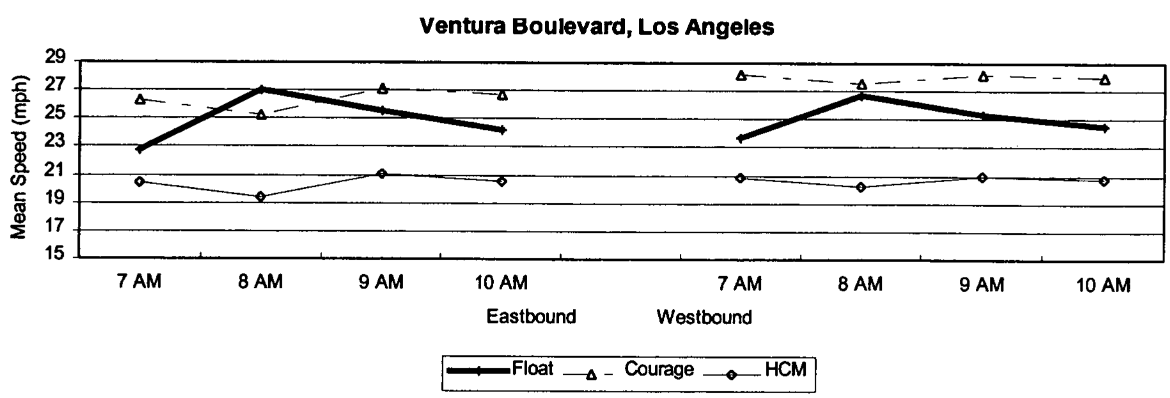

Figure 3-10. Comparison of HCM arterial speed estimates with field data.

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CHAPTER 4

CURRENT LEVEL OF SERVICE AND SERVICE VOLUME TECHNIQUES

The purpose of techniques for calculating maximum service volume is to estimate road facility level of service. The volumes are used either to measure level of service directly (in terms of v/c ratios) or as proxies for other facility performance measures (such as delay) that are believed to be a well-behaved function of volume.

This chapter explores various definitions of road facility level of service and the measures used for determining level of service. Planning methods are described for computing level of service performance measures and maximum service volume. The methods include v/c ratio and those from the HCM and Florida *Level of Service Manual.*

4.1 DEFINITION OF LEVEL OF SERVICE

The level of service concept was created to convey to the general public the general quality of traffic conditions on a road facility. Numerical results that are meaningful only to professionals (volume, v/c ratio, density, and speed) are converted to a letter grade from A to F for use in comparing planning alternatives and project impact.

The 1965 HCM (*1*) defined level of service as a combination of minimum speed and maximum v/c ratio. Cutoff values varied by facility type. Freeway weaving and ramp merge/diverge sections had separate level of service criteria based on maximum flow rates per lane. There were no level of service procedures or criteria for interrupted flow facilities (except at intersections) at that time (see Table 4-1).

Circular 212 (*2*) replaced the maximum v/c ratio criterion for freeways with maximum density and kept the minimum speed criterion. The minimum speeds for levels of service A through C were dropped by 2 mph, to 5 mph. A minimum speed criterion replaced the maximum flow rate criterion for freeway weaving sections. However, there still was no procedure or level of service criterion for interrupted flow facilities (except at the intersection level).

The 1985 HCM (*3*) and its 1994 update abandoned the combined v/c ratio and minimum speed criteria for level of service and replaced these two measures with single traffic performance measures, such as speed, percent time delay, and density (see Table 4-2).

The NCHRP 3-55(4) project to evaluate level of service standards for the year 2000 HCM is considering various modifications to these definitions of level of service, but results are not anticipated until late 1997.

The Florida Department of Transportation (FDOT) adopted the HCM level of service methodologies but substituted performance measures, such as v/c ratio and speed, that are easier for planning agencies to forecast than the HCM criteria (see Table 4-3). In addition, the cutoff levels for each level of service were modified to reflect different traffic conditions and, in some instances, the differing sensitivities to delay of rural and urban residents as they travel on different facility types. In all cases (including urban arterials), the v/c ratio must be less than 1 (1/peak-hour factor (PHF) for arterials) for the level of service to be in the A to E range.

4.2 VOLUME/CAPACITY RATIO METHOD

Level of service and maximum service volume are determined solely by the v/c ratio in this method.

4.2.1 Typical Usage

The v/c ratio method is the most popular method for predicting level of service for regional transportation plan (RTP) analyses, because the v/c ratio is an easy parameter to compute for regional traffic forecasting models. Relatively little data are required, and the results can be quickly computed and plotted for even the largest highway networks with more than 10,000 highway links. The v/c ratio method is used in all planning studies, even site impact studies. However, except for RTP analyses, the v/c ratio method is not used as frequently as HCM methods.

Planning agencies involved in the preparation of RTPs are usually less concerned with the precise level of service than with whether the v/c ratio is greater than 1. Thus, any inaccuracies in determining levels of service better than level E are usually neglected.

4.2.2 Input Requirements

The v/c ratio method requires only the following information:

- Volume
- Capacity per lane
- Number of lanes.

TABLE 4-1 Level of service criteria from 1965 HCM

Level of Service	Freeway[a]		Multi-Lane Highway[b]		Two Lane Highway[c]	
	Max V/c	Min Speed (mph)	Max V/c	Min Speed (mph)	Max V/c	Min Speed (mph)
A	0.35	60	0.30	60	0.20	60
B	0.50	55	0.50	55	0.45	50
C	0.68	50	0.75	45	0.70	40
D	0.81	40	0.90	35	0.85	35
E	1.00	30	1.00	30	1.00	30

[a] Table 9.1, 1965 HCM, assuming 0.90 peak hour factor, 70 mph Design Speed, 4-lane freeway.
[b] Table 10.1, 1965 HCM, assuming 70 mph Design Speed.
[c] Table 10.7, 1965 HCM, 100% passing sight distance greater than 1500 feet.

Capacity per lane is usually determined by means of look-up tables, based on facility type and area type. These tables, in turn, are often derived by using the HCM and assumed default parameters that vary by facility type and area type.

4.2.3 Description

Many planning agencies, when reporting the results of long-range travel forecasts, use simple v/c thresholds for level of service. Typical cutoff levels are as follows:

Level of Service	*Volume/Capacity Ratio Cutoff*
A	Less Than 60%
B	60% to Less Than 70%
C	70% to Less Than 80%
D	80% to Less Than 90%
E	90% to Less Than 100%
F	100% or Greater

4.2.4 Accuracy

To the extent that planning agencies are concerned only with determining whether a facility will be operating at level of service F, the v/c ratio is an accurate indicator of this condition. The v/c ratio, however, is relatively undependable for distinguishing the precise level of service between levels A and E, unless other information about the road facility is known.

The v/c ratio can be used as a proxy for density for determining the level of service for freeways and multilane highways. However, the planner needs to distinguish between four-lane and six-lane freeways and must know the free-flow speed in order to use Tables 3-1 and 7-1 of the HCM to determine level of service.

The v/c ratio also can be used as a proxy for level of service for two-lane rural roads if the planner knows the terrain type and the percentage of no-passing zones. Table 8-1 of the HCM can be used to convert v/c ratio to level of service with these additional data. The v/c ratio is a poor indicator of level of service for interrupted flow facilities unless it is supplemented with additional information on the signal timing characteristics of the facility.

The accuracy of the v/c ratio method is reported in Chapter 11 of this research report, in the section Standard BPR Technique.

4.2.5 User Critiques

Regional traffic modelers find that v/c ratios are a particularly convenient measure for reporting large numbers of highway links. However, modelers can cause a great deal of confusion for the public by reporting their model results in terms of letter grades rather than strictly in v/c ratios. The

TABLE 4-2 Level of service criteria from 1994 HCM

	Freeway[a]	Multi-Lane Highway[b]	Two Lane Highway[c]	Urban Arterial[d] (Class I)
Level of Service	Max Density (pc/mi/ln)	Max. Density (pc/mi/ln)	Max. % Time Delay	Min Speed (mph)
A	10	12	30%	35
B	16	20	45%	28
C	24	28	60%	22
D	32	34	75%	17
E	36.7	40	100%	13

[a] Table 3-1, 1994 HCM, assuming 4-lane freeway, 70 mph Design Speed.
[b] Table 7-1, 1994 HCM, assuming 70 mph Design Speed.
[c] Table 8-1, 1994 HCM.
[d] Table 11-1, 1994 HCM, Class I Arterial.

TABLE 4-3 Level of service criteria from Florida *Level of Service Manual* (standards shown for urban and rural)

Level of Service	Freeway[a] Maximum V/c	Multi-Lane Highway[b] Maximum V/c	Two Lane Highway[c] Maximum V/c	Urban and Rural Arterials[d] Minimum Speed (mph)
A	0.272 / 0.318	0.30 / 0.33	0.12	35 / 42
B	0.436 / 0.509	0.50 / 0.55	0.24	28 / 34
C	0.655 / 0.747	0.70 / 0.75	0.39	22 / 27
D	0.829 / 0.916	0.84 / 0.89	0.62	17 / 21
E	1.000	1.00	1.00	13 / 16

[a] Tables 5-1 and 5-3, Florida LOS Manual, assuming 4-lane freeway.
[b] Tables 5-1 and 5-3, Florida LOS Manual.
[c] Table 5-3, Florida LOS Manual, rural undeveloped area (no criteria for urban areas).
[d] Tables 5-1 and 5-3, Florida LOS Manual, Class I Arterial.

public will question why a site impact study determined the level of service to be one letter grade when the regional model determined the level of service to be a different letter grade. The result is a lack of public confidence in the regional traffic forecasting model.

4.3 HIGHWAY CAPACITY MANUAL METHODS

The HCM provides separate methods for computing level of service and service volume for freeways, multilane highways, rural two-lane roads, and urban signalized arterials.

Most planning agencies and private consultants prefer to use the HCM methods to estimate level of service for most of their planning applications. Metropolitan planning organizations (MPOs), however, prefer to use the v/c ratio method rather than HCM methods to estimate level of service for their regional transportation planning studies. HCM methods are predominantly used in site impact studies and congestion management plans.

4.3.1 Freeways (Chapters 3, 4, 5, and 6 of the HCM)

Chapter 6 of the HCM provides a procedure for analyzing the entire length of a freeway. The freeway is split into basic sections, weaving sections, and ramp merge/diverge sections. Each section is analyzed separately, and the results are tabulated and displayed separately for each section. No procedure is provided, however, for combining these results into a facility-length level of service.

The ramp and weaving analysis procedures are not well adapted to the computation of service volumes. Only the basic section analysis procedure is well suited to planning applications and the computation of service volumes.

4.3.1.1 Input Requirements

The HCM procedure for basic segments requires the following: hourly volume, number of lanes, free-flow speed, peak-hour factor, lane and shoulder widths, percent trucks, percent recreational vehicles, terrain type, and driver population type.

4.3.1.2 Description

Level of service on basic freeway sections is defined in terms of density. An explicit equation (Equation 3-2 in the HCM) is provided for determining the service flow rate for each level of service. The service volume is a function of the number of lanes, lane widths, lateral clearance, percent heavy vehicles, terrain, and driver population type.

$$SF_I = c \times v/c_I \times N \times f_W \times f_{HV} \times f_P \tag{4-1}$$

where:

SF_I = maximum service flow at level of service, i;
c = capacity per lane (from Table 3-1 of the HCM);
v/c_I = maximum v/c ratio for level of service, i (from Table 3-1 of the HCM);
N = number of lanes;
f_W = lane width and lateral clearance factor (from Table 3-2 of the HCM);
f_{HV} = heavy vehicle adjustment factor (computed according to Equation 3-5 and Table 3-3 of the HCM); and
f_P = driver population adjustment factor (from Table 3-7 of the HCM).

The level of service for weaving areas, however, is determined based on speed. No explicit procedure is provided for computing service volumes for weaving areas.

The level of service for ramp junctions is determined based on a combination of density in the two rightmost lanes of the freeway and the minimum speed of traffic in these two lanes, for a portion of the freeway 1,500 ft in length. No explicit procedure is provided for determining service flow rates, and the speed estimation procedure itself is not applicable to the entire width of the freeway in the vicinity of the freeway ramp.

4.3.2 Multilane Highways (Chapter 7 of the HCM)

The level of service for multilane highways is determined by density. Maximum service flow rates are provided in Table 7-1 of the HCM. These service flow rates also can be computed using Equation 7-5 and Figure 7-4 of the HCM. The procedure for computing maximum service volume for multilane highways is the same as that for freeways.

4.3.3 Rural Two-Lane Roads (Chapter 8 of the HCM)

Level of service is determined based on percent time delay. No explicit relationship between volume and percent time delay is provided. Table 8-1 of the HCM, however, can be used to obtain the maximum v/c ratio that corresponds to a particular level of service according to the type of terrain and percentage of no-passing zones.

4.3.3.1 Input Requirements

This procedure requires lane widths, percent heavy vehicles, directional split, percent no-passing zones, and the general terrain type for computing maximum service volumes.

4.3.3.2 Description

The following equation (taken from Equation 8-1 of the HCM) shows how to convert v/c ratios to service volumes.

$$SF_i = 2800 \times v/c_i \times f_W \times f_{HV} \times f_d \qquad (4\text{-}2)$$

where:

SF_i = maximum service flow at level of service, i;
v/c_i = maximum v/c ratio for level of service, i (from Table 8-1 of the HCM);
N = number of lanes;
f_W = lane width and lateral clearance factor (from Table 8-5 of the HCM);
f_{HV} = heavy vehicle adjustment factor (computed according to Equation 8-2 and Table 8-6 of the HCM); and
f_d = directional distribution of traffic adjustment factor (from Table 8-4 of the HCM).

4.3.4 Urban Arterials (Chapter 11 of the HCM)

Level of service is determined based on speed and arterial class. No direct procedure is provided to compute service volumes. Planners have to try different v/c ratios and see the results. Florida, however, has developed table generating spreadsheets that can compute the maximum service volume for an arterial. The input requirements and procedure were described in the previous chapter on speed estimation techniques.

4.3.5 Accuracy

HCM level of service techniques are considered the standards against which all other techniques are compared. The accuracy of these procedures is reported later, in Chapter 10 of this report.

4.3.6 User Critiques

Planners have complained about the complexity of the HCM techniques, but software has reduced this problem. There still is a problem with collecting necessary data. Planners also express frustration about the sensitivity of HCM procedures to many factors over which they have little control.

Planners, for example, may want to determine the plan lines (right-of-way width) for a future arterial. It used to be that all they needed to do was to calculate the v/c ratio. Now, they need to know the signal timing as well. If planners use the existing signal timing to compute the maximum service volume for the future, the number of lanes computed will differ from the number of lanes computed by optimizing the signal timing for the future volume.

4.4 FLORIDA LEVEL OF SERVICE MANUAL METHODS

This section describes the methods for identifying facility level of service from the Florida *Level of Service Manual* (*4*). These methods are based on HCM methods, with some extensions to enhance the application of the HCM to planning problems. The Florida *Level of Service Manual* consists of (1) generalized level of service tables planners can use to look up maximum service volume and (2) software planners can use to create customized service volumes for specific facility characteristics and areas (ARTPLAN). The software consists of (1) spreadsheets that can be used to create tables of average service volumes for the entire facility and (2) an implementation of Chapter 11 of the HCM.

4.4.1 Typical Usage

The Florida methods are used by about 6 percent of the respondents to the national user survey for this research project. These methods are being used for all planning applications, from RTPs to site impact studies. Their most frequent use is for congestion management and site impact studies. The Florida *Level of Service Manual* also facilitates the translation of v/c ratio output by regional traffic forecasting models into levels of service for all uninterrupted flow facilities.

4.4.2 Florida Generalized Level of Service Tables

The generalized level of service tables in the Florida *Level of Service Manual* provide maximum service volumes by facility type and general characteristics for three area types: urbanized, transitioning/urban, and rural.

4.4.2.1 Input Requirements

The Florida Generalized Level of Service Tables require the following input information:

- Area type (urbanized, transition, rural);
- Facility type (state, nonstate, uninterrupted, interrupted, signals/mile);
- Number of lanes (2–12);
- Median type (divided, undivided, left turn bays, no left turn bays);
- One-way or two-way; and
- Free-flow speed.

4.4.2.2 Description

As mentioned previously, the generalized level of service tables in the Florida *Level of Service Manual* provide maximum service volumes by facility type and general characteristics for the following area types:

- Urbanized areas—contiguous areas with more than 1,000 persons per square mile with a minimum total population of 50,000 (as defined by FHWA);
- Transitioning/urban areas—areas contiguous to an existing urbanized area, expected to be included within the urbanized area within 20 years, or urban areas with a population exceeding 5,000 that are not part of an urbanized area; and
- Rural areas—undeveloped areas and developed urban areas with fewer than 5,000 people.

The tables were generated using the 1994 HCM methodology and sets of agreed-on assumptions for each facility type and area type. The assumptions are averages for the entire facility being analyzed and do not take into account certain unusual facility characteristics or special problem spots within a facility.

FDOT generally followed the HCM procedures, with the following exceptions:

- Special level of service criteria were developed for rural areas.
- The v/c ratio (rather than density or percent time delay) was used for freeways, multilane highways, and two-lane roads.
- Computed service volumes were adjusted upward or downward as appropriate (generally 5 percent) to account for the effects of medians and access points on arterial performance (e.g., if such factors are not incorporated in the HCM methods).
- Similar service volume adjustments were made for the presence of passing zones on two-lane rural roads.
- The average g/C time for arterials was computed as the weighted average of the most critical intersection g/C and the average g/C for the rest of the intersections on the arterial.

Table 4-4 presents a portion of one of the generalized level of service tables. This particular example is for urbanized areas. The definitions of levels of service used in the Florida *Level of Service Manual* were described earlier in this chapter. Tables 4-5 and 4-6 show the recommended adjustments for the maximum service flows to account for the effects of medians, left turn bays, and one-way streets. Table 4-7 lists the default input data used to derive the maximum service volumes in Table 4-4. These tables illustrate the logic and assumptions necessary for creating maximum service volume look-up tables based on the 1994 HCM. Each locality can customize the assumptions and facility type groupings to suit local conditions.

4.4.3 FDOT Software

The planning-level spreadsheets developed by Elena Prassas and William McShane for FDOT provide an opportunity to use specific traffic, roadway, and signal characteristics when estimating lane requirements and levels of service.

4.4.3.1 Description

The following table generating spreadsheets assist the planner in developing tables of maximum service volumes for a facility, based on its particular characteristics. These spreadsheets are now the preferred method for estimating level of service, rather than the generalized service volume tables originally developed by FDOT.

Spreadsheet	*Application*
FREE_TAB	for freeways
ART_TAB	for arterials (interrupted or uninterrupted flow conditions)
RMUL_TAB	for rural multilane uninterrupted highways
UMMUL_TAB	for urban multilane uninterrupted highways
R2LN_TAB	for rural two-lane uninterrupted highways
U2LN_TAB	for urban two-lane uninterrupted highways
SIG_TAB	for signalized intersections and minor signalized roadways off state highway systems

The ARTPLAN program is Florida's implementation of the urban and suburban arterial method in the 1994 HCM. The program computes mean speed but not service volume.

The use of the computer software allows for a simple assessment of congestion on freeways and arterial streets, given the limited amount of data on traffic and roadway characteristics that are periodically collected by public agencies. Such software has been prepared by FDOT by using the analysis techniques in the HCM.

TABLE 4-4 Florida generalized peak-hour directional volumes for urbanized areas

Facility	lanes	Divided?	Level of Service A	B	C	D	E
Freeways[a] (Group 1)	4	n/a	1100	1760	2640	3350	4040
	6	n/a	1660	2640	3970	5030	6340
	8	n/a	2210	3530	5290	6700	8460
	10	n/a	2760	4410	6620	8380	10570
Freeways[b] (Group 2)	4	n/a	1060	1700	2550	3230	3900
	6	n/a	1600	2560	3840	4860	6130
	8	n/a	2130	3410	5110	6480	8170
	10	n/a	2670	4260	6390	8100	10210
State Multi-lane Highways	2	No	460	720	980	1280	1710
	4	Yes	1110	1850	2590	3110	3700
	6	Yes	1670	2780	3890	4660	5550
Class Ia[c] Interrupted Flow	2	No	*	660	810	880	900
	4	Yes	*	1470	1760	1890	1890
	6	Yes	*	2280	2660	2840	2840
	8	Yes	*	2840	3280	3480	3480
Class Ib[d] Interrupted Flow	2	No	*	*	460	760	840
	4	Yes	*	*	1020	1640	1800
	6	Yes	*	*	1550	2510	2710
	8	Yes	*	*	1890	3060	3320
Class II[e] Interrupted Flow	2	No	*	*	*	620	800
	4	Yes	*	*	*	1390	1740
	6	Yes	*	*	*	2130	2640
	8	Yes	*	*	*	2600	3230
Class III[f] Interrupted Flow	2	No	*	*	*	690	780
	4	Yes	*	*	*	1540	1700
	6	Yes	*	*	*	2340	2570
	8	Yes	*	*	*	2860	3140

n/a = not applicable.
* = Level of service cannot be achieved.
[a] Group 1 freeways are located within an urbanized area with over 500,000 population and the freeways lead to or are within 5 miles of the primary Central Business District.
[b] Group 2 freeways are freeways not falling within Group 1.
[c] Class Ia arterials have less than 2.50 signals per mile.
[d] Class Ib arterials have 2.50 to 4.50 signals per mile.
[e] Class II arterials have more than 4.50 signals per mile and are NOT located within a primary central business district of an urbanized area with over 500,000 population.
[f] Class III arterials have more than 4.50 signals per mile AND are located within the primary central business district of an urbanized area with over 500,000 population.

Computer models, basically Lotus 1-2-3 spreadsheet templates, that allow the generation of service volume look-up tables also have been developed. These models allow for a simple assessment of level of service based on a limited amount of data on traffic and roadway characteristics. These data, which are required for these models, usually are available from public agencies and are routinely updated.

The methodologies used to develop these look-up tables are consistent with those in the 1994 HCM. Adjustments to account for undivided roadway conditions and for consideration of one-way facilities are available. Assumptions used in developing the various look-up tables are detailed in the 1995 edition of the Florida *Level of Service Manual.*

4.4.3.2 Input Requirements

FREE_TAB is the spreadsheet model used to determine freeway level of service. This template is based on the procedures in Chapter 3 of the HCM. The data input requirements for determining peak-hour or daily level of service threshold values using FREE_TAB are as follows:

- *K* factor—percentage of daily vehicles traveling in the peak hour;
- *D* factor—percentage of vehicles traveling in the peak direction;
- *PHF* (peak-hour factor);
- Adjusted saturation flow rate—maximum hourly service flow rate of vehicles in one lane in 1 hr; and
- Free-flow speed.

The output from this software is a look-up table that provides level of service threshold values for various freeway cross-sections. The values are provided in terms of peak-hour peak direction volumes, peak-hour both direction volumes, and average annual daily traffic volumes.

TABLE 4-5 Adjustments for divided/undivided streets and left turn bays

Adjust the maximum service volumes by the following percentages			
Lanes	Median	Left Turn Bays	Adjustment Factors
2	Divided	Yes	+5%
2	Undivided	No	-20%
Multi	Undivided	Yes	-5%
Multi	Undivided	No	-25%

The RMUL_TAB table generating spreadsheet is applicable to rural multilane uninterrupted facilities. The UMUL_TAB spreadsheet is used to develop look-up tables for urban multilane uninterrupted flow facilities. These templates are based on Chapter 7 of the 1994 HCM. The data requirements for these models are the same as those for FREE_TAB, with the addition of the following:

- Presence of medians
- Presence of left turn bays.

The output from the models is look-up tables that show various threshold values for different levels of service. The values are provided in terms of peak-hour peak direction volumes, peak-hour both direction volumes, and average annual daily traffic volumes.

R2LN_TAB, the table generating spreadsheet model based on the 1994 HCM, facilitates the determination of level of service of rural two-lane uninterrupted highways. The data requirements for this spreadsheet include the following:

- *K* factor—percentage of daily vehicles traveling in the peak hour;
- *D* factor—percentage of vehicles traveling in the peak direction;
- *PHF* (peak-hour factor);
- Adjusted saturation flow rate—maximum hourly service flow rate of vehicles in both directions in 1 hr;
- Free-flow speed;
- Percent no-passing zones;
- Percent exclusive passing lanes; and
- Presence of left turn bays.

As with other table generating spreadsheets, the output of this model is a look-up table that specifies threshold volumes for different levels of service. Again, these values are available as peak-hour peak direction volumes, peak-hour both direction volumes, and average annual daily traffic volumes.

Urban two-lane uninterrupted highways are analyzed using the U2LN_TAB template, which is based on the 1994 HCM and requires the following input data:

- *K* factor—percentage of daily vehicles traveling in the peak hour;
- *D* factor—percentage of vehicles traveling in the peak direction;
- *PHF* (peak-hour factor);
- Adjusted saturation flow rate—maximum hourly service flow rate of vehicles in one lane in 1 hr;
- Posted speed limit;
- Presence of medians;
- Presence of left turn bays; and
- Area type (urban, transitioning, or rural).

As with other table generating spreadsheets, the output of this model is a look-up table that specifies threshold volumes for different levels of service. Again, these values are available as peak-hour peak direction volumes, peak-hour both direction volumes, and average annual daily traffic volumes.

TABLE 4-6 Adjustments for one-way streets

Adjust the maximum service volumes by the following percentages		
One-Way Lanes	Corresponding Two-Way Lanes	Adjustment Factor
2	4	+20%
3	6	+20%
4	8	+20%
5	8	+50%

TABLE 4-7 Default input values for urbanized areas

Input Data	Freeways		State Two-Way Arterials				
	Group 1	Group 2	Uninterrupted	Class 1a	Class 1b	Class 2	Class 3
Traffic Characteristics							
Peak Hour Factor	0.950	0.950	0.925	0.925	0.925	0.925	0.925
Adjusted Saturation Flow Rate							
2-lane facility	2125	2050	1850	1850	1850	1850	1800
4-6 lanes	2225	2150	2000	1850	1850	1850	1800
8 lanes	2225	2150	NA	1700	1700	1700	1650
10 lanes	2225	2150	NA	NA	NA	NA	NA
Turns from exclusive lanes	NA	NA	NA	0.12	0.12	0.12	0.12
Roadway Characteristics							
Through Lanes	4-12	4-12	2-6	2-8	2-8	2-8	2-8
Arterial Classification	NA	NA	NA	I	I	II	III
Free-flow speed	60	60	50	45	40	35	30
Medians	Yes	Yes	Yes	Yes	Yes	Yes	Yes
left turn bays	NA	NA	Yes	Yes	Yes	Yes	Yes
Signal Characteristics							
Signals per mile	NA	NA	NA	1.5	3.0	5.0	7.5
Arrival type	NA	NA	NA	3	4	4	4
Signal Type	NA	NA	NA	Act	Semi	Semi	Semi
Cycle length	NA	NA	NA	120	120	120	120
Weighted effective g/C	NA	NA	NA	0.45	0.45	0.45	0.45

A spreadsheet model also is available for determining signalized intersection level of service at the planning level. This spreadsheet is called SIG_TAB and is, as is all FDOT table generating software, based on the 1994 HCM. It is used to create look-up tables for minor signalized roadways off the state system. The data requirements for this model are as follows:

- *K* factor—percentage of daily vehicles traveling in the peak hour;
- *D* factor—percentage of vehicles traveling in the peak direction;
- *PHF* (peak-hour factor);
- Adjusted saturation flow rate—maximum hourly service flow rate of vehicles in one lane in 1 hr;
- Cycle length;
- Percent turns from exclusive lanes;
- Area type (urban, transitioning, or rural);
- Presence of medians;
- Presence of left turn bays;
- Arrival type (as defined in the HCM);
- Signal system type (pretimed, semiactuated, or actuated); and
- Through movement g/C ratio.

As with other table generating spreadsheets, the output of this model is a look-up table that specifies threshold volumes for different levels of service. Again, these values are available as peak-hour peak direction volumes, peak-hour both direction volumes, and average annual daily traffic volumes. In addition, the peak-hour peak direction through/right v/c ratios for the full analysis hour (the peak 15-min v/c ratio multiplied by the peak-hour factor) are output.

ART_TAB is the spreadsheet model used to determine arterial level of service. This model provides a look-up table to determine level of service on arterials, based on calculated threshold values for peak-hour or daily traffic volumes. The data input required for the ART_TAB spreadsheet is as follows:

- *K* factor—percentage of daily vehicles traveling in the peak hour;
- *D* factor—percentage of vehicles traveling in the peak direction;
- *PHF* (peak-hour factor);
- Adjusted saturation flow rate—maximum hourly service flow rate of vehicles in one lane in 1 hr;
- Free-flow speed;
- Percent turns from exclusive lanes;
- Area type (urban, transitioning, or rural);
- Arterial class (describes function, design, and free-flow speed, as defined in the HCM);
- Length of arterial;

- Presence of medians;
- Presence of left turn bays;
- Number of signalized intersections;
- Arrival type (as defined in the HCM);
- Signal system type (pretimed, semiactuated, or actuated);
- System cycle length; and
- Weighted through movement g/C (average of the critical intersection through g/C and average through g/C of all other intersections).

These data requirements are quite extensive compared with those of generalized look-up tables, which use default assumed values for most of the aforementioned data requirements. The use of arterial-specific information in the ART_TAB spreadsheet results in a customized look-up table for estimating level of service for the arterial under consideration.

4.4.4 Accuracy

The accuracy of the methods in the Florida *Level of Service Manual* is reported in Chapter 11 of this report. The Florida methods were developed and tested using an extensive set of Florida data and are based on HCM methods. Consequently, we would expect these methods to produce relatively reliable results.

4.4.5 User Critique

Users have noted that the methods in the Florida *Level of Service Manual* can (in theory) be applied to local residential streets and other low-design-speed roadways. The result of the minimum speed level of service standards for arterials, however, is that levels of service A, B, and C are unachievable for these roadways, regardless of the volume of traffic.

Arterials with short signal spacing can never achieve a level of service better than D with these methods. This is a result of the HCM level of service criteria, rather than the methods themselves. Arterials with closely spaced signals and low g/C ratios can never obtain the minimum average speed required to achieve superior levels of service.

These results are expected, given the fact that poor design causes poor levels of service. However, planners would like to be able to say that level of service A is the best that can be achieved, given the poor design features of the facility.

The methods in the Florida *Level of Service Manual* also appear to be sensitive to low g/C ratios, which cause unrealistically low estimates of facility capacities. Users, however, appreciate the ability to distinguish the effects of signal spacing, medians, and left turn bays on maximum service volumes. The 1994 HCM does not currently provide this capability.

4.5 OTHER METHODS

The other level of service methods identified by the planning agencies consisted of traffic operations software such as HCS, TRANSYT-7F, and NETSIM. These traffic operations models are not suitable for planning applications and, therefore, are not discussed here. Delaware, however, has developed a series of charts for looking up level of service for arterials.

The Delaware charts (*5*) are designed to help planners quickly look up the level of service for signalized arterials given the arterial class, free-flow speed, cycle length, number of signals per mile, and v/c ratio. The planner selects a chart based on the arterial's class, free-flow speed, and cycle length. The number of signals per mile and the v/c ratio are then used to enter the chart and determine the level of service.

The Delaware charts are based on the 1985 HCM method for signalized arterials (Chapter 11 of the HCM). These charts were developed using a variable called "signal ratio" to help reduce the number of necessary charts. The signal ratio is the ratio of signal delay to the running time (which excludes stopped delay) on the arterial. The relationship between level of service and the signal ratio was then derived and used to construct the charts.

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CHAPTER 5

TECHNIQUES SUGGESTED IN THE LITERATURE

This chapter presents various techniques for estimating speed and level of service that have been recommended by various authors in the literature but have not seen widespread application in planning practice.

5.1 AKCELIK/DAVIDSON FORMULA

Akcelik (*1*) proposed the following modification to Davidson's equation for predicting travel time on any road facility. The equation applies to v/c ratios greater than and less than 1. The equation predicts the inverse of speed, the travel time per unit distance.

$$t = t_0 + \left\{0.25T\left[(x-1) + \sqrt{(x-1)^2 + \frac{8J_A}{QT}x}\right]\right\} \quad (5\text{-}1)$$

where:

t = average travel time per unit distance (hours/mile)
t_0 = free-flow travel time per unit distance (hours/mile)
T = the flow period (typically 1 hr) (hours)
x = the degree of saturation
= v/c ratio
Q = capacity (veh/hr)
J_A = the delay parameter.

Akcelik's equation states that the travel time (t) is equal to the free-flow travel time (t_0) plus the average overflow queue (N_0) divided by the capacity (Q). The average overflow queue divided by capacity is the portion of the equation inside the brackets to the right of t_0. The equation for the average overflow queue was fitted by Akcelik to take into account variations in queue lengths caused by random variations in arrivals.

There is, in theory, no upper limit on the value of x that could be input into this equation because this equation is designed to approximate the delays caused by queuing when demand exceeds capacity. However, when this equation was incorporated into the signal delay estimation method in the HCM, it was decided to limit its application in the HCM to hourly volumes less than or equal to capacity.

The delay parameter, J_A, is a function of the number of delay causing elements in the section of road and the variability of demand. Akcelik suggests lower values of J_A for freeways and coordinated signal systems. Higher values apply to secondary roads and isolated intersections.

The value of J_A can be computed if the difference in the rate of travel (hours per mile) between capacity and free-flow conditions on the facility is known. Substituting x with 1 in the previous equation and solving for J_A yields the following:

$$J_A = \frac{2Q}{T}(t_c - t_0)^2 \quad (5\text{-}2)$$

where t_c = the rate of travel at capacity (hours per mile).

The equation explicitly takes into account the delays caused by queuing and can be applied to any facility type. The assumptions are that there is no queue at the start of the analysis period, and there is no peaking of demand within the analysis period (T).

5.2 NCHRP PROJECT 7-13 CURVES

Lomax et al. (*2*) used linear regression to fit a set of speed-flow curves for arterials and freeways to various datasets they obtained as part of their research. The curves predict speed based on the v/c ratio, signal spacing, and frequency of access points.

The following linear equations were manually smoothed into a series of curves for use in looking up speed as a function of signal density, access density, and v/c ratio.

For freeways:

$$\mathit{Speed}\text{ (mph)} = 91.4 - 0.002[\mathit{ADT/Lane}] - 2.85[\mathit{AccessPointsPerMile}] \quad (5\text{-}3)$$

For Class I arterials:

$$\mathit{Speed}\text{ (mph)} = 40.6 - 0.0002[\mathit{ADT/Lane}] - 2.67[\mathit{SignalsPerMile}] \quad (5\text{-}4)$$

For Class II and Class III arterials:

$$Speed\ (\text{mph}) = 36.4 - 0.000301[ADT/Lane] - 156[SignalsPerMile] \tag{5-5}$$

5.3 *NCHRP REPORT 255* PROCEDURES

Pedersen and Samdahl (*3*) developed a recommended set of procedures for computing speed, delay, and queue length for freeways and arterials for undercapacity and overcapacity conditions. Their recommended procedures for undercapacity conditions are almost identical to the procedures contained in the 1985 HCM for basic freeway sections and urban arterials. One difference, however, is that the researchers reduced the design speeds reported in the 1965 HCM to average speeds, using a formula developed by Makigami, Woodie, and May (*4*), as follows:

$$AS = OS - [DS/10 * (1 - v/c)] \tag{5-6}$$

where:

AS = average speed
OS = operating speed
DS = design speed
v/c = v/c ratio.

Pedersen and Samdahl's procedures for estimating average speed on freeways and arterials can be brought up to date simply by using the procedures in Chapters 3 and 11 of the 1994 HCM. There is no need to convert operating speed to average speed, because the new HCM reports average speed.

Pedersen and Samdahl, however, recommend a pair of procedures that extend the HCM methods to overcapacity conditions. These procedures were originally developed by Curry and Anderson (*5*). One procedure uses "shock wave" analysis to predict queuing on freeways. The other procedure uses deterministic queuing to predict delay on interrupted flow facilities.

5.3.1 Freeway Shock Wave Analysis Procedure

This procedure uses the lower limb of the speed-flow curve for freeways that was reported in the 1985 HCM but is not included in the 1994 HCM. The freeway is divided into three subsections (Figure 5-1). The first subsection is the bottleneck, where upstream demand exceeds capacity (often the section of freeway just downstream of an on-ramp). The second subsection is the queue immediately upstream of the bottleneck (often the section immediately upstream of an on-ramp). The third subsection is the remaining portion of the freeway upstream of the queue, which may not exist if the queue extends the full length of the freeway study section. The freeway study section must be extended if the computations indicate that the queue extends upstream beyond the initially selected freeway study section.

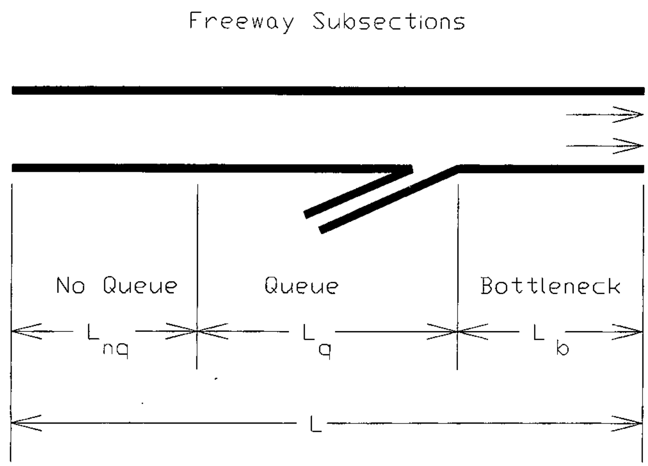

Figure 5-1. Division of freeway analysis section into subsections.

The average speed over the entire freeway section is determined by averaging the speed in each subsection, as shown in the following equation:

$$ARS = \frac{L}{\dfrac{L - L_b - L_q}{ARS_{nq}} + \dfrac{L_q}{ARS_q} + \dfrac{L_b}{ARS_b}} \tag{5-7}$$

where:

ARS = average running speed of entire freeway section
ARS_b = average running speed of bottleneck subsection of freeway
= speed at capacity
ARS_q = average running speed in queue subsection upstream of bottleneck
ARS_{nq} = average running speed in subsection upstream of queue
L = length of entire freeway section
L_b = length of bottleneck section
L_q = length of queue.

The bottleneck and nonqueuing subsection speeds can be determined from the speed-flow curves in Chapter 3 of the HCM. The average speed within the queue section must be determined from the lower limb (the forced flow) portion of the speed-flow curve in the 1985 HCM.

The following equation provides an approximate fit to the lower limb of this curve:

$$ARS_q = A * \exp[\ln B * (v/c)^{1.27}] \tag{5-8}$$

where:

$A = 5$
$B = 6$
v/c = the flow rate under queuing conditions.

This curve approaches 30 mph at $v/c = 1$ and 5 mph at $v/c = 0$. Parameters A and B can be modified according to the following if different speeds are desired:

A = speed at $v/c = 0$
B = (speed at $v/c = 1$)/A.

The length of queue (L_q) is computed as follows:

$$L_q = (QR * T)/(2DQ) \tag{5-9}$$

where:

L_q = average queue length during the analysis period (miles)
QR = queuing rate (veh/hr)
= upstream demand − bottleneck capacity
T = length of time the level of demand occurs (length of peak hour or peak period) (hours)
(Note that the queue is building, not dissipating, during this period.)
DQ = change in vehicle density between queue and upstream nonqueued subsection
= (bottleneck capacity)/ARS_q − (upstream demand)/ARS_{nq}.

5.3.2 Arterial Queuing Analysis for Overcapacity

The average running speed for the arterial is computed using the same equation as appears in Chapter 11 of the HCM:

$$Speed = \frac{[3600 * Length]}{[(RunningTimePerMile) * (Length) + D]} \tag{5-10}$$

The difference is in the calculation of intersection delay (D) for intersections on the arterial where the through movement v/c ratio is greater than 1 (overcongested intersections). The steps for this procedure follow.

Step 1. Look up the running speed for the link feeding the overcongested intersection. The speed will be based on free-flow speed and signal density.

Step 2. Adjust the vehicle arrival rate to account for the fact that as the queue extends back from the intersection, vehicles join the queue earlier than they would have if the queue were at the intersection stop line.

$$AAR = Demand * \left\{1 + \frac{(Demand - Capacity)}{Lanes * Speed * 240 - Demand}\right\} \tag{5-11}$$

where:

AAR = adjusted arrival rate (veh/hr)
$Demand$ = predicted arrival rate of vehicles at the congested intersection stop line (veh/hr)
$Capacity$ = saturation flow rate per lane times the number of through lanes times the g/C ratio for the approach (vehicles per hour per lane—vphpl)
$Lanes$ = number of through lanes on the approach (one direction)
$Speed$ = average running speed for the approach in Step 1
240 = assumed queue density of 240 vehicles per lane per mile (22 ft/veh).

Step 3. Compute the queue length:

$$Q = 0.5 * \left\{T * (AAR - Capacity) + Capacity * \frac{Cycle - Green}{3600}\right\} \tag{5-12}$$

where:

Q = mean queue length (vehicles)
T = duration of analysis period (hours)
AAR = adjusted arrival rate (veh/hr) (from Step 2)
$Capacity$ = maximum flow rate per lane times the number of lanes (veh/hr) (see Step 2)
$Cycle$ = signal cycle length (seconds)
$Green$ = effective green time for through vehicles (seconds).

Step 4. Compute average delay (D) at overcongested intersection:

$$D = 3600 * Q/Capacity \tag{5-13}$$

where:

D = average delay (seconds)
Q = mean queue length (veh) (from Step 3)
$Capacity$ = saturation flow per lane times number of lanes times g/C (veh/hr).

5.4 VAN AERDE CAR FOLLOWING MODEL

Van Aerde (*6*) proposed a single-regime car following model that can be used to predict the speed of traffic as a function of volume, given the following parameters:

c = capacity (veh/hr)
S_c = speed at capacity (mph or km/hr)
S_f = free-flow speed (mph or km/hr).

Van Aerde's model has three parameters, p_1, p_2, and p_3, which allows a great deal of flexibility in choosing the shape of the speed-flow curve. Facility capacity, speed of traffic at capacity, and free-flow speed are required for estimating these parameters as follows:

$$k = (2S_c - S_f)/(S_f - S_c)^2 \tag{5-14}$$

$$p_2 = [S_c(S_f - S_c)]/\{c[1 + k(S_f - S_c)]\} \tag{5-15}$$

$$p_1 = k * p_2 \tag{5-16}$$

$$p_3 = (1/S_c) * [S_c/c - p_1 - p_2/(S_f - S_c)] \tag{5-17}$$

where:

k = the ratio of a_1 to a_2
S_c = speed at capacity (mph or km/hr)
S_f = speed at free flow (mph or km/hr)
c = capacity (veh/hr).

Substituting parameters p_1, p_2, and p_3 into the following equation gives two speed predictions. The higher speed value is for uncongested conditions, and the lower speed value is for forced-flow (queuing) conditions.

$$s = [-b \pm \text{sqrt}(b^2 - 4ac)]/(2a) \tag{5-18}$$

where:

$a = 1 - v * p_3$
$b = v * p_3 * S_f - v * p_1 - S_f$
$c = v * p_2 + v * p_1 * S_f$

This formula cannot accommodate forecast volumes that exceed capacity. If the demand is forecasted to exceed capacity, the user must use the shock wave analysis described previously for freeways (*NCHRP Report 255*). Van Aerde's equation can be used in the shock wave analysis instead of the speed flow curves cited in the discussion of *NCHRP Report 255*.

5.5 COURAGE ET AL. MODIFICATION TO HCM ARTERIAL METHOD

Courage et al. (*7*) noted that although the HCM method for urban arterials tends to estimate running speed accurately, it tends to overestimate the delay at signals. The researchers recommended two adjustment factors to reduce the uniform and incremental delay terms in the delay equation. The incremental delay adjustment factor was designed to compensate for the effect of closely spaced signals on the incremental delay. The floating car adjustment factor was designed to compensate for the presumed bias of floating cars in measuring uniform delay. This bias is computed as a function of the quality of progression.

The revised delay formula recommended by Courage et al. follows:

$$D = 1.3 * (F_{fc} * d_u * DF + F_{ss} * d_i) \tag{5-19}$$

where:

D = total approach delay, in sec/veh;
F_{fc} = floating car adjustment factor;
d_u = approach uniform delay, in sec/veh;
F_{ss} = signal spacing adjustment factor;
d_i = approach incremental delay, in sec/veh;
DF = delay adjustment factor.

$$F_{fc} = \frac{g/C * x^2 * (1 - g/C * R_p) * (1 - g/C * x)}{[1 - g/C * x * R_p] * (1 - g/C)} \tag{5-20}$$

where:

g/C = ratio of green time to cycle length
x = v/c ratio (volume/saturation ratio divided by g/C ratio)
R_p = platoon ratio (proportion of through vehicles arriving during green phase divided by the g/C ratio).

$$F_{ss} = \min\left[\frac{signalspacing}{referencelength}, 1.0\right] \tag{5-21}$$

where:

signal spacing = distance between signals
reference length = 1/2 mile (0.8 km) for Class I arterials
= 1/4 mile (0.4 km) for Class II arterials.

The HCM method predictions of mean travel speed for Ventura Boulevard were, on the average, consistently 4.5 mph (7.2 km/hr) lower than the floating car speed measurements. Modifications recommended by Courage et al. routinely increased the HCM estimated speeds by 25 percent to 30 percent. The result was a modest overcorrection of the HCM estimated speeds. The Courage et al.–modified HCM estimates were an average of 2.2 mph (3.5 km/hr) higher than the floating car measured speeds.

5.6 DeARAZOZA/McLEOD SPEED LIMIT DEVIATION

DeArazoza and McLeod (*8*) chose to use speed instead of percent passing delay as the level of service measure for two-lane sections of U.S. 1 in the Florida Keys. They developed a novel level of service hierarchy based on deviations from the posted speed limit, which was easier for the general public to understand and perceive while driving the highway. The deviations were selected to correspond as much as possible with the thresholds contained in the

HCM. The speed level of service measure also was easier to measure in the field than percent time delay.

5.7 NCHRP PROJECT 3-45 FREEWAY SPEED-FLOW EQUATIONS

Schoen (*9*) developed the following equation for predicting free-flow speed on freeways:

$$FFS = 70 - F_n - F_{lw} - F_{lc} - F_{id} \tag{5-22}$$

where:

- FFS = free-flow speed for basic freeway segment (mph);
- F_n = adjustment factor for effect of number of lanes;
- F_{lw} = adjustment factor for effect of lane width;
- F_{lc} = adjustment factor for effect of lateral clearance; and
- F_{id} = adjustment factor for effect of interchange density.

Schoen also established recommended maximum capacities for basic freeway segments that vary according to free-flow speed. These capacities range from 2,250 vehicles per hour per lane for a freeway with a 55 mph (89 km/hr) free-flow speed to 2,400 vehicles per hour per lane for a freeway with a 70 mph (113 km/hr) free-flow speed.

5.8 CONICAL DELAY FUNCTIONS

Spiess (*10*) developed a revised speed-flow equation to enable computers to compute equilibrium traffic flow much more rapidly than with the standard BPR curve. The BPR curve is highly volatile at high v/c ratios (a slight change in the forecasted volume results in large changes in the estimated speed) and is too insensitive at low v/c ratios (a large change in volume results in minor changes in speed). The BPR curve also uses exponentials, which slows computer computations. All these characteristics of the BPR curve tend to slow down travel model computations of equilibrium traffic volumes. Spiess suggested a "conical delay function" as a more computationally efficient speed-flow curve, which still is very similar to the BPR curve. The conical delay function drops off fairly constantly over lower ranges of v/c ratios and does not increase as rapidly as the BPR curve at higher v/c ratio ranges. The equation is as follows:

$$t = t_0 * \left[2 + \sqrt{a^2 * (1 - x)^2 + b^2} - a * (1 - x) - b\right] \tag{5-23}$$

where:

- t = travel time (sec)
- t_0 = travel time under free-flow conditions (sec)
- a = a calibration parameter that must be greater than 1
- $b = (2a - 1)/(2a - 2)$
- x = v/c ratio.

Note that at capacity $x = 1$ and $t = 2t_0$, and at zero volume $x = 0$ and $t = t_0$.

5.9 MARGIOTTA SPEED DETERMINATION MODELS

Margiotta et al. (*11*) used the TRAF family of traffic simulation models to develop regression equations for predicting mean facility speeds as a function of the ratio of average daily traffic (ADT) to the hourly capacity of the facility. The functions predict the delay resulting from traffic flow and the density of traffic signals per mile. The delay is added to the free-flow travel time to obtain total travel time.

The following set of equations was developed for freeways and multilane rural highways:

$$\text{If } x \leq 8, \text{ then } d = 0.0611x + 0.00777x^2 \tag{5-24}$$

$$\text{If } 8 < x \leq 12, \text{ then } d = 28.4 - 71.6x + 0.467x^2 \tag{5-25}$$

$$\text{If } x > 12, \text{ then } d = -31.7 + 2.98x + 0.0393x^2 \tag{5-26}$$

where:

- x = the ratio of ADT to hourly capacity
- d = the ratio of hours delay to 1,000 vehicle miles traveled.

The following equations were developed for urban arterials with signals and left turn bays. (Margiotta et al. were unsatisfied with the ability of the simulation models at that time to simulate delays for intersections without turn bays.)

For $n \leq 20$ and $x \leq 7$,

$$d = (1 - \exp(-n/24.4)) * (68.6 + 16.9x) \tag{5-27}$$

For $n \leq 20$ and $7 < x \leq 18$,

$$d = (1 - \exp(-n/24.4)) * (186.9 + 14.6(x - 7) - 1.85(x - 7)^2) + 0.706(x - 7)^2 \tag{5-28}$$

where:

- n = the number of signals per mile
- x = the ratio of ADT to hourly capacity
- d = the ratio of hours delay per 1,000 vehicle miles traveled.

Similar equations also were developed for urban arterials without traffic signals and rural two-lane roads.

The delay equations are quick and simple to apply and ideal for estimating speeds for the Highway Performance Monitoring System (HPMS). These equations, unfortunately, are heuristic approximations of simulation model results from artificial datasets. Their application, therefore, is limited to the particular facility types and conditions on which the equations were developed. They cannot be relied on when signal timing, signal coordination, and peaking characteristics of demand vary from the simulated datasets used to develop the equations.

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CHAPTER 6

DESCRIPTION OF DATASETS

This chapter discusses the datasets selected for evaluation during this study. A list of selected datasets is provided, including the agencies and firms contacted to secure the data. The datasets are divided into four categories:

- Urban interrupted flow facilities,
- Urban uninterrupted flow facilities,
- Rural interrupted flow facilities, and
- Rural uninterrupted flow facilities.

Table 6-1 lists the criteria used to categorize the datasets; Table 6-2 lists the selected datasets; and Table 6-3 lists the salient characteristics of the datasets.

6.1 CRITERIA FOR DATASET SELECTION

The datasets were assembled for the purpose of developing and testing the validity of new and enhanced planning techniques for estimating speed and level of service. For this purpose, the ideal dataset would contain volumes, speeds, and sufficient data for estimating level of service by all existing and candidate techniques.

The data requirements of the most complex of the candidate methods, those based on the 1994 HCM (*1*), were selected as the basis for assembling the datasets. All planning methods would require fewer data, and the dataset could be used to verify the consistency of the planning methods with those in the HCM.

Ideally, the speeds would be 1-hr averages measured over the entire length of the study section by means of floating cars; however, it was not feasible to obtain floating car data for many freeway facilities. Consequently, loop speed data were accepted for many uninterrupted flow facilities. One hour typically is the smallest time period considered in planning applications.

The NCHRP Project 3-55(2) research oversight panel suggested a minimum length of 2 mi (3.2 km) for the study section in each dataset, based on criteria suggested in the HCM. Chapter 11, Urban and Suburban Arterials, suggests a 2-mi (3.2-km) length. Chapter 8, Rural Two-Lane Roads, suggests a minimum 2-mi (3.2-km) length for analysis, using general terrain characteristics. Chapter 7, Multilane Highways, suggests a minimum analysis length of 0.5 mi (0.8 km).

The research team attempted to meet its criterion whenever possible; however, it was necessary to relax this criterion for a few (primarily uninterrupted flow facility) datasets to obtain the necessary number of datasets.

Researchers in the field of highway capacity and level of service were contacted to determine the availability of datasets. Principal investigators for previous and ongoing NCHRP projects, such as NCHRP 3-45 and NCHRP 3-55(3), also were contacted. Several respondents to the NCHRP Project 3-55(2) User Survey volunteered datasets, which have been included in the datasets selected for this research.

6.2 CATEGORIZATION OF DATASETS

The research problem statement identified four categories of facilities for the development of planning techniques: urban interrupted, urban uninterrupted, rural interrupted, and rural uninterrupted. The NCHRP Project 3-55(2) panel and the research team discussed various approaches to defining these four facility types. Discussion took place on how the general characteristics of the area and the characteristics of frontage development (e.g., driveways, parking, and side friction) affected the operating and demand characteristics of a facility.

The research team reviewed the issues raised in the panel meeting and determined that with only four categories available to describe all possible highway facilities, one cannot be too precise in defining all possible operating and demand characteristics that might affect the speed and level of service of facilities.

6.2.1 Definition of Urban and Rural Areas

The Census Bureau provides a definition of "urbanized areas" and "urban places." The bureau defines an urbanized area as a contiguous area in which the population exceeds 1,000 persons per square mile. Urban areas are defined as areas with 2,500 or more persons. Rural areas are areas that are neither urban nor urbanized (*2*). The Florida *Level of Service Manual* defines urban areas as places with more than 5,000 persons (*3*).

The Census Bureau definitions are familiar to planners; thus, the research team decided to adopt the bureau's defini-

TABLE 6-1 Definitions of facility categories

Urban:	Areas with population greater than 5,000 persons or a population density greater than 1,000 persons per square mile.
Rural:	All areas that are not urban.
Urban Interrupted:	Urban area facilities with signals spaced 2 miles or less apart.
Urban Uninterrupted:	All other urban area facilities.
Rural Uninterrupted:	Rural area facilities with access control.
Rural Interrupted:	All other rural area facilities.

tion of urban and urbanized areas. The Census Bureau's definition of 2,500 persons for urban areas, however, is too stringent. As a result, the research team adopted the Florida modification that urban areas must exceed 5,000 persons. This definition will be used to differentiate between urban facilities and rural facilities, regardless of the presence or absence of commercial development fronting the road.

6.2.2 Definition of Interrupted Flow and Uninterrupted Flow Facilities

Chapter 11, Urban and Suburban Arterials, of the 1994 HCM provides guidance on the facility types for which its analysis techniques are appropriate. Specifically, the methods in this HCM chapter are designed for arterials with signals spaced closer than 2 mi (3.2 km) apart.

The HCM signal spacing criterion for interrupted flow facilities appears to work well for urban area facilities. Consequently, the research team adopted this definition for categorizing urban area facilities as either interrupted flow or uninterrupted flow.

The research team considered using the 2-mi (3.2-km) signal spacing criterion for rural facilities as well, but found that no facilities in rural areas met this signal density requirement. Longer signal spacing requirements for rural facilities were considered, but signals spaced farther than 2 mi apart were considered to have little significant impact on average facility speed and level of service. The use of signal spacing to define interrupted flow rural facilities also would have the disadvantage of combining most two-lane rural roads and multilane highways (with their lower capacities per lane) with freeways.

The research team, consequently, decided to adopt a separate criterion for categorizing rural facilities as either interrupted or uninterrupted flow. This criterion is called "access control." If there is no access control in a rural area, the facility is categorized as an interrupted flow facility. Otherwise, it is an uninterrupted flow facility. This definition of interrupted flow facilities in rural areas facilitates the development of separate defaults and capacity values for freeways and other roads in rural areas.

6.3 DATASET DESCRIPTIONS

6.3.1 Urban Interrupted Flow Facilities

6.3.1.1 U.S. 1, Martin County, Florida

The selected study section of U.S. 1 is a 3.8-mi (6.1-km) section of four-lane divided signalized arterial between Salerno Road and State Route 5a in Martin County, Florida. The terrain is flat. Various portions of this arterial have posted speeds of 40 mph and 45 mph (64 km/hr and 72 km/hr). These datasets were compiled by Barton-Aschman Associates, Inc., as part of the Florida Department of Transportation District Four Districtwide Capacity Analysis Study in March 1993.

6.3.1.2 State Route 808/Glades Road, Palm Beach County, Florida

The selected study section of State Route 808 is a 1.3-mi (2.1-km) section of six-lane divided signalized urban arterial in Palm Beach County, Florida. The terrain is flat. The posted speed limit is 45 mph (72 km/hr). These datasets were compiled by Barton-Aschman Associates, Inc., as part of the Florida Department of Transportation District Four Districtwide Capacity Analysis Study in April 1993.

6.3.1.3 Dodge Street, Omaha, Nebraska

The selected study section of Dodge Street is a 2.6-mi (4.2-km) section of six-lane signalized urban arterial between 90th Street and 50th Street in Omaha, Nebraska. The posted speed limit is 35 mph (56.4 km/hr). The facility carries 55,800 ADT.

Travel time runs were conducted in 1993 by researchers from the University of Nebraska at Lincoln (UNL) as part of a project to evaluate the use of CORFLO for traffic congestion management planning in the Omaha–Council Bluffs metropolitan area. Roadway geometric and signal timing data were obtained from the city of Omaha, the Omaha–Council Bluffs Metropolitan Area Planning Agency, the

TABLE 6-2 Final datasets

Facility Type	Interrupted	Uninterrupted
Urban	1. U.S. 1 (Salerno Road to State Route 5a north ramp) - Martin County, Florida (3.8 miles).	1. Interstate 5 (Interstate Bridge to Downtown Portland) - Oregon (6.0 miles).
	2. State Route 808/Glades Road, Palm Beach County, Florida (1.3 miles).	2. U.S. 101 Ventura Freeway (State Route 27 to I-405), Los Angeles, California (10.6 miles).
	3. Dodge Corridor (52nd and Farnam Street to 90th and West) - Omaha, Nebraska (2.7 miles).	3. Interstate I-880, (Marina Blvd. to Whipple Street), Hayward, California, (9.2 miles).
	4. Ventura Boulevard, (Topanga Canyon Blvd. to Sepulveda Blvd.) Los Angeles, California (8.2 miles).	4. Interstate 80 (72nd Street to I-480), Omaha, Nebraska, (4.2 miles).
	5. Burnside Road, (Eastman Parkway to Powell Valley Road), Gresham, Oregon (2.3 miles).	5. Interstate 94, (at 41st. Ave.), Minneapolis, Minnesota, (NCHRP 3-45 - MN10), (spot).
	6. Fremont Street (Canyon Del Rey Blvd. to Highway One ramps) - Seaside, California (1.9 miles)	6. Interstate 395, (at Glebe Rd.), Alexandria, Virginia, (NCHRP 3-45 - VA02), (spot).
Rural	1. State Route 28, (Route 124 to Route 6A), Cape Cod, Massachusetts, (18.1 miles).	1. State Highway 101, (Exit 3 to Exit 4), Manchester, New Hampshire. (6.4 miles).
	2. State Route 6, (Route 6a to Truro), Cape Cod, Massachusetts, (18.3 miles).	2. State Route 99, at Grantline Road (M.P. 10.07), Sacramento County, California, (spot).
	3. State Highway 82, (Island City to Enterprise), - Oregon, (44.5 miles).	3. State Route 99, at Turner Station/French Camp Road (M.P. 12.53), San Joaquin County, California, (spot).
	4. U.S. 1, (Cow Key Channel Bridge to Key Haven Blvd.), Monroe County, Florida, (1.1 miles).	4. State Route 101, at Loleta Drive (M.P. 65.54), Humboldt County, California, (spot).
	5. U.S. 1, (Ocean Blvd. to Atlantic Blvd.), Monroe County, Florida, (8.0 miles).	5. Interstate 5, Mile Post 182.05 (about 10 miles south of Eugene), Oregon, (spot).
	6. State Highway 101, (County Farm Road to State Highway 125), Brentwood Corners, New Hampshire. (2.0 miles).	6. Interstate 84, Mile Post 171.13 (approximately 3 miles east of Boardman), Oregon (spot).

Nebraska Department of Roads, and field surveys conducted by UNL.

6.3.1.4 Ventura Boulevard, Los Angeles, California

The selected study section of Ventura Boulevard is an 8.2-mi (13.1-km), four-lane and six-lane signalized arterial between Topanga Canyon Boulevard and Sepulveda Boulevard in Los Angeles, California. The terrain is flat. The posted speed limit is 35 mph (56 km/hr). The facility carries 45,000 ADT.

Hourly traffic volume data were collected for four hours (7 a.m. to 11 a.m.) on Tuesday, December 7, 1993. The count data were obtained from loop detectors on the approaches to the signalized intersections on Ventura Boulevard. Hourly turning movement volumes at the signalized intersections were estimated from morning peak-hour turning movement counts from the two previous years and from the loop detector data for the test day. The turning movement counts were adjusted to match the total approach volumes measured by the loop detectors. The average adjustment to the older turning movement counts was an 11 percent increase for westbound volumes and a 16 percent decrease for the eastbound volumes on Ventura Boulevard. Cross-street volumes generally needed no adjustment. All count data were provided by the Signal Systems and Research Section of the city of Los Angeles. (Contact Brian Gallagher at 213/485-4272 for more information.)

Twelve floating car runs were made in each direction on Ventura Boulevard between 7 a.m. and 11 a.m. on Tuesday, December 7, 1993. The mean speeds measured by the float-

TABLE 6-3 Summary of dataset facility characteristics

Route	State	AADT	Length (mi./km.)	Number of Lanes
Urban Interrupted Flow Facilities				
1. U.S. 1 (Martin County)	FL	36,500	3.8 / 6.1	4
2. SR 808/Glades	FL	45,000	1.3 / 2.1	6
3. Dodge Corridor, Omaha	NE	56,000	2.7 / 4.3	4/6
4. Ventura Boulevard, Los Angeles	CA	45,000	8.2 / 13.2	4/6
5. Burnside Road, Gresham	OR	54,000	2.3 / 3.7	4
6. Fremont Street, Seaside	CA	28,000	1.9 / 3.1	4
Urban Uninterrupted Flow Facilities				
1. Interstate 5, Portland	OR	106,000	6.0 / 9.7	4/6
2. U.S. 101 ,Los Angeles	CA	300,000	10.6 / 17.1	8
3. Interstate 880, Hayward	CA	200,000	9.2 / 14.8	6/10
4. Interstate 80, Omaha	NE	N/A	4.2 / 6.8	6
5. Interstate 94, Minneapolis	MN	N/A	spot	4/8
6. Interstate 395, Alexandria	VA	N/A	spot	4
Rural Interrupted Flow Facilities				
1. Route 28, Cape Cod	MA	13,000	18.1 / 29.1	2
2. Route 6, Cape Cod	MA	25,000	18.3 / 29.5	4
3. Highway 82, Elgin	OR	4,000	44.5 / 71.6	2
4. U.S. 1 (Monroe County #1)	FL	30,000	1.1 / 1.8	4
5. U.S. 1 (Monroe County #2)	FL	20,000	8.0 / 12.9	4
5. Highway 101 (Brentwood Corners)	NH	14,000	2.0 / 3.2	2
Rural Uninterrupted Flow Facilities				
1. Highway 101 (Candia Station)	NH	19,000	6.4 / 10.3	4
2. State Route 99 (Sacramento County)	CA	20,000	Spot	4
3. State Route 99 (San Joaquin County)	CA	22,000	Spot	4
4. U.S. 101 (Humboldt County)	CA	8,000	Spot	4
5. Interstate 5 (MP 182.05)	OR	30,000	Spot	4
6. Interstate 84 (MP 171.13)	OR	8,000	Spot	4

N/A = not available or not applicable.

ing cars were 24.9 mph (40.1 km/hr) eastbound and 25.1 mph (40.4 km/hr) westbound. The lowest speeds were observed between 7 a.m. and 8 a.m. The floating car data were collected by Dowling Associates as part of a research effort to update the DTIM2 model for the California Department of Transportation (Caltrans). (Contact Richard Dowling at 510/839-1742 for more information.)

6.3.1.5 Burnside Road, Gresham, Oregon

The selected study section of Burnside Road is a 2.3-mi (3.7-km) section of four-lane signalized arterial between Eastman Parkway and Powell Valley Road in Gresham, Oregon. The posted speed limit is 35 mph (56 km/hr). This facility carries 45,000 ADT. Travel time runs were conducted in March 1995 as part of a signal system project conducted by Kittelson & Associates, Inc., for the city. (Contact Kent Kacir at 503/228-5230 for more information.)

6.3.1.6 Fremont Boulevard, Seaside, California

The selected study section of Fremont Boulevard is a 1.9-mi (3.1-km) section of four-lane signalized urban arterial in Seaside, California. The terrain is flat. The posted speed limit is 35 mph (56 km/hr). The facility carries 28,000 ADT. This dataset was compiled by the city of Seaside and DKS Associates as part of a Fuel Efficient Traffic Signal Management project for the city.

6.3.2 Urban Uninterrupted Flow Facilities

6.3.2.1 Interstate 5, Portland, Oregon

The selected study section of I-5 is a 6.0-mi (9.7-km) section of four-lane and six-lane urban freeway between I-84 and Columbia Boulevard in Portland, Oregon. The terrain generally is rolling to flat. The posted speed limit is 55 mph (89 km/hr), except for some sections where a 50 mph (80 km/hr) speed limit is posted. This section of I-5 carries about 106,000 ADT on average. It is a major commuter route.

The dataset was assembled by Kittelson & Associates, Inc., for a 1991 study of the I-5 and I-205 corridors across the Columbia River. (Contact Wayne Kittelson at 503/228-5230 for more information.) Speed and volume data were collected in April 1989. Spot speed data were provided by the Oregon Department of Transportation from inductive loops on the I-5 mainline, which are used for ramp metering.

6.3.2.2 Interstate 880, Hayward, California

The selected study section of I-880 is a 9.2-mi (14.7-km), 6-lane and 10-lane urban freeway between Marina Boulevard and Whipple Street in Hayward, California. The terrain is flat. The posted speed limit was 55 mph (89 km/hr) at the time of the study.

This section of I-880 carries 200,000 ADT. It is a major commuter route with significant truck volumes. There is little recreational traffic.

The data were collected by the Institute of Transportation Studies, University of California, Berkeley. The field data on speed were gathered during spring and fall of 1993 using loop detectors and floating cars. Other data were gathered from loop detectors provided by Caltrans within the study segments.

6.3.2.3 U.S. 101, Ventura Freeway, Los Angeles, California

The segment of U.S. 101, Ventura Freeway, selected for study extends for 10.6 mi (17.0 km) between Topanga Canyon Boulevard and Sepulveda Boulevard in Los Angeles, California. This segment of U.S. 101 is an eight-lane freeway in an urbanized area on generally flat terrain. The posted speed limit was 55 mph (89 km/hr) at the time of the study. All on-ramps are metered during peak periods.

The facility carries 300,000 ADT. It is a major commuter route with little seasonal variation in traffic volumes.

Traffic volume and speed data were collected for a four-hour period during the morning peak hours (7 a.m. to 11 a.m.). Volume data were collected from in-place mainline loop sensors. Speed data also were collected at these mainline loop sensors and by floating car surveys.

Traffic volume data for the study segment were obtained from the Caltrans Traffic Operations Center for seven loop sensors in the northbound direction and eight loop sensors in the southbound direction. The data, which were collected as part of the Caltrans project to update the Direct Travel Impact Model (DTIM), were obtained through Dowling Associates. Floating car data were collected by Dowling Associates as part of a research effort to update the DTIM2 model for Caltrans. (Contact Richard Dowling at 510-839-1742.)

6.3.2.4 Interstate 80, Omaha, Nebraska

The selected study section of I-80 is a 3.2-mi (5.2-km) section of six-lane urban freeway between 72nd Street and I-480 in Omaha, Nebraska. The terrain is flat. The posted speed limit was 55 mph (89 km/hr) at the time of the study.

Travel time runs were conducted in 1993 by researchers from the University of Nebraska at Lincoln (UNL) as part of a project to evaluate the use of CORFLO for traffic congestion management planning in the Omaha–Council Bluffs metropolitan area. Roadway geometric and control data were obtained from the city of Omaha, the Omaha–Council Bluffs Metropolitan Area Planning Agency, the Nebraska Department of Roads, and field surveys conducted by UNL.

6.3.2.5 Interstate 94, Minneapolis, Minnesota

The selected study section of I-94 is a point in the southbound direction of a four-lane urban freeway located at the 41st Avenue interchange in Minneapolis, Minnesota. The posted speed limit was 55 mph (89 km/hr) at the time of the study. All lanes are 12 ft (3.66 m) wide. Lateral clearance is 12 ft (3.66 m) on both the left and right sides. The median is a barrier. There are 0.83 ramps per mile (0.52 per kilometer) and 0.66 interchanges per mile (0.41 per kilometer). There is ramp metering, and 1.96 percent of traffic is heavy vehicles.

There is a barrier-separated reversible pair of HOV lanes in the median, which has been excluded from the speed-flow data collection used in this study. The speed-flow data were extracted from charts in *NCHRP Project 3-45: Speed-Flow Relationships for Basic Freeway Segments*, Final Report, May 1995.

6.3.2.6 Interstate 395, Alexandria, Virginia

The selected study section of I-395 is a point in the northbound direction on a four-lane urban freeway located at the Glebe Road interchange in Alexandria, Virginia. The posted speed limit was 55 mph (89 km/hr) at the time of the study. The lanes are 12-ft (3.66-m) wide. Lateral clearance is 2 ft

(0.61 m) on the left side and 6 ft (1.83 m) on the right side. Ramp metering is present. There are 1.8 ramps per mile (1.12 per kilometer) and 0.67 interchanges per mile (0.42 per kilometer).

The speed-flow data were extracted from charts in *NCHRP Project 3-45: Speed-Flow Relationships for Basic Freeway Segments*, Final Report, May 1995.

6.3.3 Rural Interrupted Flow Facilities

6.3.3.1 Route 28, Cape Cod, Massachusetts

Route 28 is a two-lane rural road at its eastern end, between Harwich and Orleans, and parallels U.S. 6 in the Cape Cod area. The segment selected for the dataset extends for 18.1 mi (29.1 km) between Route 124 (near Harwich) and Route 6A (near Orleans). The posted speed limit is 35 mph (56 km/hr).

This segment of Route 28 carries 8,000 to 15,000 ADT. Traffic peaks during the summer months. Harwich, near the western end of the study segment, has a year-round population of 11,000. Orleans has a year-round population of 6,100. Summertime populations are double the year-round populations in this area.

Travel time runs were performed during July and August of 1995 by the Cape Cod Commission. (Contact Leo Malakhoff at 508/362-3828 for more information.)

6.3.3.2 Route 6, Cape Cod, Massachusetts

U.S. 6 is the only access route to Provincetown on the point of Cape Cod, Massachusetts. The segment included in the dataset extends from Orleans to a few miles north of Truro. The study segment is a four-lane undivided highway extending 18.3 mi (29.5 km) and carrying 25,100 ADT. Traffic peaks during the summer months on this major recreational route. Traffic signals are spaced about 3 to 5 mi apart (5 to 8 km) on this facility. The posted speed limit is 45 mph (72 km/hr).

This segment of U.S. 6 passes through the towns of Eastham (population 4,900), Wellfleet (population 2,900), and Truro (population 1,700). Provincetown has a population of 3,900; Orleans has a population of 6,100. All these population figures include year-round residents only. The population doubles during the summer months.

Travel time runs were performed during July and August of 1995 by the Cape Cod Commission. (Contact Leo Malakhoff at 508/362-3828 for more information.)

6.3.3.3 Highway 82, Northeastern Oregon, Wallowa Lake Highway

Route 82 is a two-lane rural road with a speed limit of 55 mph (89 km/hr) between Island City and Enterprise in northeastern Oregon. The two segments selected for the study extend for 16.2 mi (26.1 km) between Island City and Elgin and for 28.4 mi (45.7 km) between Minam and Enterprise.

A travel time survey was conducted on July 28, 1995, on what historically is the highway's heaviest travel day of the year, the Friday before the Chief Joseph Days festival in Joseph. Three to six floating car runs were made in each direction, depending on the length of the segment. Mean speeds ranged from 50.0 to 59.5 mph (80.5 to 95.8 km/hr) in the peak direction and from 51.1 to 57.0 mph (82.2 to 91.7 km/hr) in the off-peak direction. The standard deviation of mean speeds ranged from 0.4 to 8.1 mph (0.6 to 13.0 km/hr) eastbound and from 1.0 to 11.0 mph (1.6 to 17.7 km/hr) westbound. Hourly traffic volume data were collected by manual counters on the same day.

Kittelson & Associates, Inc., conducted the study for OTAK, Inc., and the Oregon Department of Transportation as part of a passing lane analysis for the highway using the TRARR model. (Contact Alan Danaher at 503/228-5230 for more information.)

6.3.3.4 U.S. 1, Monroe County #1, Florida

U.S. 1 in Monroe County, Florida, is a four-lane divided rural highway with posted speed limits ranging from 30 to 45 mph (48 to 72 km/hr). The segment selected for study extends for 1.1 mi (1.8 km) between the north end of the Cow Key Channel Bridge and Key Haven Boulevard. This segment carries 34,400 ADT.

The dataset was compiled by Barton-Aschman as part of the Monroe County Travel Time Survey. The data were secured during March 1995.

6.3.3.5 U.S. 1, Monroe County #2, Florida

U.S. 1 in Monroe County, Florida, is a four-lane divided rural highway with posted speed limits ranging from 35 to 55 mph (55 to 90 km/hr). The segment selected for study extends for 8.0 mi (12.9 km) between Ocean Boulevard in Tavernier and Atlantic Boulevard in Key Largo. This segment carries 20,300 ADT.

The dataset was compiled by Barton-Aschman as part of the Monroe County Travel Time Survey. The data were secured during March 1995.

6.3.3.6 State Highway 101, New Hampshire

State Highway 101 is a two-lane rural highway between County Farm Road and Highway 125, near Brentwood Corners in southeastern New Hampshire. The segment selected for study is 2.0 mi (3.2 km) long. The posted speed limit is 50 mph (81 km/hr). This segment carries 14,000 ADT.

Travel time runs were conducted in June 1990 by the New Hampshire State Department of Transportation.

6.3.4 Rural Uninterrupted Flow Facilities

6.3.4.1 State Highway 101, New Hampshire

State Highway 101 is a four-lane rural freeway in the segment selected for study between Exit 3 (near Candia Station) and Exit 4 (near Raymond) in southeastern New Hampshire. The segment selected for study is 6.4 mi (10.2 km) long. The posted speed limit is 65 mph (105 km/hr). This segment carries 18,500 ADT.

Travel time runs were conducted in June 1990 by the New Hampshire State Department of Transportation.

6.3.4.2 State Route 99 at Grantline Road, Sacramento County, California

State Route 99 at Grantline Road is a four-lane freeway in a rural area of southern Sacramento County, California. The terrain is level. The posted speed limit was 55 mph (89 km/hr) at the time of the study. The study was conducted at milepost 10.07 of the freeway, which is at the Grantline Road interchange.

The facility carried 20,250 AADT in 1992, when the survey was conducted. The facility is a major agricultural route in the Central Valley. Traffic volumes peak in summer and fall during harvest time. Recreational traffic is not a major component of daily traffic.

The study was conducted during a 24-hr period on August 15, 1992, by Caltrans, as part of its 55 mph compliance reporting effort. Speed and volume were measured by means of an inductance loop placed at the survey location. The data are reported by hour of day, with vehicles classified by 5-mph (8-km/hr) increments between zero and 80 mph (129 km/hr).

6.3.4.3 State Route 99 at French Camp Road, San Joaquin County, California

State Route 99 at French Camp Road is a four-lane freeway in a rural area of San Joaquin County, California. The terrain is flat. The posted speed limit was 55 mph (89 km/hr) at the time of the study.

The facility carried 21,500 AADT in 1992. Traffic peaks in summer. Truck traffic peaks in fall, during harvest season. Recreational traffic is not a significant component of total traffic.

The data were collected at the Turner Station–French Camp Road interchange, at milepost 12.53 of the facility. The study was conducted during 4 days in 1991 and 1992 by Caltrans, as part of its 55 mph compliance reporting effort. Each day's data were collected for a 24-hr period. Speed and volume were measured by means of an inductance loop placed at the survey location. The data are reported by hour of day, with vehicles classified by 5-mph (8-km/hr) increments between zero and 80 mph (129 km/hr).

6.3.4.4 State Route 101 at Loleta Drive, Humboldt County, California

State Route 101 at Loleta Drive is a four-lane freeway in a rural area of Humboldt County in northwestern California. The terrain is rolling. The posted speed limit was 55 mph (89 km/hr) at the time of the study.

The facility carried 8,450 AADT in 1992. Traffic peaks in summer. Truck traffic primarily consists of logging trucks. Recreational traffic is a significant component of total traffic during summer.

The data were collected at the Loleta Drive interchange, at milepost 65.54 of the facility. The study was conducted during 4 days in 1991 and 1992 by Caltrans, as part of its 55 mph compliance reporting effort. Each day's data were collected for a 24-hr period. Speed and volume were measured by means of an inductance loop placed at the survey location. The data are reported by hour of day, with vehicles classified by 5-mph (8-km/hr) increments between zero and 80 mph (129 km/hr).

6.3.4.5 Interstate 5 (Milepost 182.05), Oregon

This section of I-5 is a four-lane freeway located 10 mi south of Eugene, Oregon. The terrain is fairly level. The posted speed limit was 55 mph (89 km/hr) at the time of the study. The design speed is 75 mph (121 km/hr).

The facility carries about 30,000 AADT. The surrounding area is rural and heavily forested.

6.3.4.6 Interstate 84 (Milepost 171.13), Oregon

This section of I-84 is a four-lane freeway located 3 mi east of Boardman, Oregon. The terrain is fairly level. The posted speed limit was 55 mph (89 km/hr) at the time of the study. The design speed is 75 mph (121 km/hr).

The facility carries about 8,000 AADT. The surrounding area is rural. The nearest urbanized area is 20 mi away.

REFERENCES

1. *Special Report 209: Highway Capacity Manual.* TRB, National Research Council, Washington D.C., 1994.
2. *Census of Population and Housing: Summary Population and Housing Characteristics, California.* Bureau of the Census, U.S. Department of Commerce, Washington D.C., August 1991, p. A-11.
3. *Level of Service Manual.* Systems Planning Office, Florida Department of Transportation, Tallahassee, August 1995, p. 2-3.

CHAPTER 7

CRITIQUE OF EXISTING METHODS

This chapter evaluates existing planning techniques for predicting speed and service volume of road facilities. The purpose of this chapter is to identify the gaps and shortfalls in the techniques currently being used by planning agencies and private firms.

7.1 CRITERIA

The research problem statement suggested six criteria for the evaluation of existing planning techniques: data requirements, ease of use, reliability, user confidence, general acceptance, and significant deficiencies. The work plan suggested four additional criteria: comparability to the HCM, adaptability to the HCM, range of facilities and applications, and sensitivity to transportation control measures. These additional criteria have been included within the definitions of the original six criteria suggested in the research problem statement, as explained in the following paragraphs.

- Data requirements—Data requirements include the amount and type of data required by the technique. Ease of data collection is assessed based on the results of the user survey. The required precision of the input data is assessed speculatively at this stage.
- Ease of use—Ease of use deals with (1) the complexity of the technique, (2) how difficult it is to learn, (3) whether it can be implemented in a spreadsheet, and (4) whether it requires iterations to reach a solution.
- Reliability—Reliability is a complex criterion that consists of (1) the accuracy of the technique, (2) the range of facility types and area types to which the technique can be reliably applied, and (3) the range of planning applications of the technique. Sensitivity to transportation control measures is covered in the range of planning applications for the technique.
- User confidence and general acceptance—These two criteria have slightly different meanings, but both relate to overall user acceptance of the technique. User confidence reflects the user's experience and confidence in the reliability of results. General acceptance reflects the number of users of a technique and the number of applications to which the technique is applied. Comparability to the HCM is included under this criterion.
- Significant deficiencies—Significant deficiencies include both theoretical and practical application deficiencies of the technique.

7.2 SPEED ESTIMATION TECHNIQUES

This section evaluates existing planning techniques for predicting average vehicle speed. The existing techniques are the volume/capacity (v/c) ratio technique and HCM techniques. Conclusions are highlighted in Table 7-1.

7.2.1 Volume/Capacity Ratio Technique

Volume/capacity ratio curves such as the BPR curve are ideal for regional travel forecasting models. Speed is a function of three variables: free-flow speed, volume, and capacity. It is easy to code this limited information for the thousands of links in a typical regional model. The BPR curve also is a monotonic increasing function, which is required for finding equilibrium. The curve, however, was calibrated against old highway data from the 1965 HCM. The curve needs to be updated in light of the 1994 HCM and NCHRP Project 3-45.

The BPR curve has not been validated for queuing situations (v/c > 1), and it is not accurate for interrupted flow facilities because it does not include signal timing parameters, which can have as great an impact on signalized arterial speed as does the v/c ratio. In addition, planners use look-up tables for free-flow speed and capacity based on unclear definitions of facility type and area type.

7.2.1.1 Data Requirements

The v/c ratio method requires a minimal amount of data. Volume, area type, number of lanes, facility type, and a pair of look-up tables for determining facility capacity and free-flow speed are all that are necessary for determining level of service with this method.

The method is highly sensitive to estimated free-flow speed and estimated capacity. A 10 percent error in estimated capacity translates into a 19 percent change in predicted speed at a v/c ratio of 1. The potential difference, however, drops to zero at a low v/c ratio. A 10 percent error in estimated free-flow speed will bias the predicted mean speed by the same 10 percent.

Field measurements of free-flow speed and capacity can significantly improve the accuracy of this method. The RMS error and bias can be cut in half by field measurements of free-flow speed and capacity. In addition, all basic data are readily available to planning agencies.

TABLE 7-1 Evaluation of speed estimation techniques

Criteria	Techniques: Volume/Capacity Curves	Techniques: Highway Capacity Manual
1. Data Requirements		
Amount	-Volume, capacity, free speed	-Volume, free speed, plus numerous additional facility characteristics.
Precision	-A 10% error in volume or capacity translates into a 19% change in the estimated speed at v/c = 1.00	-Complexity of procedures makes it difficult to determine impacts of data errors.
Feasibility	-All required data are feasible for all agencies to easily obtain.	-40% of MPO's indicated it is unfeasible to obtain some of the required data items (% heavy vehicles, quality of coordination were most difficult).
2. Ease of Use		
Complexity	-Single equation	-Multiple equations
Training Required	-Few minutes to learn	-One-day training
Spreadsheet	-Spreadsheet friendly	-Adaptable to spreadsheets, but figures must be translated to look-up tables.
3. Reliability		
Accuracy	-Not accurate at high v/c ratios	-Most accurate of available techniques not in traffic model software
Facilities	-All, but not reliable for interrupted flow facilities	-No planning technique for uninterrupted flow facilities systems
Area Types	-All	-Interrupted flow technique designed for only urban application. -Rural road procedure limited to 60 mph design speed.
Planning Applications	-Good only for RTP models.	-Good for all except RTP models
4. User Confidence and Acceptance		
Overall Use	-Used by 22% of respondents.	-Used by 33% of respondents
Planning Applications	-Predominant technique for RTP's	-Predominant technique for Site Impact and Congestion Management
Agencies	-Most popular with MPO's. -Least popular with local agencies	-Most popular with state DOT's. -Least popular with MPO's
Geographic Spread	-Used throughout USA.	-Most frequently used across the country but less popular on west coast
5. Significant Strengths and Deficiencies		
Strengths	-Simple, quick, well behaved function	-Comprehensive, sensitive to many factors.
Deficiencies	1. Not accurate at v/c's > 1.00 2. Needs to be refitted to new HCM data 3. Not sensitive to signal timing.	1. Extensive data required 2. Complex procedures 3. No procedure for freeway systems 4. Can't do v/c > 1.00 5. Rural roads procedure limited

7.2.1.2 Ease of Use

The v/c ratio method is extremely simple to apply. The method, which consists of a single equation and two look-up tables, can be learned in a few minutes. This method also is spreadsheet-friendly.

7.2.1.3 Reliability

The v/c ratio method is not particularly accurate nor reliable. The method typically is applied to all facilities in all areas, but it is not particularly reliable for interrupted flow facilities because it does not include signal timing variables.

The method is ideally suited for regional traffic forecasting models because of its simplicity. It generally is not used for planning applications that do not use a regional traffic forecasting model.

7.2.1.4 User Confidence and General Acceptance

The v/c ratio method is used throughout the United States by regional transportation modelers. About 22 percent of the respondents to the user survey use the v/c ratio method to predict speed. The method is primarily used for regional transportation plan analyses.

The v/c ratio method is favored by MPOs, which must work with regional traffic forecasting models. It is used less frequently by local agencies. The method is used throughout the United States.

7.2.1.5 Significant Strengths and Deficiencies

The v/c ratio method is popular because of its simplicity. Its major deficiencies are as follows:

- The v/c ratio speed-flow curve is fitted to uncongested data from the 1965 HCM, which uses an old term, "practical capacity," that is no longer used in modeling. The speed-flow curve needs to be updated with 1994 HCM data and current terminology.
- The method does not accurately model speeds for v/c ratios greater than 1. Fixing this problem may conflict with modelers' desires for a relatively flat curve that allows models to converge to equilibrium much more rapidly. The trade-off may be convenience and tractability versus accuracy.
- The method is not sensitive to the impact of signal spacing, timing, and coordination, which can significantly influence the level of service for interrupted flow facilities.

7.2.2 1994 Highway Capacity Manual Techniques

The 1994 HCM provides three techniques for uninterrupted flow facilities: freeways, multilane highways, and two-lane rural roads. The HCM also provides one technique for interrupted flow facilities: urban and suburban arterials. These techniques are procedures rather than a single equation such as the BPR curve. They require a great deal more data on facility geometry and traffic flow characteristics.

Although the urban and suburban arterials procedure can be used to predict average travel speed over the length of a road, no equivalent procedure exists for freeways. Chapter 6 of the 1994 HCM outlines a procedure for computing level of service by sections of freeway, but not for computing average speed over the length of the facility. Current ramp analysis procedures apply only to the two rightmost lanes of the freeway; therefore, the basic section, ramp, and weaving methodologies cannot be combined by planners into an analysis over the entire length of a freeway.

Unlike the BPR curve and its variations, which can be applied over an infinite range of v/c ratios, HCM techniques are limited to v/c ratios of 1.2 or 1.0, depending on facility type. The interrupted flow (urban arterial) procedure of the 1994 HCM has been found by more then one investigator to uniformly underestimate floating car average travel speed by about 20 percent. The average speeds estimated by this procedure do not include turning traffic on the arterial. The length of queues at signalized intersections are not taken into account in the computation of average speeds in this procedure.

7.2.2.1 Data Requirements

HCM procedures can require a great deal of data not normally available to planners; however, much of this problem can be overcome through the use of defaults. HCM methods require volume and free-flow speed plus an assortment of facility-specific details for the purpose of computing capacity and level of service.

HCM methods are sensitive to a variety of factors, many of which are, in practice, estimated by the analyst instead of actually measured in the field. Analysts often rely on the default values recommended in the HCM. The use of field measured data rather than defaults can significantly improve the accuracy of results. For example, Dowling found an average 30 percent reduction in error in estimated signal delay when using field measured parameters rather than defaults (*1*).

Even though basic data (volumes and speeds) are obtainable by most agencies, some required data items (e.g., percent heavy vehicles and quality of signal coordination) are infeasible to obtain by 40 percent of the MPOs responding to the user survey.

7.2.2.2 Ease of Use

HCM procedures generally require software to assist the user in their implementation. These procedures usually require two or three steps and equations to compute level of service and maximum service flow. Training for the entire set of HCM planning and operations methods often requires 2 to 3 days. The planning procedures themselves, however, can be learned in a single day.

HCM techniques are designed as a linear sequence of steps to facilitate the use of spreadsheets in computations; however, some of the required steps involve reading data from charts for which no equations or look-up tables are provided. The user must fit equations to these charts in order to use a spreadsheet.

7.2.2.3 Reliability

HCM methods are among the most accurate methods available for estimating level of service. Unfortunately, no planning method is provided for analyzing freeway systems composed of basic sections, ramp merge sections, and weaving sections. Planners must use the basic section methodology for all freeway sections. The interrupted flow procedure (urban and suburban arterials) is addressed to urban, not rural, situations. HCM methods are applicable to most planning applications, with the exception of RTP analyses whose complexity precludes the use of HCM methods in regional traffic forecasting models.

7.2.2.4 User Confidence and General Acceptance

HCM methods have a high level of user confidence and acceptance throughout the United States. About 34 percent of the respondents to the user survey use these methods to predict level of service. HCM methods are useful in all planning applications except RTP analyses. The methods are used most frequently for site impact and congestion management studies. HCM methods are strongly favored by state DOTs, but less so by MPOs, which must work with regional traffic forecasting models.

HCM methods are used throughout the United States. The West Coast, however, appears to lag behind the rest of the nation in accepting and applying HCM methods to planning applications. Field measurements of speed are preferred to HCM methods by West Coast agencies.

7.2.2.5 Significant Strengths and Deficiencies

HCM methods are comprehensive and sensitive to many system operation parameters. Their major deficiencies are as follows:

- HCM methods require extensive data to ensure reliable results.
- HCM methods are complex and difficult to apply without specialized software.
- There is no planning procedure in the HCM for analyzing the overall level of service of a freeway containing more than just basic sections. The state of Florida, however, has developed service volume tables and software based on the HCM for evaluating overall level of service for freeways.
- The planning procedure for interrupted flow facilities is oriented toward urban conditions. No separate procedure or modification is provided for rural facilities. Florida, however, has developed planning procedures for rural facilities.

7.3 LEVEL OF SERVICE AND SERVICE VOLUME ESTIMATION TECHNIQUES

This section evaluates the three major techniques currently being used to estimate level of service: v/c ratio, HCM methods, and the Florida *Level of Service Manual* method. Table 7-2 highlights the conclusions.

7.3.1 Volume/Capacity Ratio

Volume/capacity ratios are easy to obtain from regional traffic forecasting models; therefore, they are used for RTP analyses. There are general correspondences between v/c ratios and level of service for uninterrupted flow facilities (freeways, multilane highways, and rural two-lane roads) if the information necessary for selecting the appropriate level of service cutoff is known. However, there is no direct correspondence between v/c ratios and level of service for interrupted flow facilities (urban and suburban arterials) unless signal timing is known.

7.3.1.1 Data Requirements

The v/c ratio method requires a minimal amount of data. Volume, area type, number of lanes, facility type, and a lookup table for determining facility capacity are all that are necessary for determining level of service by means of this method.

Volume/capacity ratios are highly correlated to level of service for most facility types. The v/c ratio, however, is not a very reliable indicator of level of service for urban arterials. A wide range of levels of service may occur at a given v/c ratio. The appropriate average v/c ratio for a facility is another issue to be considered. For instance, is the mean v/c ratio, median v/c ratio, or critical (highest) v/c ratio the best indicator of overall facility level of service?

All basic data are readily available to planning agencies.

7.3.1.2 Ease of Use

The v/c ratio method is extremely simple to apply. The method consists of a single measure of effectiveness and a single equation, can be learned in a few minutes, and is spreadsheet-friendly.

7.3.1.3 Reliability

The v/c ratio method is not particularly accurate nor reliable. The v/c ratio cutoffs are arbitrary and unrelated to the level of service criteria in the HCM. The method typically is applied to all facilities in all areas, but it is not particularly reliable for interrupted flow facilities because it does not include

TABLE 7-2 Evaluation of level of service estimation techniques

	Techniques		
Criteria	Volume/Capacity Curves	Highway Capacity Manual	Florida LOS Manual
1. Data Requirements			
Amount	-Volume, area type, and facility type.	-volume, free speed, plus numerous additional data on facility characteristics.	- Volume, facility type, area type, and a few additional facility characteristics
Precision	-A 10% error in volume or capacity translates into one level of service error	-Complexity of procedures make it difficult to determine impacts of data errors.	-6% error in arterial volume (20% for freeways) causes error of one level of service.
Feasibility	-All required data are feasible for all agencies to easily obtain.	-40% of MPO's indicated it is infeasible to obtain some of the required data items (% heavy vehicles, quality of coordination were most difficult).	-all input data are feasible except for signal spacing. About 25% of MPO's said they could not get signal spacing.
2. Ease of Use			
Complexity	-Single equation, -Single LOS measures	-Multiple equations, -Multiple LOS measures	-Volume look-up table.
Training Required	-Few minutes to learn	-One-day training	-Half-day training
Spreadsheet	-Spreadsheet friendly	-Figures must be translated to equations.	-Spreadsheet friendly
3. Reliability			
Accuracy	-Not accurate	-Most accurate of available techniques	-Same accuracy as HCM
Facilities	-All, but not reliable for interrupted flow	-No technique for uninterrupted flow facilities systems	-Applicable to all facilities
Area Types	-All	-Interrupted flow technique only for urban application.	-Applicable to all areas
Planning App's	-Good only for RTP models.	-Good for all except RTP models	-Applicable for all except RTP models
4. User Confidence and Acceptance			
Overall Use	-Used by 28% of respondents.	-Used by 34% of respondents	-Florida method used by 6% of respondents, but service volume techniques used by additional 23%.
Planning Applications	-Used mostly for RTP's	-Predominant technique for Site Impact and Congestion Management	-Used for site impact and congestion management
Agencies	-Most popular with MPO's. -Least popular with local agencies	-Most popular with state DOT's. -Least popular with MPO's	-Used mostly by local agencies.
Geographic Spread	-Used across the USA	-Used throughout USA but less popular on west coast	-Predominantly used in Florida.
5. Significant Strengths and Deficiencies			
Strengths	-Simple, quick	-Comprehensive, sensitive to many factors.	-Simple, adapts to unique conditions, based on HCM.
Deficiencies	1. v/c not related to level of service. 2. No signal timing and other parameters.	1. Extensive data required, Complex procedures 2. No procedure for freeway systems	1. LOS standards for low design speed roads not achievable.

signal timing variables. The method is ideally suited for regional traffic forecasting models because of its simplicity.

7.3.1.4 User Confidence and General Acceptance

The v/c ratio method is used throughout the United States by regional transportation modelers seeking a simple way to translate their traffic forecasts into level of service. About 28 percent of the respondents to the user survey use the v/c ratio method to predict level of service. The method is primarily used for RTP analyses.

The v/c ratio method is favored by MPOs, which must work with regional traffic forecasting models. It is used less frequently by local agencies. The method is used throughout the United States.

7.3.1.5 Significant Strengths and Deficiencies

The v/c ratio method is popular because of its simplicity. The method's major deficiencies are as follows:

- Because the v/c ratio is not used as a measure of level of service in the HCM, there is little or no correspondence between the levels of service predicted using the v/c ratio method and HCM methods. There is a relationship between v/c ratios and level of service for uninterrupted flow facilities, but the planner must take into account additional data on facility characteristics (such as number of lanes) to correctly translate v/c ratios into level of service. The standard 60, 70, 80, and 90 percent v/c ratio cutoffs often used by planners do not apply if they want to match the HCM level of service results.
- The method is not sensitive to the impact of signal spacing, timing, and coordination, which can significantly influence the level of service for interrupted flow facilities.

7.3.2 1994 HCM

HCM methods require more data and computational steps than the v/c ratio method to obtain level of service and service volume. Explicit procedures for computing service volume are provided for uninterrupted flow facilities in urban areas. However, no such procedure is provided for interrupted flow facilities.

7.3.2.1 Data Requirements

HCM methods can require a great deal of data not normally available to planners. Much of this problem, however, can be overcome through the use of defaults. HCM methods require volume and free-flow speed plus an assortment of facility-specific data for the purpose of computing capacity and level of service.

The complexity of HCM methods makes it difficult to assess the impact of input data errors on the accuracy of results. As previously discussed, the use of field measurements instead of default values in the HCM method for estimating signalized intersection delay can improve the accuracy of the estimated signal delay by 30 percent.

Although basic data (volume and speed) are obtainable by most agencies, some required data items (e.g., percent heavy vehicles and quality of signal coordination) are infeasible to obtain by 40 percent of the MPOs responding to the user survey.

7.3.2.2 Ease of Use

HCM methods generally require software to help users with their implementations. These methods usually require two or three steps and equations to compute level of service and maximum service flow.

Training for the entire set of HCM planning and operations methods often requires a 2- to 3-day course. The planning procedures themselves, however, can be learned in a single day.

HCM techniques are designed as a linear sequence of steps to facilitate the use of spreadsheets in making computations; however, some of the required steps involve reading data off charts for which no equations or look-up tables are provided. The user, therefore, must fit equations to these charts in order to use a spreadsheet.

7.3.2.3 Reliability

HCM methods are among the most accurate methods available for estimating level of service. Unfortunately, no planning method is provided for analyzing freeway systems composed of basic sections, ramp merge sections, and weaving sections. Planners must use the basic section methodology for all freeway sections. The interrupted flow procedure (urban and suburban arterials) is addressed to urban, not rural, situations. HCM methods are applicable to most planning applications, with the exception of RTP analyses, for which their complexity precludes their use in regional traffic forecasting models.

7.3.2.4 User Confidence and General Acceptance

HCM methods have a high level of user confidence and acceptance throughout the United States. About 34 percent of the respondents to the user survey use HCM methods to predict level of service.

HCM methods are useful in all planning applications, except RTP analyses. The methods are used most frequently for site impact and congestion management studies. HCM methods are strongly favored by state DOTs and less so by MPOs, which must work with regional traffic forecasting models.

HCM methods are used throughout the United States. The West Coast, however, appears to lag behind the rest of the United States in accepting and applying HCM methods to planning applications. Field measurements of speed are preferred to HCM methods by West Coast agencies.

7.3.2.5 Significant Strengths and Deficiencies

HCM methods are comprehensive and sensitive to many system operation parameters. The methods' major deficiencies are as follows:

- HCM methods require extensive data to provide reliable results.
- HCM methods are complex and difficult to apply without specialized software.
- There is no planning procedure in the HCM for analyzing the overall level of service of a freeway containing more than just basic sections.
- The planning procedure for interrupted flow facilities is oriented toward urban conditions. No separate procedure or modification is provided for rural facilities.

7.3.3 Florida *Level of Service Manual*

The Florida *Level of Service Manual* and supporting software greatly facilitate the use of HCM methods for determining service volume and level of service, through the use of default values for most input data and the preparation of tables of maximum service volumes based on these defaults.

7.3.3.1 Data Requirements

The Florida Generalized Level of Service Tables are ideal for most planning applications because they require few data and are adaptable (through the use of spreadsheets supplied with the tables) to situations in which the planning agency has more data available for a specific facility. Thus, the Florida method can be used for a wide range of planning applications, from the most general analyses to the most specific. Minimum data requirements for using the look-up tables are volume, facility type, area type, signal density, median type, and presence of left turn bays.

It would take an error in volume on the order of 20 percent to 25 percent to cause the Florida method to predict Level of Service E rather than D for a freeway. An error of 6 percent to 10 percent in the predicted volume for an interrupted flow facility would cause the method to predict Level of Service E rather than D. The Florida method cannot distinguish between Levels of Service D and E for six-lane and eight-lane Class IA arterials in urbanized areas (the same service volume applies to both levels of service) because of intersection capacity constraints.

All minimum required input data are readily available to local agencies and state DOTs. Only 25 percent of MPOs indicated that it is infeasible for them to obtain average signal spacing.

7.3.3.2 Ease of Use

The generalized level of service tables are easy to use. Their only drawbacks are the footnotes and warnings the user must consider before making a final determination. These drawbacks can be easily overcome with training.

The Florida method consists of simple look-up tables. The Florida Department of Transportation currently gives 2-day training sessions on the entire *Level of Service Manual* and supporting software. The generalized level of service tables and spreadsheets require only a half day of training.

The Florida method is performed using a spreadsheet-based program on a computer. The software program necessary for the application of this method is the commonly used spreadsheet program Lotus 1-2-3. The method provides straightforward answers and is easy to learn and use by individuals familiar with common computer spreadsheet programs.

7.3.3.3 Reliability

The Florida method is almost as accurate as the methods in the HCM on which it is based. The method is applicable to all facilities and area types. The method can be used in all planning applications, but requires some minor adaptations (e.g., fitting a set of equations to the look-up tables) for use in RTP analyses.

The Florida ARTPLAN method of analysis for interrupted flow facilities is sensitive to many input parameters, including signal spacing, peak-hour factor, traffic volumes, and g/C ratios. The method, in essence, provides an excellent way of testing level of service improvements that could result from a wide variety of improvements to the arterial. Conversely, it facilitates assessment of deteriorating conditions as traffic volumes increase and improvements are not made.

7.3.3.4 User Confidence and General Acceptance

The Florida method has a high level of user confidence and acceptance within the state of Florida; however, it is relatively unknown elsewhere. About 6 percent of the respondents to the user survey use the Florida method; an additional 23% use some kind of service volume method. Service volume methods (including the Florida service volume method) are the second most popular methods for predicting level of service in the United States.

The Florida method is useful in all planning applications, with the exception of long-range transportation planning (LRTP) analyses. It is used most frequently for site impact and congestion management studies.

The Florida method is used by all agencies in Florida, and outside Florida, it is used predominantly by local agencies. The method is used by a few agencies and private firms scattered throughout the United States, but its use is concentrated in the state of Florida.

7.3.3.5 Significant Strengths and Deficiencies

The Florida method and supporting software are simple to apply and adaptable to special conditions. The method has the advantage of being based on the HCM. The method's major deficiencies are as follows:

- Special level of service criteria developed for rural areas may need to be verified for acceptability in other parts of the country.
- HCM level of service criteria for arterials make it impossible for low-design-speed roads to achieve acceptable levels of service, even at zero flows.

The following minor technical weaknesses of the method have been observed:

- The ARTPLAN model is not particularly sensitive to saturation flow rate, and it is usual practice to use a default saturation flow rate rather than one that is field measured. It also has been found that the model is not very sensitive to arrival type. Use of the default arrival type instead of a field observed type does not result in significant variation in calculated speed. It has been common practice to use Arrival Type 3 if the traffic signal is not on a coordinated system. It also has been common practice to use Arrival Type 4 for the peak direction and Arrival Type 2 for the off-peak direction if the signal is on a coordinated system that allows good progression in the peak direction of travel.
- The ARTPLAN model is especially sensitive to g/C ratios for the through movement of traffic and is one of the primary factors that influence the determination of speed that ARTPLAN calculates. The weighted g/C ratio of 0.40 or greater calculated and used in the analysis provides a good estimate of speed. If the weighted g/C ratio is less than 0.40, ARTPLAN calculates an arterial speed that is much lower than observed. If individual g/C ratios are used for through movements at all signalized intersections, an improved result is obtained. The intersections with through g/C ratios of less than 0.40 act as bottlenecks and fail first in ARTPLAN analysis.
- Intersection spacing becomes critical input when intersections are close to each other. Cases have been observed in which, because of signal proximity, the arterial will fail despite very low traffic volumes. The threshold signal spacing below which failure will occur, despite other favorable conditions when the ARTPLAN model is used, is approximately 700 ft. This is a result of HCM level of service criteria for arterials, which set specific minimum average speeds for each level of service. If physical constraints make it impossible for traffic to flow at the minimum speed, the level of service cannot be achieved.
- The ARTPLAN model also is sensitive to the fact that an arterial can fail as a result of a nondissipating queue being formed at an intersection when the v/c ratio exceeds 1/PHF.
- ARTPLAN analysis can be used to test typical transportation system management (TSM) measures such as intersection restriping to yield exclusive turn lanes and signal optimization. Benefits of TSM measures can be tested to the extent that they lower peak-hour volumes input into the method.

7.4 RECOMMENDED ENHANCEMENTS FOR LONG-RANGE TRANSPORTATION PLANNING AND MANUAL APPLICATIONS

Planners require relatively simple and rapid techniques for estimating speed and level of service for LRTP and manual evaluation purposes. LRTP requires the use of extremely complex demand forecasting models that, in turn, require relatively simple speed estimation procedures so that the thousands of road links in a typical urbanized area demand model can be processed. The speed estimating technique used in these models, therefore, ideally should consist of a single equation with relatively few variables that outputs travel time as a monotonic increasing function of volume.

Planners also want a simple "back of the envelope" type technique that allows them to quickly assess the desirability of numerous planning options without investing a great deal of effort in developing extensive input data or complex models. Simple techniques allow the planning professional to check the accuracy and reliability of the datasets they input into the complex software and models they may use to predict level of service and speed. Apparent discrepancies help the planner focus on the specific portion of the demand model's input dataset that may contain erroneous entries.

The simple v/c ratio technique for estimating speed and level of service fills the need for LRTP travel demand forecasting and simple manual techniques. However, current v/c speed-flow curves (the BPR curve and its variations) generally have poor accuracy and are not sensitive to the operational improvements often considered in the development of TSM measures and transportation control measures (TCMs). Table 7-3 highlights the problems with current v/c ratio techniques and the research team's proposed enhancements.

7.5 RECOMMENDED ENHANCEMENTS FOR OTHER PLANNING APPLICATIONS (NON-LRTP)

Non-LRTP applications do not have as stringent a requirement for simplicity as do LRTP applications. The methods for estimating speed and level of service can be more flexible and sensitive to more facility-specific factors.

The 1994 HCM describes methods for estimating speed and level of service that are among the most accepted in practice. These methods, however, generally are oriented toward traffic operations analysis and are poorly suited to most planning applications.

The Florida *Level of Service Manual* and supporting software translate many HCM methods into planning methods that require less data and provide improved output that is more useful to planners. The Florida methods, however, need to be reviewed and expanded to cover conditions elsewhere in the country. These methods also need to be expanded to cover gaps in the underlying HCM methods on which they are based. The following discussion describes the proposed enhancements to the Florida methods for interrupted and uninterrupted flow facilities.

7.5.1 Interrupted Flow Facilities

The urban and suburban arterials method in Chapter 11 of the HCM is the best available planning procedure for estimating speed (and therefore level of service) of interrupted flow facilities. However, the method requires some data (such as signal timing) not readily available to planners, and it is limited to urban areas and v/c ratios less than 1.2 at intersections. The predicted average link speed is for through traffic only and excludes consideration of the speed of traffic turning onto and off the facility.

The Florida *Level of Service Manual* and software overcome the data requirements problem through the development and use of defaults by area type and facility type. However, the Florida methods are designed for Florida conditions and need to be expanded to other terrain types and conditions. Also, being based on HCM methods, the Florida methods suffer from many of the same shortcomings of the HCM, namely, the limitation to conditions in which demand is less than capacity.

7.5.2 Uninterrupted Flow Facilities

The level of service and speed of traffic on uninterrupted flow facilities can be significantly influenced by queuing at ramp merging sections. Yet there is no method for incorporating these effects that is suitable for planning applications. The Florida *Level of Service Manual* and the HCM do not provide a planning procedure that takes into account the effects of queuing on overall average speed or level of service of a facility.

A new method is proposed to make up for the lack of an existing planning method for predicting level of service and average travel speed for an entire uninterrupted flow facility. This new procedure would be similar to the HCM procedure for urban and suburban arterials. The method in Chapter 3 of the HCM, Basic Freeway Sections, would be used to compute average speed for sections of freeway between ramp termini. A special procedure then would be used to estimate queuing delay at ramp termini.

A new averaging technique will be needed for computing an average level of service over the length of a facility.

TABLE 7-3 Recommended enhancements to the v/c ratio method

Problem	Recommended Enhancement
A. The BPR curve was calibrated to old data.	1. Refit to 1994 HCM and NCHRP 3-45 data.
B. Look-up tables for capacity and free-flow speed are not accurate.	2. Develop procedure for developing local capacity and free-flow speed look-up tables. Recommend use of facility specific free-flow speed and capacity whenever feasible.
C. The BPR and related curves are unreliable for signalized facilities.	3. Add signal timing variable(s) and signal density factor to account for signal spacing and coordination.
D. The BPR and related curves are not accurate for queuing situations.	4. Develop queuing delay estimates using NCHRP 255 procedure for situations where demand exceeds capacity.
E. The BPR and related curves predict speed and not LOS.	5. Document how v/c ratios and mean speed can be used to estimate level of service.

TABLE 7-4 Recommended enhancements to Florida level of service methods (enhanced ARTPLAN procedure)

Problem	Recommended Enhancement
A. Florida defaults need confirmation and extension of validity outside Florida.	1. Compare performance of Florida Generalized Service Volume Tables against validation data sets. Identify need for determining localized defaults.
B. The urban interrupted flow procedure for signalized arterials underestimates speeds by 20%.	2. Revise procedure for estimating segment running time.
C. The Florida and HCM procedures can't deal with conditions where demand is greater than capacity.	3. Add queuing procedure from NCHRP 255.
D. The Florida and HCM procedures for rural roads are in some cases limited to 60 mph design speeds.	4. Create speed flow formula that can be applied to two-lane rural roads with lower design speeds.
E. There is no planning procedure for uninterrupted flow facilities that takes into account delays and capacity reductions at weaving and merge sections.	5. Create procedure for uninterrupted flow facilities that is similar in concept to HCM Chapter 11 method. Ramps and weaves become delay points on freeway. Current knowledge of weaving sections, however, is weak and may require that weaving be excluded from this research effort.

This will follow the HCM arterial method of computing total section travel time, including delay, and dividing this total time into the total section length to obtain average travel speed.

Table 7-4 highlights the research team's recommendations for enhancing the Florida methods.

REFERENCE

1. Dowling, R.G. The Use of Default Parameters for Estimating Signalized Intersection Level of Service. In *Transportation Research Record 1457,* TRB, National Research Council, Washington, D.C., 1994.

CHAPTER 8

DERIVATION OF TECHNIQUES FOR LONG-RANGE TRANSPORTATION PLANNING AND SKETCH PLANNING

This chapter explains the derivation of techniques for predicting speed and level of service for two specific planning applications: long-range transportation planning (LRTP) and sketch planning. These two seemingly different applications have the same basic requirements: the techniques they employ must be quick and simple and require very little information about the road facility.

8.1 DERIVATION OF SPEED ESTIMATION TECHNIQUE

The recommended speed estimation technique for use in LRTP studies is an update of the BPR speed-flow curve. The new curve has been fitted to updated speed-flow data for freeways from the 1994 HCM and has been validated against speed-flow data for both uninterrupted flow and interrupted flow facilities.

The analysis found that the accuracy of the BPR technique is highly dependent on the accuracy of the free-flow speed and capacity used in the computations. Therefore, a technique is presented for local agencies to develop customized free-flow speed and capacity look-up tables for use with the updated BPR curve. This technique standardizes the process used by local agencies to customize the look-up tables to local conditions and ensures greater consistency with the 1994 HCM.

Through the appropriate choice of free-flow speeds and capacities, the updated BPR curve can be applied to the full range of highway facilities, from urban local streets to rural freeways, where detailed segment by segment analysis is not wanted or needed. The recommended procedure incorporates the following improvements to the standard BPR technique:

- The parameters of the BPR curve are updated to better represent recent speed-flow research. Separate parameters were developed for urban interrupted flow facilities and for all other facilities.
- The look-up table for free-flow speeds is replaced with an equation using the posted speed limit. The free-flow speed equation for urban interrupted flow facilities includes a signal delay term.
- The look-up table for capacity is replaced with 1994 HCM equations that have been adapted for planning applications.

8.1.1 Objectives of the Technique

The traffic forecasting models used to evaluate LRTP studies require relatively simple and rapid techniques for estimating speed and level of service. LRTP models must be able to quickly process highway networks containing thousands of street links. The planning agency must obtain and forecast facility characteristics (such as number of lanes, capacity, and free-flow speed) for each of these links. This is why planners working with LRTP models employ techniques that simplify the collection and processing of data for the highway network.

Similarly, in sketch planning, the planner is interested in quickly assessing the relative merits of many alternatives, many of which need to be rapidly ruled out to spare planning resources for more feasible alternatives. Thus, sketch planning also requires simple techniques that can be quickly implemented in a spreadsheet (or calculator) environment with a minimum of field data.

Speed estimation techniques for LRTP studies and sketch planning applications, therefore, should have the following characteristics:

- The techniques should be simple and quick to apply by calculator, spreadsheet, or computer.
- The required input data should be limited to variables that are easily obtainable by planning agencies.
- The techniques must predict travel time as an increasing function of volume. This is required for equilibrium assignment procedures to reach closure in LRTP models.

Ideally, the technique would consist of a single equation with relatively few variables that can be computed rapidly.

8.1.2 Candidate Procedures

The candidate procedures consist of the BPR curve, the Akcelik curve, and the Van Aerde curve. All three procedures share the desirable characteristic of simplicity in both required data and execution. More elaborate techniques such as those in the HCM and simulation models such as FREQ and TRANSYT were rejected as being too complex and data intensive for LRTP application.

8.1.3 Evaluation of Standard BPR Curve

The standard BPR curve, with its look-up tables of capacity and free-flow speed, greatly facilitates data coding, storage, and manipulation of geometric data for the highway network. Rather than obtaining capacity and free-flow speed data for each of the thousands of links in the network, the planner determines the functional class of the facility and the area type in which it is located. The functional class and area type are then used to look up the facility's capacity per lane and free-flow speed.

The standard BPR curve (with parameters $a = 0.15$ and $b = 4$), however, does not fit current knowledge of speed-flow characteristics on freeways (see Figure 8-1). This figure illustrates the results of a simulation designed to compare all techniques on a similar basis over a range of traffic demand conditions on a real-world facility. The simulation was necessary to obtain conditions in which demand exceeds capacity, which are difficult to measure accurately in the field. The v/c ratio used in this figure is the "critical v/c ratio" for the facility, which is defined as the highest v/c ratio occurring on any segment of the facility during any single hour of the peak period. (See Chapter 11 for more information on the simulation model tests.)

The figure compares the BPR curve with the 1994 HCM speed-flow curve (Figure 3-3 in the HCM) and a FREQ (*1*) model simulation for various v/c ratio values for Interstate 880 in Hayward, California. As illustrated in the figure, the BPR curve (with standard parameters) drops too soon at low v/c ratios (compared with the 1994 HCM) and does not drop fast enough at v/c ratios greater than 1 (compared with the FREQ results).

The standard BPR curve also has difficulty matching speed estimates produced by the 1994 HCM and the TRANSYT-7F (*2*) simulation model for arterials (see Figure 8-2). (This figure also shows the results of a simulation of different demand volumes on a real-world facility. Capacity is defined as the saturation flow times the g/C ratio (ratio of effective green per cycle) for the signal approach on the segment with the highest peak-hour v/c ratio. Mean speed is the a.m. peak-hour space mean speed for the eastbound through movement only, averaged over the entire facility. The 1994 HCM speed estimates were obtained using ARTPLAN. The HCM states that its methodology should not be applied when the v/c ratio at any one intersection exceeds 1.2 or the ratio of 1.0 over the peak-hour factor. The dashed line in the figure shows what the speed prediction would be if this advice were ignored. (See Chapter 13 for more information on the simulation model tests.) The standard BPR curve generally drops too soon before capacity is reached and then drops too slowly when demand exceeds capacity.

Tests using the BPR curve with alternative definitions of facility v/c ratio, such as the mean or median v/c ratio, demonstrated that the BPR curve performs best when the critical v/c ratio for the facility is used. For example, the mean v/c ratio for the facility would be the average volume of the segments of the facility divided by their average capacity. The BPR curve drops the fastest when the v/c ratio is in the vicinity of 1. The average speed for the facility also drops the fastest when the capacity of one of its segments is exceeded. Thus, the BPR curve is able to estimate the average speed over the length of a facility best when the maximum observed v/c ratio along the length of the facility is used to compute the average speed.

Testing the sensitivity of the BPR speed estimates to the accuracy of the input data revealed that the accuracy of the BPR curve could be significantly improved with the use of field data on the capacity and free-flow speed of the facility (see Table 8-1). The one exception is urban interrupted facilities, where the increased accuracy of the field data did not improve the results. This is because the free-flow speeds used in these field tests of the standard BPR curve were the mid-block (speed limit) free-flow speeds. (Simulation tests of the BPR method on Ventura Boulevard (in Los Angeles) used the TRANSYT-7F computed free-flow speed in the BPR curve so that errors caused by the form of the BPR curve could be isolated from errors related to the estimation of the free-flow speed that is input into the BPR curve.) These mid-

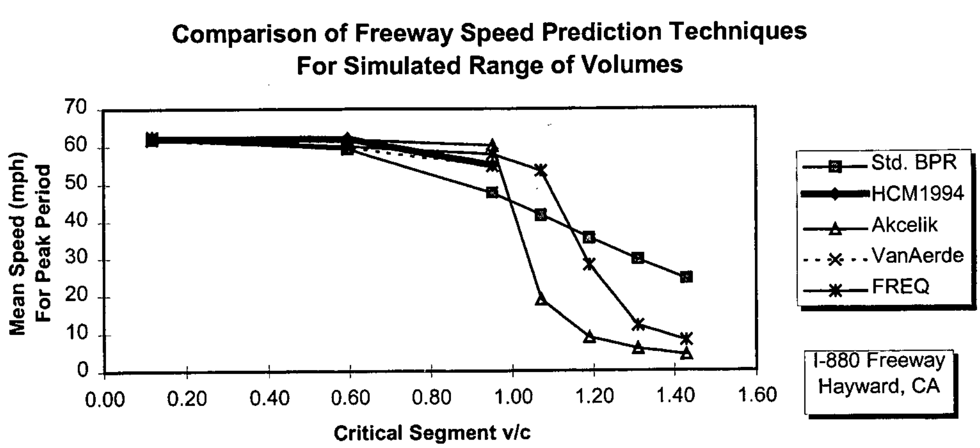

Figure 8-1. Comparison of BPR, Akcelik, and Van Aerde curves for freeways.

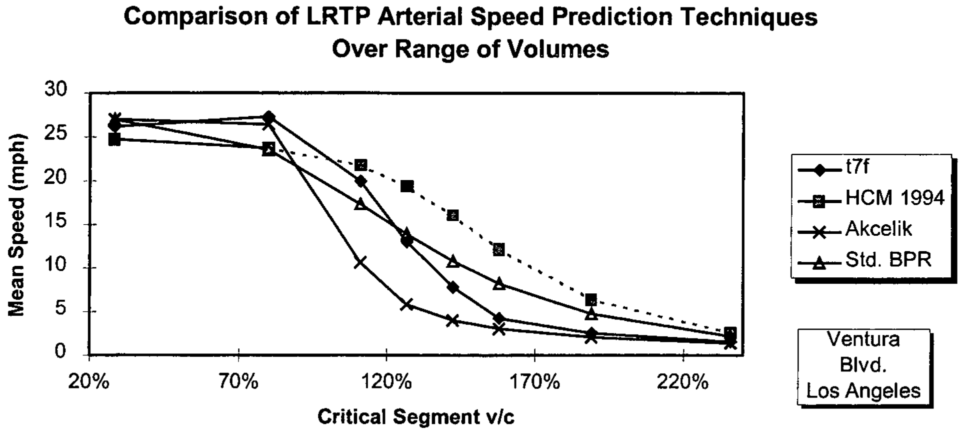

Figure 8-2. Fit of BPR and Akcelik curves to simulated arterial data.

block free-flow speeds do not accurately represent the average speed of traffic over the length of the facility (including signal delay) under low flow conditions.

8.1.4 Evaluation of Van Aerde Model

The Van Aerde model is the most mathematically complex of the three methods evaluated here. The model requires four parameters: free-flow speed, speed at capacity, capacity, and jam density. These parameters give the model a great deal of flexibility to shape itself to most any speed-flow condition. As shown in Figure 8-1, the Van Aerde model (similar to the BPR curve) can be calibrated to fit the 1994 HCM speed-flow curve for freeways closely. The fatal flaw in the Van Aerde model, as far as its utility for planning applications, is its inability to forecast speeds for v/c ratios greater than 1. For this reason, the Van Aerde model must be rejected from further consideration.

8.1.5 Evaluation of Akcelik Equation

The Akcelik speed-flow equation is slightly more mathematically complex than the BPR curve, but has the advantage of being similar to the signalized intersection delay equation in Chapter 9 of the 1994 HCM. The J_a parameter is calibrated based on the estimated facility speed at capacity (much like the a parameter for the BPR curve, which is calibrated for the facility speed at capacity). The Akcelik equation also has the advantage of being based on queuing theory, and thus can be expected to be more robust than the heuristic BPR curve.

Figure 8-1, however, shows that the Akcelik equation does not drop as fast as the 1994 HCM speed-flow curve for freeways for v/c ratios less than 1. The Akcelik equation tends to overestimate the delay caused by congestion for v/c ratios greater than 1 (when compared with the FREQ simulation). In fairness, however, it should be pointed out that the Akcelik equation is making its speed estimate based on a single critical v/c ratio for the entire peak period over the entire facility, whereas FREQ is making its estimate based on disaggregated 15-min data for the peak period for each segment of the facility.

A similar comparison for arterials (see Figure 8-2) demonstrates that the Akcelik curve accurately estimates speeds at low v/c ratios and overestimates the impact of queuing on arterial speeds. Again, this overestimate of delay is probably the result of using a single critical v/c ratio to compute aver-

TABLE 8-1 Impact of field data on accuracy of standard BPR (all entries in mph)

	Look-Up Data		Field Data		Percent Improvement	
Facility Type	**Bias**	**RMS**	**Bias**	**RMS**	**Bias**	**RMS**
Urban Uninterrupted	-3.7	6.8	+1.8	4.0	+51%	+42%
Urban Interrupted	-7.3	9.4	+8.8	10.3	-21%	-10%
Rural Uninterrupted	+3.6	5.4	+0.9	3.1	+75%	+43%
2-Lane Rural Interrupted	-19.7	22.0	+0.3	1.7	+98%	+92%
4-Lane Rural Interrupted	-19.8	21.0	-4.1	6.0	+79%	+71%

Notes:
Bias is the average difference between the estimated and actual mean facility speed (mph). A negative value means the method underestimates the actual speed.
RMS is the root mean square error between the estimated and actual mean facility speed.
Percent improvement is the percent reduction in bias and RMS when using field data.
Tests made using the validation data set described in Chapter 7 of this research report.

age speed instead of using the segment-specific v/c ratios used in the TRANSYT-7F simulation.

The Akcelik equation appears to be better suited to more detailed analyses when the facility is divided into segments for analysis. The method is appealing because of its sound theoretical basis in queuing theory and its similarity to the HCM signal delay equation. However, the Akcelik equation does not function well under the more stringent data limitations we have imposed in these tests (using a single critical v/c ratio to represent the entire facility). Consequently, we will focus on updating the BPR equation, a form more familiar to LRTP modelers, rather than focusing on the Akcelik equation.

8.1.6 Update of the BPR Curve

The standard BPR curve was refitted to the 1994 HCM freeway speed-flow curves contained in Figure 3-3 of the publication. The best results were obtained with a value of 0.20 for a and 10 for b. The curve was fit using capacity rather than the previous concept of practical capacity.

The enhanced BPR equation (labeled "updated" in Figures 8-3 and 8-4) is as follows:

$$s = \frac{s_f}{1 + a(v/c)^b} \qquad (8\text{-}1)$$

where:

s = predicted mean speed
s_f = free-flow speed
v = volume
c = capacity
a = 0.05 for signalized facilities
= 0.20 for all other facilities
b = 10.

A signalized facility is one with signals spaced 2 mi (3.2 km) apart or closer. The resulting curve is flatter than the original BPR curve for v/c ratios less than 0.70, and the new curve drops much faster in the vicinity of capacity (v/c = 1.00). Although the flatness of the curve is less desirable for achieving closure in equilibrium assignment, it is a necessary compromise for obtaining more accurate and reliable estimates of speeds for air quality purposes. The results of using the updated BPR curve are shown in Figures 8-5 and 8-6.

As was found in testing the standard BPR curve, the two keys to success in applying the updated BPR curve are to have accurate estimates of the free-flow speed and capacity for the facility. Once those two key parameters are known, the BPR curve can estimate speeds for both arterials and freeways with accuracies approaching those of the HCM and simulation models.

The following sections describe the derivation of procedures for obtaining more accurate estimates of free-flow speed and capacity.

8.2 ESTIMATION OF FREE-FLOW SPEED

The free-flow speed of a facility is defined as the space mean speed of traffic when volumes are so light that their effect on speed is negligible. The best technique for estimating free-flow speed is to measure it in the field under light traffic conditions, but this is not feasible when several thousand streets links must be analyzed. The paragraphs that follow provide a recommended procedure for estimating free-flow speed in the absence of field measurements of free-flow speed.

8.2.1 Candidate Procedures

Chapter 7 of the 1994 HCM and the final report of NCHRP Project 3-45 provide equations for computing free-flow

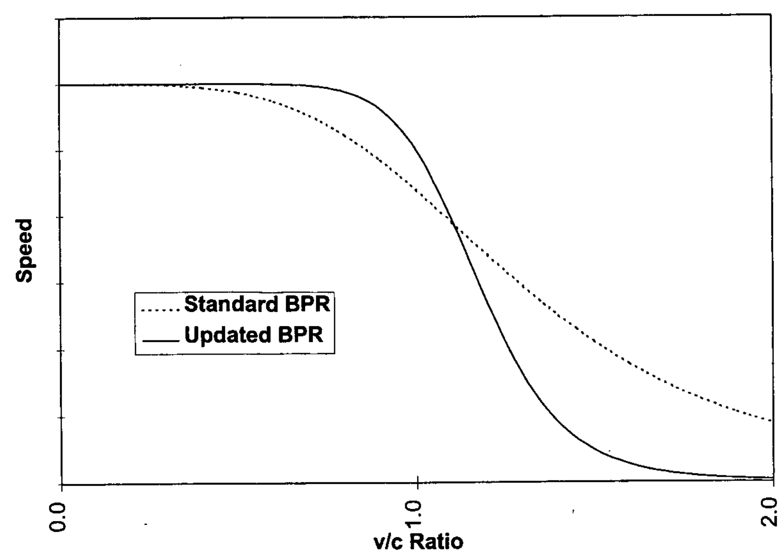

Figure 8-3. Speed-flow curves for unsignalized facilities.

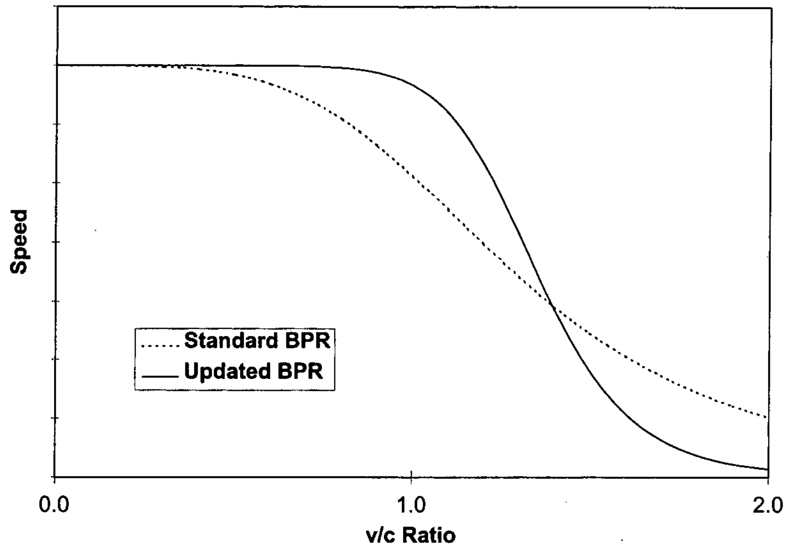

Figure 8-4. *Speed-flow curves for signalized facilities.*

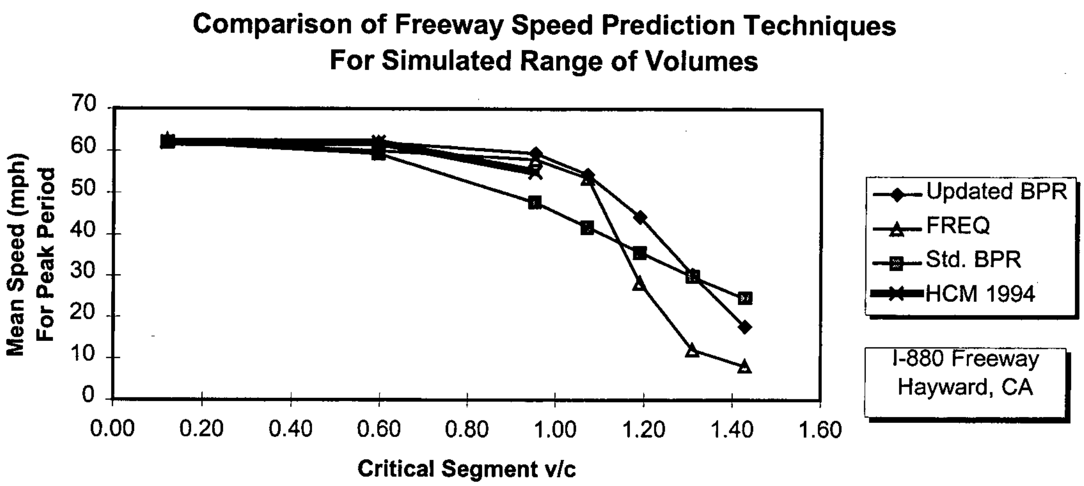

Figure 8-5. *Updated BPR curve versus simulated freeway data.*

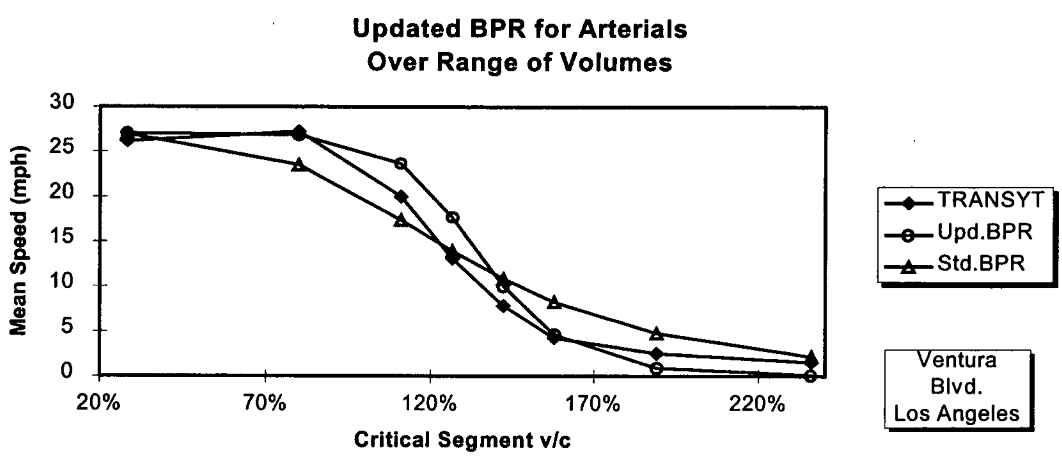

Figure 8-6. *Fit of updated BPR to simulated arterial data.*

speed for freeways and multilane highways based on facility design characteristics (ideal speed, lane width, lateral clearance, number of lanes, and number of access points or interchanges per mile). These methods, however, require a great deal of facility design data not readily available to planning agencies.

Tignor and Warren (*3*) plotted the relationship between posted speed limit, 85th percentile speeds, and mean speeds for 52 roads during 24-hr periods in Delaware, North Carolina, Colorado, and Arizona (see Figure 8-7). Holland (*4*) assembled a dataset comparing 85th percentile speeds with mean speeds for 205 spot speed surveys conducted in various urban areas in the San Joaquin Valley of California (see Figure 8-8).

Table 8-2 compares the measured free-flow speeds with posted speed limits for four rural freeways in California, Oregon, and New Hampshire. The mean free-flow speeds are on the average 5.6 mph higher than the posted speed limits for rural freeways.

8.2.2 Recommended Procedure (Urban Uninterrupted, Rural Uninterrupted, and Rural Interrupted)

Two linear equations were fitted to the available datasets. One equation is for facilities whose posted speed limits exceed 50 mph (80 km/hr). The other equation is for facilities with lower posted speed limits.

The two equations are based on the posted speed limits of the facility. Facility design speed and the 85th percentile speed were two other input variables considered for these equations, which were eventually rejected. Facility design speed was ruled out because it is difficult to obtain and is relatively meaningless for straight roads on level terrain. Holland's data demonstrated a strong and consistent relationship between the 85th percentile speed and the mean speed of traffic. However, the 85th percentile speed would require field measurements, and if field measurements are going to be made, the analyst might as well directly measure the mean free-flow speed.

The posted speed limit usually is based on the 85th percentile speed of traffic and thus can be a useful proxy for field measurements of free-flow speed. However, this general practice can be overruled by safety and policy considerations; therefore, the posted speed limit is not always a reliable indicator of the 85th percentile speed. If the analyst believes that the posted speed limit is an unreliable indicator of average travel speed (because the speed limit was set at another level other than the 85th percentile speed), a field measurement of the mean free-flow speed may be necessary.

The recommended equations in all cases are based on spot speed data. The time mean speed data have not been converted to space mean speed because of the lack of necessary information on the distribution of observed speeds. Therefore, these equations may overestimate the space mean speed by 1 to 5 mph (0.5 to 8 km/hr). This bias is considered tolerable for most planning applications. If greater accuracy is wanted, the analyst should consider field measurements of free-flow speed.

The spot speed data on urban roads do not include delays resulting from signalization. Thus, these equations cannot be applied to urban interrupted facilities without adding an adjustment to account for the effects of signal delay on free-flow travel time and mean speed.

8.2.2.1 Facilities with Posted Speed Limits Greater Than 50 mph (80 km/hr)

The following linear equation was fitted to the rural freeway dataset obtained for this research project. A total of 10 data points were obtained for six facilities in Oregon, California, and New Hampshire. The equation can be used for all facilities with posted speed limits that exceed 50 mph.

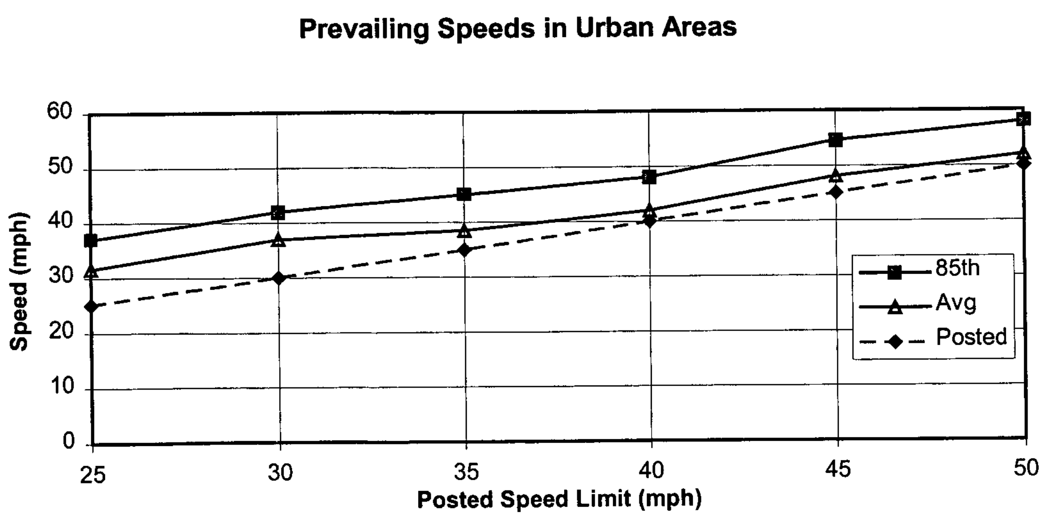

Figure 8-7. Speed limit, 85th percentile speeds, and mean speeds in urban areas (Tignor and Warren).

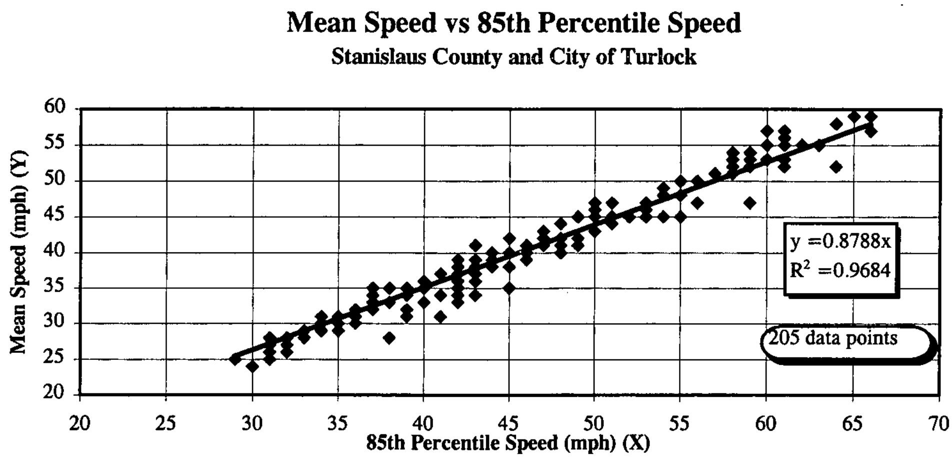

Figure 8-8. 85th percentile and mean speeds in urban/rural areas of California (Holland).

Mean Speed (mph)
= 0.88 × (Posted Speed Limit in mph) + 14 (8-2a)

Mean Speed (km/hr)
= 0.88 × (Posted Speed Limit in km/hr) + 22 (8-2b)

8.2.2.2 Facilities with Posted Speed Limits Equal To or Less Than 50 mph (80 km/hr)

The following linear equation was fitted to the Tignor and Warren dataset. It has a regression coefficient (R^2) of 0.98 and can be used to predict free-flow speed as a function of the posted speed limit for all roadways whose posted speed limits are 50 mph (80 km/hr) or less.

Mean Speed (mph)
= 0.79 × (Posted Speed Limit in mph) + 12 (8-3a)

Mean Speed (km/hr)
= 0.79 × (Posted Speed Limit in km/hr) + 19 (8-3b)

8.2.3 Recommended Procedure (Urban Interrupted)

The previous equations are based on spot speed studies of urban and rural roads. Spot speed studies in urban areas are made at midblock locations, away from the influence of upstream and downstream signals. Consequently, estimated free-flow speeds obtained by applying these equations will give an average midblock speed for urban streets, but not the average speed, including signal delays, along the street (which occur even at low volumes). Thus, it is necessary to adjust the estimated midblock free-flow speed for the estimated signal delay at low volumes to obtain a more accurate estimate of free-flow speed over the length of an urban street.

The 1994 HCM provides a method for estimating signal delay on urban arterials, which can be adapted to our needs (see page 11-9 of the HCM). We can obtain the signal delay for low flow conditions by setting the volume to zero in these equations. The incremental delay (d_2) term drops out, and we are left with a simple equation for uniform delay (d_1) as follows:

TABLE 8-2 Posted and free-flow speeds for four rural freeways

Facility	Location	Date	Speed Limit (mph)	Free Speed (mph)	Difference (mph)
I-5	Creswell, OR	1993	65	65.8	0.8
SR 99	San Joaquin Co., CA	11/6/91	55	64.1	9.1
SR 99	San Joaquin Co., CA	3/26/92	55	65.6	10.6
SR 99	San Joaquin Co., CA	5/18/92	55	65.6	10.6
SR 99	Sacramento Co., CA	8/15/92	55	64.0	9.0
SR 101	Humbolt Co., CA	11/20/91	55	58.0	3.0
SR 101	Humbolt Co., CA	1/14/92	55	58.3	3.3
SR 101	Humbolt Co., CA	4/7/92	55	58.6	3.6
SR 101	Humbolt Co., CA	7/12/92	55	58.1	3.1
NH 101	Candia-Raymond, NH	6/90	65	68.0	3.0
	Mean		57	62.6	5.6

$$d_1 = 0.38 * C(1 - g/C)^2 \tag{8-4}$$

Multiplying by the delay adjustment factor (DF) and 1.3 to obtain total delay yields the following equation:

$$D = DF * 0.5 * C(1 - g/C)^2 \tag{8-5}$$

where:

D = total signal delay per vehicle (sec)
$DF = (1 - P)/(1 - g/C)$
P = proportion of vehicles arriving on green
g = effective green time (sec)
C = cycle length (sec).

If signal timing data are not available, the planner can use the following default values:

C = 120 sec
g/C = 0.45
DF = 0.9 for uncoordinated traffic actuated signals
= 1.0 for uncoordinated fixed time signals
= 1.2 for coordinated signals with unfavorable progression
= 0.90 for coordinated signals with favorable progression
= 0.60 for coordinated signals with highly favorable progression.

The total delay per signal (D) is multiplied by the total number of signals on the facility to determine the total delay resulting from signalization under low volume conditions. This total signal delay is added to the total travel time (at the estimated midblock free-flow speed) to obtain total free-flow travel time for the facility with signal delay. The total free-flow travel time is divided into the total facility length to obtain the mean free-flow speed, including signal delay, for the facility. The resulting free-flow speed with signal delay then can be used in the updated BPR equation to compute speeds at other volume levels.

$$S_f = \frac{L}{L/S_{mb} + N * (D/3600)} \tag{8-6}$$

where:

S_f = free-flow speed for urban interrupted facility (mph or km/hr)
L = length of facility (mi or km)
S_{mb} = midblock free-flow speed (mph or km/hr)
= 0.79 (posted speed limit in mph) + 12 (mph)
= 0.79 (posted speed limit in km/hr) + 19 (km/hr)
N = number of signalized intersections on length (L) of the facility
D = average delay per signal per Equation 8-5 (sec).

8.3 ESTIMATION OF CAPACITY

The 1994 HCM provides a set of procedures for estimating facility capacity for operations analysis purposes. These procedures vary by facility type and generally require a great deal of information about the facility. The following sections provide recommended procedures that attempt to simplify the application of HCM methods for use in planning applications.

8.3.1 Procedure for Urban and Rural Uninterrupted Facilities

Chapter 3 of the HCM provides a procedure for converting observed traffic volumes into ideal passenger car equivalent volumes that can be used to evaluate the speed and level of service of a basic freeway section. The same ideal capacity is used for the freeway, regardless of its design characteristics.

The procedure that follows applies these volume adjustments to the calculation of capacity rather than to the calculation of ideal vehicle equivalents. This avoids the creation of fictitious vehicle equivalents, which can complicate the later computation of vehicle miles traveled, vehicle hours traveled, average delay, fuel consumption, and air pollutant emissions unless the planner remembers to back out these adjustments when calculating these measures of effectiveness.

Converting the volume adjustments to capacity adjustments results in the following equation derived from Equations 3-3 and 3-4 of the HCM.

HCM equation:

$$\textit{Capacity}\text{ (vph)} = \textit{Ideal Cap} * N * F_w * F_{hv} * F_{pop} * PHF \tag{8-7a}$$

where:

$\textit{Ideal Cap}$ = 2,300 (pcphl) for six-lane freeways
= 2,200 (pcphl) for four-lane freeways
F_w = lane width and lateral clearance factor
F_{hv} = heavy vehicle adjustment factor
F_{pop} = driver population adjustment factor
PHF = peak-hour factor (ratio of the peak 15-min flow rate to the average hourly flow rate).

Chapter 3 in the draft report produced for NCHRP Project 3-45, however, incorporates lane width and lateral clearance factors in the estimation of free-flow speed. The free-flow speed is then used to determine ideal capacity.

The recommended capacity procedure for uninterrupted flow facilities adopts the NCHRP Project 3-45 approach, with a few simplifying modifications to reduce data and computational requirements. The lane width adjustment factor was dropped because it is a minor adjustment and rarely comes into play in planning studies. The driver population adjustment factor was dropped consistent with the recommendations of NCHRP Project 3-45. The recommended equation is as follows:

$$Capacity\ (\text{vph}) = Ideal\ Cap * N * F_{hv} * PHF \qquad (8\text{-}7b)$$

where:

Ideal Cap = 2,400 (pcphl) for freeways with 70 mph (110 km/hr) or greater free-flow speeds (based on Chapter 3 in the draft report produced by NCHRP Project 3-55 (page 3-8), which recommends ideal capacities of 2,250 to 2,400 for freeways, depending on free-flow speed) (The 1994 HCM (page 3-5) recommends an ideal capacity of 2,300 for six-lane freeways.)
= 2,300 (pcphl) for all other freeways (free-flow speed < 70 mph (110 km/hr))

N = number of through lanes (Ignore auxiliary lanes and "exit only" lanes.)

F_{hv} = heavy vehicle adjustment factor
= $1.0/(1.0 + 0.5 * HV)$ for level terrain
= $1.0/(1.0 + 2.0 * HV)$ for rolling terrain
= $1.0/(1.0 + 5.0 * HV)$ for mountainous terrain
(HV = proportion of heavy vehicles, including trucks, buses, and recreational vehicles, in the traffic flow. If HV is unknown, use 0.05 heavy vehicles as the default (1994 HCM, page 3-13, Table 3-3, page 9-14, Table 9-6).)

PHF = peak-hour factor (ratio of peak 15-min flow rate to average hourly flow rate) (If unknown, use default of 0.90.).

8.3.2 Procedure for Multilane Rural Interrupted Facilities

Chapter 7 of the HCM provides a volume adjustment procedure and a free-flow speed computation procedure, both of which are used to estimate speed and level of service. The maximum service volume for the facility then is a function of free-flow speed (Table 7-1, page 7-8, of the 1994 HCM).

The recommended capacity estimation procedure for multilane rural interrupted flow facilities is similar to the procedure recommended for uninterrupted flow facilities:

$$Capacity\ (\text{vph}) = Ideal\ Cap * N * F_{hv} * PHF \qquad (8\text{-}8)$$

where:

Ideal Cap = 2,200 (pcphl) for multilane rural roads with 60 mph free-flow speed (based on Table 7-1 of 1994 HCM, which identifies a range of maximum service flow rates (at Level of Service E) from 1,900 to 2,200 passenger cars per hour, depending on the free-flow speed)
= 2,100 (pcphl) for multilane rural roads with 55 mph free-flow speed
= 2,000 (pcphl) for multilane rural roads with 50 mph free-flow speed

N = number of through lanes (Ignore exclusive turn lanes.)

F_{hv} = heavy vehicle adjustment factor
= $1.0/(1.0 + 0.5 * HV)$ for level terrain
= $1.0/(1.0 + 2.0 * HV)$ for rolling terrain
= $1.0/(1.0 + 5.0 * HV)$ for mountainous terrain
(HV = proportion of heavy vehicles, including trucks, buses, and recreational vehicles, in the traffic flow. If HV is unknown, use 0.05 heavy vehicles as the default (adapted from Equation 7-4, 1994 HCM).)

PHF = peak-hour factor (ratio of the peak 15-min flow rate to the average hourly flow rate) (If unknown, use default of 0.90.).

This equation was derived from the maximum service flow rates for Level of Service E shown in Table 7-1 of the HCM and the volume adjustment factors in Equation 7-3 of the HCM. All volume adjustment factors were converted to capacity adjustment factors.

8.3.3 Procedure for Two-Lane Rural Interrupted Facilities

Chapter 8 of the 1994 HCM presents a procedure for adjusting the volumes on a two-lane rural road. The adjusted volumes are compared with the ideal capacity of 2,800 vehicles per hour (total two-way traffic) to determine level of service (Figure 8-1 and Equation 8-1, 1994 HCM).

The recommended procedure is based on Equation 8-1 of the HCM, with a v/c ratio of 1 and the two-way capacity cut in half to give a capacity for a single direction. The peak-hour factor adjustment shown on page 8-7 of the HCM has been converted to a capacity adjustment.

The recommended equation for computing the capacity in a single direction for two-lane rural roads is as follows:

$$Capacity\ (\text{vph}) = Ideal\ Cap * N * F_w * F_{hv} * PHF * F_{dir} * F_{nopass} \qquad (8\text{-}9)$$

where:

Ideal Cap = 1,400 (pcphl) for all two-lane rural roads (taking half of the ideal two-directional capacity of 2,800 in Table 8-1, 1994 HCM)

F_w = lane width and lateral clearance factor
= 0.80 if narrow lanes and/or narrow shoulders are present
= 1.00 otherwise
(Narrow lanes are less than 12 ft (3.6 m) wide; narrow shoulders are less than 3 ft (1.0 m) wide.)

F_{hv} = heavy vehicle adjustment factor
= $1.0/(1.0 + 1.0 * HV)$ for level terrain
= $1.0/(1.0 + 4.0 * HV)$ for rolling terrain
= $1.0/(1.0 + 11.0 * HV)$ for mountainous terrain
(HV = proportion of heavy vehicles, including trucks, buses, and recreational vehicles,

in the traffic flow. If HV is unknown, use 0.18 heavy vehicles as the default (Equation 8-2, 1994 HCM; default HV taken from 0.14 trucks plus 0.04 recreational vehicles recommended on page 8-8 of the HCM).)

PHF = peak-hour factor (ratio of the peak 15-min flow rate to the average hourly flow rate (If not known, use default of 0.90.)

F_{dir} = directional adjustment factor
= 0.71 + 0.58 * (1.0 − peak direction proportion)
(Peak direction proportion is the percent of two-way traffic going in peak direction. If not known, use default of 0.55 peak direction.)

F_{nopass} = no-passing zone factor
= 1.00 for level terrain
= 0.97 − 0.07 * (*NoPass*) for rolling terrain
= 0.91 − 0.13 * (*NoPass*) for mountainous terrain
(*NoPass* is the proportion of length of facility for which passing is prohibited (1994 HCM, page 8-5, Table 8-1; equations fitted using log-linear regression to maximum allowable v/c ratios for Level of Service E). If *NoPass* is unknown, use 0.60 *NoPass* for rolling terrain and 0.80 for mountainous terrain.)

8.3.4 Procedure for Urban Interrupted Flow Facilities

The recommended capacity equation for urban interrupted flow facilities is derived from Equation 9-12 of the 1994 HCM. The peak-hour factor shown in Equation 9-9 of the HCM has been converted to a capacity adjustment factor. The lane utilization factor shown in Equation 9-10 of the HCM has been dropped because it generally has a minor effect on capacity (less than 10 percent). The grade adjustment factor was dropped because it affects capacity by 5 percent or less.

$$Capacity\text{ (vph)} = Ideal\ Sat * N * F_w * F_{HV} * PHF * F_{park} * F_{bay} * F_{CBD} * g/C * F_c \qquad (8\text{-}10)$$

where:

Ideal Sat = ideal saturation flow rate (vehicles per lane per hour of green)
= 1,900 (ideal saturation flow rate, page 9-14, 1994 HCM)

N = number of lanes at critical intersection (Exclude exclusive turn lanes and short lane additions.)

F_w = lane width factor
= 0.93 if narrow lanes (lanes < 12 ft (3.6 m) wide) (derived from Table 9-5, 1994 HCM, assuming 9-ft (2.8-m) lane widths)
= 1.00 otherwise

F_{HV} = heavy vehicle adjustment factor
= 1.0/(1.0 + HV) (Table 9-6, 1994 HCM)
(HV = proportion of heavy vehicles, including trucks, buses, and recreational vehicles, in the traffic flow. If HV is unknown, use 0.02 heavy vehicles as default (Table 9-3, 1994 HCM).)

PHF = peak-hour factor (ratio of the peak 15-min flow rate to the average hourly flow rate) (Use 0.90 as default if PHF not known (Table 9-3, 1994 HCM).)

F_{park} = on-street parking adjustment factor
= 0.90 if on-street parking is present and parking time limit is 1 hr or less (derived from Table 9-8 of 1994 HCM, assuming 10 parking spaces on one side of two-lane street and average turnover rate of two vehicles per hour per space)
= 1.00 otherwise

F_{bay} = exclusive left turn bay or lane adjustment factor
= 1.10 if exclusive left turn lanes present
= 1.00 otherwise (page 11-16, 1994 HCM)

F_{CBD} = central business district (CBD) adjustment factor
= 0.90 if located in CBDs
= 1.00 elsewhere (Table 9-10, 1994 HCM)

g/C = ratio of effective green time per cycle
If no data are available, use the following defaults, which are based on Florida default for g/C of 0.45 for state arterials and 0.42 to 0.32 for other arterials (Table E-1, Florida *Level of Service Manual*, 1995):

- Protected left turn phase present: g/C = 0.40
- Protected left turn phase not present: g/C = 0.45

Other defaults may be developed by the local planning agency based on local conditions. Additional defaults might be developed based on the functional classes of the major and crossing streets.

F_c = optional user-specified calibration factor to match the estimated capacity to the observed capacity (This factor is provided in lieu of a more complex calculation of left turn and right turn adjustment factors for a through lane group with shared left and right turn lanes.).

The left turn and right turn adjustment factors in the HCM involve a complex series of calculations or require data (pedestrian volumes) typically not available to planners. These factors are replaced here with an optional user-specified calibration factor (F_c) the planner can use to adjust the capacity downward for situations in which left and right turns are made from the through lanes.

TABLE 8-3 Maximum v/c ratios for freeways (1994 HCM, Table 3-1, page 3-9)

	Four Lanes (2 each direction)				Six Lanes (3 each direction)			
	Free-Flow Speed (mph)				Free-Flow Speed (mph)			
Level of Service	70	65	60	55	70	65	60	55
A	0.32	0.30	0.27	0.25	0.30	0.28	0.26	0.24
B	0.51	0.47	0.44	0.4	0.49	0.45	0.42	0.38
C	0.75	0.70	0.65	0.60	0.71	0.67	0.63	0.57
D	0.92	0.89	0.83	0.80	0.88	0.85	0.79	0.77
E	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

8.4 LEVEL OF SERVICE ESTIMATION TECHNIQUES

The recommended level of service estimation techniques for use in LRTP use v/c ratios or mean speed to estimate level of service. Consistent with the 1994 HCM, mean speed is used to determine level of service for urban interrupted facilities, whereas v/c ratios (as a proxy for density or percent time delay) are used to determine level of service for all other facilities.

8.4.1 Correspondence to HCM Facility Types

This research employs four facility types: urban uninterrupted flow, rural uninterrupted flow, urban interrupted flow, and rural interrupted flow facilities. The 1994 HCM uses the following facility types: basic freeway section, rural multilane highways, rural two-lane highways, and urban and suburban arterials. The following paragraphs explain the equivalencies between the two systems.

8.4.1.1 Urban and Rural Uninterrupted

Both urban uninterrupted and rural uninterrupted facilities as defined in this research are equivalent to the single 1994 HCM (page 3-2) facility type of freeways. This facility type is defined in the HCM as a divided highway with full control of access and two or more lanes for the exclusive use of traffic in each direction.

8.4.1.2 Rural Interrupted

Rural interrupted facilities are divided into multilane and two-lane facilities. To maintain consistency with the facility definitions in the 1994 HCM, any rural road with signals spaced 2 mi apart or less must be redefined as an "urban" interrupted facility, even if it is located in a rural area. This is because the level of service procedures for rural roads in the 1994 HCM do not provide for the analysis of the effects of frequent signalization on rural roads.

8.4.1.3 Urban Interrupted

The urban interrupted category includes urban and suburban arterials defined by the 1994 HCM. However, to ensure that planners have a procedure for as many facility types as possible, the urban interrupted category also includes rural roads with signals spaced 2 mi apart or less, as well as all signalized streets regardless of functional class (e.g., collectors and locals are included). The HCM level of service criteria for arterials, therefore, has been extended as part of this current research effort to include a broader class of facilities.

8.4.2 Urban and Rural Uninterrupted Level of Service Criteria

The 1994 HCM provides a table of maximum v/c ratios by level of service (Table 8-3). These maximum v/c ratios are derived in the HCM from the maximum density criteria and the basic relationship between density, flow, and speed (Density = Volume/Speed). The table can be interpolated for different free-flow speeds.

8.4.3 Multilane Rural Interrupted Level of Service Criteria

The HCM provides a table of maximum v/c ratios for multilane highways as a function of free-flow speed (Table 8-4).

TABLE 8-4 Maximum v/c ratios for multilane highways (1994 HCM, Table 7-1, page 7-8)

	Free Flow Speed			
Level of Service	60 mph	55 mph	50 mph	45 mph
A	0.33	0.31	0.30	0.28
B	0.55	0.52	0.50	0.47
C	0.75	0.72	0.70	0.66
D	0.89	0.86	0.84	0.79
E	1.00	1.00	1.00	1.00

TABLE 8-5 Maximum v/c ratios for two-lane road level of service (1994 HCM, Table 8-1, page 8-5)

L	Level Terrain						Rolling Terrain						Mountainous Terrain					
O	Percent No Passing						Percent No Passing						Percent No Passing					
S	0	20	40	60	80	100	0	20	40	60	80	100	0	20	40	60	80	100
A	0.15	0.12	0.09	0.07	0.05	0.04	0.15	0.10	0.07	0.05	0.04	0.03	0.14	0.09	0.07	0.04	0.02	0.01
B	0.27	0.24	0.21	0.19	0.17	0.16	0.26	0.23	0.19	0.17	0.15	0.13	0.25	0.20	0.16	0.13	0.12	0.10
C	0.43	0.39	0.36	0.34	0.33	0.32	0.42	0.39	0.35	0.32	0.30	0.28	0.39	0.33	0.28	0.23	0.20	0.16
D	0.64	0.62	0.60	0.59	0.58	0.57	0.62	0.57	0.52	0.48	0.46	0.43	0.58	0.50	0.45	0.40	0.37	0.33
E	1.00	1.00	1.00	1.00	1.00	1.00	0.97	0.94	0.92	0.91	0.90	0.90	0.91	0.87	0.84	0.82	0.80	0.78

8.4.4 Two-Lane Rural Interrupted Level of Service Criteria

The percent time delay is the level of service measure for rural two-lane highways. A look-up table of maximum v/c ratios is provided in the 1994 HCM in lieu of percent time delay (Table 8-5).

8.4.5 Urban Interrupted Level of Service Criteria

The 1994 HCM sets speed level of service criteria for arterials by arterial class (see Table 8-6) (1994 HCM, Table 11-1, page 11-4). The definition of arterial class, however, requires additional information on the facility's characteristics, which may be difficult to assemble for several thousand street links in a region. Also, the criteria need to be extended to the other facility types in the urban interrupted category.

TABLE 8-6 HCM level of service criteria for arterials

Arterial Class:	I	II	III
Free-Flow Speed:	40 mph	33 mph	27 mph
L.O.S. A	35 mph (0.88)	30 mph (0.91)	25 mph (0.93)
L.O.S. B	28 mph (0.70)	24 mph (0.73)	19 mph (0.70)
L.O.S. C	22 mph (0.55)	18 mph (0.55)	13 mph (0.48)
L.O.S. D	17 mph (0.43)	14 mph (0.42)	9 mph (0.33)
L.O.S. E	13 mph (0.33)	10 mph (0.30)	7 mph (0.26)

(values in parentheses are ratio of cut-off speed to free-flow speed)

TABLE 8-7 Recommended level of service criteria for urban interrupted facilities

Level of Service	Minimum Speed as a Percent of Free-Flow Speed
A	90%
B	70%
C	50%
D	40%
E	30%

As can be seen in Table 8-6, although the cutoff speed for each level of service varies by arterial class, the ratio of the cutoff speed to the free-flow speed is comparatively more stable. Consequently, we will use the average of these ratios to extend the HCM level of service concept to nonarterial facilities. To simplify its application for planning purposes, we will use these ratios for urban interrupted facilities that fall outside of Class I, II, and III arterials defined by the HCM. These criteria are shown in Table 8-6.

Generalized v/c look-up tables cannot be constructed so easily for signalized arterials, because the level of service on these facilities is a function of many factors. The following equation for determining maximum v/c ratios was derived by solving the updated BPR equation for v/c. The maximum acceptable v/c ratio is a function of the signalization and free-flow speed characteristics of the arterial.

$$v/c \leq \sqrt[10]{20\left(\frac{S_f}{a * S_{mb}} - 1\right)} \qquad (8\text{-}11)$$

where:

S_{mb} = midblock free-flow speed (mph or km/hr)

a = minimum acceptable percent of the midblock free-flow speed (S_{mb}) for the desired level of service according to Table 8-7

S_f = free-flow speed for signalized arterial (mph or km/hr), which is computed according to Equation 8-6.

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CHAPTER 9

RECOMMENDED PROCEDURE FOR LONG-RANGE TRANSPORTATION PLANNING AND SKETCH PLANNING

This chapter presents the recommended procedure for predicting speed and level of service for two specific planning applications: long-range transportation planning (LRTP) and sketch planning. These two seemingly different applications have the same basic requirements: the techniques they employ must be quick and simple and require very little information on the facility.

The procedure is performed in three stages:

1. Identify study section and critical point
2. Estimate speed
3. Estimate level of service and service volumes.

9.1 IDENTIFICATION OF STUDY SECTION AND CRITICAL POINT

The recommended procedure uses data for a single critical point to estimate the average speed and level of service for a specific study section on the facility. The critical point is defined as the location within the study section where the demand to capacity ratio has its highest value. The critical point is the bottleneck of the study section.

A study section is the portion of the facility to be analyzed. A study section is equivalent to the link used in LRTP travel demand models. The study section or link can be of any length. However, the procedure works best when the demand and capacity conditions at the critical point are similar to conditions on the rest of the study section. If this is not the case, greater accuracy can be obtained by splitting the study section into a series of subsections, each of which has its own critical point and is analyzed separately.

The following illustrates how to estimate the average speed over the study section as shown in Figure 9-1. One portion of the facility has demand (D_1) and capacity (C_1). The other portion has demand (D_2) and capacity (C_2). Because the ratio of D_2/C_2 is greater than D_1/C_1, the critical point on the facility is located in the second portion of the facility. If the ratio D_1/C_1 is significantly different from the ratio D_2/C_2, it may be desirable to split the facility into two separate study sections, each of which is analyzed separately. The average speed for each section would be converted to travel time. The travel times then would be added and divided into the entire facility length to obtain the average facility speed.

9.2 SPEED ESTIMATION TECHNIQUE

The recommended speed estimation technique for use in LRTP studies is an update of the BPR speed-flow curve. The new curve has been fitted to updated speed-flow data in the 1994 HCM and has been validated against speed-flow data for both uninterrupted flow and interrupted flow facilities.

The facility space mean speed is computed in three steps:

1. Estimate free-flow speed
2. Estimate link capacity
3. Compute average speed.

Look-up tables of defaults can be used to skip the first two steps, but poor choices of free-flow speed and capacity can seriously compromise the accuracy of the technique.

Step 1. Estimate Free-Flow Speed

The free-flow speed of a facility is defined as the space mean speed of traffic when volumes are so light that their effect on speed is negligible. The best technique for estimating free-flow speed is to measure it in the field under light traffic conditions; however, this is not feasible when several thousand streets links must be analyzed. The paragraphs that follow provide a recommended set of equations for estimating free-flow speed in the absence of field measurements of free-flow speed.

Option 1a. Equations for Facilities Without Signals

Two separate linear equations are provided for estimating free-flow speed for facilities with less than one signal every 2 mi (3.2 km). One equation is for facilities with posted speed limits that exceed 50 mph (80 km/hr). The other equation is for facilities with lower posted speed limits.

High-speed facilities (posted speed limits that exceed 50 mph (80 km/hr)):

$$S_f\,(\text{mph}) = 0.88 * S_p + 14 \quad (9\text{-}1a)$$

$$S_f\,(\text{km/hr}) = 0.88 * S_p + 22 \quad (9\text{-}1b)$$

Low-speed facilities (posted speed limit is 50 mph (80 km/hr) or less):

$$S_f\,(\text{mph}) = 0.79 * S_p + 12 \quad (9\text{-}2a)$$

$$S_f\,(\text{km/hr}) = 0.79 * S_p + 19 \quad (9\text{-}2b)$$

where:

S_f = free-flow speed in either mph or km/hr
S_p = posted speed limit in either mph or km/hr.

Option 1b. Equations for Signalized Facilities

The free-flow speed for signalized facilities must take into account both the free-flow speed measured midblock between signals and the signal delays along the street (which occur even at low volumes). The mean free-flow speed (including signal delay) is computed using the following equation, which adds the free-flow travel time between signals and the delay time at signals (under free-flow conditions).

$$S_f = \frac{L}{L/S_{mb} + N * (D/3600)} \quad (9\text{-}3)$$

where:

S_f = free-flow speed for urban interrupted facility (mph or km/hr)
L = length of facility (mi or km)
S_{mb} = midblock free-flow speed (mph or km/hr)
= 0.79 (posted speed limit in mph) + 12 (mph)
= 0.79 (posted speed limit in km/hr) + 19 (km/hr)
N = number of signalized intersections on length, L, of facility
D = average delay per signal per Equation 9-4, which follows (sec).

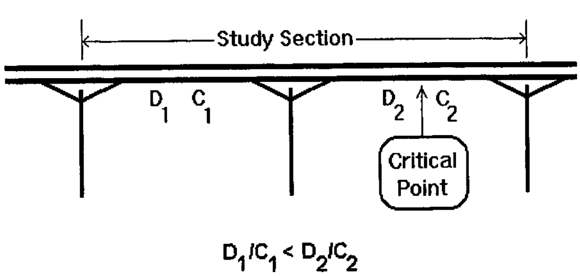

Figure 9-1. Illustration of study section and critical point.

The average delay per signal is computed using the following equation:

$$D = DF * 0.5 * C(1 - g/C)^2 \quad (9\text{-}4)$$

where:

D = total signal delay per vehicle (sec)
g = effective green time (sec)
C = cycle length (sec)
If signal timing data are not available, the planner can use the following default values:
C = 120 sec
g/C = 0.45
$DF = (1 - P)/(1 - g/C)$, where P = proportion of vehicles arriving on green
If P is unknown, the following defaults can be used for DF:
DF = 0.9 for uncoordinated traffic actuated signals
= 1.0 for uncoordinated fixed time signals
= 1.2 for coordinated signals with unfavorable progression
= 0.90 for coordinated signals with favorable progression
= 0.60 for coordinated signals with highly favorable progression.

Option 1c. Default Free-Flow Speeds

To simplify the estimation of free-flow speeds, planners may want to develop a look-up table of free-flow speeds based on the facility type and area type in which it is located. Depending on local conditions, the planning agency may wish to add terrain type (e.g., level, rolling, or mountainous) and frontage development type (commercial, residential, or undeveloped) to the general development types used in Tables 9-1 and 9-2.

The accuracy of the speed estimation procedure is highly dependent on the accuracy of the free-flow speed and capacity used in the computations. Great care should be taken in creating local look-up tables so that they accurately reflect the free-flow speeds in the locality.

Step 2. Estimate Link Capacity

The 1994 HCM provides a set of procedures for estimating facility capacity for operations analysis purposes. These procedures vary by facility type and generally require a great deal of information on the facility. The following equations simplify the application of HCM methods for use in planning applications.

TABLE 9-1 Example default free-flow speeds (mph)

Area Type	Freeway	Expressway	Arterial	Collector	Local
Central Business District	50	45	40	35	30
Urban	55	50	45	40	35
Suburban	60	55	50	45	40
Rural	65	60	55	50	45

Option 2a. Capacity Equation for Freeways

The following equation is used to compute the capacity of a freeway at its critical point:

$$\textit{Capacity}\ (\text{vph}) = \textit{Ideal Cap} * N * F_{hv} * \textit{PHF} \tag{9-5}$$

where:

Ideal Cap = 2,400 (pcphl) for freeways with 70 mph (110 km/hr) or greater free-flow speed
= 2,300 (pcphl) for all other freeways (free-flow speed < 70 mph (110 km/hr))

N = number of through lanes (Ignore auxiliary lanes and "exit only" lanes.)

F_{hv} = heavy vehicle adjustment factor
= $100/(100 + 0.5 * HV)$ for level terrain
= $100/(100 + 2.0 * HV)$ for rolling terrain
= $100/(100 + 5.0 * HV)$ for mountainous terrain
(HV = proportion of heavy vehicles, including trucks, buses, and recreational vehicles, in the traffic flow. If HV is unknown, use 0.05 heavy vehicles as default.)

PHF = peak-hour factor (ratio of the peak 15-min flow rate to the average hourly flow rate) (If unknown, use default of 0.90.).

Option 2b. Capacity Equation for Unsignalized Multilane Roads

The following equation is used to compute the capacity of a multilane road with signals (if any) spaced more than 2 mi apart:

$$\textit{Capacity}\ (\text{vph}) = \textit{Ideal Cap} * N \times F_{hv} * \textit{PHF} \tag{9-6}$$

where:

Ideal Cap = 2,200 (pcphl) for multilane rural roads with 60 mph free-flow speed
= 2,100 (pcphl) for multilane rural roads with 55 mph free-flow speed
= 2,000 (pcphl) for multilane rural roads with 50 mph free-flow speed

N = number of through lanes (Ignore exclusive turn lanes.)

F_{hv} = heavy vehicle adjustment factor
= $100/(100 + 0.5 * HV)$ for level terrain
= $100/(100 + 2.0 * HV)$ for rolling terrain
= $100/(100 + 5.0 * HV)$ for mountainous terrain
(HV = proportion of heavy vehicles, including trucks, buses, and recreational vehicles, in the traffic flow. If HV is unknown, use 0.05 heavy vehicles as default.)

PHF = peak-hour factor (ratio of the peak 15-min flow rate to the average hourly flow rate) (If unknown, use default of 0.90.).

Option 2c. Capacity Equation for Two-Lane Unsignalized Roads

The following equation is used to compute the capacity (in one direction) for a two-lane (total of both directions) road with signals (if any) more than 2 mi apart:

$$\textit{Capacity}\ (\text{vph}) = \textit{Ideal Cap} * N * F_w * F_{hv} * \textit{PHF} * F_{dir} * F_{nopass} \tag{9-7}$$

TABLE 9-2 Example default free-flow speeds (km/hr)

Area Type	Freeway	Expressway	Arterial	Collector	Local
Central Business District	80	72	64	56	50
Urban	88	80	72	64	56
Suburban	96	88	80	72	64
Rural	104	96	88	80	72

where:

Ideal Cap = 1,400 (pcphl) for all two-lane rural roads
N = number of lanes
F_w = lane width and lateral clearance factor
= 0.80 if narrow lanes and/or narrow shoulders are present
= 1.00 otherwise
(Narrow lanes are less than 12 ft (3.6 m) wide; narrow shoulders are less than 3 ft (1.0 m) wide.)
F_{hv} = heavy vehicle adjustment factor
= $1.00/(1.00 + 1.0 * HV)$ for level terrain
= $100/(100 + 4.0 * HV)$ for rolling terrain
= $100/(100 + 11.0 * HV)$ for mountainous terrain
(HV = proportion of heavy vehicles, including trucks, buses, and recreational vehicles, in the traffic flow. If HV is unknown, use 0.02 heavy vehicles as default.)
PHF = peak-hour factor (ratio of the peak 15-min flow rate to the average hourly flow rate) (If not known, use default of 0.90.)
F_{dir} = directional adjustment factor
= $0.71 + 0.58 * (1.00 -$ peak direction proportion) (Peak direction proportion is the proportion of two-way traffic going in peak direction. If not known, use default of 0.55 peak direction.)
F_{nopass} = no-passing zone factor
= 1.00 for level terrain
= $0.97 - 0.07 * (NoPass)$ for rolling terrain
= $0.91 - 0.13 * (NoPass)$ for mountainous terrain (*NoPass* is the proportion of length of facility for which passing is prohibited. If *NoPass* is unknown, use 0.60 *NoPass* for rolling terrain and 0.80 for mountainous terrain.).

Option 2d. Capacity Equation for Signalized Arterials

The following equation is used to compute the one-directional capacity of any signalized road with signals spaced 2 mi apart or less:

$$Capacity\ (\text{vph}) = Ideal\ Sat * N * F_{hv} * PHF * F_{park} * F_{bay} * F_{CBD} * g/C * F_c \tag{9-8}$$

where:

Ideal Sat = ideal saturation flow rate (vehicles per lane per hour of green)
= 1,900
N = number of lanes (Exclude exclusive turn lanes and short lane additions.)
F_{hv} = heavy vehicle adjustment factor
= $1.00/(1.00 + HV)$
(HV = proportion of heavy vehicles, including trucks, buses, and recreational vehicles, in the traffic flow. If HV is unknown, use 2 percent heavy vehicles as default.)
PHF = peak-hour factor (ratio of the peak 15-min flow rate to the average hourly flow rate) (Use 0.90 as default if *PHF* unknown.)
F_{park} = on-street parking adjustment factor
= 0.90 if on-street parking is present and parking time limit is 1 hour or less
= 1.00 otherwise
F_{bay} = left turn bay adjustment factor
= 1.10 if exclusive left turn lanes (often as a left turn bay) are present
= 1.00 otherwise
F_{CBD} = central business district (CBD) adjustment factor
= 0.90 if located in CBDs
= 1.00 elsewhere
g/C = ratio of effective green time per cycle
If no data are available, use the following defaults.
Protected left turn phase present: $g/C = 0.40$
Protected left turn phase not present: $g/C = 0.45$
Other defaults may be developed by the local planning agency based on local conditions. Additional defaults might be developed based on the functional classes of the major and crossing streets.
Fc = optional user-specified calibration factor necessary to match estimated capacity with field measurements or other independent estimates of capacity (no units) (can be used to account for the capacity-reducing effects of left and right turns made from through lanes).

Option 2e. Construction of Localized Capacity Look-Up Table

The accuracy of speed estimates are highly dependent on the accuracy of the estimated capacity for the facility. Consequently, it is recommended that each planning agency use capacities that are specific to the critical point of the selected study section whenever possible. However, it is recognized that this is not always feasible for planning studies. Consequently, the following two tables illustrate a procedure for selecting default values and computing a look-up table of capacities by facility type, area type, and terrain type. Other classification schemes may be appropriate, depending on the nature of local roadway conditions.

Table 9-3 contains a set of selected default parameters for the calculation of capacity for freeways, divided arterials, undivided arterials, and collectors. Each facility type is further subclassified according to area type (urban or rural), ter-

TABLE 9-3 Example table for entering default values for computing capacity by functional class and area/terrain type

Functional Class	Area Type	Terrain Type	Lanes	Free Speed	Lane Width	PHF	% Heavy Vehicles	Direction Split	% No Pass	Parking	Left Turn Bay	g/C
Freeway	Rural	Level	all	> 70 mph		0.85	5%					
		Rolling	all	> 70 mph		0.85	5%					
		Mountain	all	< 70 mph		0.85	5%					
	Urban	all	all	< 70 mph		0.90	2%					
Divided Arterial	Rural	Level	>2	60 mph		0.85	5%					
		Rolling	>2	55 mph		0.85	5%					
		Mountain	>2	50 mph		0.85	5%					
	Suburban	all	all			0.90	2%			no	yes	0.45
	Urban	all	all			0.90	2%			yes	yes	0.45
	CBD	all	all			0.90	2%			yes	yes	0.45
Undivided Arterial	Rural	Level	2		standard	0.85	5%	55%	0%			
		Rolling	2		standard	0.85	5%	55%	60%			
		Mountain	2		narrow	0.85	5%	55%	80%			
	Suburban	all	all			0.90	2%			no	no	0.45
	Urban	all	all			0.90	2%			yes	no	0.45
	CBD	all	all			0.90	2%			yes	no	0.45
Collector	Urban	all	all			0.85	2%			yes	no	0.40

rain type (level, rolling, or mountainous), and number of lanes (total of two lanes both directions or more). A separate set of default parameters is then selected for each subclassification of each facility type.

For example, a rural freeway in level or mountainous terrain is assumed to have a free-flow speed that exceeds 70 mph (112 km/hr), 5 percent heavy vehicles, and a peak-hour factor of 0.85. An urban freeway is assumed to have a free-flow speed below 70 mph (112 km/hr), 2 percent heavy vehicles, and a peak-hour factor of 0.90 to reflect the lower design speeds, heavier passenger car volumes, and flatter peak volumes in urban areas.

Divided arterials in rural areas are assumed to have free-flow speeds that decrease as the difficulty of the terrain increases. The assumed free-flow speed for level terrain is 60 mph (96 km/hr); for rolling terrain, 55 mph (88 km/hr); and for mountainous terrain, 50 mph (80 km/hr).

Any road in a rural area is assumed in this table to have signals (if any) spaced more than 2 mi apart. Urban area roads are assumed in this table to have signals at least 2 mi apart. The local planning agency should modify these assumptions if they are not appropriate for its particular jurisdiction.

Table 9-3 shows assumptions only for two-lane rural undivided arterials, but the planning agency can add additional rows of data for multilane rural undivided arterials.

Table 9-4 shows the computation of capacities by facility type based on the assumptions in Table 9-3. The results have been rounded off to the nearest 50 or 100 vehicles per hour per lane. The capacities per lane in this table then would be multiplied by the number of lanes (in one direction) at the critical point to obtain the critical point capacity for the facility.

Step 3. Compute Average Speed

Once the link capacity and free-flow speed are known, the following updated BPR equation can be used to predict the space mean vehicle speed for the link at forecasted traffic volumes. The same equation is used for both metric and customary units.

$$s = \frac{s_f}{1 + a(v/c)^b} \tag{9-9}$$

where:

s = predicted space mean speed
s_f = free-flow speed
v = volume
c = capacity
a = 0.05 for facilities with signals spaced 2 mi apart or less
= 0.20 for all other facilities
b = 10.

The two keys to success in applying the updated BPR curve are to have an accurate estimate of the free-flow speed

TABLE 9-4 Example computation of default capacities by functional class and area/terrain type

Functional Class	Area Type	Terrain Type	Lanes	Ideal Cap	PHF	Fhv	Fw	Fdir	Fnopass	Fpark	Fleft	Fcbd	g/C	Cap/Lane
Freeway	Rural	Level	all	2400	0.85	0.98								2000
		Rolling	all	2400	0.85	0.91								1900
		Mountain	all	2300	0.85	0.80								1600
	Urban	all	all	2300	0.90	0.98								2000
Divided Arterial	Rural	Level	>2	2200	0.85	0.98								1800
		Rolling	>2	2100	0.85	0.91								1600
		Mountain	>2	2000	0.85	0.80								1400
	Suburban	all	all	1900	0.90	0.98				1.00	1.10	1.00	0.45	850
	Urban	all	all	1900	0.90	0.98				0.90	1.10	1.00	0.45	750
	CBD	all	all	1900	0.90	0.98				0.90	1.10	0.90	0.45	650
Undivided Arterial	Rural	Level	2	1400	0.85	0.95	1.00	0.97	1.00					1100
		Rolling	2	1400	0.85	0.83	1.00	0.97	0.93					900
		Mountain	2	1400	0.85	0.65	0.80	0.97	0.81					500
	Suburban	all	all	1900	0.90	0.98				1.00	1.00	1.00	0.45	750
	Urban	all	all	1900	0.90	0.98				0.90	1.00	1.00	0.45	700
	CBD	all	all	1900	0.90	0.98				0.90	1.00	0.90	0.45	600
Collector	Urban	all	all	1900	0.85	0.98				0.90	1.00	1.00	0.40	550

and capacity for the facility. Once those two key parameters are known, the updated BPR curve can estimate speeds for both arterials and freeways with accuracies approaching those of the HCM and simulation models.

9.3 LEVEL OF SERVICE ESTIMATION TECHNIQUE

The recommended level of service estimation technique involves computing the v/c ratio at the critical point on the facility and comparing the v/c ratio with the maximum acceptable v/c ratio for the desired level of service. Maximum service volumes can be computed by multiplying the maximum acceptable v/c ratio by the capacity at the critical point on the facility. The procedure is performed in three steps:

1. Computer critical point v/c ratio
2. Compare with maximum service v/c ratios
3. Compute maximum service volumes.

Step 1. Compute Critical Point v/c Ratio

The first step is to use the procedures described in Section 9.1 to identify the critical point of the facility. Then, the procedures described in Step 2 of the speed estimation technique are used to compute the one-directional capacity of the critical point.

Step 2. Compare to v/c Ratio Cutoffs

The critical point v/c ratio is then compared to the maximum acceptable v/c ratio for each level of service to determine the level of service.

Option 2a. v/c Look-Up Tables for Unsignalized Facilities

The maximum acceptable v/c ratio for a desired level of service for freeways, multilane highways, and two-lane roads can be obtained from Tables 9-5, 9-6, and 9-7, respectively. The facility types are defined as follows:

- Freeways—Facilities that are completely access controlled.
- Multilane highways—Roads with two or more lanes in one direction with traffic signals spaced no closer than 2 mi apart.
- Two-lane roads—Roads with only one lane in each direction and with traffic signals spaced no closer than 2 mi apart.
- Signalized Arterials—Roads with traffic signals spaced no farther than 2 mi apart.

The maximum v/c ratios for freeways are categorized by number of lanes and the free-flow speed. The maximum v/c ratios for multilane highways are categorized by free-flow speed. The maximum v/c ratios for two-lane roads are cate-

TABLE 9-5 Maximum v/c ratios for freeways (1994 HCM, Table 3-1, page 3-9)

	Four Lanes (2 each direction)				Six + Lanes (3 each direction)			
	Free-Flow Speed (mph)				Free-Flow Speed (mph)			
Level of Service	70	65	60	55	70	65	60	55
A	0.32	0.30	0.27	0.25	0.30	0.28	0.26	0.24
B	0.51	0.47	0.44	0.4	0.49	0.45	0.42	0.38
C	0.75	0.70	0.65	0.60	0.71	0.67	0.63	0.57
D	0.92	0.89	0.83	0.80	0.88	0.85	0.79	0.77
E	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

The above table can be interpolated for different free-flow speeds.

gorized by general terrain type and the percent of the study section length in which passing is prohibited.

Option 2b. Maximum v/c Equation for Signalized Arterials

Generalized v/c look-up tables cannot be constructed so easily for signalized arterials, because the level of service on these facilities is a function of many more factors. The following equation is used to compute the maximum acceptable v/c ratio for a desired level of service based on the signalization and free-flow speed of the arterial.

$$v/c \leq \sqrt[10]{20\left(\frac{S_f}{a * S_{mb}} - 1\right)} \tag{9-10}$$

where:

S_{mb} = midblock free-flow speed (mph or km/hr)
= 0.79 (posted speed limit in mph) + 12 (mph)
= 0.79 (posted speed limit in km/hr) + 19 (km/hr)

a = minimum percent of the midblock free-flow speed (S_{mb}) according to the following table.

Level of Service	Minimum a value (%)
for LOS A	90
for LOS B	70
for LOS C	50
for LOS D	40
for LOS E	30

$$S_f = \frac{L}{L/S_{mb} + N * (D/3600)}$$

where:

S_f = free-flow speed for signalized arterial (mph or km/hr)
L = length of facility (mi or km)
N = number of signalized intersections on length, L, of facility
D = average delay per signal (sec)
$D = DF * 0.5 * C(1 - g/C)^2$

where:

g = effective green time (sec)
C = cycle length (sec)
If signal timing data are not available, the planner can use the following default values:
C = 120 sec
g/C = ratio of effective green time per cycle
If no data are available, use the following defaults, which are based on Florida default for g/C of 0.45 for state arterials and 0.42 to 0.32 for other arterials (Table E-1, Florida *Level of Service Manual*, 1995).

- Protected left turn phase present: $g/C = 0.40$
- Protected left turn phase not present: $g/C = 0.45$

$DF = (1 - P)/(1 - g/C)$, where P = proportion of vehicles arriving on green
If P is unknown, the following defaults can be used for DF:
DF = 0.9 for uncoordinated traffic actuated signals
= 1.0 for uncoordinated fixed time signals
= 1.2 for coordinated signals with unfavorable progression
= 0.9 for coordinated signals with favorable progression

TABLE 9-6 Maximum v/c ratios for multilane highways (1994 HCM, Table 7-1, page 7-8)

	Free Flow Speed Category			
Level of Service	60 mph	55 mph	50 mph	45 mph
A	0.33	0.31	0.30	0.28
B	0.55	0.52	0.50	0.47
C	0.75	0.72	0.70	0.66
D	0.89	0.86	0.84	0.79
E	1.00	1.00	1.00	1.00

The above table can be interpolated for different free-flow speeds.

TABLE 9-7 Maximum v/c ratios for two-lane-road level of service (1994 HCM, Table 8-1, page 8-5)

LOS	Level Terrain						Rolling Terrain						Mountainous Terrain					
	Percent No Passing						Percent No Passing						Percent No Passing					
	0	20	40	60	80	100	0	20	40	60	80	100	0	20	40	60	80	100
A	0.15	0.12	0.09	0.07	0.05	0.04	0.15	0.10	0.07	0.05	0.04	0.03	0.14	0.09	0.07	0.04	0.02	0.01
B	0.27	0.24	0.21	0.19	0.17	0.16	0.26	0.23	0.19	0.17	0.15	0.13	0.25	0.20	0.16	0.13	0.12	0.10
C	0.43	0.39	0.36	0.34	0.33	0.32	0.42	0.39	0.35	0.32	0.30	0.28	0.39	0.33	0.28	0.23	0.20	0.16
D	0.64	0.62	0.60	0.59	0.58	0.57	0.62	0.57	0.52	0.48	0.46	0.43	0.58	0.50	0.45	0.40	0.37	0.33
E	1.00	1.00	1.00	1.00	1.00	1.00	0.97	0.94	0.92	0.91	0.90	0.90	0.91	0.87	0.84	0.82	0.80	0.78

The above table can be interpolated for different percent no-passing.

TABLE 9-8 Maximum level of service v/c ratios for a 1-mi long arterial

	Smb= 25 mph					Smb= 35 mph					Smb= 45 mph				
LOS	I	II	III	IV	V	I	II	III	IV	V	I	II	III	IV	V
A	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
B	1.13	1.11	1.07	1.13	1.17	1.05	1.01	0.77	1.05	1.14	0.88	n/a	n/a	0.88	1.09
C	1.29	1.28	1.27	1.29	1.31	1.26	1.25	1.23	1.26	1.29	1.24	1.22	1.19	1.24	1.28
D	1.35	1.35	1.34	1.35	1.37	1.33	1.33	1.31	1.33	1.36	1.32	1.31	1.29	1.32	1.34
E	1.42	1.42	1.41	1.42	1.44	1.41	1.40	1.39	1.41	1.43	1.39	1.38	1.37	1.39	1.42

(2 signals/mile, 120 second Cycle, 45% g/C Ratio)
n/a = not attainable with 4 signals per mile and the assumed cycle length and g/C.
Smb = mid-block free-flow speed.
Case I = uncoordinated traffic actuated signals.
Case II = uncoordinated pre-timed signals.
Case III = coordinated signals with unfavorable progression.
Case IV = coordinated signals with favorable progression.
Case V = coordinated signals with highly favorable progression.

= 0.6 for coordinated signals with highly favorable progression.

For example, Table 9-8, which contains maximum v/c values, was constructed using the previous equation for a 1-mi-long arterial with half-mile signal spacing. (When interpreting these v/c values, note that capacity for signalized arterials is equal to the saturation flow times the g/C ratio.)

Step 3. Compute Maximum Service Volumes

The maximum service volume for a study section is computed by multiplying the maximum v/c ratio by the capacity of the critical point of the study section.

TABLE 9-9 Maximum service volumes for a four-lane, 70 mph freeway

Level of Service	Max. Service Volume (vph)
A	0.32 * 4400 = 1400
B	0.51 * 4400 = 2200
C	0.75 * 4400 = 3300
D	0.92 * 4400 = 4000
E	1.00 * 4400 = 4400

Freeway

The maximum service volumes (maximum flow in direction for 1 hour) for a four-lane freeway with a capacity of 4,400 vehicles per hour in one direction and a free-flow speed of 70 mph would be shown as in Table 9-9. These values are rounded to the nearest 100 vehicles per hour to be consistent

TABLE 9-10 Sample peak-hour service volumes table for signalized arterial

	Smb= 35 mph				
LOS	I	II	III	IV	V
A	n/a	n/a	n/a	n/a	n/a
B	1,600	1,500	1,200	1,600	1,700
C	1,900	1,900	1,900	1,900	2,000
D	2,000	2,000	2,000	2,000	2,100
E	2,200	2,100	2,100	2,200	2,200

(2 signals/mile, 120 second Cycle, 45% g/C Ratio, sat. flow = 1700 vphgl)
n/a = not attainable with 4 signals per mile and the assumed cycle length and g/C.
Smb = mid-block free-flow speed.
Case I = uncoordinated traffic actuated signals.
Case II = uncoordinated pre-timed signals.
Case III = coordinated signals with unfavorable progression.
Case IV = coordinated signals with favorable progression.
Case V = coordinated signals with highly favorable progression.

with the accuracy of the computations and the assumptions used to arrive at the facility capacity.

Signalized Arterial

Table 9-10 contains the results of a computation of a table of service volumes (for different degrees of signal coordination) for a four-lane signalized arterial (two lanes each direction) with a saturation flow of 1,700 vphgl (vehicles per hour of green per lane), a midblock free-flow speed of 35 mph (56 km/hr), and the signalization characteristics shown in Table 9-8. The capacity is equal to $1{,}700 * 2 * 0.45 = 1{,}530$. The maximum acceptable v/c values in Table 9-8 are multiplied by the computed capacity (1,530 vph) to obtain maximum acceptable service volumes. Note that the quality of progression generally affects the maximum service volume of signalized arterials by no more than plus or minus 10 percent; therefore, it is acceptable for planning purposes to use approximate estimates of general progression quality without resorting to field measurements of the percentage of vehicles arriving on green.

CHAPTER 10

DERIVATION OF PROCEDURES FOR OTHER PLANNING APPLICATIONS

The LRTP–sketch planning techniques described in the previous chapter require a minimum amount of data and analysis to quickly estimate approximate speeds and levels of service. These techniques, however, do not provide information on the performance of individual segments of a facility. They compute average facility performance using data on only the most critical segment of the facility. The techniques' accuracy is quite good considering how little information is required to apply them; however, their accuracy can be significantly improved with the addition of segment- and intersection-specific information.

The planning techniques described in this chapter are designed for more detailed planning evaluations of the performance of specific facilities (such as a traffic impact analysis for a new development project or a major investment study) to determine not only the average performance of the facility but also the specific locations where the facility breaks down. These techniques generally require that the facility be split into subsections for analysis and require more input data for each subsection. They provide for the analysis of multihour peak periods by allowing the peak period to be split into a sequence of 1-hour time periods. The techniques can be applied manually, but are best applied with the aid of a spreadsheet or custom software.

The techniques described here generally are not suitable for LRTP work because they require knowledge of intersection turning movements and evaluate the facility at the segment-specific level (each segment and node is evaluated individually and the results are summed to obtain facility averages). However, some LRTP model software that can use node delay and capacity calculations may be able to take advantage of parts of the techniques.

10.1 OBJECTIVES OF PROCEDURES

The purpose of the proposed speed and level of service estimation procedures for non-LRTP planning applications is to analyze the performance of individual segments of a facility and specific intersections over several hours of a peak period. It is recognized that this level of analysis requires more information. Intersection turning movement counts are needed for all significant intersections. Ramp volumes are needed for all interchanges. Count data have to extend the length of the peak period, which may last several hours.

Another purpose of these procedures is to improve the accuracy of the updated BPR method, which depends on data for a single critical segment of a facility. Additional segment- and intersection-specific data for the facility collected over several hours of the peak period allow for better accuracy in the estimated speeds and levels of service.

10.2 CANDIDATE PROCEDURES

Several simulation models for evaluating the operations of a single facility are available: FREQ, INTRAS, and FREESIM for freeways; NETSIM, TRANSYT-7F, and PASSER for arterials; and TRARR and TWO-PASS for two-lane rural roads. These models, however, require a great deal of data and are too complex to apply for most planning situations.

The HCM, which tends to focus on techniques for evaluating highway operations at a single point, does provide a planning procedure for evaluating the operation of signalized arterials. This procedure, described in Chapter 11 of the HCM and implemented in Florida's ARTPLAN software, divides the arterial into segments and intersections. The average running time is computed for the segments between the intersections. The delay at the intersections is then added to the total segment travel time to obtain total travel time in one direction for the length of the facility. This travel time is used to compute average speed and level of service. This procedure, however, cannot deal with overcapacity conditions or unsignalized arterials.

The HCM procedure for freeways (Chapter 6 of the HCM) is out of date and infeasible to apply in a planning environment.

10.3 OVERVIEW OF PROPOSED IMPROVEMENTS

A new procedure for summing and averaging level of service over several segments of a facility is created, along with a new procedure for estimating the delay caused when demand exceeds capacity. Other improvements vary by facility type.

10.3.1 Urban and Rural Uninterrupted Flow Facilities

A new procedure (in lieu of the outdated procedure in Chapter 6 of the HCM) is developed based on the analysis procedures in Chapter 3 of the HCM. The uninterrupted flow facility is divided into segments within which demand and capacity are relatively constant. The peak period demand is divided into a sequence of hourly demand rates. A simplified analysis is then applied to each segment for each hour of the peak period. Excess demand in 1 hour on one segment is carried over to the following hour (but the queue is not propagated to upstream segments in order to avoid computational complexity).

The weaving and ramp merge analyses in the HCM are not applied because of their physical limitations (weaving is limited to less than 2,500 ft and ramp merge analysis is limited to 1,500 ft of the two right lanes) and application limitations (ramp merge speeds cannot be estimated for speeds below 42 mph). Comparisons with simulation model results suggest that ramp merge analyses may not be critical for obtaining reasonably accurate speed estimates for a freeway.

10.3.2 Multilane Rural Interrupted Flow Facilities

The recommended procedure for multilane rural interrupted flow facilities is similar to the procedure for uninterrupted flow facilities. The facility is divided into segments. The peak period demand is divided into a sequence of hourly demand rates. A simplified procedure from Chapter 7 of the HCM is applied to each segment for each hour of the peak period. Excess demand in 1 hour on one segment is carried over to the following hour (but there is no queue propagation to upstream segments).

Signalized intersections or stop sign–controlled intersections are treated using the general procedure for arterials described in Chapter 11 of the HCM. Intersection delay is added to the total segment running time and divided into the facility length to obtain average facility speed.

10.3.3 Two-Lane Rural Interrupted Flow Facilities

The recommended procedure for two-lane rural interrupted flow facilities is similar to the procedure for multilane interrupted flow facilities. The difference is that a new procedure for estimating segment speed is used because no such procedure is currently available in the HCM. The multilane procedure is adapted for the special capacity considerations associated with two-lane roads.

10.3.4 Urban Interrupted Flow Facilities

The recommended procedure for urban interrupted flow facilities is an extension of the procedure in Chapter 11 of the HCM and implemented in Florida's ARTPLAN. The procedure is extended to situations in which demand exceeds capacity. The procedure also is extended to estimate average speeds over multihour peak periods. The HCM segment running time table is replaced with an equation to compute midblock free-flow speed.

10.4 DERIVATION OF PROCEDURE FOR ESTIMATING AVERAGE SPEED

This section describes the derivation and theoretical foundation for the recommended procedure for estimating space mean speed over the length of the facility and over the length of the peak period. The recommended speed estimation procedure estimates the space mean speed for one direction on the facility, including all delays for traffic moving in the subject direction along the length of the facility.

The procedure requires the analyst to divide the facility into segments, with nodes at the end of each segment. Segments are stretches of the facility where the traffic demand and capacity conditions are constant over the length of the segment. Nodes are points where traffic enters, leaves, or crosses the facility (or points where the capacity of the facility is changed by a lane drop, grade change, passing lane, and the like). The definitions of segments and nodes vary by facility type.

Each segment has one node at its downstream endpoint. The upstream node of the segment is associated with the upstream segment. Nodes will be treated as geometric points with zero length for the purpose of calculating segment speeds. However, it is recognized that an intersection or ramp junction can influence flow conditions for several hundred feet upstream and downstream of the intersection/junction itself. Thus, the node influence area will be taken into account in the computation of the delay associated with a node.

The speed estimation procedure derives the space mean speed for the facility from estimates of individual segment speeds and node delays, using the basic procedure given in Chapter 11 of the HCM. The segment running time (excluding delays at points such as signals) is added to the delay at the endpoint of the segment. The segment running times and the point delays are summed to obtain the total travel time along the length of the facility. The total travel time is converted from seconds to hours and then divided into the total facility length to obtain the space mean speed for the facility.

The basic speed estimation equation for all facility types is as follows:

$$Speed = \frac{3600 * \sum L_i}{\sum R_i * L_i + \sum D_j + \sum DQ_j} \quad (10\text{-}1)$$

where:

$Speed$ = average travel speed for facility (space mean speed) (mph or km/hr)
L_i = length of segment i (mi or km)
R_i = segment running time per unit distance for segment i (sec/mi or sec/km)
D_j = delay at node (point) j (sec)
DQ_j = delay due to queuing at node j (sec).

The delay due to queuing (demand greater than capacity) is added to the node delay and segment running time to obtain total travel time over the length of the facility.

10.5 DERIVATION OF PROCEDURE FOR ESTIMATING QUEUE DELAYS

Pedersen and Samdahl (*1*) developed a recommended set of procedures for computing speed, delay, and queue length for freeways and arterials for undercapacity and overcapacity conditions. The delay calculation adds the delay caused by excess demand (that part of demand that exceeds capacity) to the delay calculated for demand equal to capacity.

$$DQ = 0.5 * 3600 * T * (Demand - Capacity)/Capacity \quad (10\text{-}2)$$

where:

DQ = mean delay caused by excess demand (sec)
T = duration of analysis period (hr)
$Demand$ = through vehicle demand rate in subject direction (veh)
$Capacity$ = maximum segment flow rate in subject direction (veh).

Pedersen and Samdahl also suggest adjusting the demand rate to reflect the upstream propagation of the queue (vehicles arrive in the queue faster because the queue extends backward to meet the arriving vehicles); however, this adjustment will be neglected for the sake of simplicity.

The Pedersen and Samdahl equation has been modified to allow for the presence of a queue at the start of the analysis period, as follows:

$$DQ = 1800 * T * \left(\frac{V_{t-1} + V_t}{Capacity} - 1\right) \quad (10\text{-}3)$$

where:

V_{t-1} = queue (veh) at end of previous time period ($t - 1$)
V_t = additional demand (veh) occurring in current time period (t).

10.6 DERIVATION OF PROCEDURE FOR COMPUTING LEVEL OF SERVICE

The computation of level of service for a single direction of a facility with several segments and nodes with varying capacities requires a procedure for combining level of service results for individual segments and nodes. The averaging procedure varies by facility type because level of service measures vary by facility type.

In all cases, the maximum acceptable v/c ratio or minimum acceptable speed for a desired level of service is determined from look-up tables derived from the 1994 HCM (see Chapter 8 of this report). The maximum segment service volume then can be easily determined for unsignalized facilities by multiplying the maximum acceptable segment v/c ratio by the capacity of the segment. Signalized facilities require a more complex calculation, which is discussed later in the section Special Notes for Urban Interrupted Flow Facilities.

10.6.1 Uninterrupted Flow Facilities and Multilane Rural Interrupted Flow Facilities

The level of service measure for freeways (urban and rural uninterrupted flow facilities) and multilane rural roads is density. Thus, to obtain the mean level of service over the length of the facility, it is necessary to average the density of traffic over each segment of the facility. This is done by summing the number of vehicles in each segment and dividing by the total length of the facility.

The mean density of vehicles on the facility is a weighted average of the per lane densities of the individual segments, as follows:

$$\bar{D} = \frac{\sum_i D_i * L_i * N_i}{\sum_i L_i * N_i} \quad (10\text{-}4)$$

where:

D = mean density over the length of the study section of the facility (single direction) (veh/lane/mi or veh/lane/km)
D_i = density of segment i (veh/lane/mi or veh/lane/km)
L_i = length of segment i (mi or km)
N_i = number of lanes in one direction of segment i.

When a proxy, such as v/c ratio, is used instead of density, the proxy must be weighted as if it were density, because mean density is the ultimate service measure.

$$v/c_{mean} = \frac{\sum_i (v/c)_i * L_i * N_i}{\sum_i L_i * N_i} \quad (10\text{-}5)$$

where:

v/c_{mean} = mean v/c ratio for the facility in one direction
v/c_i = v/c ratio in one direction for segment i
L_i = length of segment i (mi or km)
N_i = number of lanes in subject direction on segment i.

10.6.2 Two-Lane Rural Interrupted Flow Facilities

The measure of level of service for two-lane rural roads is percent time delay, which is approximated by a maximum v/c ratio for each level of service. The mean v/c ratio for the facility can be obtained by taking a weighted average of the segment v/c ratios, with each segment's v/c ratio weighted by its length. The length of the segment is a proxy for the amount of time a vehicle is exposed to the percent time delay for that segment.

The weighted average v/c ratio for a two-lane rural road is computed as follows:

$$v/c_{mean} = \frac{\sum_i (v/c)_i * L_i}{\sum_i L_i} \tag{10-6}$$

where:

v/c_{mean} = mean v/c ratio for the facility in one direction
v/c_i = v/c ratio in one direction for segment i
L_i = length of segment i (mi or km).

10.6.3 Urban Interrupted Flow Facilities

In the case of urban arterials, the averaging of level of service is done automatically, because the level of service measure is the facility mean speed.

10.7 SPECIAL NOTES FOR UNINTERRUPTED FLOW FACILITIES

This section discusses special modifications to the general speed and level of service procedures, as part of their application to urban and rural uninterrupted flow facilities.

10.7.1 Speed Estimation Procedure

The general procedure for estimating mean speed over the length of a facility would allow for the computation of node delay at ramp merge-diverge points and at weaving sections. However, the procedures for analyzing weaving and ramp merge operations are not well suited for planning applications. In addition, the impact of ramp merge and weaving on average speed is minor compared with the impact of demand exceeding capacity. Consequently, the node delay term, D_j, will be ignored. This results in the following simplified equation for freeways:

$$Speed = \frac{3600 * \sum L_i}{\sum R_i * L_i + \sum DQ_i} \tag{10-7}$$

The updated BPR formula, derived in Chapter 8 of this report, is used to estimate the mean segment speed based on free-flow speed.

$$R_i = 3600 * \frac{(1 + a(v/c)^b)}{S_f} \tag{10-8}$$

where:

R_i = mean segment running time per unit length (sec/mi or sec/km)
S_f = mean segment free-flow speed (mph or km/hr) (space mean speed)
v/c = ratio of volume to capacity for the segment
a = 0.20
b = 10.

This equation requires the determination of the segment capacity ratio. The capacity is determined using the capacity equation for uninterrupted flow facilities defined in Chapter 8 of this report. The mean segment free-flow speed is estimated based on the posted speed limit, using the equations derived in Chapter 8.

10.7.2 Level of Service Procedure

The procedure for computing the level of service for freeways requires computing the weighted average v/c ratio for the facility (in the subject direction) and looking up the equivalent level of service. The v/c ratio is weighted both by segment lanes and length because the v/c ratio is being used here as a proxy for density, the actual level of service measure. Table 10-1 is used to determine the maximum acceptable average v/c ratio for the facility for the desired level of service.

10.7.3 Computation of Service Volumes

The mean facility service volumes (for one direction) are computed by averaging the segment service volumes. The segment service volumes are computed using the following equation:

$$SV_{LOS,i} = v/c_{maxLOS} * c \tag{10-9}$$

where:

$SV_{LOS,i}$ = maximum service volume for desired level of service on segment i

TABLE 10-1 Maximum v/c ratios for freeways (1994 HCM, Table 3-1, page 3-9)

	Four Lanes (2 each direction)				Six Lanes (3 each direction)			
	Free-Flow Speed (mph)				Free-Flow Speed (mph)			
Level of Service	70	65	60	55	70	65	60	55
A	0.32	0.30	0.27	0.25	0.30	0.28	0.26	0.24
B	0.51	0.47	0.44	0.4	0.49	0.45	0.42	0.38
C	0.75	0.70	0.65	0.60	0.71	0.67	0.63	0.57
D	0.92	0.89	0.83	0.80	0.88	0.85	0.79	0.77
E	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

This table can be interpolated for different free-flow speeds.

v/c_{maxLOS} = maximum v/c ratio from table for desired level of service

c = capacity (veh/hr) in one direction on segment i.

10.8 SPECIAL NOTES FOR MULTILANE RURAL INTERRUPTED FLOW FACILITIES

This section discusses special modifications to the general procedures that may be required for rural multilane interrupted flow facilities. The general procedures used for freeways are used for multilane rural roads. The free-flow speed and capacity are computed for each segment. The maximum service volume for each level of service is determined for each segment, using the computed capacity and the v/c ratios in Table 10-2. The resulting segment service volumes are averaged to determine the mean service volumes for the entire study section of the facility.

10.8.1 Speed Estimation Procedure

The procedures for freeways can be used to estimate speed for multilane rural interrupted flow facilities. Delays at traffic signals more than 2 mi apart usually can be ignored; therefore, node delay usually is zero for these facilities.

10.8.2 Level of Service Estimation Procedure

The procedures for estimating level of service for multilane rural interrupted flow facilities are identical to those for uninterrupted flow facilities, except that the v/c ratios in Table 10-2 are used to convert v/c ratio to level of service. The procedure for estimating maximum service volumes is identical to that for uninterrupted flow facilities.

10.9 SPECIAL NOTES FOR TWO-LANE RURAL INTERRUPTED FLOW FACILITIES

The speed and level of service procedures for two-lane rural interrupted flow facilities are the same as those for multilane rural interrupted flow facilities, with the exception of the level of service look-up table. The procedures are designed to be applied to a single direction of the facility.

10.9.1 Speed Estimation Procedure

The procedure for multilane rural interrupted flow facilities is used for two-lane roads.

10.9.2 Level of Service Estimation Procedure

The procedures for estimating level of service for two-lane rural interrupted flow facilities are identical to those for uninterrupted flow facilities, except that the v/c ratios in Table 10-3 are used to convert v/c ratio to level of service. The procedure for estimating maximum service volumes is identical to that for uninterrupted flow facilities.

10.10 SPECIAL NOTES FOR URBAN INTERRUPTED FLOW FACILITIES

This section discusses necessary variations on the general procedures for their application to urban interrupted flow facilities. The procedures are designed to be applied to a single direction of the facility.

10.10.1 Speed Estimation Procedure

The procedure for estimating mean speed over the length of a facility uses the basic equation described previously

TABLE 10-2 Maximum v/c ratios for multilane highways (1994 HCM, Table 7-1, page 7-8)

	Free Flow Speed			
Level of Service	60 mph	55 mph	50 mph	45 mph
A	0.33	0.31	0.30	0.28
B	0.55	0.52	0.50	0.47
C	0.75	0.72	0.70	0.66
D	0.89	0.86	0.84	0.79
E	1.00	1.00	1.00	1.00

TABLE 10-3 Maximum v/c ratios for two-lane-road level of service (1994 HCM, Table 8-1, page 8-5)

LOS	Level Terrain						Rolling Terrain						Mountainous Terrain					
	Percent No Passing						Percent No Passing						Percent No Passing					
	0	20	40	60	80	100	0	20	40	60	80	100	0	20	40	60	80	100
A	0.15	0.12	0.09	0.07	0.05	0.04	0.15	0.10	0.07	0.05	0.04	0.03	0.14	0.09	0.07	0.04	0.02	0.01
B	0.27	0.24	0.21	0.19	0.17	0.16	0.26	0.23	0.19	0.17	0.15	0.13	0.25	0.20	0.16	0.13	0.12	0.10
C	0.43	0.39	0.36	0.34	0.33	0.32	0.42	0.39	0.35	0.32	0.30	0.28	0.39	0.33	0.28	0.23	0.20	0.16
D	0.64	0.62	0.60	0.59	0.58	0.57	0.62	0.57	0.52	0.48	0.46	0.43	0.58	0.50	0.45	0.40	0.37	0.33
E	1.00	1.00	1.00	1.00	1.00	1.00	0.97	0.94	0.92	0.91	0.90	0.90	0.91	0.87	0.84	0.82	0.80	0.78

If the percent of the facility length where passing is not allowed is not known, use 40% no passing for level terrain, 60% no passing for rolling terrain and 80% no passing for mountainous terrain.

(Equation 10-1). Running time and node delay, however, require special equations to reflect the impact of signal control.

10.10.1.1 Running Time

Running time is computed based on midblock free-flow speed, which, in turn, is computed based on the posted speed limit, using the following equation fitted to the Tignor and Warren dataset (see Chapter 8 of this report).

$$R_i = \frac{3600}{S_{mb}} \qquad (10\text{-}10)$$

where:

R_i = running time per unit distance (sec/mi or sec/km)
S_{mb} = midblock free-flow speed
= 0.79 * (posted speed limit in mph) + 12 mph
= 0.79 * (posted speed limit in km/hr) + 19 km/hr.

10.10.1.2 Node Delay

The node delay for signalized intersections is computed based on the following equations from Chapter 11 of the HCM:

$$D = 1.3 * (d_u * DF + d_i) \qquad (10\text{-}11)$$

$$d_u = (0.38) * C * \frac{[1 - (g/C)]^2}{[1 - (g/C) * \min(X, 1.0)]} \qquad (10\text{-}12)$$

$$d_i = 173 * X^2 * \left\{(X - 1) + \sqrt{(X - 1)^2 + m * (X/c)}\right\} \qquad (10\text{-}13)$$

where:

D = approach total delay (sec/veh)
d_u = approach uniform delay (sec/veh)
d_i = approach incremental delay (sec/veh)
DF = delay adjustment factor
C = cycle length (sec)
g = effective green time for lane group (sec)
X = v/c ratio for subject lane group
c = capacity for through lane group
m = a calibration term.

The HCM equations require as input the cycle length (C), green time per cycle (g/C), and v/c ratio (X) for each approach. Signal timing parameters can be estimated using the planning analysis procedure in Chapter 9 of the 1994 HCM (pages 9-49 to 9-57), Appendix II of Chapter 9 of the HCM, and other techniques.

The procedure in Chapter 9 of the 1994 HCM is implemented in Florida's ARTPLAN software and McTrans HCS software. The user should refer to the HCM for the necessary details. The steps are as follows:

1. Determine the lane volumes for each turning movement on all approaches.
2. Determine the type of left turn protection for each approach.
3. Select the phase plan that gives the desired degree of left turn protection.
4. Sum the critical volumes for each phase and determine intersection status.
5. Determine the cycle length (C).
6. Compute the g/C ratio for each phase.

To make it easier to apply these delay equations in planning situations, two modifications have been made:

- The 1.3 factor used to convert stopped delay to total delay has been multiplied through the delay equations.
- The length of the analysis period (T) has been added back into the HCM delay equations.

The time factor (T) was reintroduced to allow for analysis periods other than the peak 15 min of the peak hour used in the HCM. This allows planners to analyze full 1-hour conditions rather than the peak 15 min.

Akcelik's research report on traffic signals was used for guidance (2). The factor of 173 in the incremental delay term (d_i) was multiplied by the 1.3 total delay factor to obtain the factor of 225. The 225 factor is equal to the analysis period (15 min or 900 sec) divided by 4. Increasing the analysis period to 1 hour (3,600 sec) increases the 225 factor to 900 and allows the introduction of the variable T, which gives the desired length of the analysis period in terms of hours.

The following equations result from these modifications (a value for T of 0.25 hours would match the original HCM equations):

$$d_n = d_u + d_r \tag{10-14}$$

$$d_u = \frac{C * [1 - (g/C)]^2}{2 * [1 - (g/C) * X]} * DF \tag{10-15}$$

$$d_r = 900 * T * X^2 * \left\{(X - 1) + \sqrt{(X - 1)^2 + 4m * \left(\frac{X}{sat * g/C * T}\right)}\right\} \tag{10-16}$$

Note that capacity has been replaced with its equivalent (saturation flow times the g/C ratio) in the incremental delay equation. Also note that we have replaced the subscript i for incremental delay with r because the subscript i already is being used in the procedure to designate the segment number.

10.10.2 Level of Service Procedure

The level of service for an urban street is defined by the average speed of traffic compared with its free-flow speed. The look-up table of minimum acceptable speeds was derived in Chapter 8 of this report.

The HCM does not provide for additional level of service considerations beyond the facility average speed in one direction; however, the planner may want to tally the number of intersections operating at a level of service that is worse than the facility average. Facilities with a significant proportion of intersections operating at a level of service that is worse than average may be degraded to a lower service level for planning evaluation purposes.

Service volumes are meaningful only if the planning agency wants to control the level of service of a facility so that no single segment exceeds the level of service goal during the peak hour. Otherwise, there are an infinite number of segment service volume combinations that will provide the same average level of service for the facility. The following procedure is for the computation of individual segment service volumes, with the objective that the level of service goal for each segment not be exceeded, regardless of the average level of service on the entire facility.

Direct equations for estimating intersection delay and, therefore, segment space mean speed cannot be easily solved to determine the service volume that will yield the desired speed.

$$Speed = \frac{3600 * \sum L_i}{\sum R_i * L_i + \sum D_j + \sum DQ_j} \tag{10-17}$$

where:

R_i (segment running time between signals) is constant with respect to volume,

D_j (intersection delay) is a complex function of volume, and

DQ_j (excess demand delay) is either zero or a linear function difference between the volume and capacity.

This procedure uses a two-piece linear approximation to the intersection delay equation that slightly overestimates delay in order to estimate the volume that will give the desired segment speed (Figure 10-1). The break point between the two linear approximations is at the capacity of the through movement at the intersection.

The slopes of the two lines are computed using the travel time at zero volume, travel time at capacity, and incremental travel time when demand exceeds capacity. If the desired speed for the desired level of service (S_{LOS}) is less than the speed at capacity (S_{cap}), the following equation derived from

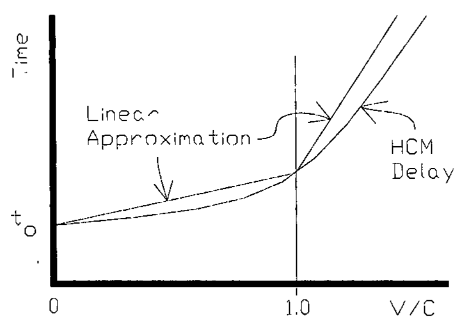

Figure 10-1. Linear approximation to HCM delay curve.

Pedersen and Samdahl's queue delay equation is used to estimate maximum service volume:

$$V_{LOS} = \frac{Cap}{1800T} * \left[\frac{L}{S_{LOS}} - \frac{L}{S_{cap}} + 1800T\right] \qquad (10\text{-}18)$$

where:

V_{LOS} = maximum service volume (vph)
S_{LOS} = minimum speed at desired level of service (mph or km/hr)
S_{cap} = speed at capacity (mph or km/hr)
Cap = capacity of through movement at intersection (vph)
L = length of the segment (mi or km)
T = length of time period (hr).

Otherwise, if $S_{LOS} \geq S_{cap}$, use the following equation to estimate service volume:

$$V_{LOS} = Cap * \left[\frac{S_0 - S_{LOS}}{S_0 - S_{cap}}\right] \qquad (10\text{-}19)$$

where S_0 = speed at zero volume (mph or km/hr).

All other variables are defined in Equations 10-18 and 10-19.

The average segment speed for each condition is computed using the previous segment speed equation and node delay equations and substituting the appropriate value of v/c for each condition.

$$S_0 = \frac{L}{3600\ L/S_{mb} + \tfrac{1}{2}DF * C * (1 - g/C)^2}$$

$$S_{cap} = \frac{L}{3600 * L/S_{mb} + \tfrac{1}{2}DF * C(1 - g/C) + 900T * \sqrt{\dfrac{4m}{T * Cap}}} \qquad (10\text{-}20)$$

where:

S_{cap} = speed at capacity (mph or km/hr)
S_0 = speed at zero volume (mph or km/hr)
L = length of segment (mi or km)
S_{mb} = midblock free-flow speed (mph or km/hr)
DF = delay adjustment factor (see notes to Equation 10-15)
C = cycle length (sec)
g = green time for through movement (sec)
T = length of time period being analyzed (hr)
m = calibration constant (see notes to Equation 10-16)
Cap = capacity (vph) = saturation flow times g/C ratio.

REFERENCES

1. Pedersen, N.J., and D.R. Samdahl. *NCHRP Report 255: Highway Traffic Data for Urbanized Area Project Planning and Design.* TRB, National Research Council, Washington, D.C., December 1982.
2. Akcelik, R. *Traffic Signals: Capacity and Timing Analysis.* Research Report ARR 123. Australian Road Research Board, Victoria, Australia, March 1981.

CHAPTER 11

RECOMMENDED PROCEDURES FOR OTHER PLANNING APPLICATIONS

The LRTP–sketch planning techniques described in the previous chapters require a minimum amount of data and analysis to quickly estimate approximate speeds and levels of service. These techniques, however, do not provide information on the performance of individual segments of the facility. They compute average facility performance using data only on the most critical segment of the facility. The techniques' accuracy is quite good considering how little information is required to apply them; however, their accuracy can be significantly improved with the addition of segment-and intersection-specific information.

The planning techniques described in this chapter are designed for more detailed planning evaluations of the performance of specific facilities (such as a traffic impact analysis for a new development project or a major investment study to determine not only the average performance of the facility but also the specific locations where and the times of day when the facility breaks down. These techniques generally require that the facility be split into subsections for analysis and require more input data for each subsection. The techniques can be applied manually, but are best applied with the aid of a spreadsheet or custom software.

The techniques described here generally are not suitable for LRTP work because they require knowledge of intersection turning movements and evaluate the facility at the segment-specific level (each segment and node is evaluated individually and the results are summed to obtain facility averages). However, some LRTP model software that can use node delay and capacity calculations may be able to take advantage of parts of the techniques described here.

11.1 OVERVIEW OF PROCEDURES

The recommended procedures estimate the space mean speed and level of service for one direction on a facility over the entire peak period. The analysis takes into account delays resulting from signal control and queuing.

The recommended procedures vary according to whether the study facility is signal controlled. Signal control is defined as signals spaced 2 mi apart or less. A facility with signals spaced farther than 2 mi apart is defined as an "unsignalized facility" for the purposes of this analysis.

11.1.1 Unsignalized Facilities

The recommended procedure for unsignalized facilities is based on the analysis procedures in Chapters 3, 7, and 8 of the 1994 HCM. The facility is divided into subsections (within which demand and capacity are relatively constant). The traffic demand in the peak period (if more than 1 hour long) is divided into a sequence of hourly demand rates. A simplified HCM analysis is then applied to each segment for each hour of the peak period. Excess demand in 1 hour on one segment is carried over to the following hour (but the queue is not propagated to upstream segments to avoid computational complexity).

11.1.2 Signalized Facilities

The recommended procedure for signalized facilities (those whose signals are 2 mi apart or closer) is an extension of the procedure in Chapter 11 of the HCM. The Chapter 11 procedure is extended to situations in which demand exceeds capacity and to the analysis of multihour peak periods. The impact of stop signs on nonarterials is ignored in this planning method. The HCM segment running time table is replaced with an equation to compute midblock free-flow speed.

11.2 DEFINING FACILITY TYPE AND DIVIDING THE FACILITY INTO SUBSECTIONS

This section provides guidance on defining the facility type to be analyzed and dividing the facility into subsections.

11.2.1 Defining Facility Types

The definitions of facility types are as follows:

- Freeways—Facilities that are completely access controlled.
- Multilane highways—Roads with two or more lanes in one direction, with traffic signals spaced no closer than 2 mi apart.

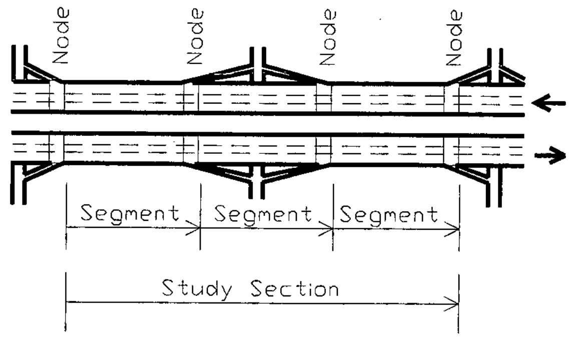

Figure 11-1. Segments and nodes for a freeway.

- Two-lane roads—Roads with only one lane in each direction, with traffic signals spaced no closer than 2 mi apart.
- Signalized arterials—Roads with traffic signals spaced no farther than 2 mi apart.

11.2.2 Dividing the Facility into Subsections

The procedure requires the analyst to divide the study section of the facility into subsections (segments), with nodes at the end of each segment. Segments are stretches of the facility where the traffic demand and capacity conditions are relatively constant (within 10 percent) over the length of the segment. Nodes are points where traffic enters, leaves, or crosses the facility or points where the capacity of the facility is changed by a lane drop, grade change, passing lane, and the like. The definitions of segments and nodes vary by facility type.

Each segment has one node at its downstream end point. The upstream node of the segment is associated with the upstream segment. There is no minimum or maximum length for a segment.

Nodes will be treated as geometric points with zero length for the purpose of calculating segment speeds. However, it is recognized that an intersection or ramp junction can influence flow conditions for several hundred feet upstream and downstream of the intersection/junction itself. Thus, the node influence area will be taken into account in the computation of the delay associated with a node.

Figures 11-1 and 11-2 illustrate the division of a freeway and a signalized arterial into nodes and segments.

Figure 11-3 illustrates the division of a two-lane rural road into segments. The beginning and end point of each segment are determined by a major change in the geometric characteristics of the facility, such as a change from a 10 percent no-passing segment to a lengthy 100 percent no-passing segment or a change from short grades in rolling terrain to extended grades in mountainous terrain.

11.3 PROCEDURE FOR UNSIGNALIZED FACILITIES

This section presents the recommended speed and level of service estimation procedures for facilities without signals or stop signs for the through traffic (or signals or stop signs spaced more than 2 mi apart). The procedures are designed to be applied to a single direction of the facility.

Step 1. Compute Capacity for Each Segment

The hourly capacity of each segment in one direction is determined using the capacity equations by facility type in this section. The objective is to fill in a table of capacities by segment and hour, such as Table 11-1, for a facility divided into three segments over a 3-hr peak period. Tables 11-2,

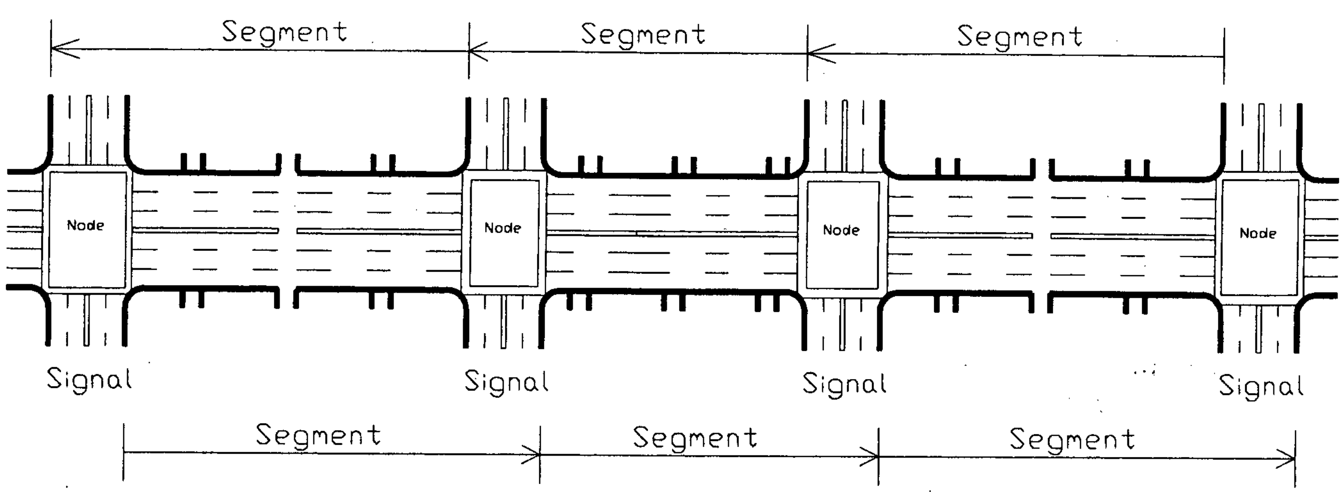

Figure 11-2. Segments and nodes for an arterial.

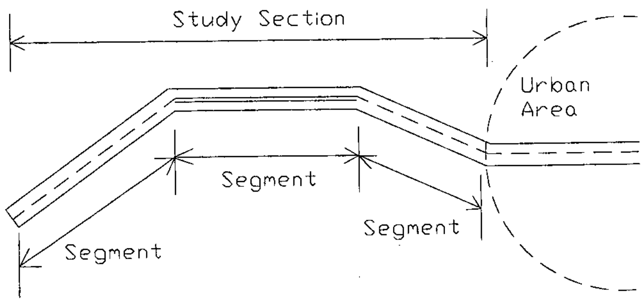

Figure 11-3. Segments for a two-lane rural road.

11-3, and 11-4 show the maximum v/c ratios for freeways, multilane highways, and two-lane rural roads, respectively.

Normally, the same capacity can be used for each hour in the peak period, but if the vehicle mix (percent trucks) or peaking (percent of hourly demand occurring in peak 15 min) changes by more than 20 percent over the length of the peak period, it may be necessary to compute separate capacities for each hour within the peak period.

Option 1a. Capacity Equation for Freeways

The following equation is used to compute the capacity of a freeway at its critical point:

$$\textit{Capacity}\ (\text{vph}) = \textit{Ideal Cap} * N * F_{HV} * PHF \qquad (11\text{-}1)$$

where:

Ideal Cap = 2,400 (pcphl) for freeways with 70 mph (110 km/hr) or greater free-flow speed
= 2,300 (pcphl) for all other freeways (free-flow speed < 70 mph (110 km/hr))
N = number of through lanes (Ignore auxiliary lanes and "exit only" lanes.)
F_{HV} = heavy vehicle adjustment factor
= $1.0/(1.0 + 0.5 * HV)$ for level terrain
= $1.0/(1.0 + 2.0 * HV)$ for rolling terrain
= $1.0/(1.0 + 5.0 * HV)$ for mountainous terrain
HV = proportion of heavy vehicles (including trucks, buses, and recreational vehicles) in the traffic flow (If the HV is unknown, use 0.05 heavy vehicles as default.)
PHF = peak-hour factor (ratio of the peak 15-min flow rate to the average hourly flow rate) (If unknown, use default of 0.90.).

TABLE 11-1 Example segment capacities table

Capacities	Segment 1	Segment 2	Segment 3
Hour 1	3600	4000	3500
Hour 2	3600	4000	3500
Hour 3	3600	4000	3500

Option 1b. Capacity Equation for Unsignalized Multilane Roads

The following equation is used to compute the capacity of a multilane road with signals (if any) spaced more than 2 mi apart:

$$\textit{Capacity}\ (\text{vph}) = \textit{Ideal Cap} * N * F_{HV} * PHF \qquad (11\text{-}2)$$

where:

Ideal Cap = 2,200 (pcphl) for multilane rural roads with 60 mph free-flow speed
= 2,100 (pcphl) for multilane rural roads with 55 mph free-flow speed
= 2,000 (pcphl) for multilane rural roads with 50 mph free-flow speed
N = number of through lanes (Ignore exclusive turn lanes.)
F_{HV} = heavy vehicle adjustment factor
= $1.0/(1.0 + 0.5 * HV)$ for level terrain
= $1.0/(1.0 + 2.0 * HV)$ for rolling terrain
= $1.0/(1.0 + 5.0 * HV)$ for mountainous terrain
HV = proportion of heavy vehicles (including trucks, buses, and recreational vehicles) in the traffic flow. (If HV is unknown, use 0.05 heavy vehicles as default.)
PHF = peak-hour factor (ratio of peak 15-min flow rate to average hourly flow rate) (If unknown, use default of 0.90.).

Option 1c. Capacity Equation for Two-Lane Unsignalized Roads

The following equation is used to compute the capacity for a two-lane (total of both directions) road with signals (if any) more than 2 mi apart:

$$\textit{Capacity}\ (\text{vph}) = \textit{Ideal Cap} * N * F_w * F_{HV} * PHF * F_{dir} * F_{nopass} \qquad (11\text{-}3)$$

where:

Ideal Cap = 1,400 (pcphl) for all two-lane rural roads
F_w = lane width and lateral clearance factor
= 0.80 if narrow lanes and/or narrow shoulders are present
= 1.00 otherwise
(Narrow lanes are less than 12 ft (3.6 m) wide; narrow shoulders are less than 3 ft (1.0 m) wide.)
F_{HV} = heavy vehicle adjustment factor
= $1.0/(1.0 + 1.0 * HV)$ for level terrain
= $1.0/(1.0 + 4.0 * HV)$ for rolling terrain
= $1.0/(1.0 + 11.0 * HV)$ for mountainous terrain
HV = proportion of heavy vehicles (including trucks, buses, and recreational vehicles)

TABLE 11-2 Maximum v/c ratios for freeways (1994 HCM, Table 3-1, page 3-9)

	Four Lanes (2 each direction)				Six Lanes (3 each direction)			
	Free-Flow Speed (mph)				Free-Flow Speed (mph)			
Level of Service	70	65	60	55	70	65	60	55
A	0.32	0.30	0.27	0.25	0.30	0.28	0.26	0.24
B	0.51	0.47	0.44	0.4	0.49	0.45	0.42	0.38
C	0.75	0.70	0.65	0.60	0.71	0.67	0.63	0.57
D	0.92	0.89	0.83	0.80	0.88	0.85	0.79	0.77
E	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

This table can be interpolated for different free-flow speeds.

in the traffic flow. (If HV is unknown, use 0.18 heavy vehicles as default.)

PHF = peak-hour factor (ratio of the peak 15-min flow rate to average hourly flow rate) (If not known, use default of 0.90.)

F_{dir} = directional adjustment factor
= $0.71 + 0.58 * (1.0 -$ peak direction proportion)
(Peak direction proportion is the proportion of two-way traffic going in peak direction. If not known, use default of 0.55 peak direction.)

F_{nopass} = no-passing zone factor
= 1.00 for level terrain
= $0.97 - 0.07 * (NoPass)$ for rolling terrain
= $0.91 - 0.13 * (NoPass)$ for mountainous terrain
($NoPass$ is the proportion of length of facility for which passing is prohibited. If $NoPass$ is unknown, use 0.60 $NoPass$ for rolling terrain and 0.80 for mountainous terrain.).

TABLE 11-3 Maximum v/c ratios for multilane highways (1994 HCM, Table 7-1, page 7-8)

	Free Flow Speed			
Level of Service	60 mph	55 mph	50 mph	45 mph
A	0.33	0.31	0.30	0.28
B	0.55	0.52	0.50	0.47
C	0.75	0.72	0.70	0.66
D	0.89	0.86	0.84	0.79
E	1.00	1.00	1.00	1.00

Step 2. Check for Excess Demand Conditions

It is necessary to check whether the demand on any segment exceeds its capacity for any hour within the peak period. If so, the excess demand must be carried over to the following hour and the queue delay computed for the current hour.

The current hour demand for each segment is compared with the capacity. If the current hour demand exceeds capacity, the difference (the excess demand) must be added to the demand for the following hour on that segment. This step is repeated for all segments for the current hour, the excess demand that must be carried over to the next hour is computed, after which the capacity checks are repeated for the next hour using the new demand. See example calculation in Figure 11-4.

The queuing delay (resulting from demand exceeding capacity) for each segment and time period is computed using the following equation only if demand is greater than capacity:

$$d_q = 3600 * T * \left(\frac{V_{t-1} + V_t}{2c} - 1\right) \qquad (11\text{-}4)$$

where:

d_q = mean delay resulting from excess demand (sec)
T = duration of time period (hr)
3600 = converts hours to seconds

TABLE 11-4 Maximum v/c ratios for two-lane-road level of service (1994 HCM, Table 8-1, page 8-5)

L	Level Terrain						Rolling Terrain						Mountainous Terrain					
O	Percent No Passing						Percent No Passing						Percent No Passing					
S	0	20	40	60	80	100	0	20	40	60	80	100	0	20	40	60	80	100
A	0.15	0.12	0.09	0.07	0.05	0.04	0.15	0.10	0.07	0.05	0.04	0.03	0.14	0.09	0.07	0.04	0.02	0.01
B	0.27	0.24	0.21	0.19	0.17	0.16	0.26	0.23	0.19	0.17	0.15	0.13	0.25	0.20	0.16	0.13	0.12	0.10
C	0.43	0.39	0.36	0.34	0.33	0.32	0.42	0.39	0.35	0.32	0.30	0.28	0.39	0.33	0.28	0.23	0.20	0.16
D	0.64	0.62	0.60	0.59	0.58	0.57	0.62	0.57	0.52	0.48	0.46	0.43	0.58	0.50	0.45	0.40	0.37	0.33
E	1.00	1.00	1.00	1.00	1.00	1.00	0.97	0.94	0.92	0.91	0.90	0.90	0.91	0.87	0.84	0.82	0.80	0.78

If the percent of the facility length where passing is not allowed is not known, use 40% no passing for level terrain, 60% no passing for rolling terrain and 80% no passing for mountainous terrain.

Step One: Initial Segment Demand Table

Demand	Segment 1	Segment 2	Segment 3
Hour 1	3200	3800	2700
Hour 2	3500	4200	3400
Hour 3	2800	3500	2300

Step Two: Segment Capacities Table

Capacities	Segment 1	Segment 2	Segment 3
Hour 1	3600	4000	3500
Hour 2	3600	4000	3500
Hour 3	3600	4000	3500

Step Three: Compute Excess Demand

Demand - Capacity	Segment 1	Segment 2	Segment 3
Hour 1	0	0	0
Hour 2	0	200	0
Hour 3	0	0	0

Step Four: Compute Revised Demand Table

Revised Demand	Segment 1	Segment 2	Segment 3
Hour 1	3200	3800	2700
Hour 2	3500	**4200**	3400
Hour 3	2800	**3700**	2300

Note that the 4200 vehicles per hour demand in Hour 2 on Segment 2 is retained, even though it exceeds the capacity by 200 vph. The excess 200 vehicles per hour demand in Hour 2 on Segment 2 is retained in Hour 2 and also added to the demand that must be served in Hour 3, since it occurs in Hour 2 but could not be served until Hour 3.

Figure 11-4. Example calculation of excess demand.

V_{t-1} = leftover demand from previous time period $(t-1)$
V_t = additional demand occurring in current time period (t)
c = capacity of segment in subject direction (veh/hr).

Step 3. Compute Segment Running Times

The segment running times are computed for each segment (i) and time period (t) using the following equation:

$$R_{i,t} = 3600 * \frac{(1 + a(v/c)^b_{i,t})}{S_f} \tag{11-5}$$

where:

$R_{i,t}$ = mean segment running time per unit length for segment, i, and time period, t (sec/mi, sec/km)
S_f = mean segment free-flow speed (mph or km/hr)
= 0.88 * (posted speed limit in mph) + 14 mph
= 0.88 * (posted speed limit in km/hr) + 22 km/hr
$v/c_{i,t}$ = ratio of volume to capacity for the segment (If v/c is greater than 1.0, use 1.0, because excess demand already has been dealt with in the queue analysis step.)
a = 0.20
b = 10.

Step 4. Compute Mean Speed

The space mean speed over the entire peak period and the total study section length of a freeway, multilane highway, or two-lane rural road is estimated using the following equation. Delays resulting from demand exceeding capacity on any one segment are added to the individual segment travel times, which are then summed over the entire study section to obtain the total travel time over the length of the study section. The total travel time is then divided into the total study section length to obtain the space mean speed for the study section.

$$s = \frac{3600 * N_t * \sum L_i}{\sum_{i,t} R_{i,t} * L_i + \sum_{i,t} dq_{i,t}} \tag{11-6}$$

where:

s = space mean speed over the length of the facility (mph or km/hr)
L_i = length of segment, i (mph or km/hr)
$R_{i,t}$ = running time for segment, i, during time period, t (sec/mi or sec/km)
$dq_{i,t}$ = delay resulting from queuing on segment, i, and time period, t (sec)
N_t = number of time periods being analyzed.

Step 5. Estimate Level of Service

The procedure for computing the level of service for unsignalized facilities requires computing the weighted average v/c ratio for the facility (in the subject direction) and looking up the equivalent level of service.

The weighted average v/c ratio for the facility is computed as follows:

$$v/c_{mean} = \frac{\sum_{i,t} v/c_{i,t} * L_i * N_i}{N_t * \sum_i L_i * N_i} \tag{11-7}$$

where:

v/c_{mean} = mean v/c ratio for the facility in one direction
$v/c_{i,t}$ = v/c ratio in one direction for segment, i, for time period, t
L_i = length of segment, i (mi)
N_i = number of through lanes in one direction of segment, i
N_t = number of time periods included in analysis.

This equation assumes that each time period (t) within the peak period being analyzed has the same duration.

The v/c ratio is weighted by segment lanes and length because the v/c ratio is being used here as a proxy for density, the actual level of service measure.

Service volumes are meaningful only if the planning agency wants to control the level of service of a facility so that no single segment exceeds the desired level of service goal during the peak hour. Otherwise, there are an infinite number of segment service volume combinations that will still provide the same average level of service for the facility. The following procedure provides for the computation of individual segment service volumes with the objective that the level of service goal for each segment not be exceeded, regardless of the average level of service on the entire facility.

The segment service volumes are computed by multiplying the maximum acceptable v/c ratio for the desired level of service (obtained from the previous tables) by the segment capacity:

$$SV_{LOS,i} = v/c_{maxLOS} * c_i \tag{11-8}$$

where:

$SV_{LOS,i}$ = maximum service volume for desired level of service (LOS) on segment, i
v/c_{maxLOS} = maximum v/c ratio from table for desired level of service
c_i = capacity (veh/hr) in one direction on segment, i.

Unlike the procedure recommended for LRTP and sketch planning applications (which computes a single critical point service volume), this procedure computes a set of service volumes for the facility, none of which can be exceeded for the segments to maintain the desired level of service.

11.4 PROCEDURE FOR SIGNALIZED FACILITIES

This section presents the recommended speed and level of service estimation procedures for urban interrupted flow facilities. The procedures are designed to be applied to a single direction of the facility.

Step 1. Compute Saturation Flow for Each Signalized Segment

The following equation is used to compute the one-directional saturation flow rate for through traffic for each signal on the facility:

$$\begin{aligned} Saturation \text{ (vphg)} &= Ideal\ Sat * N * F_{HV} * PHF \\ &\quad * F_{park} * F_{bay} * F_{CBD} * F_c \end{aligned} \tag{11-9}$$

where:

Ideal Sat = ideal saturation flow rate (vehicles per lane per hour of green)
= 1,900
N = number of through lanes (Exclude exclusive turn lanes and short lane additions.)
F_{HV} = heavy vehicle adjustment factor
= 1.0/(1.0 + HV)
HV = proportion of heavy vehicles (including trucks, buses, and recreational vehicles) in the traffic flow (If HV is unknown, use 0.02 heavy vehicles as default.)
PHF = peak-hour factor (ratio of peak 15-min flow rate to average hourly flow rate)
(Use 0.90 as default if PHF is unknown.)

F_{park} = on-street parking adjustment factor
= 0.90 if on-street parking is present and parking time limit is 1 hr or less
= 1.00 otherwise

F_{bay} = left turn bay adjustment factor
= 1.10 if exclusive left turn lanes (often as turn bays) are present
= 1.00 otherwise

F_{CBD} = central business district (CBD) adjustment factor
= 0.90 if located in CBDs
= 1.00 elsewhere

F_c = optional user-specified calibration factor necessary to match estimated capacity with field measurements or other independent estimates of capacity (no units).

Step 2. Estimate Signal Timing

The purpose of this step is to estimate the g/C ratio (green time per cycle) for the through movement and the cycle length (C) for each intersection of the facility. If the signal system is traffic responsive, it will be necessary to compute a separate g/C ratio and cycle length for each of the time periods within the analysis period. Otherwise, the g/C ratio and cycle length need be computed only for the peak hour within the peak period.

The g/C and cycle length can be determined using the planning procedure in Chapter 9 of the 1994 HCM, as implemented in the HCS software by McTrans, or the following simplified procedure adapted from Appendix II, Chapter 9, of the HCM.

$$C = \frac{t_L}{1 - F * \sum_{crit} V_{crit}/S_{crit}} \tag{11-10}$$

where:

C = cycle length (in sec)
t_L = loss time (in sec)
V_{crit} = volume (vph) on critical movement
S_{crit} = saturation flow (vphg) for critical movement
F = factor that provides some excess capacity at the intersection, typically 1.10.

At high v/c ratios this equation may yield infinite or negative cycle lengths. In these cases, the planning agency should set a policy maximum cycle length (such as 180 sec) to use in the analysis.

The excess capacity factor (F) is equal to the inverse of the desired v/c ratio for the intersection (X_c). Thus, a factor of 1.10 implies a goal of a 90 percent v/c ratio for the intersection.

The loss time (t_L) can be obtained from the following table based on whether left turns are protected or permitted for the main street and the cross street. A "protected" left turn is one that has a special signal phase that allows it to move at a different time than the opposing through traffic.

Main Street	*Cross Street*	*Phases*	t_L
Protected	Protected	4	12 sec
Protected	Permitted	3	9 sec
Permitted	Protected	3	9 sec
Permitted	Permitted	2	6 sec

If it is not known whether the left turns are or will be protected, the following rule can be used:

Assume that left turns are protected if

1. There is an exclusive left turn lane, or
2. There are more than 50 vph left turns and (left turns/hr) * (opposing through movements/hour) > 100,000 (*1*).

Unopposed left turns (left turns made from a one-way street or from the unopposed leg of a "T" intersection) are treated as permitted for the purpose of computing loss time in the previous table. Protected plus permitted left turns can be treated as protected for the purposes of this table.

The table of loss times assumes 3 sec of loss time between phases of the signal. The term "phase" is used here as it is defined in the HCM and should not be confused with the term "NEMA phase," which is used in traffic actuated control to refer to the green time for a single movement. Thus, an eight-phase NEMA controller (which has protected left turns for all four approaches) has four HCM phases for the purposes of the previous table.

The critical movements at the signal are determined by first computing the v/c ratio for each through and left turn movement at the intersection. Then the left turn and opposing through v/c ratios are summed to determine the pair of movements for each street that have the highest sum. The pair of critical movements for each street is the pair of left and opposing through movements with the highest total v/c ratio. The sum of the critical movement v/c ratios for each street is the value used to compute the cycle length for the intersection.

Figure 11-5 is an example of the identification of critical movements. The critical pair of left and through movements in the east/west street are the eastbound through and the westbound left. The critical north/south pair are the southbound left and northbound through. Together, the critical movements sum to 1.00 (0.50 N/S + 0.50 E/W) for this example.

This simplified procedure is not designed to address the subtleties of left turns from a shared lane, permitted lefts opposed by heavy through volumes, permitted plus protected phasing, or other unusual signal phasing strategies or street geometries. In many cases these subtleties can be safely ignored. If these issues are important, the planner should use another method to determine signal timing.

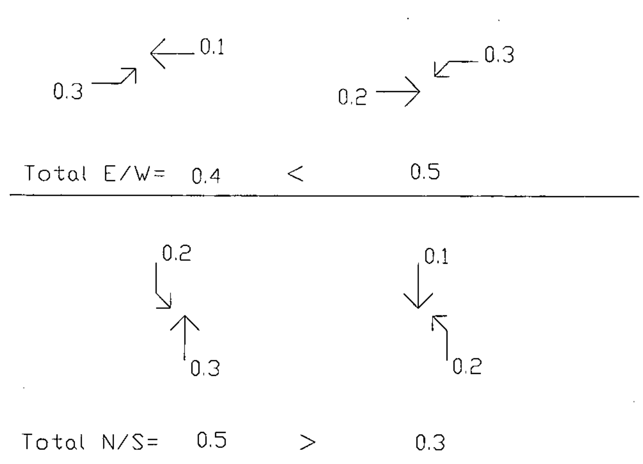

Figure 11-5. Critical movement example.

The g/C ratio for each through movement can be computed using the following equation that equalizes the v/c ratio for each through movement:

$$g/C = \frac{V/S}{X_c} \tag{11-11}$$

where:

g/C = ratio of green per cycle for a through movement
V = volume (vph) of movement
S = saturation flow (vphg) for movement
X_c = desired v/c ratio for intersection, typically 0.90.

This method will underestimate the g/C ratio for the noncritical through movement on each street, but should be satisfactory for planning purposes. This method of estimating g/C ratios will not minimize the overall delay at the intersection. A signal operations analysis should be conducted if such a solution is desired.

Step 3. Check for Excess Demand Conditions

This procedure is the same as the excess demand procedure described previously for unsignalized facilities. Refer to that discussion for an example of the procedure.

The current hour demand for each segment is compared to the capacity. If the current hour demand exceeds capacity, the difference (the excess demand) must be added to the demand for the following hour on that segment. This step is repeated for all segments for the current hour, the excess demand that must be carried over to the next hour is computed, after which the capacity checks are repeated for the next hour using the new demand.

The queuing delay (resulting from demand exceeding capacity) for each segment and time period is computed using the following equation only if demand is greater than capacity. (This is the same equation used for unsignalized facilities.)

$$d_q = 1800 * T * \left(\frac{V_{t-1} + V_t}{cap} - 1\right) \tag{11-12}$$

where:

d_q = mean delay resulting from excess demand (sec)
T = duration of time period (hr)
V_{t-1} = leftover demand (veh) from previous time period $(t-1)$
V_t = additional demand (veh) occurring in current time period (t)
cap = capacity of segment in subject direction (veh).

Step 4. Compute Running Time

The running time is computed based on the midblock free-flow speed, which in turn is computed based on the posted speed limit.

$$R_i = \frac{3600}{S_{mb}} \tag{11-13}$$

where:

R_i = running time per unit distance (sec/mi or sec/km)
S_{mb} = midblock free-flow speed
= 0.79 * (posted speed limit in mph) + 12 mph
= 0.79 * (posted speed limit in km/hr) + 19 km/hr.

This equation replaces the running time look-up table in Chapter 11 of the HCM, which tends to overestimate running times.

Step 5. Compute Node Delay

The node delay for signalized intersections is computed using the following equations adapted from Chapter 11 of the HCM.

$$d_n = d_u + d_r \tag{11-14}$$

$$d_u = \frac{C * [1 - (g/C)]^2}{2 * [1 - (g/C) * X]} * DF \tag{11-15}$$

$$d_r = 900 * T * X^2 * \left\{(X - 1) + \sqrt{(X - 1)^2 + 4m * \left(\frac{X}{sat * g/C * T}\right)}\right\} \tag{11-16}$$

where:

d_n = approach total delay for node (in sec/veh)
d_u = approach uniform delay (in sec/veh)
d_r = approach incremental (random) delay (in sec/veh)
DF = delay adjustment factor
= (1−P)/(1−g/C)
Use the following defaults for DF if the proportion of vehicles arriving on green is unknown:
0.9 for uncoordinated traffic actuated signals,
1.0 for uncoordinated fixed time signals,
1.2 for coordinated signals with unfavorable progression,
0.9 for coordinated signals with favorable progression, and
0.6 for coordinated signals with highly favorable progression (adapted from Table 9-13, 1994 HCM).
P = proportion of vehicles arriving on green
C = cycle length (in sec)
g = effective green time for the lane group (in sec)
X = v/c ratio for the subject lane group
= minimum of (v/*sat*)/(g/C),1.00
v = volume per hour (vph)
sat = saturation flow for the through lane group
m = a calibration term
= 16 for uncoordinated signals
= 12 for coordinated signals
T = length of a time period in hours (usually 1 hr) (The analysis period (usually the peak period) may contain several 1-hr time periods. The signal delay is computed separately for each time period (each having length (T) and summed over all the time periods to obtain the total signal delay over the full length of the analysis period.)

Step 6. Compute Mean Speed

The space mean speed in one direction over the length of a signalized facility and over an entire analysis period is computed using the following equation:

$$Speed = \frac{3600 * N_t * \sum L_i}{\sum_{i,t} R_{i,t} * L_i + \sum_{j,t} dn_{j,t} + \sum_{j,t} dq_{j,t}} \tag{11-17}$$

where:

$Speed$ = space mean speed over the length of the facility (mph or km/hr)
N_t = number of time periods (t) within analysis period
L_i = length of segment, i (mi or km)
$R_{i,t}$ = running time for segment, i (sec/mi or sec/km)
$dn_{j,t}$ = delay at node, j, for through traffic in the subject direction during time, t
$dq_{j,t}$ = delay resulting from demand exceeding capacity at node, j, during time period, t.

Step 7. Look Up Level of Service

The level of service for urban streets is defined by the average speed of traffic as compared with its free-flow speed. Specific service speeds are defined for Arterial Classes I, II, and III.

The minimum ratio of average speed to free-flow speed shown in the rightmost column of Table 11-5 can be used for looking up the level of service for all urban streets with higher or lower free-flow speeds not fitting the HCM definition of arterial class.

Service volumes are meaningful only if the planning agency wants to control the level of service of a facility so that no single segment exceeds the desired level of service goal during the peak hour. Otherwise, there are an infinite number of segment service volume combinations that will still provide the same average level of service for the facility. The following procedure provides for the computation of individual segment service volumes with the objective that the level of service goal for each segment not be exceeded, regardless of the average level of service on the entire facility.

The direct equations for estimating intersection delay cannot be solved easily to determine the service volume that will yield the desired speed. Therefore, this procedure uses a two-piece linear approximation to the intersection delay equation that slightly overestimates delay in order to estimate the volume that will give the desired segment speed. First, the speed at capacity must be computed to determine which linear approximation will be used. Then the service volume can be computed directly from the minimum speed for the desired level of service.

Step 7a. Compute Speed at Capacity

$$S_{cap} = \frac{L}{3600 * L/S_{mb} + {}^{1}\!/\!_{2}DF * C(1 - g/C) + 900T * \sqrt{\frac{4m}{T * Cap}}} \tag{11-18}$$

TABLE 11-5 HCM level of service criteria for arterials (1994 HCM, Table 11-1, page 11-4)

Arterial Class:	I	II	III	Others
Mid-Block Free-Flow Speed (Smb):	40 mph (64 kph)	33 mph (53 kph)	27 mph (43 kph)	Smb
L.O.S. A	35 mph (56 kph)	30 mph (48 kph)	25 mph (40 kph)	0.90 * Smb
L.O.S. B	28 mph (45 kph)	24 mph (39 kph)	19 mph (31 kph)	0.70 * Smb
L.O.S. C	22 mph (35 kph)	18 mph (29 kph)	13 mph (21 kph)	0.50 * Smb
L.O.S. D	17 mph (27 kph)	14 mph (23 kph)	9 mph (14 kph)	0.40 * Smb
L.O.S. E	13 mph (21 kph)	10 mph (16 kph)	7 mph (11 kph)	0.30 * Smb

where:

S_{cap} = speed at capacity (mph or km/hr)
L = length of segment (mi or km)
S_{mb} = midblock free-flow speed (mph or km/hr)
DF = delay adjustment factor (see notes to Equation 11-15)
C = cycle length (sec)
g = green time for through movement (sec)
T = length of time period being analyzed (hr)
m = calibration constant (see notes to Equation 11-16)
cap = capacity (vph) = saturation flow times g/C ratio.

Step 7b. Compute Segment Service Volume

The computed segment speed at capacity (S_{cap}) is compared with the minimum speed for the desired level of service (S_{LOS}) obtained from Table 11-5. If $S_{LOS} < S_{cap}$, use the following equation to estimate the service volume:

$$V_{LOS} = \frac{Cap}{1800T} * \left[\frac{L}{S_{LOS}} - \frac{L}{S_{Cap}} + 1800T \right] \tag{11-19}$$

where:

V_{LOS} = maximum service volume (vph)
S_{LOS} = minimum speed at desired level of service (mph or km/hr)
All other variables as defined previously in Equation 11-18.

Otherwise, if $S_{LOS} \geq S_{cap}$, use the following equation to estimate the service volume:

$$V_{LOS} = Cap * \left[\frac{S_0 - S_{LOS}}{S_0 - S_{cap}} \right] \tag{11-20}$$

where:

$$S_0 = \frac{L}{3600\ L/S_{mb} + \frac{1}{2}DF * C(1 - g/C)^2}$$

All other variables as defined previously in Equations 11-18 and 11-19.

REFERENCE

1. Kell, James H., and I.J. Fullerton. *Manual of Traffic Signal Design.* Prentice Hall, Englewood Cliffs, NJ, 1982.

CHAPTER 12

ACCURACY OF RECOMMENDED PROCEDURES

This chapter evaluates the accuracy of the recommended planning procedures for estimating facility speed and level of service.

12.1 OVERVIEW OF ACCURACY TESTS

The objective of the accuracy tests was to determine the relative accuracy of the recommended planning procedures for predicting mean facility speed and mean facility level of service under both congested and uncongested conditions. The test results could be used to give planners an idea of the likely variation of the predicted mean speed and level of service from the true results.

Three series of tests were performed: tests against real-world validation datasets, tests against simulation models, and tests against the HCM. The simulation model tests allowed the planning techniques to be evaluated against conditions that could not be found in the validation datasets. The HCM tests made it possible to test the planning techniques for their consistency with the HCM.

12.1.1 Tests on Real-World Datasets

Real-world datasets were tested against the 24 datasets (6 datasets for each of the four facility types) in the validation dataset, as described in Chapter 6. Input data in each dataset were used in each planning procedure to predict mean facility speed. The predicted speed was then compared with the field-measured mean speed for each dataset.

Spot speed survey data were the only data available for the urban and rural uninterrupted flow facility datasets; therefore, the planning techniques were tested for their ability to predict spot speeds (unadjusted to space mean speed or to facility length average speed) for uninterrupted flow facilities. For all other facility types (urban and rural interrupted), it was possible to verify the facility space mean speeds estimated by the planning techniques with floating car field measurements of mean speed over the length of the facility.

The real-world datasets, however, could not be used to test how well the planning procedures predicted delay caused by queuing. The datasets measured only actual volumes and not demand; thus, they were of limited use in evaluating the procedures under congested conditions. Simulation models were used to fill this gap in the available data.

12.1.2 Simulation Model Tests

The simulation model tests involved the expansion of a real-world dataset for a single freeway and a real-world dataset for a single urban arterial into a series of datasets for different levels of traffic demand. This allowed the research team to assess the performance of the planning techniques under a wide range of undercapacity and overcapacity demand conditions, while controlling for all other variations in the test data. Although the real-world dataset tests demonstrated the overall accuracy of the planning techniques, the simulation model tests illustrated the robustness of the planning techniques for overcapacity demand conditions.

The selected real-world datasets for expansion into simulation datasets were those for the I-880 freeway in Hayward, California, and Ventura Boulevard in Los Angeles, California. These two datasets were selected because of the availability of most of the data necessary for coding the simulation models. The necessary data included peak-hour turning movement counts and signal timing for each signalized intersection on the urban arterial and on-ramp, off-ramp, and mainline volumes for several hours of the peak period for the freeway.

Simulated volumes for a range of demand levels were obtained by factoring the real-world data volumes by factors ranging from 10 percent to 150 percent of the counts. The same percentage factor for that demand level test was applied to all ramp and mainline counts for the freeway dataset. For the arterial dataset, the percentage factor was applied only to the through movements on the arterial. This was done because the arterial simulation model, TRANSYT-7F, provides a convenient way to quickly factor the through movements on the arterial.

The University of California FREQ (*1*) model was selected to simulate the average facility speeds for the I-880 freeway for a range of volumes. The FREQ model is a deterministic, macroscopic simulation model that divides the freeway into a series of segments and the analysis period into a series of time slices. The demand for each time slice is loaded onto each segment of the facility. When demand exceeds capacity, the excess demand is queued in the upstream segment and discharged in a later time slice. The model employs macroscopic compressible fluid equations for computing queue propagation derived from the 1965 HCM. Speed and density are computed by these same equations.

The user has the option of inputting more up-to-date speed-flow curves, but this was not considered crucial for the purpose of the simulation tests. The 1994 HCM speed-flow curves do not increase as fast with increasing v/c ratios as the 1965 HCM curves, but the difference in the forecasted speeds between the two sets of curves is relatively minor, compared with differences between the HCM and the planning technique forecasts.

The FREQ simulation used 15-min volumes, whereas the planning techniques used 1-hr volumes. The FREQ-computed speeds are the space mean speed over the entire facility, averaged over the entire 6-hr peak period (2 p.m. to 8 p.m.) that was simulated in each FREQ model run.

The McTrans TRANSYT-7F (*2*) model was used to simulate average facility speeds for Ventura Boulevard for a range of through volume conditions. TRANSYT is a deterministic, macroscopic simulation model that divides the arterial into a series of segments between signals and divides the peak hour into time slices equal to the cycle length of the arterial signal system. The signal cycle is further divided into a sequence of steps. Conditions are simulated for each step within the cycle and then summed to determine conditions for the entire cycle. The results for the cycle are assumed to be identical for each cycle in the hour; therefore, the simulation results for the cycle are simply factored to the total hour.

TRANSYT computes the average delay at signals based on the number of vehicles stopped during each step of the signal cycle. An incremental delay term (*d2*), taken from the 1985 HCM, is added to the computed average delay to obtain the total signal delay. TRANSYT assumes that vehicles travel at the midblock free-flow speed between signals, but that vehicles in the platoon are delayed as they spread out behind the leading vehicles in the platoon.

The TRANSYT simulation runs were performed for all directions of traffic for Ventura Boulevard; however, the simulation results are reported only for the peak eastbound direction. (The results for the westbound direction were not analyzed to conserve analysis effort.) The TRANSYT-simulated mean speed is the AM peak-hour space mean speed for the eastbound through movement only, averaged over the entire facility.

12.1.3 Higway Capacity Manual Consistency Tests

The 1994 HCM speed and level of service estimates for urban interrupted flow facilities were obtained using the Florida DOT spreadsheet program ARTPLAN. ARTPLAN is an implementation of the urban and suburban arterial procedures documented in Chapter 11 of the 1994 HCM.

The 1994 HCM speed and level of service estimates for all other facilities (all uninterrupted flow facilities and rural interrupted flow facilities) were obtained by coding the HCM procedures for basic freeway sections (Chapter 3 of the HCM), multilane highways (Chapter 7 of the HCM), and two-lane rural roads (Chapter 8 of the HCM) into spreadsheets. The use of custom spreadsheets for the HCM methods facilitated the interchange of data between the real-world dataset spreadsheets and evaluation spreadsheets.

12.2 MEASURES OF ACCURACY

Different measures of accuracy were adopted for the speed and level of service estimates. Speed is a continuous variable susceptible to the usual statistical analysis such as mean error and root mean square (RMS) error. Level of service is a discrete variable taking only five values and, thus, required special treatment.

12.2.1 Accuracy Measures for Speed Estimation Procedures

The two statistical measures of accuracy used for the speed estimation techniques are bias and RMS error. They are computed as follows:

$$Bias = \frac{\sum_i^N (y_i - x_i)}{N} \tag{12-1}$$

$$RMS = \sqrt{\frac{\sum_i^N (y_i - x_i)^2}{N}} \tag{12-2}$$

where:

y_i = speed estimated by procedure, y, for facility, i
x_i = measured speed for facility, i
N = number of facilities in dataset.

Bias is an indicator of how much the speed estimation technique overestimates or underestimates the facility space mean speed on the average. Percent bias is the bias divided by the mean field-measured speed.

The RMS error indicates roughly the average error in the speed estimation for an individual facility. If the error (the difference between the observed and predicted speeds) follows a normal distribution, the RMS can be used to obtain the confidence interval for the speed estimation. Percent RMS is the RMS error divided by the mean field-measured speed.

12.2.2 Level of Service Accuracy Measures

Three measures of accuracy were selected for evaluating the level of service procedures: measure of agreement, percentage of observations resulting in the same level of service, and percentage of observations within one level of service.

12.2.2.1 Establishing "True" Level of Service

The evaluation of planning techniques for predicting level of service required that a base or "true" level of service be established for comparing the techniques. The 1994 HCM is used as the true method for predicting level of service for urban and rural uninterrupted flow facilities and rural interrupted flow facilities because the primary level of service measures for these facilities were not available from the datasets. Field measurements of level of service, rather than HCM-estimated levels of service, were used as the true level of service measures for urban interrupted flow facilities because field measurements of speed were available.

The percentage of observations in which the method produced the same level of service or produced level of service estimates within one level of the HCM or field measurements is self-explanatory. The statistical measure of agreement requires more explanation.

12.2.2.2 Definition of Measure of Agreement

The measure of agreement tests the agreement between test results measured by two observers. The test is described in the book *Discrete Multivariate Analysis* (*3*). The measure of agreement test requires that the level of service estimates made by the two different methods be organized in a table. Each study facility in the dataset generates a pair of level of service observations. One level of service is estimated using the HCM method; the other is estimated using the "test" method. For example, in Table 12-1, there were three facilities in which both the HCM method and the test method yielded Level of Service A. There was one facility in which the test method resulted in Level of Service B and the HCM method resulted in Level of Service A.

There would be 100 percent agreement between the two methods if all observations fell in the diagonal of the table (the cells in which A = A, B = B, C = C, and so on). In our example, 3 + 2 + 4 + 2 or 11 out of 24 (46 percent) of our observations fall in the diagonal of agreement. Is this good agreement or the same as we would get with random chance?

The number of observations we would obtain in the diagonal from random chance is equal to the row total times the column total divided by the total observations in the table. For example, for the A = A cell, the number of observations we would obtain by random chance is $6 * 6/24 = 1.5$ observations.

To compute the "strength" of agreement between the two methods, we take the agreement we observed (11 observations) and subtract from it the number of agreeing observations we would have gotten by random chance. We then normalize this result so that the measure of agreement will range between 0 and 1.00.

The normalized measure of agreement equals zero if we obtain the same number of agreeing observations as we would have gotten by random chance. A negative result means we obtained less agreement than we would have obtained from random chance. The measure of agreement equals 1.00 if all of the observations we obtained fall in the diagonal of agreement (cells in which A = A). The normalized measure of agreement, A, is computed as follows:

$$A = \frac{N\sum x_{ii} - \sum x_{i*} * x_{*i}}{N^2 - \sum x_{i*} * x_{*i}} \qquad (12\text{-}3)$$

where:

A = measure of agreement
N = total number of observations in the table
x_{ii} = number of observations falling in the cell located along the diagonal in row number, i, and column number, i (the row number equals the column number)
x_{i*} = total number of observations in row, i
x_{*i} = total number of observations in column, i.

For our preceding example:

$$A = \frac{24(3 + 2 + 4 + 2) - (6 * 6 + 5 * 7 + 8 * 7 + 5 * 4)}{24^2 - (6 * 6 + 5 * 7 + 8 * 7 + 5 * 4)}$$

where $A = 0.2727$.

We have 27 percent agreement between the two methods. This is better than random chance (because it is greater than zero) but it is not strong agreement (because it is quite a bit less than 100 percent). Among the tests performed as part of this research, measures of agreement on the order of 0.30 occurred when two methods predicted the same level of service slightly more than 50 percent of the time and within one level of service 85 percent of the time. Thus, a measure of agreement of 0.30 would be quite satisfactory for planning purposes.

TABLE 12-1 Tabulation of level of service results

Test (across) vs HCM (down)	A	B	C	D	Total
HCM LOS "A"	3	1	1	1	6
HCM LOS "B"	2	2	2	1	7
HCM LOS "C"	1	1	4	1	7
HCM LOS "D"	0	1	1	2	4
Total	6	5	8	5	24

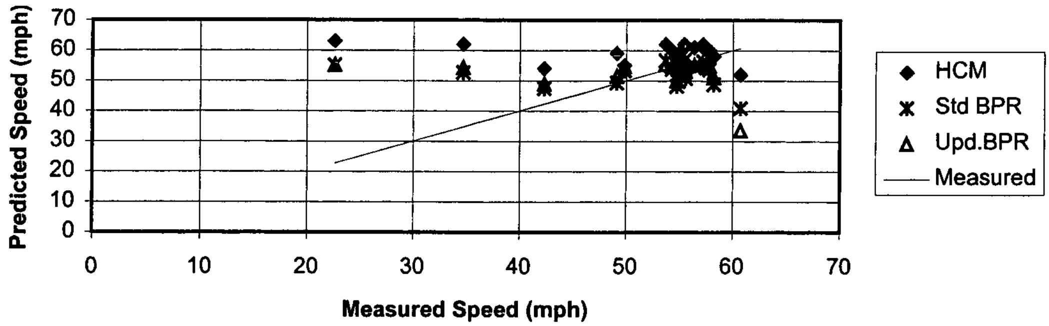

Figure 12-1. Accuracy of speed estimation procedures for urban uninterrupted flow facilities.

12.3 FIELD DATA TEST RESULTS—SPEED

The field data tests evaluated the following speed estimation techniques against field measurements of mean facility speed for 24 datasets:

- The standard BPR technique (described in Chapter 3),
- The updated BPR technique (described in Chapter 8),
- HCM techniques (described in Chapter 3), and
- The expanded ARTPLAN technique (described in Chapter 9.

The datasets were split into five categories by facility and area type:

- Urban uninterrupted,
- Rural uninterrupted,
- Multilane rural interrupted,
- Two-lane rural interrupted, and
- Urban interrupted.

These five categories were selected to correspond with the different techniques used to estimate speed for each facility and area type.

The results of the tests are summarized in Figures 12-1 through 12-5 and Tables 12-2 and 12-3, which follow. (The enhanced BPR technique is referred to as "upd. BPR" in this series of figures.) The straight line in each figure shows where the predicted speeds would equal the actual speeds.

The standard and enhanced BPR methods were applied to the datasets using a custom spreadsheet. The enhanced ARTPLAN was applied by modifying the original ARTPLAN spreadsheet as follows:

1. The running time look-up table was replaced with the free-flow speed equation, which is based on the posted midblock speed limit.
2. The estimated queuing delay resulting from overcapacity conditions at any of the intersections was added to the computed total travel time and divided into the facility length to obtain mean facility speed.
3. Exact signal timing, signal spacing, and volumes were used for each segment rather than averages for the entire facility.

Figure 12-1 shows the results for 14 speed and volume data points for six urban uninterrupted facilities. Two of the data points, where the measured speed is less than 40 mph (64 km/hr), reflect conditions in which demand exceeds capacity. For these conditions, the validation dataset has volumes only and not demand data. Thus, none of the techniques are able to predict correct congested speeds. These two data points were dropped from further evaluation.

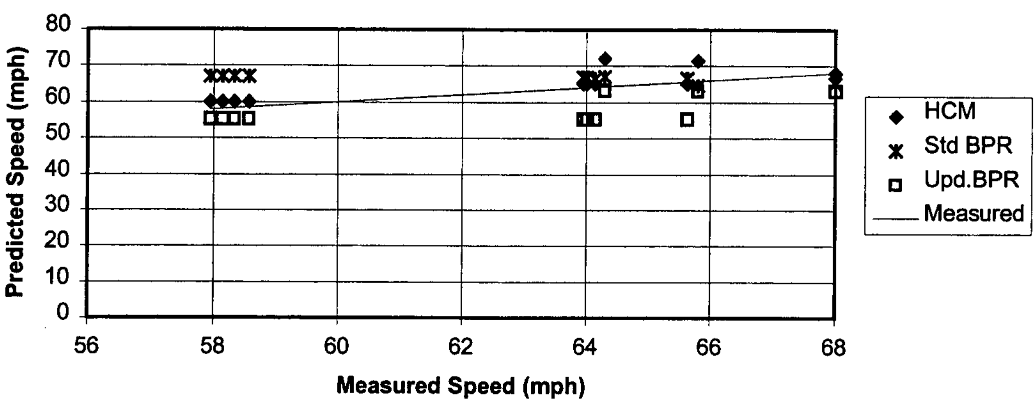

Figure 12-2. Accuracy of speed estimation procedures for rural uninterrupted flow facilities.

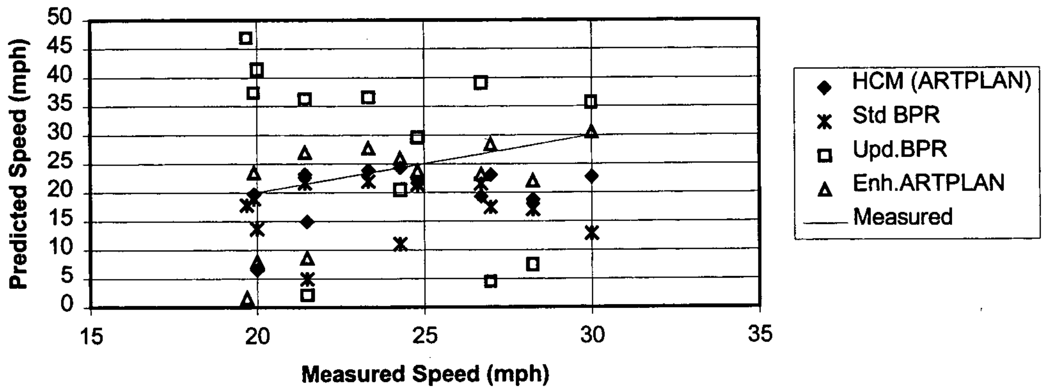

Figure 12-3. Accuracy of speed estimation procedures for urban interrupted flow facilities.

There is one data point where the measured speed exceeds 60 mph (96 km/hr) and where the validation dataset appears to have a capacity that is lower than actual capacity. This underestimate of the capacity causes all the techniques to underestimate the true mean speed.

Figure 12-1 illustrates the results for 10 observations made on six rural uninterrupted flow facilities. All techniques fit the data well, but the range of observed speed data is narrow (58 mph (93 km/hr) to 68 mph (109 km/hr)). Two outlier speed observations on Florida State Route 808 were dropped from the chart because of apparent errors in the estimated capacity of the facility.

Figure 12-2 shows the results for 12 observations made on six urban interrupted flow facilities. The spread in the predictions is five times greater than the range of measured speeds. The standard and enhanced BPR techniques performed poorly. The HCM technique, which is implemented in these tests using the original ARTPLAN and enhanced ARTPLAN methods, is significantly closer to the observed speeds, but still shows quite a bit of variation. The HCM-estimated speed is always lower than the measured speed. The enhanced ARTPLAN speed estimates tend to straddle the measured speeds better than the HCM estimates.

Figure 12-3 shows the results for six observations on three multilane rural interrupted flow facilities. The standard BPR technique underestimates the speeds because it is based on a look-up table that underestimates the capacity for rural roads. The HCM and enhanced BPR both fit the observed data well. The enhanced ARTPLAN technique produced the same results as the enhanced BPR technique and consequently is not plotted separately in this figure.

Figure 12-4 shows the results for 19 observations made on three two-lane rural interrupted flow facilities. Again, the standard BPR technique underestimates speeds because it is using a look-up table value for capacity that underestimates the capacity of rural roads. All other methods cluster around the measured speeds. The HCM method was unable to estimate speeds for facilities with free-flow speeds less than 60 mph (96 km/hr).

Table 12-2 summarizes the results of the tests by facility type in terms of the average bias and RMS error.

The HCM methods are the best overall performers of the methods evaluated. They have the lowest bias and RMS error in almost all cases. The HCM methods have the greatest difficulty in predicting mean speeds for urban interrupted flow facilities.

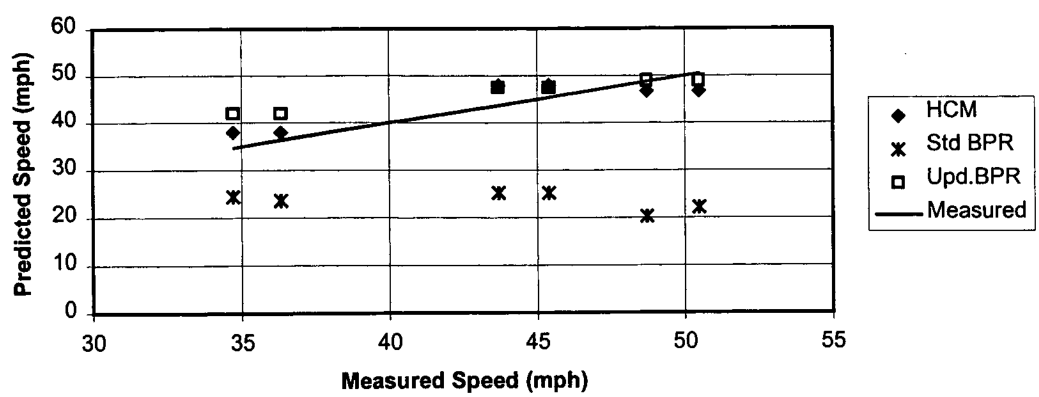

Figure 12-4. Accuracy of speed estimation procedures for multilane rural interrupted flow facilities.

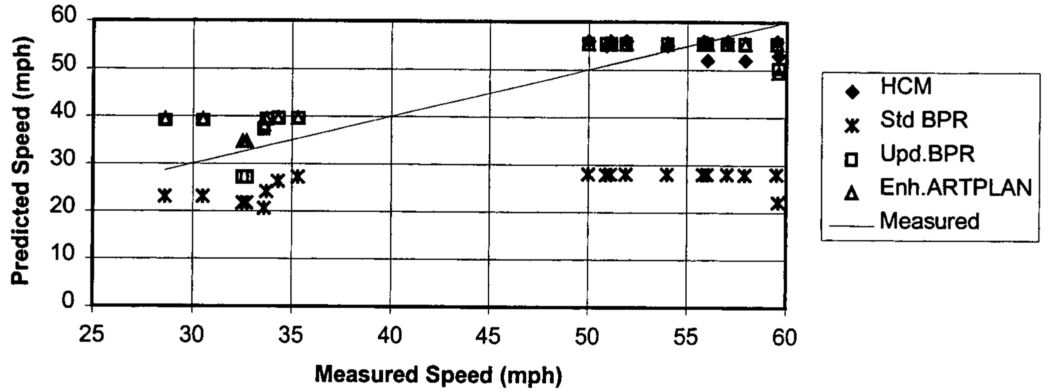

Figure 12-5. Accuracy of speed estimation procedures for two-lane rural interrupted flow facilities.

The enhanced BPR method is a significant improvement over the standard BPR method for rural interrupted flow facilities and urban interrupted flow facilities. The performance of the two techniques are similar on the uninterrupted flow facility datasets because of the lack of speed observations in the validation dataset for conditions on uninterrupted flow facilities in which demand is approaching capacity.

The enhanced ARTPLAN technique is second only to the HCM techniques in terms of its performance.

In many cases, the apparent superiority of the HCM methods resulted from the fact that they could not be applied to some data points (thus reducing the number of observations on which the RMS error is calculated), which created an RMS error that appears to be lower than that for the enhanced ARTPLAN technique. This is particularly true for two-lane rural roads where the HCM methods could not be applied to six of the 19 observations. The HCM methods also could not be applied to one of the urban interrupted observations because demand exceeded capacity at one of the intersections.

The uninterrupted flow datasets were spot speed observations; therefore, both the enhanced ARTPLAN and enhanced BPR techniques yielded the same speed results in cases in which the facility has, in effect, only one segment.

12.4 SIMULATION RESULTS—SPEED

None of the available datasets in the validation dataset allowed for testing of conditions in which demand exceeded capacity. Consequently, simulation model runs with differing demand volumes were used to test the planning methods on overcapacity conditions.

The simulation tests artificially constrained all the planning techniques to use the same free-flow speed and capacity as were used in the simulation model. The purpose was to test how well the planning techniques could predict queuing delay resulting from overcapacity conditions and to separate this effect from errors that would be introduced if the planning techniques used differing capacities and free-flow speeds.

Figure 12-6 illustrates the results of the simulation runs for the freeway. The enhanced ARTPLAN technique produces speed estimates for overcapacity conditions that are close to the FREQ simulation results. The enhanced BPR technique underestimates the delay caused by overcapacity conditions for freeways.

Figure 12-7 shows that the enhanced ARTPLAN estimates of speed for each hour within the peak period are comparable to those of FREQ. This is an important conclusion because FREQ uses compressible fluid theory to compute

TABLE 12-2 Accuracy of existing and recommended speed estimation techniques

	Standard BPR		Enhanced BPR		Enhanced ARTPLAN		1994 HCM	
Facility Type	Bias	RMS	Bias	RMS	Bias	RMS	Bias	RMS
Urban Uninterrupted	-7%	12%	-6%	14%	-6%	14%	+5%	9%
Rural Uninterrupted	+6%	9%	-9%	10%	-9%	10%	+3%	5%
Multi-Lane Rural Int.	-46%	49%	+7%	10%	+7%	10%	+3%	7%
Two-Lane Rural Int.	-43%	48%	+3%	12%	+6%	11%	+0%	8%
Urban Interrupted	+31%	39%	+15%	35%	+13%	33%	-19%	26%

Entries are percent of mean observed speed for that facility type.

TABLE 12-3 Mean and range of observed mean speeds for validation dataset

Facility Type	Mean (mph)	Range (mph)	Mean (kph)	Range (kph)
Urban Uninterrupted	55.8	(50-61)	89.3	(80-98)
Rural Uninterrupted	62.9	(58-68)	100.6	(93-109)
Multi-Lane Rural Interrupted	43.2	(35-51)	69.1	(56-82)
Two-Lane Rural Interrupted	54.9	(51-60)	87.8	(82-96)
Urban Interrupted	24.3	(20-30)	38.9	(32-48)

queuing, whereas the enhanced ARTPLAN uses classical deterministic queuing theory to compute delay. This figure shows that both techniques result in similar predicted mean speeds for each hour in the peak period.

Figure 12-8 shows the results of the arterial simulation. The enhanced BPR technique underestimates the impact of overcapacity conditions until extreme congestion is reached. Then the technique overestimates delay (estimated speeds are lower than the TRANSYT model estimate).

The HCM technique (ARTPLAN modified to produce speed estimates even when the v/c ratio exceeds 1/PHF) would overestimate speeds for overcapacity conditions if it were used to predict speeds under these conditions. The HCM recommends against its use under these conditions.

The enhanced ARTPLAN (labeled ARTPLAN + Queue in the figure) produces speed estimates closer to the TRANSYT estimate for overcapacity conditions.

Figure 12-9 shows the sensitivity of the planning techniques to changes in signal timing for constant demand conditions. ARTPLAN and enhanced ARTPLAN (labeled ARTPLAN + Queue in the figure) come closer to matching TRANSYT results than the enhanced BPR technique (labeled "Upd. BPR" in the figure).

These simulation results cannot be used to state that one method is more accurate than the other, because they are based on simulated data. However, the results do show that the enhanced ARTPLAN technique is superior to the standard and enhanced BPR techniques in reproducing the likely speed effects of conditions in which demand exceeds capacity.

12.5 HCM CONSISTENCY RESULTS—SPEED

Table 12-4 contains the results of tests on the consistency of the planning techniques' speed estimates with the HCM speed estimates for the validation dataset. The enhanced ARTPLAN method comes closest to matching the HCM estimates.

12.6 LEVEL OF SERVICE ACCURACY

This section evaluates the accuracy of the level of service estimation methods. For uninterrupted flow facilities and rural interrupted flow facilities, the planning techniques are compared with HCM estimates of level of service. For urban interrupted flow facilities, it was possible to use field measurements of level of service (mean facility speed) to evaluate the planning techniques.

Table 12-5 shows the results of the measure of agreement tests. Most of the techniques performed well on urban uninterrupted flow facilities (measure of agreement $\geq$ 0.30), but none of the techniques did well for rural uninterrupted flow facilities.

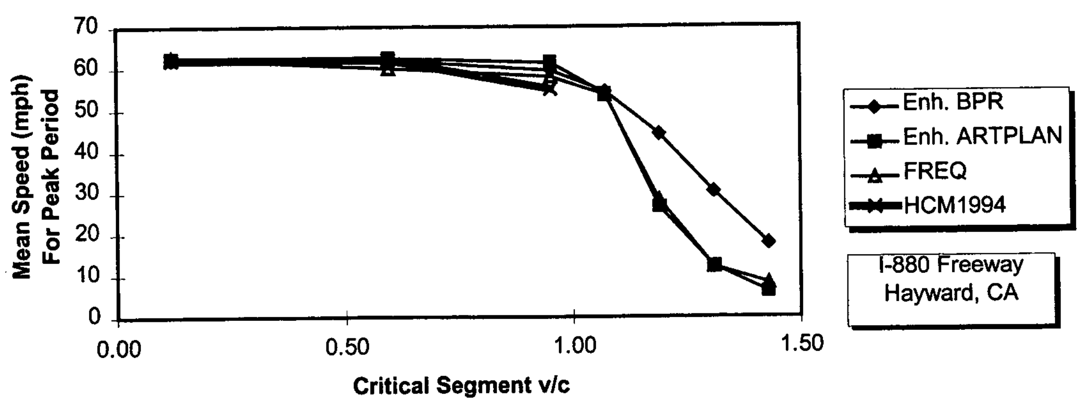

Figure 12-6. Comparison of enhanced BPR and ARTPLAN techniques with FREQ simulation results.

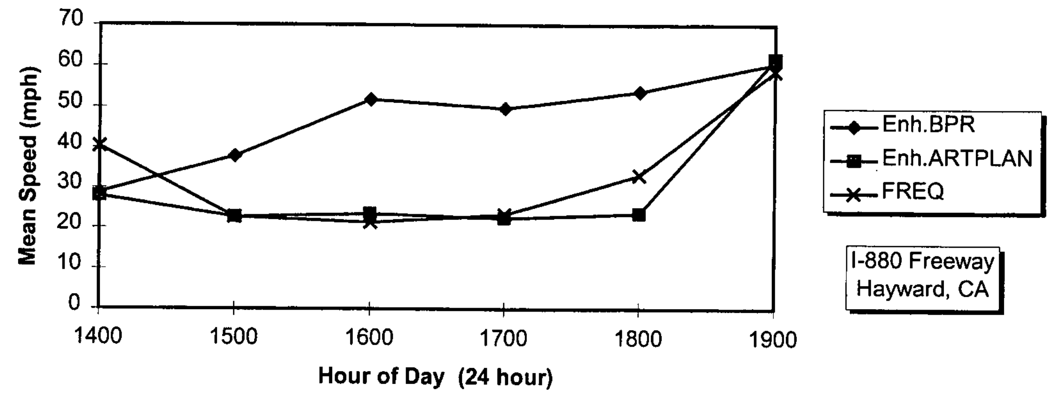

Figure 12-7. Performance of enhanced BPR and ARTPLAN techniques within the peak period for a freeway.

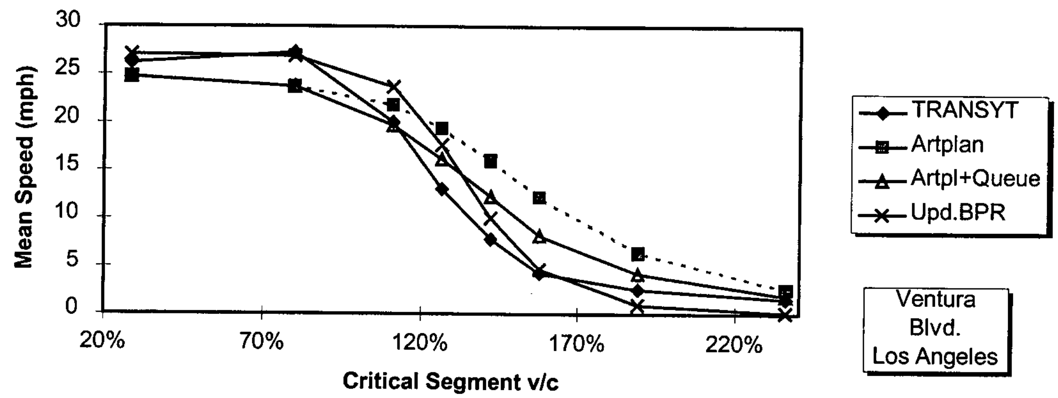

Figure 12-8. Simulation of ARTPLAN, enhanced ARTPLAN, and updated BPR over range of volumes.

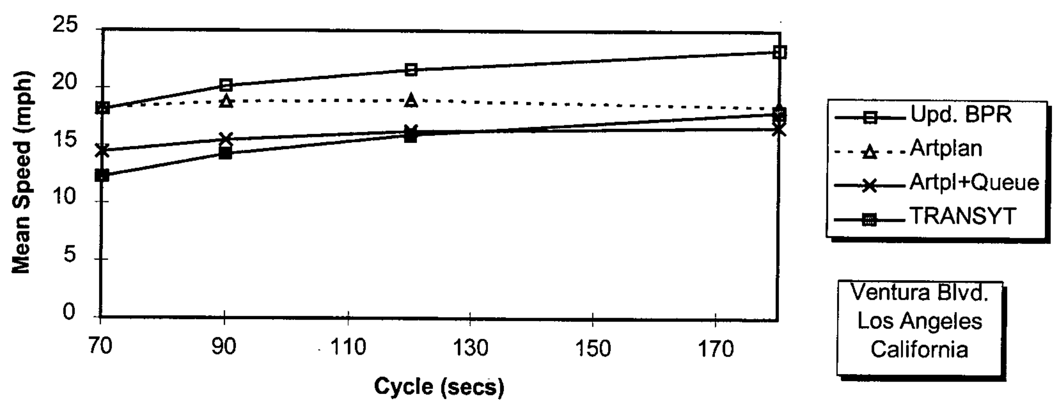

Figure 12-9 Simulation of ARTPLAN, enhanced ARTPLAN, and updated BPR over range of cycles.

TABLE 12-4 Consistency of existing and recommended speed estimation techniques with 1994 HCM

	Standard BPR		Enhanced BPR		Enhanced ARTPLAN	
Facility Type	Bias	RMS	Bias	RMS	Bias	RMS
Urban Uninterrupted	-11%	12%	-11%	13%	-11%*	13%*
Rural Uninterrupted	+3%	8%	-11%	12%	-11%*	12%*
Multi-Lane Rural Int.	-27%	50%	+4%	6%	+4%	6%
Two-Lane Rural Int.	-47%	49%	0%	3%	0%	3%
Urban Interrupted	-16%	33%	0%	38%	+14%	22%

Entries are percent of mean HCM estimated speed for that facility type.
*The enhanced ARTPLAN estimates are identical to the enhanced BPR estimates for the uninterrupted flow datasets because these datasets consist of only a single element, and there is no congestion present.

TABLE 12-5 Comparison of measures of agreement for level of service methods

	Urban	Rural	Rural	Urban
Method	Uninterrupted	Uninterrupted	Interrupted	Interrupted
Standard BPR	0.10	-0.06	0.15	0.04
Florida General. Tables	0.30	-0.04	-0.01	-0.07
Florida Table Gen. Software	0.39	0.09	0.24	0.04
Updated BPR	0.36	0.09	0.66	0.06
Enhanced ARTPLAN	n/a	n/a	0.40	0.29

TABLE 12-6 Percentage of tests for which estimated level of service is equal to true level of service

	Urban	Rural	Rural	Urban
Method	Uninterrupted	Uninterrupted	Interrupted	Interrupted
Standard BPR	28%	8%	21%	8%
Florida General. Tables	54%	8%	21%	17%
Florida Table Gen. Software	62%	33%	43%	33%
Updated BPR	62%	33%	71%	17%
Enhanced ARTPLAN	n/a	n/a	50%	50%

TABLE 12-7 Percentage of tests for which estimated level of service is within one level of true level of service

	Urban	Rural	Rural	Urban
Method	Uninterrupted	Uninterrupted	Interrupted	Interrupted
Standard BPR	89%	67%	64%	17%
Florida General. Tables	85%	58%	86%	50%
Florida Table Gen. Software	85%	92%	100%	67%
Updated BPR	92%	92%	100%	33%
Enhanced ARTPLAN	n/a	n/a	100%	67%

Table 12-6 shows the percentage of tests for which the planning techniques obtained the same level of service as was measured in the field or estimated by the HCM method. The enhanced BPR and enhanced ARTPLAN techniques are the best performing techniques across the board. ARTTAB, the Florida table generating software for arterials, is superior to the enhanced BPR curve for urban interrupted flow facilities.

Table 12-7 shows the percentage of tests for which the planning techniques obtained results within one level of service (better or worse) of the HCM or field estimates. Again, the enhanced BPR and enhanced ARTPLAN techniques are the best across the board. ARTTAB, the Florida table generating software for arterials, is superior to the enhanced BPR curve for urban interrupted flow facilities.

For comparison purposes, here is how the HCM method (the original ARTPLAN) performed against the field data :

- Measure of agreement: −0.08
- Percentage of time HCM predicted same level of service as in field: 17 percent
- Percentage of time HCM predicted worse level of service than in field: 83 percent
- Percentage of time HCM predicted better level of service than in field: 0 percent
- Percentage of time HCM was within one level of service of field: 58 percent.

The HCM method did not perform well in these tests.

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CHAPTER 13

EVALUATION OF RECOMMENDED PROCEDURES

This chapter provides a comprehensive evaluation of the suitability of the recommended speed and level of service estimation procedures for planning applications.

13.1 EVALUATION CRITERIA

The speed and level of service estimation techniques are evaluated against seven criteria:

- Data requirements
- Ease of use
- Accuracy of results
- Consistency with the HCM
- Applications
- Ease of incorporation into the HCM
- Sensitivity to transportation system management (TSM) and transportation control measures (TCMs).

Each of these criteria is explained in the following paragraphs. The level of service estimation techniques are evaluated separately from the speed estimation techniques, because some of the level of service techniques are not adaptable to speed estimation.

13.1.1 Data Requirements

Data requirements include the amount and type of data that are required by the technique. The ease of data collection is assessed based on the results of the user survey. The required precision of the input data is assessed based on the results of the accuracy evaluation.

The difficulty of data collection is rated according to the following general results from the user survey:

Data Type	*Difficulty*
Traffic Counts	Easy
Lanes	Easy
Speed Limit	Medium
Signal Timing	Hard
Design Data (grades, curvature, widths)	Hard
Miscellaneous (trucks, parking)	Hard

13.1.2 Ease of Use

Ease of use deals with the complexity of the technique, how difficult it is to learn, whether it can be implemented in a spreadsheet, and whether it requires iterations to reach a solution. Sensitivity to quality of input data is included in this criterion.

13.1.3 Accuracy of Results

The accuracy of the method is evaluated in comparison with existing techniques. The results of the validation dataset accuracy tests discussed in the previous chapter are summarized here in terms of the general terms in Table 13-1, based on the maximum bias and RMS error observed for each technique.

13.1.4 Consistency with the HCM

The accuracy test results using the validation datasets are used to assess the consistency of the planning application results with the results that would be obtained using the appropriate HCM method. This analysis was discussed in the previous chapter and is summarized here using the same qualitative terms (very good, good, fair, and poor) as for the overall accuracy assessment tabulated earlier.

13.1.5 Applications

This criterion discusses the range of planning applications and facility types addressed by the planning technique. The range of planning applications are described in Chapter 1. They consist of the following:

- Long-range transportation planning (LRTP)
- Transportation improvement programs (TIPs)
- Major investment studies (MISs)
- Congestion management systems (CMSs)
- Growth management programs (GMPs)
- Air quality conformity studies (AQCSs)
- Intermodal planning studies (IPSs)
- Highway performance monitoring systems (HPMSs)
- Site impact analyses.

The facility types include the following:

- Urban uninterrupted flow facilities (urban freeways)
- Urban interrupted flow facilities (urban streets)

TABLE 13-1 Definition of qualitative terms for accuracy

Qualitative Term	Maximum Bias	Maximum RMS Error
Very Good	5%	10%
Good	10%	20%
Fair	20%	40%
Poor	> 20%	> 40%

- Rural uninterrupted flow facilities (rural freeways)
- Rural interrupted flow facilities (multilane and two-lane rural roads).

13.1.6 Ease of Incorporation into the HCM

This is a qualitative criterion that assesses the relative amount of effort that would be required to incorporate the planning technique into the next update of the HCM. This criterion also addresses the likely difficulty of updating the technique to keep abreast of future changes in the HCM techniques for operations analysis.

13.1.7 Sensitivity to TSM and TCMs

This criterion qualitatively assesses the relative sensitivity of the planning techniques to typical TSM and TCMs such as demand management (staggered work hours, ridesharing promotion); highway operations improvements (signal optimization, peak-hour turn prohibitions); and high occupancy vehicle incentives (HOV lanes).

13.2 EVALUATION OF SPEED ESTIMATION TECHNIQUES

This section evaluates the standard BPR, enhanced BPR, and expanded ARTPLAN speed estimation techniques. These techniques are evaluated in comparison with the existing HCM techniques, where appropriate.

13.2.1 Data Requirements

The data requirements for the standard BPR, HCM, enhanced BPR, and enhanced ARTPLAN techniques are shown in Tables 13-2 through 13-5.

Data requirements are minimal for the standard BPR curve (see Table 13-2), and the data generally are easy for planning agencies to obtain. Defaults are not available for any of the required data; however, because of their basic nature (volume, facility type, area type, and so on), they likely would not be useful even if they were available. The standard BPR curve derives hourly Level of Service C capacity per lane and free-flow speed from look-up tables based on area and facility type and, for arterials, on-street parking and one- or two-way operation. Speed predictions are based on hourly volume, capacity, and free-flow speed.

Unlike the standard BPR curve, which provides one methodology for all facility types, the 1994 HCM provides separate methods for freeways, multilane highways, arterials, and rural two-lane highways. Table 13-3 lists the data requirements for the HCM methods, the availability of defaults for each item, and the feasibility of planning agencies to obtain each item.

Table 13-3 illustrates that HCM methods require considerably more data than the standard BPR curve and that the data are less feasible for agencies to obtain. However, default values are available for most of the items that are more difficult to obtain, with the exception of percent heavy vehicles.

Data requirements for the enhanced BPR and ARTPLAN methods are greater than those for the basic methods. However, defaults are available for all items that determine capacity in the two methods. As shown in Table 13-4 and Table 13-5, the factors for which defaults are not available are, for the most part, basic in nature and easy to obtain. Data requirements for the enhanced ARTPLAN method are similar to those for 1994 HCM methods; however, for a given facility type, the enhanced ARTPLAN method requires fewer data than does the corresponding HCM method.

13.2.2 Ease of Use

The methods analyzed differ greatly in the amount of training required, their ease of use, and the computational

TABLE 13-2 Data requirements for standard BPR method

Item	Defaults Available?	Ease to Obtain[a]
Hourly volume	No	Easy
Number of lanes	No	Easy
Free-Flow Speed	Look-Up Table[b]	Medium
Capacity per lane	Look-Up Table[b]	Hard

[a] If defaults are not used.
[b] Look-up table requires data on facility type, area type, presence of on-street parking, one-way/two-way operation.

TABLE 13-3 Data requirements for 1994 HCM methods

Item	Facility Type[a]	Defaults Available?	Ease to Obtain
Hourly volume	F,M,A,R	No	Easy
Peak hour factor		Yes	Easy
Lane/shoulder width	F,M,R	Yes	Hard
Population factor	F	Yes	Easy
Percent heavy vehicles	F,M,A,R	Yes (A,R)	Hard
Number of lanes	F,M,A	No	Easy
Terrain	F,M,R	No	Easy
Directional distribution	R	Yes	Easy
Arterial classification (I, II, or III)	A	No	Easy
Signal controller type (pretimed, actuated)	A	No	Hard
Effective green time and cycle	A	Procedure[b]	Hard
Coordination quality	A	Yes	Hard
Percent turns from exclusive lanes	A	No	Easy
Medians	A	No	Easy
Left-turn bays or exclusive left-turn lanes	A	No	Easy

[a]Facility Types: F—Freeway, M—Multilane, A—Arterial, R—Rural Two-Lane
[b]Procedure provided in Chapter 9 Planning Method for calculating effective green and cycle time.

TABLE 13-4 Data requirements for enhanced BPR curve

Item	Facility Type[a]	Defaults Available?	Ease to Obtain
Hourly volume	all	No	Easy
Number of lanes	all	No	Easy
Posted speed limit	all	No	Medium
Capacity per lane (or following data)	all	Procedure[b]	Hard
Peak hour factor	all	Yes	Easy
Facility Type	all	No	Easy
Percent heavy vehicles	all	Yes	Hard
Terrain	UU,RU,RI	No	Easy
Lane/shoulder width	2-RI	Yes	Hard
Percent no-passing zones	2-RI	Yes	Hard
Directional distribution	2-RI	Yes	Easy
Left turn bays	UI	No	Easy
Left turn phase protection	UI	No	Hard
Effective green time and cycle time	UI	Yes	Hard
Area type (CBD or not)	UI	No	Easy

[a]UU=Urban Uninterrupted, UI=Urban Interrupted, RU=Rural Uninterrupted, RI=Rural Interrupted, 2-RI=Two-Lane Rural Interrupted.
[b]Procedure provided that requires data listed in rest of table.
Note: All data items are for only the critical segment of the facility.

TABLE 13-5 Data requirements for enhanced ARTPLAN

Item	Facility Type[a]	Defaults Available?	Ease to Obtain
Hourly volume	all	No	Easy
Number of lanes	all	No	Easy
Posted speed limit	all	No	Medium
Segment lengths	all	No	Easy
Peak hour factor	all	Yes	Easy
Facility Type	all	No	Easy
Percent heavy vehicles	all	Yes	Medium
Terrain	UU,RU,RI	No	Easy
Lane/shoulder width	2-RI	Yes	Hard
Percent no-passing zones	2-RI	Yes	Hard
Directional distribution	2-RI	Yes	Easy
Intersection turn moves	UI	No	Medium
Left turn bays	UI	No	Easy
Left turn phase protection	UI	No	Medium
Effective green time and cycle time	UI	Yes	Hard
Area type (CBD or not)	UI	No	Easy

Note: All data items are for each segment of the facility.
[a]UU=Urban Uninterrupted, UI=Urban Interrupted, RU=Rural Uninterrupted, RI=Rural Interrupted, 2-RI=Two-Lane Rural Interrupted.

time needed. Table 13-6 summarizes the differences among the four methods.

The standard BPR curve is easiest to use in all respects. Because it consists of a single equation that relies on a single look-up table for all of its inputs other than volume, it takes little time to learn. The enhanced BPR curve takes longer to set up because of the need to develop a customized capacity look-up table, but otherwise it is as easy to use as the standard BPR curve. The 1994 HCM uses a different method for each facility type, a process that requires anywhere from a 2- to 3-day training class to a college-level course to learn. The HCM methods' data also take considerably longer to collect and input, compared with the BPR curves. The enhanced ARTPLAN method is easier to use than the HCM methods, because it provides one process for all facility types, although certain equations differ depending on facility type. At the same time, because of the amount of data required, the enhanced ARTPLAN method is considerably more difficult to use than the BPR curves.

Even though all of the methods lend themselves to computerization, the HCM and enhanced ARTPLAN methods require significantly more calculations, which means that large-scale applications using these methods take substantially longer. The BPR curve is easily calculated on a spreadsheet. The enhanced ARTPLAN method also can be calculated on a spreadsheet, but it takes more time to set up the spreadsheet. The HCM methods require specialized software or considerable time and expertise to program on a spreadsheet.

13.2.3 Accuracy of Results

The most appropriate measures for comparing the accuracy of each speed prediction method examined are considered to be the statistics of mean bias and RMS error, both with respect to actual measured speeds. The mean bias provides the best indication of whether the method is likely to underestimate or overestimate speed. The RMS error provides the best indication of whether the method is likely to provide consistent estimates of speed.

Table 13-7 summarizes qualitatively the findings of the investigation into the accuracy of each method for each facility type using the mean biases and RMS errors detailed in the previous chapter and the qualitative criteria described at the beginning of this chapter.

Table 13-7 demonstrates that the existing HCM procedures for rural roads, when they are applicable, produce very good results. Results for the urban uninterrupted facilities, when they are applicable, are good. Although only fair, the

TABLE 13-6 Comparison of ease of use of methods

Criterion	Standard BPR	Enhanced BPR	1994 HCM	Enhanced ARTPLAN
Complexity	two steps, one equation for all facility types	two steps, one equation for all facility types	multiple steps, multiple processes varying by facility type	multiple steps, one process for all facility types
Training Time	minutes	hours	days	one day
Application Time	minimal to gather data and perform calculation	moderate to gather data and construct lookup table, minimal to perform calculation	substantial to gather data and input for calculation	substantial to gather data and input for calculation
Required Software	spreadsheet	spreadsheet	HCS or equivalent spreadsheet	revised Florida spreadsheets (i.e., ART_PLAN)

HCM procedure for urban interrupted facilities produces the best results. Overall, the HCM procedures are useful tools for predicting speed for planning purposes.

The BPR curve using the look-up values provided for capacity at Level of Service C and free-flow speed is poor overall. This principally is a function of the limit in the applicability of the look-up values provided. The enhanced BPR method is mostly an improvement over the BPR look-up method; however, it is not as accurate as the HCM methods. Its advantage over HCM methods is its ease of use.

The enhanced ARTPLAN method gives results that are equivalent in terms of accuracy to the results from the enhanced BPR method for rural uninterrupted facilities. The enhanced ARTPLAN method gives results that are better in terms of accuracy than results from the enhanced BPR method for urban uninterrupted facilities. An advantage of the enhanced ARTPLAN method is that it can be used to estimate average speeds along an entire corridor, also accounting for delays at nodes between individual links, as can the HCM method. The method gives slightly closer estimates than the HCM method, although more spread is evident.

The validation datasets for uninterrupted flow facilities consisted of spot speed data only. Thus, the accuracy of the various methods for these facilities is compared in terms of their spot speed estimation capabilities, not their facility mean speed estimation capabilities. The available data did not allow the application of the enhanced ARTPLAN method to the uninterrupted flow datasets.

13.2.4 Consistency with 1994 HCM

Again, the most appropriate measures for assessing agreement between the HCM and each of the alternative methods examined are considered to be the statistics of mean bias and RMS error, both with respect to predicted HCM speeds. Table 13-8 summarizes qualitatively the findings of the

TABLE 13-7 Accuracy of each method

Facility Type	1994 Highway Capacity Manual	BPR Curve	Enhanced BPR Curve	Enhanced ARTPLAN Method
Urban Uninterrupted	Good	Good	Good	Good[a]
Rural Uninterrupted	Very Good	Good	Good	Good[a]
Rural 2 Lane Interrupted	Very Good	Poor	Good	Good
Rural 4 Lane Interrupted	Very Good	Poor	Good	Good
Urban Interrupted	Fair	Poor	Fair	Fair

[a] The Enhanced ARTPLAN method predicts facility mean speed based upon segment data. The validation data sets for uninterrupted flow facilities consisted of spot speed data only and therefore do not contain the necessary segment information. In the absence of segment data for the freeways, the Enhanced ARTPLAN yields the same estimates as the Enhanced BPR Method.

TABLE 13-8 Consistency of methods with the 1994 HCM

Facility Type	BPR Curve	Enhanced BPR Curve	Enhanced ARTPLAN Method
Urban Uninterrupted	Fair	Fair	Fair[a]
Rural Uninterrupted	Very Good	Fair	Fair[a]
Rural 2 Lane Interrupted	Poor	Very Good	Very Good
Rural 4 Lane Interrupted	Poor	Very Good	Very Good
Urban Interrupted	Fair to Poor	Fair	Fair

[a] The Enhanced ARTPLAN method predicts facility mean speed based upon segment data. The validation data sets for uninterrupted flow facilities consisted of spot speed data only and therefore do not contain the necessary segment information. In the absence of segment data for freeways, the Enhanced ARTPLAN yields the same estimates as the Enhanced BPR Method.

investigation into the agreement of each method with the HCM for each facility type using the mean biases and RMS errors detailed in the previous chapter. The results were again stratified from very good to poor, based on the previously described criteria for maximum bias and RMS error.

The table indicates that the BPR method is largely inconsistent with the HCM procedures. For all facilities, the enhanced BPR method generally provides better agreement with the HCM than the standard BPR method. However, the enhanced BPR still has poor agreement for the highly variable urban interrupted facilities. Although this is not considered to be a disadvantage of this method with respect to the HCM procedure, it does highlight the difficulty in reliably predicting corridor speeds on this type of facility. The enhanced ARTPLAN is mostly the same on a link basis as the enhanced BPR method for rural interrupted facilities and produces results that are equally consistent. The enhanced ARTPLAN had the best agreement with the HCM for urban interrupted facilities.

13.2.5 Applications

Table 13-9 summarizes the four methods' suitability for various planning applications.

Although the standard BPR curve can theoretically be applied to any facility type, it is not particularly suited to interrupted flow facilities because it does not take signal timing into account. The standard look-up table used in the standard BPR method underestimates the capacity of rural interrupted flow facilities. The method's ease of calculation makes it a frequent choice for regional transportation planning, but its lack of detail limits its usefulness for other planning applications.

TABLE 13-9 Applicability of speed estimation techniques

Criterion	Standard BPR	Enhanced BPR	1994 HCM	Enhanced ARTPLAN
Number of Procedures Needed	one equation fits all facility types	one equation fits all facility types	specialized method for each facility type	one process applies to all facility types
Suitability by Facility Type	Not suitable for Urban or Rural Interrupted.	Weak on Urban Interrupted.	No procedure for low speed 2-lane rural roads.	Suitable for all facilities
Planning Applications[a]	LRTP & Sketch Planning	LRTP & Sketch Planning	CMS, GMP, site impact analysis Can't be applied when demand exceeds capacity	TIP, MIS, CMS, GMP, AQCS, IPS, HPMS, site impact analysis

[a] Planning applications: Long Range Transportation Plans (LRTP's) & Sketch Planning, Transportation Improvement Programs (TIP's), Major Investment Studies (MIS), Congestion Management Systems (CMS), Growth Management Programs (GMP), Air Quality Conformity Studies (AQCS), Intermodal Planning Studies (IPS), Highway Performance Monitoring Systems (HPMS), and site impact analysis.

The HCM methods, on the other hand, provide specialized methods for each facility type that take into account more factors than does the standard BPR curve. The time required to perform calculations and the amount of data required make HCM methods difficult to apply to regional transportation plans, but do not hinder most other planning applications, which typically are conducted on smaller scales.

The HCM procedures for speed prediction are limited. They are only applicable when flow is less than capacity and when operations are uncongested. Generally, these conditions are sought in planning; however, some situations may arise now and more so in the future when it would be useful to predict speed under congested operations, even though it is highly variable. Research of "adaptive forecasting" techniques may be beneficial to this end.

The rural two-lane section of the HCM includes only one speed-flow relationship, corresponding to an ideal free-flow speed of 60 mph. However, in many rural settings, free-flow and operating speeds less than 60 mph frequently occur. The HCM method is inadequate for this type of facility.

The BPR method is applicable to all facility types; however, the curve assumes uncongested operation. The applicability of the enhanced BPR method is the same as that of the BPR look-up method. It was found to be reliable for all facilities where operations were in the uncongested regime of flow.

The enhanced ARTPLAN method can be reliably applied to more facility types under a wider range of conditions than any of the other methods (including the HCM methods) considered here.

13.2.6 Ease of Incorporation into the HCM

This section addresses the ease of incorporating either the enhanced BPR method or the enhanced ARTPLAN method into the next edition of the HCM.

The enhanced BPR method would stand separate from the rest of the HCM because its procedures are generally unrelated to the procedures contained in the current HCM. The enhanced BPR method would need to be presented in a separate planning section of the HCM with an explanation of the conditions in which it might be appropriate to apply the enhanced BPR in lieu of the more detailed procedures already available in the HCM.

The enhanced ARTPLAN method could be relatively easy to incorporate into the next edition of the HCM because ARTPLAN is an implementation of the Chapter 11 (urban and suburban arterials) procedures of the HCM. The concepts of the enhanced ARTPLAN method for freeways and rural roads would need to be added to the respective chapters for these facility types. The minor changes in the enhanced ARTPLAN method for urban arterials would require some editorial modifications to Chapter 11 of the HCM.

The enhanced ARTPLAN method would require the following changes to the current HCM:

- Adding the ARTPLAN planning method to the freeway systems chapter (Chapter 6) of the HCM;
- Adding the ARTPLAN planning method to the multilane rural and suburban highways chapter (Chapter 7) of the HCM;
- Adding the ARTPLAN planning method to the two-lane highways chapter (Chapter 8) of the HCM; and
- Modifying Chapter 11 (urban and suburban arterials) of the HCM to
  - Include queuing analysis delay calculation for conditions in which demand exceeds capacity and
  - Replace running time table (Table 11-4 of HCM) with a free-flow speed equation using posted speed limit.

13.2.7 Sensitivity to TSM and TCMs

Both the enhanced BPR and enhanced ARTPLAN methods have limited sensitivity to TSM and TCMs in general. Like the HCM itself, the enhanced BPR and ARTPLAN methods do not provide explicit techniques for evaluating HOV lanes. They cannot forecast the impact of ridesharing incentives and pricing changes on demand, but once given the new demand levels, these methods can predict the impact on speed and level of service. Both the enhanced BPR and enhanced ARTPLAN techniques are sensitive to signal operation improvements, although the enhanced ARTPLAN technique will provide a more accurate forecast of the impact of signal timing changes on speed and level of service. The enhanced BPR technique also is sensitive to the impact of signal timing on the estimated free-flow speed for urban interrupted flow facilities.

13.3 EVALUATION OF LEVEL OF SERVICE ESTIMATION TECHNIQUES

This section evaluates the existing and proposed planning techniques for predicting level of service that have not been covered as one of the speed estimation techniques. The Florida Generalized Service Volume Tables and the Florida Table Generating Software (not discussed under the speed estimation techniques) are compared with the HCM, standard BPR, enhanced BPR, and enhanced ARTPLAN techniques. (Note that the HCM technique for urban interrupted facilities is the original ARTPLAN.)

13.3.1 Data Requirements

This section describes and evaluates the data requirements of the two Florida techniques and compares them with the other techniques described previously in the evaluation of speed estimation techniques.

The Florida Generalized Service Volume Tables require the following data:

- Traffic volume (AADT, two-way peak hour, or one-way peak hour)
- Area type (urbanized, transitioning, or rural)
- Facility type (uninterrupted or interrupted)
- Number of lanes
- Median type (divided or undivided)
- Functional classification (freeways, state two-way arterials, non-state roadways, or other signalized roadways)
- Traffic signal density (signals per mile).

The use of these tables in rural areas requires the following additional data:

- Provision of left turn bays
- Posted speed limit (for uninterrupted roads)
- Percent miles with exclusive passing lanes (for two-lane roads).

These data items usually are easy to obtain from DOTs, MPOs, and local agencies for most roads (see Table 13-10). Knowledge of all of these items allows an analyst to enter the correct table to determine level of service. Signal density can be calculated easily with the knowledge of the length of the study section and number of signals on it. Percent miles with exclusive passing lanes need to be known only to the extent of general ranges—greater than or equal to 60 percent, 20 to 59 percent, 5 to 19 percent, or 1 to 4 percent.

Data requirements for the use of different spreadsheet models depend on the spreadsheet model to be used (see Table 13-11). Data requirements, therefore, are discussed in association with the type of facility the specific spreadsheet software is designed to address.

13.3.1.1 Rural Uninterrupted Facilities

These types of facilities are addressed by two spreadsheet templates. One spreadsheet addresses rural multilane uninterrupted highways (RMUL_TAB); the other addresses rural two-lane uninterrupted highways (R2LN_TAB). Data requirements for the RMUL_TAB spreadsheet are as follows:

- Traffic volume (AADT, peak-hour peak direction or peak-hour volume—both directions)
- Number of lanes
- Length of roadway segment under study
- K factor (K100—planning analysis hour factor)
- Directional distribution factor
- Peak-hour factor
- Adjusted saturation flow rate
- Free-flow speed
- Presence of medians
- Presence of left turn bays.

Default values that were used to assess the rural uninterrupted dataset if the information was not available included the following:

- Adjusted saturation flow rate (1,750)
- Free-flow speed (same as posted speed on roadway segment).

TABLE 13-10 Data requirements for Florida Generalized Service Volume Tables

Item	Facility Type[a]	Defaults Available?	Ease to Obtain
Daily or Hourly volume	all	No	Easy
Number of lanes	all	No	Easy
Area Type	all	No	Easy
Facility Type	all	No	Easy
Functional Class	all	No	Easy
Median Type	UI, RI	No	Easy
One-Way/Two-Way	UI	No	Easy
Signal Spacing	UI	No	Easy
Left Turn Bays	RI	No	Easy
Posted Speed Limit	RI	No	Easy
Percent Passing Lanes	2-RI	No	Easy

Note: All data items are averaged over length of the facility.
[a] UU=Urban Uninterrupted, UI=Urban Interrupted, RU=Rural Uninterrupted, RI=Rural Interrupted, 2-RI=Two-Lane Rural Interrupted.

TABLE 13-11 Data requirements for Florida Table Generating Software

Item	Facility Type[a]	Defaults Available?	Ease to Obtain
Daily or Hourly volume	all	No	Easy
Directional and K Factors	all	Yes	Easy
Peak Hour Factor	all	Yes	Easy
Number of lanes	all	No	Easy
Length of Segments	all	No	Easy
Area Type	all	No	Easy
Facility Type	all	No	Easy
Median Type	all	No	Easy
Functional Class	all	No	Easy
Adjusted Saturation Flow Rate	all	Yes	Hard
Left Turn Bays	UI,RI	No	Easy
Posted Speed Limit	UI, RI	No	Easy
One-Way/Two-Way	UI	No	Easy
Number of Signals	UI	No	Easy
Signal Control Type	UI	Yes	Hard
Effective Green and Cycle Time	UI	Yes	Hard
Arrival Type	UI	Yes	Hard
Percent Turns	UI	Yes	Hard
Percent Passing Lanes	2-RI	No	Medium

Note: All data items are for each segment of the facility.
[a] UU=Urban Uninterrupted, UI=Urban Interrupted, RU=Rural Uninterrupted, RI=Rural Interrupted, 2-RI=Two-Lane Rural Interrupted.

These data usually are available from most DOTs, MPOs, and local agencies. The K100—planning analysis hour factor, however, may be available only from the DOT. Procedures to collect the data necessary for calculating the input data are described in the Florida *Level of Service Manual.*

Data requirements to estimate the level of service of a two-lane uninterrupted rural facility using R2LN_TAB follow:

- Traffic volume (AADT, peak-hour peak direction or peak-hour volume—both directions)
- Number of lanes
- Length of roadway segment under study
- K factor (K100—planning analysis hour factor)
- Directional distribution factor
- Peak-hour factor
- Bidirectional adjusted saturation flow rate
- Posted speed limit
- Percent no-passing zones
- Percent exclusive passing lanes
- Presence of left turn bays.

A default value of 2,600 was used for the bidirectional adjusted saturation flow rate to assess the rural uninterrupted dataset. The default value for percent exclusive passing lanes is zero if such information is not available. These data usually are available from most DOTs, MPOs, and local agencies. The K100—planning analysis hour factor, however, may be available only from the DOT. Procedures to collect the data necessary for calculating the input data are described in the Florida *Level of Service Manual.*

13.3.1.2 Rural Interrupted Facilities

Rural interrupted facilities may be analyzed using RMUL_TAB, R2LN_TAB, or ART_TAB, as appropriate. RMUL_TAB and R2LN_TAB are used if traffic signal spacing is greater than 2 mi. Data requirements for RMUL_TAB and R2LN_TAB were discussed in the section dealing with rural uninterrupted facilities.

Rural interrupted arterials are analyzed using the ART_TAB spreadsheet if traffic signals on the facility are spaced closer than 2 mi apart. Data requirements for ART_TAB follow:

- Traffic volume (AADT, two-way peak hour, or one-way peak hour)

- K factor (K100—planning analysis hour factor)
- Directional distribution factor
- Number of lanes
- Presence of medians
- Peak-hour factor
- Adjusted saturation flow rate
- Provision of left turn bays
- Free-flow speed
- Percent turns from exclusive lanes
- Area type (urban, transitioning, or rural)
- Arterial classification (per Chapter 11 of 1994 HCM)
- Length of arterial segment under study
- Number of signalized intersections on segment under study
- Arrival type
- Signal system type (pretimed, semiactuated, or actuated)
- Signal system cycle length
- Weighted through movement g/C.

Because this list of data items is so extensive, most local agencies and some MPOs may not have a database that contains all the items. State DOTs, however, are likely to have a database that contains most of these items for roads under their jurisdiction. The HCM (Chapter 11) and Florida's *Level of Service Manual* discuss possible default values for some of these items. The Florida manual, in addition, recommends appropriate procedures for collecting the information necessary to calculate various input parameters.

Default values that were used to assess the rural interrupted dataset if the information was not available include the following:

- Adjusted saturation flow rate (1,700)
- Percent turns from exclusive lanes (12 percent for rural)
- Number of signals (minimum allowable if unknown for rural interrupted facilities)
- Arrival type (3)
- Signal system type (actuated)
- System cycle length (60 sec)
- Weighted through movement g/C (0.45).

13.3.1.3 Urban Uninterrupted Facilities

The level of service of urban uninterrupted facilities may be estimated using the spreadsheet model FREE_TAB for freeways and UMUL_TAB for other urban multilane uninterrupted highways. Data requirements for these two models are similar and are as follows:

- Traffic volume (AADT, peak-hour peak direction, or peak-hour volume—both directions)
- Number of lanes
- Length of roadway segment under study
- K factor (K100—planning analysis hour factor)
- Directional distribution factor
- Peak-hour factor
- Adjusted saturation flow rate
- Free-flow speed.

The UMUL_TAB model requires the following two data items:

- Presence of medians
- Presence of left turn bays.

All of these data items, except for adjusted saturation flow rate, usually are available from DOTs and MPOs for urban uninterrupted facilities. The default value for adjusted saturation flow rate for a four- to six-lane freeway is 2,225 vehicles per hour per lane. The default value for urban multilane uninterrupted roadways that are not freeways is 1,900 vehicles per hour per lane. Local agencies that may not have the data necessary to make a level of service assessment using this technique need to obtain the data from the DOT or MPO. The Florida *Level of Service Manual* prescribes procedures for collecting appropriate data and computing required input data.

13.3.1.4 Urban Interrupted Facilities

Urban interrupted arterials are analyzed using the ART_TAB spreadsheet. The spreadsheet's data requirements were listed in the section on rural interrupted facilities. As discussed previously, data requirements for using the ART_TAB spreadsheet model are extensive, and most local agencies and some MPOs may not have all of the data items. State DOTs, however, are likely to have a database that contains most of these data items for roads under their jurisdiction. The HCM (Chapter 11) and Florida's *Level of Service Manual* discuss possible default values for some of the items. The Florida manual, in addition, recommends appropriate procedures for collecting the information necessary to calculate various input parameters. Default values that were used to assess the control data points in the urban interrupted dataset include the following:

- Adjusted saturation flow rate (1,850)
- Percent turns from exclusive lanes (12 percent).

The standard BPR method requires the least amount of data, whereas the HCM method for urban interrupted facilities requires the most data. Florida's Table Generating Software also requires more data for interrupted facilities. The amount of data required for rural interrupted facilities will drop to a moderate level if RMUL_TAB and R2LN_TAB are the appropriate table generating models to use. The enhanced ARTPLAN technique requires a large amount of data for all facility types. All other techniques require a moderate amount of data.

The data required for the standard BPR method are easiest to obtain, whereas the HCM technique for urban interrupted facilities is classified as being the most difficult technique for which to obtain input data. Data for the enhanced BPR technique are easy to obtain if the look-up tables are prepared beforehand or the analyst decides to use the default look-up tables. The Florida generalized tables technique received a score of 4 for this criterion.

In general, obtaining data for the Florida Table Generating Software technique was classified as more difficult than obtaining data for the Florida generalized tables technique, but as easier than obtaining data for the HCM techniques. Input data required for the enhanced ARTPLAN technique are more difficult to obtain than data for HCM techniques, with the exception of urban interrupted facilities.

13.3.2 Ease of Use

Criteria used to evaluate the ease of use of the level of service estimating techniques are as follows:

- Complexity
- Training time
- Application time
- Required software.

13.3.2.1 Complexity

The techniques are evaluated based on how many equations are involved in the level of service computation, number of measures of effectiveness that are used across different types of facilities, and general methodology of applying the technique.

The standard BPR method uses a single equation (standard BPR curve) and is based on a single measure of effectiveness (v/c ratio). The analyst who uses the standard BPR method need only use two look-up tables to estimate level of service. This technique is the simplest technique available.

The Florida generalized tables use volume look-up tables, and the underlying multiple equations are transparent to the user. Level of service determination is based on one measure, traffic volumes. There are, however, multiple tables and look-up areas within each table that depend on other input data parameters. This technique is simple in its application, though not as simple as the standard BPR method, and its complexity is similar to that of the enhanced BPR technique.

The Florida Table Generating Software is based on multiple spreadsheet templates or models. These spreadsheet models have multiple underlying equations that result in customized volume look-up tables. In the application of this technique, one must choose the appropriate model to use and then compare volume in the look-up table to estimate level of service. All facilities use a single measure, volume, in the assessment of level of service.

The HCM techniques are from the 1994 HCM and are based on multiple equations and multiple measures for computing level of service. The 1994 HCM does not specify a planning technique for uninterrupted flow facilities, and the benchmark level of service was computed using the operational technique identified for such facilities. The 1994 HCM techniques are the most complex of the techniques evaluated.

The enhanced BPR technique is based on a single equation, a modified BPR curve, and a single measure of level of service, v/c ratio. There are two look-up tables an analyst must use, which are constructed using two additional equations. The construction of these look-up tables using the additional equations makes this technique comparable in complexity to the Florida generalized tables technique.

The enhanced ARTPLAN technique is based on multiple equations and two measures of effectiveness. All facilities other than urban interrupted use v/c ratios to assess level of service. Urban interrupted facilities use average running speed as the measure of level of service. This technique is almost as complex as HCM techniques.

13.3.2.2 Training Time

Training time is an estimate of the time it will take to teach a planner how to use a technique. Training time includes an explanation of theoretical underpinnings of the technique that are necessary for ensuring that the technique is applied correctly. The amount of training time required is used as an indicator of the technique's ease of use. Estimates of training time required for the various techniques are as follows:

- Standard BPR method—a half hour
- Florida generalized tables—1 hr
- Florida table generating software—4 hr
- HCM techniques—3 days
- Enhanced BPR technique—2 hr
- Enhanced ARTPLAN technique—1 day.

13.3.2.3 Application Time

Application time is an estimate of the amount of time it will take an analyst to apply the technique to a data point. Data for the application of the v/c ratio technique can be assembled in a few minutes, and the method can be applied within a half hour. The Florida generalized tables and table generating software and the enhanced BPR technique take only a few minutes to apply to a data point.

Data requirements for the Florida techniques are similar; therefore, data assembly takes about the same time for both techniques. The data assembly time for the enhanced BPR technique also is about the same as that for the Florida techniques. This data assembly time, which is more than that required for the standard BPR method but less than that required for the HCM and enhanced HCM techniques, is moderate.

The HCM and enhanced HCM techniques take a few hours to apply manually to a data point. This significant amount of time may be reduced through the use of custom software or a spreadsheet. The time required for data assembly for the HCM and enhanced HCM techniques is significant because of the extent of data items and amount of detail required. The few minutes estimated for applying the enhanced BPR technique do not include time for development of custom look-up tables, which can take an entire day to develop.

13.3.2.4 Required Software

There is no requirement for using software for the application of any level of service estimation technique, except for the Florida Table Generating Tables. The Florida Table Generating Software is implemented and distributed as Lotus 1-2-3 spreadsheet files. These spreadsheets should be accessible and can be used by means of any other spreadsheet program. The HCM and enhanced HCM techniques require the use of custom software or a spreadsheet to ensure that the techniques are implemented quickly and efficiently. All the techniques are spreadsheet-friendly and can be implemented on a spreadsheet if necessary.

13.3.3 Accuracy of Results

Each technique's ability to accurately and reliably estimate level of service is assessed by the following steps:

- Estimate the level of service of various datasets by using the technique.
- Compare these estimates with the level of service estimates computed using the HCM techniques or with the field-measured level of service, if such measurement is possible.

The only dataset for which level of service can be measured in the field is the urban interrupted dataset. For all other datasets, the benchmark level of service was taken as the HCM-computed level of service.

The measures of effectiveness based on which technique the benchmark level of service is determined are density for uninterrupted facilities, average travel speed for urban interrupted facilities, density for multilane rural highways, and v/c ratio for two-lane rural roads. The primary level of service measure of effectiveness for two-lane roads in the 1994 HCM is percent time delay, not v/c ratio.

The accuracy of each technique is assessed by the percentage of correct level of service estimates made by using the technique. The reliability of the technique is assessed by the confidence interval, the percentage of estimates that fall within one level of service of the benchmark level of service. Any bias in an estimate made by using a particular technique is noted, and a test of agreement is made to quantify the extent to which the estimate matches the benchmark level of service. Any obvious sensitivity to data input errors is noted, and a subjective assessment is made as to the possible integration of the technique into HCM 2000.

A summary of the accuracy and reliability of different techniques is presented in Table 13-12. The qualitative descriptions of accuracy are based on the percentage of the tests in which the estimated level of service is within one level of the HCM-estimated or field-measured level of service:

- Very Good = 90 percent or more of the tests were within one level of service;
- Good = 80 percent to 89 percent;
- Fair = 60 percent to 79 percent; and
- Poor = less than 60 percent.

TABLE 13-12 Accuracy of level of service estimation methods

Facility Type	Standard BPR	FDOT Tables	FDOT Software	1994 HCM	Enhanced BPR	Enhanced ARTPLAN
Urban Uninterrupted	Good	Good	Good	N/A	Very Good	Very Good[a]
Rural Uninterrupted	Fair	Poor	Very Good	N/A	Very Good	Very Good[a]
Rural Interrupted	Fair	Good	Very Good	N/A	Very Good	Very Good
Urban Interrupted	Poor	Poor	Fair[b]	Poor[c]	Poor	Fair

[a] The Enhanced ARTPLAN method predicts facility mean level of service based upon segment data. The validation data sets for uninterrupted flow facilities do not contain the necessary segment information. In the absence of segment data for the freeways, the Enhanced ARTPLAN yields the same estimates as the Enhanced BPR Method.

[b] ARTTAB (FDOT Software) performed better against field conditions (67% within one LOS) than ARTPLAN (HCM method) (58% within one LOS) for the urban arterials contained in the validation data set.

[c] The original ARTPLAN software (which is the HCM method for urban interrupted) was compared to field measurements of speed for urban interrupted facilities and was found to predict LOS within one level of the field measure of LOS only 58% of the time. By the standards used in this table, the HCM method would perform "Poorly" compared to field measurements. The HCM estimated worse than measured levels of service in 83% of the tests for urban interrupted facilities.

A description of the ability of the different techniques to accurately and reliably estimate level of service follows.

13.3.3.1 Rural Uninterrupted Facilities

The Florida Table Generating Software technique was found to do the best job in estimating the same level of service as computed by the HCM technique. The enhanced ARTPLAN technique was not used on this dataset because data points with the level of detail necessary were not available. The Florida Table Generating Software estimated the benchmark level of service for 42 percent of the test data points within one level of service of the benchmark level of service for 92 percent of the test data points and returned a score of 0.22 on the test of agreement.

13.3.3.2 Rural Interrupted Facilities

The enhanced BPR technique was found to make the most accurate estimate of the benchmark level of service. This technique, which estimated the benchmark level of service for 71 percent of the test dataset, was within one level of service of the benchmark level of service for all data points in the dataset. The test of agreement returned a score of 0.66.

The enhanced ARTPLAN/HCM technique and the Florida Table Generating Software also estimated within one level of service of the benchmark level of service for all test data points. The enhanced ARTPLAN/HCM technique was used without dividing the data point section into segments, as the technique requires. This may have resulted in a poorer estimate of level of service than that made by using the enhanced v/c ratio technique.

13.3.3.3 Urban Uninterrupted Facilities

The enhanced BPR technique does the best job of estimating the benchmark level of service for urban uninterrupted facilities. The enhanced ARTPLAN/HCM technique was not assessed because data points with the level of detail necessary were unavailable. The Florida Table Generating Software estimated the benchmark level of service for the same number of data points as the enhanced BPR technique (62 percent). The level of service estimate made by using the enhanced BPR technique, however, was within one level of service of the benchmark level of service for 92 percent of the dataset, compared with 85 percent of the dataset when using the Florida Table Generating Software technique. The test of agreement returned a score of 0.39 for the table generating software technique and 0.36 for the enhanced v/c ratio technique. The Florida generalized tables did not do as well in estimating the benchmark level of service (54 percent), and the standard BPR method did worse, estimating the benchmark level of service for only 38 percent of the data points.

13.3.3.4 Urban Interrupted Facilities

The enhanced ARTPLAN/HCM technique was found to be the best technique for estimating level of service. The technique estimated the field-measured level of service for 42 percent of the test data points. The technique, which was biased toward estimating a poorer level of service than the field-measured level of service, estimated within one level of service of the field-measured benchmark level of service for 67 percent of the test dataset. The test of agreement for the test dataset using the enhanced ARTPLAN/HCM technique was found to be 0.22. All other techniques performed poorly in estimating level of service for urban interrupted facilities.

The HCM technique did not do as well as the Florida Table Generating Software, specifically the ART_TAB model, in estimating level of service. A review of the HCM technique reveals that it penalized data points with a Level of Service F if the v/c ratio for the critical intersection exceeded the ratio of 1/PHF. This penalty reflects the fact that 1/PHF greater than v/c ratio implies that the critical intersection has more traffic than it can handle in an hour and, therefore, that the intersection fails over the hour under consideration. The field-measured average travel speed indicates, however, that traffic is proceeding through the critical intersection, albeit at a crawl. The overall section average travel speed, therefore, may still yield a level of service better than F.

ART_TAB, the Florida Table Generating Software model for urban interrupted facilities, is not similarly constrained. To test this hypothesis, the HCM technique was used to estimate the level of service of the test dataset without the v/c ratio less than 1/PHF constraint. This resulted in a level of service estimate identical in accuracy and reliability between the HCM technique and the Florida table generating technique.

Sensitivity analysis for these techniques was not conducted; therefore, each technique's sensitivity to error in input data was not ascertained. In applying the techniques, it was obvious that in the case of the v/c ratio technique, a 10 percent error in traffic volume would change the level of service estimate.

The standard BPR method possibly can be used as a new screening technique in HCM 2000, but it does not give satisfactory results. The enhanced BPR technique, on the other hand, is a much better estimator of level of service and would make a good addition to HCM 2000 as a new first-level level of service screening analysis technique.

The enhanced ARTPLAN/HCM technique is based on the 1994 HCM; therefore, it is compatible with the existing HCM. The technique, therefore, can be included in HCM 2000 as a new planning technique to use when more detailed analysis is required, compared with the analysis produced by the enhanced v/c ratio technique. Although the enhanced ARTPLAN technique is clearly a superior technique for urban interrupted facilities, a better technique using more detailed data is required for other facilities. The level of detail in the test dataset did not allow for the proper estimate of level of service using the enhanced ARTPLAN/HCM technique.

CHAPTER 14

CONCLUSIONS

The enhanced BPR technique is recommended for long-range transportation planning (LRTP) and sketch planning applications. The enhanced ARTPLAN technique is recommended for all other planning applications in which facility-specific analyses can be performed.

14.1 ENHANCED BPR TECHNIQUE

The enhanced BPR technique is recommended for LRTP and sketch planning applications because of its simplicity. LRTP applications frequently require the evaluation of thousands of street facilities over an entire region. The enhanced BPR technique is ideal for these applications because of its limited data requirements and its ability to rapidly compute speeds for thousands of facilities.

The enhanced BPR technique makes the following improvements over current techniques:

- The enhanced BPR technique updates speed-flow curve parameters to recent speed-flow research results.
- The technique fits separate curve for urban interrupted flow facilities.
- The look-up table of free-flow speeds is replaced by an equation based on posted speed limits and signalization characteristics.
- The look-up table on capacities is replaced with 1994 HCM–derived equations for computing facility-specific capacity.
- The technique uses data on critical segment of facility instead of facility averages.

The enhanced BPR technique had RMS errors of less than 15 percent of the true mean speed for all facility types except urban interrupted. For urban interrupted flow facilities, the RMS error was found to be 35 percent. The enhanced BPR technique was able to predict the correct level of service more than 60 percent of the time for urban uninterrupted and rural interrupted facilities. It performed less well on urban interrupted facilities, predicting the correct level of service only 17 percent of the time.

Thus, although the enhanced BPR technique is a significant improvement over current techniques, it is not accurate enough for facility-specific analyses. The enhanced BPR technique should be limited to LRTP and sketch planning applications in which precision is an acceptable trade-off for reduced data requirements and increased computational efficiency.

14.2 ENHANCED ARTPLAN TECHNIQUE

The enhanced ARTPLAN technique is recommended for all planning applications in which facility-specific analyses are to be performed and in which available resources allow for the collection of the additional segment and intersection-specific data required by the ARTPLAN technique.

The enhanced ARTPLAN technique makes the following major improvements over current techniques:

- The technique creates a planning method for assessing freeway speed and level of service on a segment-by-segment basis, accounting for the effects of conditions in which demand exceeds capacity on specific segments of the facility for a portion of the peak period.
- The technique creates a similar planning method for rural interrupted flow facilities and incorporates the impact of widely spaced signals on facility speed and level of service.
- The technique expands the current ARTPLAN technique for arterials to urban streets with stop sign control and to conditions in which demand exceeds capacity.
- The technique improves the bias toward underestimating arterial speed in the current HCM/ARTPLAN technique by replacing the running time look-up table (in Chapter 11 of the HCM) with an equation to compute midblock free-flow speed.

The enhanced ARTPLAN technique was found to estimate delay for conditions in which demand exceeds capacity more accurately. The improved technique had RMS errors of less than 22 percent for urban interrupted flow facilities and RMS errors of 13 percent or less for all other facilities.

The enhanced ARTPLAN technique is able to predict the correct level of service between 33 percent and 62 percent of the time for all facility types. This compares with the current HCM method for arterials (as implemented in ARTPLAN), which is able to predict the correct level of service only 17 percent of the time.

APPENDIXES A AND B

UNPUBLISHED MATERIAL

Appendixes A and B contained in the research agency's final report are not published herein. For a limited time, loan copies are available on request to NCHRP, Transportation Research Board, Box 289, Washington, D.C. 20055. The appendixes are titled as follows:

Appendix A: Validation Data Set
Appendix B: User Survey Results

Appendix C
Example Problems

This appendix presents example problems showing the application of the recommended planning techniques for estimating speed and level of service, the Enhanced BPR Technique and the Enhanced ARTPLAN Technique.

C.1 Enhanced BPR Technique

The Enhanced BPR technique is illustrated with four example problems for: an uninterrupted flow facility, a multi-lane rural interrupted flow facility, a two-lane rural interrupted flow facility, and an urban interrupted flow facility.

Sample Problem #1 - Uninterrupted Flow Facility

The sample problem for an uninterrupted flow facility, is a section of the Interstate 80 freeway in Omaha, Nebraska. The input data (shown below) is taken from the validation data set.

Table C-1 Data for Uninterrupted Flow Facility

Roadway Name	I-80 Freeway
From	I-480 merge
To	42nd St. off-ramp
Location	Omaha, NE
Facility Length (miles)	3.2
Facility Type	Uninterrupted
Area Type	Urban
Terrain Type	Level
AADT (Average Annual Daily Traffic)	114800
K Factor	0.085
D Factor	0.58
Volume	5670
Peak/Off-Peak Direction	WB
PHF	0.94
Number of Through Lanes in Critical Section	3
% No Passing Zones (2-Lane Only)	N/A
% Exclusive Passing Lanes (2-Lane Only)	N/A
% Trucks, R.V.'s, and Buses	9%
Posted Speed Limit (mph)	55

Step 1. Determine Facility Type

The analyst must determine which of four facility type categories are appropriate for the subject facility so that the appropriate analytical technique can be applied. The categories are:

- Uninterrupted Flow Facilities;
- Multi-Lane Rural Interrupted Flow Facilities;
- Two-Lane Rural Interrupted Flow Facilities;
- Urban Interrupted Flow Facilities.

Use the following definitions to determine the facility type:

An **uninterrupted flow facility,** is defined as a divided highway with full control of access with one or more lanes for the exclusive use of traffic in each direction. This facility type corresponds to freeways in the Highway Capacity Manual. The type of area in which the facility is located (e.g. urban or rural) is not important in selecting the analytical technique.

A **multi-lane rural interrupted flow facility** is a roadway with two or more lanes of traffic in each direction with "controlled intersections" spaced more than 2 miles (3.2 km) apart. A **controlled intersection** is defined as an intersection where a traffic signal or stop signs cause traffic on the subject roadway to stop for traffic on the side street. The type of area in which the facility is located (e.g. urban or rural) is not important in selecting the analytical technique. The controlling factor is the spacing of the controlled intersections. If the controlled intersections are spaced 2 miles (3.2 km) or closer apart, then the facility should be analyzed as an urban interrupted facility, even if it is located in a rural area.

A **two-lane rural interrupted flow facility** is an undivided roadway with only one lane of traffic in each direction with "controlled intersections" spaced more than 2 miles (3.2 km) apart. If the controlled intersections are spaced 2 miles (3.2 km) or closer apart, then the facility should be analyzed as an urban interrupted facility, even if it is located in a rural area. If the two-lane road is divided (e.g. with two way left turn lanes or a median) it should be analyzed as a multi-lane rural interrupted flow facility.

An **urban interrupted flow facility** is a divided or undivided roadway with any number of lanes of traffic in each direction with "controlled intersections" spaced more than 2 miles (3.2 km) apart.

Note that the words, "urban" and "rural", are used here to characterize the relative density of controlled intersections, and not the type of development in the vicinity of the facility. Thus a facility with a high density of controlled intersections is characterized as an "urban" facility, even though it may be located in an otherwise rural area.

In our example, the facility is a divided highway with complete access control.. It is therefore an "uninterrupted flow facility".

Step 2. Identify Critical Segment

The critical segment of the facility must be identified for the purpose of computing the facility's critical volume/capacity ratio.

The critical segment is the "bottleneck" of the facility where the ratio of demand to capacity is highest. In the absence of segment specific demand data, the demand can be assumed to be relatively constant over the facility. The "bottleneck" can then be identified by locating the segment with the least number of through lanes or other characteristics, such as steep grades, that cause it to have the lowest throughput capacity of the facility.

For our example facility, the number of through lanes is constant over the length of the facility. The terrain is also level. So there are no capacity bottlenecks. The critical segment would then be determined by the segment with the highest demand. We do not have demand data by segment (over the three mile length of the facility) to determine the bottleneck segment, consequently we will use the demand data we do have for one segment to estimate the mean speed for that segment alone.

Step 3. Compute Free Flow Speed

The facility free-flow speed is computed using the following equation for facilities with posted speed limits in excess of 50 mph (80 kph) (if the speed limit were 50 mph or less, then see the example problem for multi-lane rural interrupted flow facility for the appropriate equation).

Mean Free-Flow Speed (mph) = 0.88 * (Speed Limit in mph) + 14 mph **(Customary Units)**
Mean Free-Flow Speed (kph) = 0.88 * (Speed Limit in kph) + 22 mph **(SI Units)**

For our example:

Speed Limit = 55 mph (88 kph);

Mean Free-Flow Speed = 0.88 * 55 + 14 = 62 mph (100 kph).

Step 4. Compute Capacity

The critical segment capacity is computed using the following equation for uninterrupted flow facilities:

Capacity (vph) = Ideal Cap * N * F_{hv} * PHF

where:

Ideal Cap = 2400 (pcphl) for freeways with 70 mph (110 kph) or greater free-flow speed.
= 2300 (pcphl) for all other freeways (free flow speed < 70 mph (110 kph)).

N = Number of through lanes. Ignore auxiliary lanes and "exit only" lanes.

F_{hv} = Heavy vehicle adjustment factor.
= 1.0 / (1.0 + 0.5 *HV) for level terrain
= 1.0 / (1.0 + 2.0 *HV) for rolling terrain
= 1.0 / (1.0 + 5.0 *HV) for mountainous terrain
HV is the proportion of heavy vehicles (including trucks, buses, and recreational vehicles) in the traffic flow. If the HV is unknown, use 0.05 heavy vehicles as default.

PHF = Peak hour factor (ratio of the peak 15 minute flow rate to the average hourly flow rate) If unknown, use default of 0.90.

For our example:

Ideal Cap = 2300 vphpl (the free flow speed is below 70 mph (110 kph))

N = 3

F_{hv} = 1.0/(1.0+0.5 * 0.09) = 0.96 (level terrain)

PHF = 0.94

Capacity = 2300 * 3 * 0.96 * 0.94 = 6221 vph

Step 5. Compute Mean Facility Speed

The mean facility speed is computed using the following equation:

$$s = \frac{s_f}{1 + a(v/c)^b}$$

where:

s = predicted mean speed

s_f = free flow speed

v = volume

c = capacity,

a = 0.05 for urban interrupted flow facilities,
= 0.20 for all other facilities

b = 10

For our example:

s_f = 62 mph (100 kph)

v = 5,670

c = 6,221

a = 0.20 (not urban interrupted)

b = 10

$$\text{Mean Facility Speed} = \frac{62}{1 + 0.20(5670/6221)^{10}} = 58 \text{ mph (92 kph)}$$

Step 6. Determine Level of Service

The volume/capacity ratio for the critical segment is computed for the facility. The v/c ratio is then used to look up the level of service in the following table taken from the 1994 Highway Capacity Manual.

Table C-2 Maximum Volume/Capacity Ratios for Freeways[1]

	Four Lanes (2 each direction)				Six Lanes (3 each direction)			
	Free-Flow Speed (mph)				Free-Flow Speed (mph)			
Level of Service	70	65	60	55	70	65	60	55
A	0.32	0.30	0.27	0.25	0.30	0.28	0.26	0.24
B	0.51	0.47	0.44	0.40	0.49	0.45	0.42	0.38
C	0.75	0.70	0.65	0.60	0.71	0.67	0.63	0.57
D	0.92	0.89	0.83	0.80	0.88	0.85	0.79	0.77
E	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

The above table can be interpolated for different free-flow speeds.

[1] 1994 HCM, Table 3-1, page 3-9.

For our example:

Six-lane freeway;

Free-flow speed = 62 mph;

v/c = 5670/6221 = 0.91;

Interpolating between 60 mph and 65 mph for a six lane freeway, the maximum v/c ratio is 0.85 for level of service "D" and 1.00 for level of service "E". Thus the level of service for our facility is "E".

Sample Problem #2 - Multi-Lane Rural Interrupted Flow Facility

Sample Problem #2 is State Route 6 in the Cape Cod area of Massachusetts. It is a multi-lane rural interrupted flow facility. The input data (shown below) is taken from the validation data set.

Table C-3 Data for Multi-Lane Rural Interrupted Flow Facility

Roadway Name	Route 6
From	Route 6A
To	Orleans Rotary
Location	Wellfleet, MA
Facility Length (miles)	18.28
Facility Type	Interrupted
Area Type	Rural
Controlled Access Facility	No
Divided / Undivided	Undivided
Terrain	Level
Provision of Left-Turn Bays	No
AADT (Average Annual Daily Traffic)	25,100
K Factor	0.12
D Factor	0.50
PHF	0.90
Number of lanes (continuous through lanes)	2
Volume per hour	1,500
% No Passing Zones (2-Lane Only)	N/A
% Exclusive Passing Lanes (2-Lane Only)	N/A
% Trucks, buses, and recreational vehicles	6%
Posted Speed Limit (mph)	45

Step 1. Determine Facility Type

See the first example problem for definitions of facility type. In our example, the facility is a multi-lane highway without access control. Controlled intersections are spaced more than 2 miles (3.2 km) apart. It is therefore a "multi-lane rural interrupted flow facility".

Step 2. Identify Critical Segment

See the first example problem for a discussion of how to identify the critical segment.

For our example facility, the number of through lanes is constant over the length of the facility. The terrain is also level. So there are no capacity bottlenecks. The critical segment would then be determined by the segment with the highest demand. We do not have demand data by segment to determine the bottleneck segment, consequently we will use the demand data we do have for one segment to estimate the mean speed for that segment alone.

Step 3. Compute Free Flow Speed

The facility free-flow speed is computed using the following equation for facilities with posted speed limits equal to or below 50 mph (80 kph). (see the equation for Example Problem #1, if the posted speed limit is greater than 50 mph).

Free-Flow Speed (mph) = 0.79 * (Speed Limit in mph) + 12 mph **(Customary Units)**
Free-Flow Speed (kph) = 0.79 * (Speed Limit in kph) + 19 kph **(SI Units)**

For our example:

Speed Limit = 45 mph (72 kph)

Mean Free-Flow Speed = 0.79 * 45 + 12 = 48 mph (76 kph)

Step 4. Compute Capacity

The critical segment capacity is computed using the following equation for multi-lane rural interrupted flow facilities:

Capacity (vph) = Ideal Cap * N * F_{hv} * PHF

where:

Ideal Cap = 2200 (pcphl) for multi-lane rural roads with 60 mph free-flow speed.
= 2100 (pcphl) for multi-lane rural roads with 55 mph free-flow speed.
= 2000 (pcphl) for multi-lane rural roads with 50 mph free-flow speed.

N = Number of through lanes at critical segment. Ignore exclusive turn lanes.

F_{hv} = Heavy vehicle adjustment factor.
= 1.0 / (1.0 + 0.5 *HV) for level terrain
= 1.0 / (1.0 + 2.0 *HV) for rolling terrain
= 1.0 / (1.0 + 5.0 *HV) for mountainous terrain
HV = the proportion of heavy vehicles (including trucks, buses, and recreational vehicles) in the traffic flow. If the HV is unknown, use 0.05 heavy vehicles as default.

PHF = Peak hour factor (ratio of peak 15 minute flow rate to the average hourly flow rate)
If unknown, use default of 0.90.

For our example:

Ideal Cap	= 2000 vphpl (the free flow speed is below 50 mph (80 kph), so use the lowest ideal capacity of 2000 vph per lane. No extrapolation below 2000)
N	= 2 (lanes in one direction).
F_{hv}	= 1.0/(1.0+0.5 * 0.06) = 0.97 (level terrain. 6% heavy vehicles)
PHF	= 0.90
Capacity	= 2000 * 2 * 0.97 * 0.90 = 3492 vph

Step 5. Compute Mean Facility Speed

The mean facility speed is computed using the same equation as given in Example Problem #1.

For our example:

s_f = 48 mph (76 kph)

v = 1,500

c = 3,492

a = 0.20

b = 10

$$\text{Mean Facility Speed} = \frac{48}{1 + 0.20(1500/3492)^{10}} = 48 \text{ mph (76 kph)}$$

Step 6. Determine Level of Service

The volume/capacity ratio for the critical segment is computed for the facility. The v/c ratio is then used with the following table taken from the 1994 Highway Capacity Manual to determine the level of service.

Table C-4 Maximum Volume/Capacity Ratios for Multi-Lane Highways[2]

	Free Flow Speed			
Level of Service	60 mph	55 mph	50 mph	45 mph
A	0.33	0.31	0.30	0.28
B	0.55	0.52	0.50	0.47
C	0.75	0.72	0.70	0.66
D	0.89	0.86	0.84	0.79
E	1.00	1.00	1.00	1.00

The values in this table can be interpolated for different free-flow speeds.

For our example:

Free-flow speed = 48 mph;

v/c = 1500/3492 = 0.43;

[2] 1994 HCM, Table 7-1, page 7-8.

Interpolating between 50 mph and 45 mph, the maximum v/c ratio is 0.29 for level of service "A" and 0.49 for level of service "B". Thus the level of service for our facility is "B".

Sample Problem #3 - Two-Lane Rural Interrupted Flow Facility

Sample Problem # 3 is a two-lane rural highway, State Route 82, located in eastern Oregon. It is a recreational route to the Wallowa Mountains. The table below shows the input data. The data is taken from the validation data set.

Table C-5 Data for Two-Lane Rural Interrupted Flow Facility

Roadway Name	Highway 82
From	Island City (MP 2.62)
To	Imbler (MP 11.81)
Location	Oregon
Facility Length (miles)	9.19
Facility Type	Interrupted
Controlled intersections/mile	0
Controlled Access Facility	No
Divided / Undivided	Undivided
Terrain	Level
Provision of Left-Turn Bays	Yes
AADT (Average Annual Daily Traffic)	4000
K Factor	0.1
D Factor	0.77
PHF	0.96
Number of lanes (continuous through lanes)	1
Volume per hour (one direction)	306
% No Passing Zones (2-Lane Only)	17
% Exclusive Passing Lanes (2-Lane Only)	0
% Trucks, buses, and recreational veh.	20%
Posted Speed Limit (mph)	55

Step 1. Determine Facility Type

See the first example problem for definitions of facility type. In our example, the facility is a two-lane highway without access control. Controlled intersections are spaced more than 2 miles (3.2 km) apart. It is therefore a "two-lane rural interrupted flow facility".

Step 2. Identify Critical Segment

See the first example problem for a discussion of how to identify the critical segment.

For our example facility, the number of through lanes is constant over the length of the facility. The terrain is also level. So there are no capacity bottlenecks. The critical segment would then be determined by the segment with the highest demand. We do not have demand data by segment to determine the bottleneck segment, consequently we will use the demand data we do have for one segment to estimate the mean speed for that segment alone.

Step 3. Compute Free Flow Speed

The facility free-flow speed is computed using the same equation as used in Example Problem #1 for facilities with posted speed limits in excess of 50 mph (80 kph).

For our example:

Speed Limit = 55 mph (88 kph)

Mean Free-Flow Speed = 0.88 * 55 + 14 = 62 mph (100 kph)

Step 4. Compute Capacity

The critical segment capacity is computed using the following equation for two-lane rural interrupted flow facilities:

Capacity (vph) = **Ideal Cap * N * F_w * F_{hv} * PHF * F_{dir} * F_{nopass}**

where:

Ideal Cap = 1400 (pcphl) for all two-lane rural roads.

N = Number of lanes (if there were passing lanes present on a portion of the facility, these would be excluded from the lane count anyway, because we need the number of through lanes on the critical segment of the facility).

F_w = Lane width and lateral clearance factor.
= 0.80 if narrow lanes and/or narrow shoulders are present.
= 1.00 otherwise.
Narrow lanes are less than 12 feet (3.6 m) wide. Narrow shoulders are less than 3 feet wide (1.0 m).

F_{hv} = Heavy vehicle adjustment factor.
= 1.0 / (1.0 + 1.0 * HV) for level terrain
= 1.0 / (1.0 + 4.0 * HV) for rolling terrain
= 1.0 / (1.0 + 11.0 * HV) for mountainous terrain
HV = the proportion of heavy vehicles (including trucks, buses, and recreational vehicles) in the traffic flow. If the HV is unknown, use 0.18 heavy vehicles as default.

PHF = Peak hour factor (the ratio of the peak 15 minute flow rate to the average hourly flow rate). If not known, use default of 0.90.

F_{dir} = Directional Adjustment Factor.
= 0.71 + 0.58 * (1 - Peak Direction % / 100%)
Peak Direction % is the percent of two-way traffic going in peak direction. If not known, use default of 55% peak direction.

F_{nopass} = No-Passing Zone Factor.
= 1.00 for level terrain
= 0.97 - 0.07 * (NoPass) for rolling terrain
= 0.91 - 0.13 * (NoPass) for mountainous terrain
NoPass is the proportion of length of facility for which passing is prohibited. If NoPass is unknown, use 0.60 NoPass for rolling terrain and 0.80 for mountainous terrain.

For our example:

Ideal Cap = 1400 vph
N = 1 (lanes in one direction).
F_w = 1.00 (lanes are standard 12 foot (3.6 m) width);
F_{hv} = 100/(100+1.0 * 20%) = 0.83 (level terrain, 20% heavy vehicles)
PHF = 0.96
F_{dir} = 0.71 + 0.58 * (1 -0.77) = 0.84
F_{nopass} =1.00 (level terrain)
Capacity = 1400 * 1 * 1.00* 0.83 * 0.96 * 0.84 * 1.00 = 937 vph

Step 5. Compute Mean Facility Speed

The mean facility speed is computed using the same equation as in Example Problem #1.

For our example:

s_f = 62 mph (100 kph)
v = 306 vph
c = 937 vph
a = 0.20
b = 10

$$\text{Mean Facility Speed} = \frac{62}{1 + 0.20(306/937)^{10}} = 62 \text{ mph (100 kph)}$$

Step 6. Determine Level of Service

The volume/capacity ratio for the critical segment is computed for the facility. The v/c ratio is then used with the following table taken from the 1994 Highway Capacity Manual to determine the level of service.

Table C-6 Maximum Volume/Capacity Ratios for Two-lane Road Level of Service[3]

L O S.	Level Terrain						Rolling Terrain						Mountainous Terrain					
	Percent No Passing						Percent No Passing						Percent No Passing					
	0	20	40	60	80	100	0	20	40	60	80	100	0	20	40	60	80	100
A	0.15	0.12	0.09	0.07	0.05	0.04	0.15	0.10	0.07	0.05	0.04	0.03	0.14	0.09	0.07	0.04	0.02	0.01
B	0.27	0.24	0.21	0.19	0.17	0.16	0.26	0.23	0.19	0.17	0.15	0.13	0.25	0.20	0.16	0.13	0.12	0.10
C	0.43	0.39	0.36	0.34	0.33	0.32	0.42	0.39	0.35	0.32	0.30	0.28	0.39	0.33	0.28	0.23	0.20	0.16
D	0.64	0.62	0.60	0.59	0.58	0.57	0.62	0.57	0.52	0.48	0.46	0.43	0.58	0.50	0.45	0.40	0.37	0.33
E	1.00	1.00	1.00	1.00	1.00	1.00	0.97	0.94	0.92	0.91	0.90	0.90	0.91	0.87	0.84	0.82	0.80	0.78

The values in this table can be interpolated for different "no-passing" percentages.

[3] 1994 HCM, Table 8-1, page 8-5.

For our example:

- Terrain = Level
- %No Pass = 17% (from data)
- v/c = 306/937 = 0.33.

The maximum v/c ratio is 0.25 for level of service "B" and 0.40 for level of service "C" for 17% no passing in level terrain. Thus the level of service for our facility is "C".

Sample Problem #4 - Urban Interrupted Flow Facility

Sample Problem #4 is an eight mile long section of Ventura Boulevard in Los Angeles, California. The input data is taken from the validation data set. The arterial contains 40 signals and therefore 39 segments. Data is shown for the critical segment only. All analysis is for the eastbound direction only.

On-street parking is present on the arterial but generates relatively few interruptions to through traffic. Consequently, on-street parking is considered to have an insignificant impact on capacity.

Table C-7 Input Data for Urban Interrupted Flow Facility

Roadway Name	Ventura Blvd
From	Topanga Canyon Rd
To	Sepulveda Blvd
Location	Los Angeles, Ca
Facility Length (miles)	8.2
Facility Type	Interrupted
Area Type	Urban
Arterial Class	II
Critical Section Data (#33-34)	
AADT (Average Annual Daily Traffic)	40,000
K Factor	0.100
D Factor	0.641
Peak Hour Volume (vph)	2691
PHF	0.94
% Heavy Vehicles (trucks, buses, R.V.'s)	? (use default)
Narrow Lanes?	No
Left-Turn Bay ?	Yes
% Turns in Turn Bay	1%
g/C	? (use default)
Cycle (sec)	? (use default)
Left Turns Protected?	No
Number of Through Lanes	2
On street parking?	Yes (but not significant)
Located in CBD?	No
Posted Speed Limit (mph)	35

Table C-8 Determination of Critical Segment for Enhanced BPR Technique - Ventura Blvd.

Link	Link Volume	% Turns Exc.Lanes	Thru Volume	Thru Lanes	Thru Vol/Lane
10-11	1638	1%	1616	3	528
11-12	1244	37%	779	3	255
12-13	1136	2%	1111	3	363
13-14	1524	2%	1490	3	487
14-15	1201	1%	1187	3	388
15-16	1201	1%	1187	3	388
16-17	1201	0%	1201	3	393
17-18	1149	16%	965	2	483
18-19	2511	0%	2501	2	1251
19-20	2153	11%	1921	2	961
20-21	2042	0%	2032	2	1016
21-22	2743	7%	2564	2	1282
22-23	1778	28%	1286	2	643
23-24	1299	6%	1217	2	609
24-25	1994	9%	1805	2	903
25-26	1694	2%	1661	2	784
26-27	1302	6%	1230	2	615
27-28	2668	1%	2654	3	868
28-29	2641	0%	2631	2	1316
29-30	1610	5%	1531	3	501
30-31	2721	1%	2687	3	878
31-32	1600	2%	1564	3	511
32-33	2773	5%	2631	3	877
33-34	2691	1%	2674	2	1337
34-35	2304	1%	2272	3	743
35-36	2800	2%	2746	3	915
36-37	2938	2%	2893	3	946
37-38	2789	4%	2681	3	876
38-39	2454	1%	2438	3	797
39-40	2362	0%	2352	3	769
40-41	2527	4%	2430	3	794
41-42	2562	1%	2531	3	827
42-43	2640	1%	2624	3	858
43-44	2759	1%	2740	3	896
44-45	3266	4%	3125	3	1022
45-46	1834	1%	1814	3	571
46-47	2332	0%	2322	3	731
47-48	1925	9%	1747	3	571
48-49	1358	13%	1176	3	370

Critical segment is 33-34 where through volume per lane equals 1337

Step 1. Determine Facility Type

See the first example problem for definitions of facility type. In our example, the facility is a four-lane to 6 lane arterial without access control.. Controlled intersections are spaced less than 2 miles (3.2 km) apart. It is therefore a "urban interrupted flow facility".

Step 2. Identify Critical Segment

See the first example problem for a discussion of how to identify the critical segment.

For our example facility, the number of through lanes varies between 2 lanes and 3 lanes in one direction. The facility includes 40 signalized intersections.

Table C-8 lists the volumes and through lanes by segment. By taking the ratio of volume to lanes, one can find that the critical section is between intersections #33 and #34. If signal timing data were available, then the volume/capacity ratio for each segment should be used to find the critical segment.

Step 3. Compute Free Flow Speed

The facility free flow speed is computed using the following equation for urban interrupted flow facilities.

$$S_f = \frac{L}{L/S_{mb} + N*\left(D/3600\right)}$$

Where:

S_f = Free flow speed for urban interrupted facility (mph or kph)

L = Length of facility (miles or km)

S_{mb} = Mid-block free flow speed (mph or kph)

If Speed Limit >= 50 mph (80 kph)

S_{mb} = 0.88 (Speed Limit in mph) + 14 (mph)
S_{mb} = 0.88 (Speed Limit in kph) + 22 (kph)

If Speed Limit < 50 mph (80 kph)

S_{mb} = 0.79 (Speed Limit in mph) + 12 (mph)
S_{mb} = 0.79 (Speed Limit in kph) + 19 (kph)

N = Number of signalized intersections on length "L" of facility (exclude signal at start of first segment)

D = Average delay per signal per equation 9-5 (sec)
$D = DF * 0.5 * C(1-g/C)^2$

where:
g = The effective green time (sec) (default = C * 0.45)
C = The cycle length (sec) (default = 120 seconds)
DF = (1-P) / (1-g/C)
P = The proportion of vehicles arriving on green.

Default Values for DF:
DF = 0.90 for uncoordinated traffic actuated signals,
DF = 1.00 for uncoordinated fixed time signals,
DF = 1.20 for coordinated signals with unfavorable progression,
DF = 0.90 for coordinated signals with favorable progression,
DF = 0.60 for coordinated signals with highly favorable progression.

For our example:

Speed Limit = 35 mph (56 kph)
Mid-Block Free-Flow Speed (S_{mb})= 0.79 * 35 + 12 = 40 mph (63 kph)
Length (L) = 43,097 feet/5280 = 8.16 miles (13 km)
Number of signals (N) = 40 -1 = 39
Cycle = 120 seconds (default)
Effective green time (g) = 0.45 * 120 = 54 seconds (default)
DF = 0.90 (default for favorable coordination)
Signal delay (D) = 0.90 * 0.50 * 120 * $(1 - 0.45)^2$ = 16.34 seconds per signal

$$S_f = \frac{8.16}{8.16/40 + 39*\left(16.34/3600\right)} = 21 \text{ mph}$$

Step 4. Compute Capacity

The critical segment capacity is computed using the following equation for urban interrupted flow facilities:

Capacity (vph) = Ideal Sat *N* F_w * F_{hv} * PHF *F_{park} *F_{bay} * F_{CBD} * g/C

where:

Ideal Sat = Ideal saturation flow rate (vehicles per lane per hour of green).
= 1900.

N = Number of lanes (exclude exclusive turn lanes and short lane additions)

F_w = Lane width factor.
= 0.93 if narrow lanes (lanes < 12 feet wide (3.6 m))
= 1.00 otherwise.

F_{hv} = Heavy vehicle adjustment factor
= 1.0 / (1.0 + HV)
HV = the proportion of heavy vehicles (including trucks, buses, and recreational vehicles) in the traffic flow. If the HV is unknown, use 0.02 heavy vehicles as default

PHF = Peak hour factor (the ratio of the peak 15 minute flow rate to the average hourly flow rate). Use 0.90 as default if PHF not known.

F_{park} = On-street parking adjustment factor.
= 0.90 if on-street parking present and parking time limit is one hour or less;
= 1.00 otherwise.

F_{bay} = Exclusive left turn bay or lanes adjustment factor
= 1.10 if exclusive left turn lanes present
= 1.00 otherwise.

F_{CBD} =Central Business District (CBD) Adjustment Factor
=0.90 if located in CBD's,
=1.00 elsewhere.

g/C =Ratio of effective green time per cycle. If no data available, use following defaults:
= 0.40 if Protected left turn phase present,
= 0.45 if Protected left turn phase NOT present.

F_C = optional user specified capacity calibration factor to account other factors (such as shared turn lanes) that affect capacity.

For our example:

- Ideal Cap = 1900 vph
- N = 2 (lanes in one direction, critical segment).
- F_w = 1.00 (lanes are standard 12 foot (3.6 m) width;
- F_{hv} = 100% / (100% + 2%) = 0.98 (using default = 2% heavy vehicles)
- PHF = 0.94 (from data)
- F_{park} = 1.00 (on-street parking does not significantly impact capacity)
- F_{bay} = 1.00 (exclusive left turn lanes present)
- F_C = 1.00 (no user adjustment needed).
- $F_{CBD=}$ = 1.00 (not in CBD)
- g/C = 0.45 (default)

Capacity = 1900 * 2 * 1.00 * 0.98 * 0.94 * 1.00 * 1.00 * 1.00 * 1.00 * 0.45 = 1575 vph

Step 5. Compute Mean Facility Speed

The mean facility speed is computed using the same equation as in Example Problem #1, but with the "a" parameter at 0.05, since this is an urban interrupted flow facility.

For our example:

- s_f = 21 mph (34 kph)
- v = 2691 link vph - (1% Turns from Exclusive Lanes * 2691) = 2674 through vph
- c = 1575 vph
- a = 0.05
- b = 10

$$\text{Mean Facility Speed} = \frac{21}{1 + 0.05(2674/1575)^{10}} = 2 \text{ mph (3 kph)}$$

Step 6. Determine Level of Service

The ratio of the actual speed to the mid-block free-flow speed is used to determine the level of service for urban interrupted flow facilities.

S/Smb = 2 mph / 40 mph = 5% of the free-flow speed.

According to the table below, the facility is operating at level of service "F".

Table C-9 Recommended Level of Service Criteria for Urban Interrupted Facilities

Level of Service	Minimum Speed as a Percent of Mid-Block Free-Flow Speed
A	90%
B	70%
C	50%
D	40%
E	30%

C.2 Enhanced ARTPLAN Technique

The following two example problems illustrate the application of the Enhanced ARTPLAN technique to an uninterrupted flow facility and to an interrupted flow facility.

Sample Problem #5 - uninterrupted Flow Facility

The sample problem is performed for a 6.1 mile (9.8 km) length of the I-880 freeway in Hayward, California. The facility is divided into 12 segments. The analysis is conducted for the southbound direction during the afternoon peak period extending from 2 PM to 8 PM. In this example the capacity is already known (2000 vehicles per hour per lane). If the capacity were not known, then the equations listed in Chapter 12 would be employed to compute capacity for each segment.

Table C-10 Description of I-880 Freeway Segments

Segment	Lanes	Length (ft)	Design Speed	On-Ramp (O) Off-Ramp (D)	Description
1	5	500	65		Mainline To Hesperian On
2	5	7500	65	OD	Hesperian On To "A" St. Off
3	5	1500	65		"A" St. Off To "A" St. On
4	5	2800	65	OD	"A" St. On To Winton Off
5	5	1500	65		Winton Off To Winton On
6	5	3600	65	OD	Winton On To Sr 92 Off
7	4	2000	65		Sr. 92 Off To Sr. 92 On
8	4	3900	65	OD	Sr. 92 Onto Tennyson Off
9	4	1500	65		Tennyson Off Tennys. On
10	5	4400	65	OD	Tennys. On To Indus. Off
11	4	1000	65		Indust.Off To Lane Drop
12	3	2000	65		Mainline Out

Table C-11 Input Data for Enhanced ARTPLAN, Uninterrupted Flow Facility Example (I-880 SB, Hayward, CA)

	Volumes:											
Time	seg 1	seg 2	seg 3	seg4	seg 5	seg6	seg 7	seg 8	seg 9	seg 10	seg 11	seg 12
1400	6777	7510	6436	7491	6844	7882	5954	7699	6943	7390	6649	6649
1500	7073	7832	6703	7656	7048	7986	5922	7454	6613	7095	6420	6420
1600	6641	7314	6284	7268	6794	7763	5704	6772	6035	6462	5762	5762
1700	6739	7402	6455	7334	6861	7727	5672	6785	6089	6518	5842	5842
1800	5609	6243	5550	6242	5852	6533	4896	6016	5377	5742	5171	5171
1900	4593	5050	4357	4924	4535	5094	3781	4921	4355	4688	4430	4430

Volumes in vehicles per hour, Length in feet, Capacity in vehicles per hour.

Step 1. Compute Capacity

The capacity for each segment is computed using equation 12-1.

Capacity (vph) = Ideal Cap * N * F_{hv} * PHF **(12-1)**

where:

Ideal Cap = 2400 (pcphl) for freeways with 70 mph (110 kph) or greater free-flow speed.
= 2300 (pcphl) for all other freeways (free flow speed < 70 mph (110 kph)).

N = Number of through lanes. Ignore auxiliary lanes and "exit only" lanes.

F_{hv} = Heavy vehicle adjustment factor.
= 1.0 / (1.0 + 0.5 *HV) for level terrain
= 1.0 / (1.0 + 2.0 *HV) for rolling terrain
= 1.0 / (1.0 + 5.0 *HV) for mountainous terrain
HV = the proportion of heavy vehicles (including trucks, buses, and recreational vehicles) in the traffic flow. If the HV is unknown, use 0.05 heavy vehicles as default.

PHF = Peak hour factor (the ratio of the peak 15 minute flow rate to the average hourly flow rate)
If unknown, use default of 0.90.

The ideal capacity of 2300 vph/lane is multiplied by 0.87 heavy vehicle adjustment factor (7.5% heavy vehicles in rolling terrain) to obtain a capacity of 2000 vph/lane before applying the peak hour factor (PHF).

In this case a separate segment capacity is computed for each hour because the available 15 minute data indicated that the PHF varied significantly by hour (0.93, 0.95, 0.96, 0.95, 0.88, 0.91). The use of the correct PHF for each hour was crucial in obtaining reasonably reliable predictions of overflow demand conditions. An average PHF for the peak period did not produce as accurate forecasts of queuing conditions.

Table C-12 Computed Capacity by Segment and Time Period

c	sec 1	sec 2	sec 3	sec4	sec 5	sec6	sec 7	sec 8	sec 9	sec 10	sec 11	sec 12
1400	9300	9300	9300	9300	9300	9300	7400	7400	7400	7400	7400	5600
1500	9500	9500	9500	9500	9500	9500	7600	7600	7600	7600	7600	5700
1600	9600	9600	9600	9600	9600	9600	7700	7700	7700	7700	7700	5800
1700	9500	9500	9500	9500	9500	9500	7600	7600	7600	7600	7600	5700
1800	8800	8800	8800	8800	8800	8800	7100	7100	7100	7100	7100	5300
1900	9100	9100	9100	9100	9100	9100	7300	7300	7300	7300	7300	5500

Step 2. Compute Volume/Capacity Ratio

The volume capacity ratio is computed for each segment for each hour of the peak period. The purpose of this computation is to identify segments and hours when demand exceeds capacity.

Table C-13 Step 2 - Compute Volume/Capacity Ratios

v/c	seg 1	seg 2	seg 3	seg4	seg 5	seg6	seg 7	seg 8	seg 9	seg 10	seg 11	seg 12
1400	0.73	0.81	0.69	0.81	0.74	0.85	0.80	1.03	0.93	0.99	0.89	1.19
1500	0.74	0.82	0.70	0.80	0.74	0.84	0.78	0.98	0.87	0.93	0.84	1.12
1600	0.69	0.76	0.65	0.76	0.71	0.81	0.74	0.88	0.79	0.84	0.75	1.00
1700	0.71	0.78	0.68	0.77	0.72	0.81	0.74	0.89	0.80	0.86	0.77	1.02
1800	0.63	0.71	0.63	0.71	0.66	0.74	0.69	0.85	0.76	0.81	0.73	0.97
1900	0.50	0.56	0.48	0.54	0.50	0.56	0.52	0.68	0.60	0.64	0.61	0.81

Step 3. Carry Over Excess Demand to Following Hour

For those segments and hours (time slices) where demand exceeds capacity, compute the excess demand that cannot be served during that hour and must be carried over to the following hour. You do not need to show the propagation of the queue to upstream segments. Add the excess demand to the demand in the following hour. Recompute the v/c ratio for that following hour and see if new excess must be carried over to the next hour. Repeat the process until you have reached the end of the peak period. All remaining excess demand is assigned to the last hour of the peak period.

Table C-14 Step 3 - Carry Over Excess Demand to Following Hour (when v/c > 1.00 for segment)

vols	sec 1	sec 2	sec 3	sec4	sec 5	sec6	sec 7	sec 8	sec 9	sec 10	sec 11	sec 12
1400	6777	7510	6436	7491	6844	7882	5954	7699	6943	7390	6649	6649
1500	7073	7832	6703	7656	7048	7986	5922	7753	6613	7095	6420	7469
1600	6641	7314	6284	7268	6794	7763	5704	6925	6035	6462	5762	7531
1700	6739	7402	6455	7334	6861	7727	5672	6785	6089	6518	5842	7573
1800	5609	6243	5550	6242	5852	6533	4896	6016	5377	5742	5171	7044
1900	4593	5050	4357	4924	4535	5094	3781	4921	4355	4688	4430	6174

Step 4. Re-Compute Volume/Capacity Ratios

Recompute the v/c ratio for each segment and time slice (hour) using the new demands computed in Step 3.

Table C-15 Step 4 - Re-Compute Volume/Capacity Ratios With Carry-Over Volumes

v/c	sec 1	sec 2	sec 3	sec4	sec 5	sec6	sec 7	sec 8	sec 9	sec 10	sec 11	sec 12
1400	0.73	0.81	0.69	0.81	0.74	0.85	0.8	1.04	0.94	1	0.9	1.19
1500	0.74	0.82	0.71	0.81	0.74	0.84	0.78	1.02	0.87	0.93	0.84	1.31
1600	0.69	0.76	0.65	0.76	0.71	0.81	0.74	0.9	0.78	0.84	0.75	1.3
1700	0.71	0.78	0.68	0.77	0.72	0.81	0.75	0.89	0.8	0.86	0.77	1.33
1800	0.64	0.71	0.63	0.71	0.67	0.74	0.69	0.85	0.76	0.81	0.73	1.33
1900	0.5	0.55	0.48	0.54	0.5	0.56	0.52	0.67	0.6	0.64	0.61	1.12

Step 5. Compute Segment Running Speed

Compute segment running speed (without queue delays) using the Enhanced BPR curve:

$$s = \frac{s_f}{1 + a(X)^b}$$

where:

s = predicted mean speed

s_f = free flow speed

X = Minimum (volume/capacity ratio, 1.00)

a = 0.20 for all non-signalized facilities

b = 10

The volume/capacity ratio or 1.00 is used to compute the speed, whichever is lower. The impact of queuing on speed is determined later, so v/c ratios in excess of 1.00 are not used in the running speed calculation.

Table C-16 Step 5 - Compute Segment Speed Using BPR Curve and Max. V/C of 1.00

mph	sec 1	sec 2	sec 3	sec4	sec 5	sec6	sec 7	sec 8	sec 9	sec 10	sec 11	sec 12
1400	61.5	60.5	61.7	60.5	61.4	59.7	60.7	51.7	56	51.7	58	51.7
1500	61.4	60.3	61.6	60.5	61.4	59.9	61	51.7	59.1	56.5	59.9	51.7
1600	61.7	61.2	61.8	61.2	61.6	60.5	61.4	58	61	59.9	61.3	51.7
1700	61.6	61	61.7	61.1	61.5	60.5	61.3	58.4	60.7	59.4	61.1	51.7
1800	61.9	61.6	61.9	61.6	61.8	61.4	61.7	59.7	61.2	60.5	61.5	51.7
1900	62	62	62	62	62	62	62	61.8	61.9	61.9	61.9	51.7

Step 6. Compute Segment Running Times

Convert segment running speeds to running times by dividing the running speed into the segment length. Convert hours to seconds.

Table C-17 Step 6 - Compute Segment Running Times (Without Queue Delay) (sec)

Segment	sec 1	sec 2	sec 3	sec4	sec 5	sec6	sec 7	sec 8	sec 9	sec 10	sec 11	sec 12	Total
1400	5.5	84.5	16.6	31.6	16.7	41.1	22.5	51.4	18.3	58.0	11.8	26.4	384.3
1500	5.6	84.8	16.6	31.6	16.7	41.0	22.4	51.4	17.3	53.1	11.4	26.4	378.1
1600	5.5	83.6	16.5	31.2	16.6	40.6	22.2	45.8	16.8	50.1	11.1	26.4	366.4
1700	5.5	83.8	16.6	31.2	16.6	40.6	22.2	45.5	16.8	50.5	11.2	26.4	367.1
1800	5.5	83.0	16.5	31.0	16.5	40.0	22.1	44.5	16.7	49.6	11.1	26.4	363.0
1900	5.5	82.5	16.5	30.8	16.5	39.6	22.0	43.0	16.5	48.5	11.0	26.4	358.7

Step 7. Compute Queue Delay

Compute the delay caused by queuing using the following equation:

$$d_q = 3600 * T * \left(\frac{V_{t-1} + V_t}{2c} - 1 \right) \qquad \textbf{(12-4)}$$

where:

d_q	= Mean delay due to excess demand (sec).
T	= Duration of time period (hrs)
3600	= Converts hours to seconds.
V_{t-1}	= Leftover demand from previous time period (t-1).
V_t	= Additional demand occurring in current time period (t).
c	= Capacity of segment in subject direction (veh/hr)

Table C-18 Step 7 - Compute Queue Delay (sec)

delay	sec 1	sec 2	sec 3	sec4	sec 5	sec6	sec 7	sec 8	sec 9	sec 10	sec 11	sec 12	Sum
1400	0.0	0.0	0.0	0.0	0.0	0.0	0.0	72.7	0.0	0.0	0.0	337.2	409.9
1500	0.0	0.0	0.0	0.0	0.0	0.0	0.0	36.2	0.0	0.0	0.0	558.6	594.9
1600	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	537.2	537.2
1700	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	591.5	591.5
1800	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	592.3	592.3
1900	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	220.6	220.6

Step 8. Compute Total Travel Time

Compute total travel time by summing the segment running speed and the queuing delay.

Table C-19 Step 8 - Compute Total Travel Time (sec)

Total Time	sec 1	sec 2	sec 3	sec4	sec 5	sec6	sec 7	sec 8	sec 9	sec 10	sec 11	sec 12	Sum (secs)	Mean Speed (mph)
1400	6	85	17	32	17	41	22	124	18	58	12	364	796	28
1500	6	85	17	32	17	41	22	88	17	53	11	585	974	23
1600	6	84	17	31	17	41	22	46	17	50	11	564	906	24
1700	6	84	17	31	17	41	22	46	17	51	11	618	961	23
1800	6	83	17	31	17	40	22	45	17	50	11	619	958	23
1900	5	82	16	31	16	40	22	43	17	48	11	247	578	38

Step 9. Compute Mean Facility Speed

Compute facility speed for each hour of peak period. Divide the total travel time for each hour (Sum) by the length of the facility (6.1 miles) to obtain the average speed for each hour. If the mean speed over the whole peak period is desired, compute the total vehicle-hours traveled over the peak period (multiply the total travel time for each hour by the total number of vehicles (VHT) on the facility during each hour and sum over the peak period) and divide the VHT into the total vehicle-miles traveled (VMT) (the

number of vehicle on each segment times the length of each segment, summed over all hours and segments).

For example, for the first hour (14:00), 32,200 feet / 796 seconds * 3600/5280 = 28 mph (45 kph).

The results should be rounded off to the nearest whole mph or kph.

Step 10. Determine Level of Service

The mean level of service for the facility is determined by averaging the segment volume/capacity ratios over the facility using the following equation:

$$\frac{v}{c}_{mean} = \frac{\sum_{i,t} \left(\frac{v}{c}\right)_{i,t} * L_i * N_i}{N_t * \sum_i L_i * N_i} \qquad \textbf{(12-7)}$$

where:

v/c_{mean} = the mean volume/capacity ratio for the facility in one direction.
v/c_i = the volume capacity ratio in one direction for segment "i".
L_i = The length of segment "i" (miles).
N_i = the number of through lanes in one direction of segment "I".
N_t = Number of time periods included in analysis.

The table below shows the computation of mean v/c ratio.

Table C-20 Computation of Mean V/C

v/c*L*N	sec 1	sec 2	sec 3	sec4	sec 5	sec6	sec 7	sec 8	sec 9	sec 10	sec 11	sec 12	Sum
1400	1825	30375	5175	11340	5550	15300	6400	16224	5640	17600	3600	7140	127569
1500	1850	30750	5325	11340	5550	15120	6240	15912	5220	16368	3360	7860	126395
1600	1725	28500	4875	10640	5325	14580	5920	14040	4680	14784	3000	7800	117469
1700	1775	29250	5100	10780	5400	14580	6000	13884	4800	15136	3080	7980	119465
1800	1600	26625	4725	9940	5025	13320	5520	13260	4560	14256	2920	7980	111531
1900	1250	20625	3600	7560	3750	10080	4160	10452	3600	11264	2440	6720	87401
												Sum:	689830
6*L*N	15000	225000	45000	84000	45000	108000	48000	93600	36000	105600	24000	36000	865200

The mean volume/capacity ratio is.

v/c = 689830/ 865200 = 0.80

The level of service can then be looked up in the following table. The facility is a 6 to 10 lane facility with a free flow speed of 62 mph. By interpolation we get the result that the facility is operating at a borderline Level of Service is "D/E" averaged over the 6 hour peak period.

Table C-21 Maximum Volume/Capacity Ratios for Freeways[4]

	Four Lanes (2 each direction) Free-Flow Speed (mph)				Six Lanes (3 each direction) Free-Flow Speed (mph)			
Level of Service	70	65	60	55	70	65	60	55
A	0.32	0.30	0.27	0.25	0.30	0.28	0.26	0.24
B	0.51	0.47	0.44	0.4	0.49	0.45	0.42	0.38
C	0.75	0.70	0.65	0.60	0.71	0.67	0.63	0.57
D	0.92	0.89	0.83	0.80	0.88	0.85	0.79	0.77
E	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

This table can be interpolated for different free-flow speeds.

4 1994 HCM, Table 3-1, page 3-9.

Sample Problem #6 - Interrupted Flow Facility

Sample Problem #4 is repeated for Ventura Boulevard, but this time the analysis is performed for all of the street segments, rather than for just the most critical segment.

Table C-22 lists the required input data for applying the Enhanced ARTPLAN method to an interrupted flow facility.

Table C-23 shows the computation of mean speed according to the enhanced ARTPLAN process. The following text explains each of the columns in this table. Once the mean speed is known, the level of service can be determined based upon the ratio of the mean speed to the free-flow speed, using the level of service table presented in Example Problem #4.

The **Thru Volume** is computed by multiplying the link volume by 1.00 minus the proportion of traffic making turns from any exclusive left or right turn lanes.

The **Capacity** is computed by multiplying the saturation flow rate by the number of lanes by the g/c ratio.

The **v/c** ratio is computed by dividing the capacity into the thru volume.

The uniform delay term, **du**, is computed using the following equation:

$$d_u = (0.38) * C * \frac{[1-(g/C)]^2}{[1-(g/C) * \mathrm{Min}(X,1.0)]}$$

The incremental delay term, **di**, is computed using the following equation:

$$d_i = 173 * X^2 * \left\{(X-1) + \sqrt{(X-1)^2 + m*(X/c)}\right\}$$

subject to X = Minimum of [volume/capacity, 1.00].

The total **Signal Delay** (exclusive of overflow delay caused by demand exceeding capacity) is computed using the following equation:

$$D = 1.3 * (d_u * DF + d_i)$$

The **Queue Delay** is the excess delay caused when demand exceeds the capacity of the signal approach (this is different from the normal queue delay that occurs each time the signal turns red). It is computed using equation 12-12:

$$d_q = 1800 * T * \left(\frac{V_{t-1} + V_t}{cap} - 1\right) \qquad \textbf{(12-12)}$$

The mid-block Free-Flow Speed is computed using the following equation:

Sf (mph) = 0.79 * (Posted Speed Limit in mph) + 12 mph (Customary Units)

Table C-22 Input Data For Enhanced ARTPLAN Example - Ventura Blvd. Eastbound, AM Peak Hour

Link	Link Volumes	%Turns Exc.Lanes	Saturation Flow/Lane	Thru Lanes	Effective g/C	Arrival Type	Cycle (sec)	Segment Length (ft)	Speed Limit (mph)
10-11	1638	1%	1700	3	0.64	4	90	640	35
11-12	1244	37%	1700	3	0.59	4	90	1943	35
12-13	1136	2%	1700	3	0.63	3	90	1429	35
13-14	1524	2%	1700	3	0.33	2	90	1385	35
14-15	1201	1%	1700	3	0.62	4	90	732	35
15-16	1201	1%	1700	3	0.62	3	90	1770	35
16-17	1201	0%	1700	3	0.62	3	90	1745	35
17-18	1149	16%	1700	2	0.42	2	90	1350	35
18-19	2511	0%	1700	2	0.63	4	90	1618	35
19-20	2153	11%	1700	2	0.63	3	90	1119	35
20-21	2042	0%	1700	2	0.64	3	90	1310	35
21-22	2743	7%	1700	2	0.48	4	90	1342	35
22-23	1778	28%	1700	2	0.52	4	90	1031	35
23-24	1299	6%	1700	2	0.61	4	90	1628	35
24-25	1994	9%	1700	2	0.67	4	90	640	35
25-26	1694	2%	1700	2	0.64	3	90	690	35
26-27	1302	6%	1700	2	0.54	3	90	1361	35
27-28	2668	1%	1700	3	0.62	3	90	1759	35
28-29	2641	0%	1700	2	0.68	4	90	363	35
29-30	1610	5%	1700	3	0.64	4	90	805	35
30-31	2721	1%	1700	3	0.68	4	90	932	35
31-32	1600	2%	1700	3	0.66	4	90	550	35
32-33	2773	5%	1700	3	0.48	4	90	1140	35
33-34	2691	1%	1700	2	0.68	3	90	1414	35
34-35	2304	1%	1700	3	0.62	4	90	1330	35
35-36	2800	2%	1700	3	0.68	4	90	1340	35
36-37	2938	2%	1700	3	0.68	4	90	685	35
37-38	2789	4%	1700	3	0.49	3	90	673	35
38-39	2454	1%	1700	3	0.68	3	90	1575	35
39-40	2362	0%	1700	3	0.67	4	90	462	35
40-41	2527	4%	1700	3	0.49	5	90	662	35
41-42	2562	1%	1700	3	0.67	4	90	1331	35
42-43	2640	1%	1700	3	0.67	4	90	1334	35
43-44	2759	1%	1700	3	0.67	4	90	653	35
44-45	3266	4%	1700	3	0.66	4	90	681	35
45-46	1834	1%	1700	3	0.67	4	90	550	35
46-47	2332	0%	1700	3	0.66	4	90	673	35
47-48	1925	9%	1700	3	0.68	4	90	1607	35
48-49	1358	13%	1700	3	0.34	3	90	845	35

Table C-23 Computations of V/C, Delay, Travel Time - Enhanced ARTPLAN Example - Ventura Blvd.

Link	Thru Vol.	Capacity (vph)	v/c	du (sec)	m	di (sec)	DF	Sig.Delay (sec)	Que.Delay (sec)	FreeSpd (mph)	Run-Time (sec)	Seg. Time (sec)
10-11	1616	3351	0.48	6.27	12	0.07	0.41	2.61	0.00	39.65	11.01	13.62
11-12	779	3062	0.25	6.80	12	0.01	0.75	5.10	0.00	39.65	33.41	38.51
12-13	1111	3293	0.34	5.85	16	0.02	1.00	5.87	0.00	39.65	24.57	30.44
13-14	1490	1733	0.86	21.30	12	2.53	1.05	24.89	0.00	39.65	23.82	48.71
14-15	1187	3236	0.37	6.32	12	0.02	0.34	2.15	0.00	39.65	12.59	14.73
15-16	1187	3236	0.37	6.32	16	0.03	1.00	6.36	0.00	39.65	30.44	36.79
16-17	1201	3236	0.37	6.35	16	0.03	1.00	6.38	0.00	39.65	30.01	36.39
17-18	965	1436	0.67	15.94	12	0.66	1.21	20.02	0.00	39.65	23.21	43.23
18-19	2501	2153	1.16	12.54	12	12.91	0.37	17.56	290.62	39.65	27.82	336.00
19-20	1921	2153	0.89	10.57	16	3.75	1.00	14.32	0.00	39.65	19.24	33.57
20-21	2032	2191	0.93	10.75	16	5.53	1.00	16.27	0.00	39.65	22.53	38.80
21-22	2564	1624	1.58	17.86	12	14.87	0.87	30.39	1041.09	39.65	23.08	1094.56
22-23	1286	1776	0.72	12.56	12	0.79	0.62	8.62	0.00	39.65	17.73	26.34
23-24	1217	2078	0.59	8.06	12	0.24	0.30	2.66	0.00	39.65	27.99	30.65
24-25	1805	2267	0.80	8.10	12	1.11	0.48	4.96	0.00	39.65	11.01	15.97
25-26	1661	2320	0.72	8.03	16	0.76	1.00	8.79	0.00	39.65	11.87	20.65
26-27	1230	1851	0.66	11.12	16	0.65	1.00	11.77	0.00	39.65	23.40	35.17
27-28	2654	3236	0.82	9.97	16	1.27	1.00	11.24	0.00	39.65	30.25	41.49
28-29	2631	2304	1.14	11.02	12	12.48	0.51	18.11	255.07	39.65	6.24	279.43
29-30	1531	3351	0.46	6.13	12	0.05	0.41	2.54	0.00	39.65	13.84	16.38
30-31	2687	3524	0.76	7.35	12	0.54	0.51	4.30	0.00	39.65	16.03	20.32
31-32	1564	3409	0.46	5.80	12	0.05	0.44	2.61	0.00	39.65	9.46	12.07
32-33	2631	2437	1.08	17.86	12	12.14	0.87	27.66	143.56	39.65	19.60	190.82
33-34	2674	2304	1.16	11.02	16	14.42	1.00	25.44	288.66	39.65	24.32	338.41
34-35	2272	3236	0.70	8.67	12	0.37	0.34	3.28	0.00	39.65	22.87	26.15
35-36	2746	3457	0.79	7.69	12	0.72	0.51	4.65	0.00	39.65	23.04	27.69
36-37	2893	3524	0.82	8.00	12	0.89	0.51	4.98	0.00	39.65	11.78	16.76
37-38	2681	2542	1.05	17.48	16	13.72	1.00	31.20	98.26	39.65	11.57	141.04
38-39	2438	3524	0.69	6.69	16	0.42	1.00	7.10	0.00	39.65	27.08	34.19
39-40	2352	3467	0.68	6.94	12	0.29	0.48	3.59	0.00	39.65	7.94	11.53
40-41	2430	2542	0.96	16.77	8	4.15	0.53	13.01	0.00	39.65	11.38	24.39
41-42	2531	3467	0.73	7.40	12	0.43	0.48	3.95	0.00	39.65	22.89	26.84
42-43	2624	3467	0.76	7.67	12	0.53	0.48	4.18	0.00	39.65	22.94	27.12
43-44	2740	3467	0.79	8.03	12	0.69	0.48	4.52	0.00	39.65	11.23	15.74
44-45	3125	3409	0.92	10.17	12	2.55	0.44	7.03	0.00	39.65	11.71	18.74
45-46	1814	3600	0.50	5.72	12	0.07	0.48	2.80	0.00	39.65	9.46	12.25
46-47	2322	3540	0.66	7.12	12	0.24	0.44	3.38	0.00	39.65	11.57	14.95
47-48	1747	3524	0.50	5.35	12	0.07	0.51	2.80	0.00	39.65	27.63	30.44
48-49	1176	1860	0.63	18.79	16	0.51	1.00	19.30	0.00	39.65	14.53	33.83
							Total:	396.35	2117.26			3254.71

The **m** and **DF** terms are obtained from the following table:

Table C-24 Delay Adjustment Factor (Df) And Incremental Delay Calibration Term (M)

Delay Adjustment Factor (DF)	g/C	Quality of Progression					
		Very Poor	Unfavorable	Random Arrivals	Favorable	Highly Favorable	Exceptionally Good
	0.20	1.167	1.007	1.000	1.000	0.833	0.750
	0.30	1.286	1.063	1.000	0.986	0.714	0.571
	0.40	1.445	1.136	1.000	0.895	0.555	0.333
	0.50	1.667	1.240	1.000	0.767	0.333	0.000
	0.60	2.001	1.395	1.000	0.576	0.000	0.000
	0.70	2.556	1.653	1.000	0.256	0.000	0.000
Delay Calibration Term (m)		8	12	16	12	8	4

Source: 1994 Highway Capacity Manual

The **Segment Time** is computed by dividing the mid-block free-flow speed into the segment length.

The **Total Travel Time** for each segment is computed by summing the Signal Delay, the Queue Delay, and the Segment Time.

The Total Travel Time summed over all links (3,254 seconds) is divided into the total length (43,097 feet) to obtain the **mean speed** for the arterial of 13 feet per second or 9 mph (14 kph). While it is tempting to report the results to the nearest tenth of a mile per hour, the method is not considered accurate to anything more precise than the nearest 2 or 3 mph (3 to 5 kph).

The ratio of the mean speed (9 mph) to the mid-block free-flow speed 40 mph (derived in Example Problem #4) is used to determine the **level of service**. Note that the posted speed limit (35 mph) is not used to determine level of service.

S / Sf = 9 mph / 40 mph = 22% of the free-flow speed.

The facility is operating at level of service "F" in the eastbound direction during the morning peak hour (see Table C-9 in sample problem 4).

APPENDIX D
GLOSSARY

Average Running Speed The length of a road segment divided by the average "running" time of vehicles traversing the segment. Running time excludes all stopped time. Running speed is always equal to or higher than the average travel speed. Running speed equals average travel speed if there are no stops.

Average Travel Speed The length of a segment of road divided by the average travel time of the vehicles traversing that segment. It includes stopped time. It is also the "space mean speed."

Capacity The maximum sustainable flow on the facility.

Design Speed As defined in AASHTO's *A Policy on Geometric Design of Highways and Streets* (1994), design speed is "the maximum safe speed that can be maintained over a specified section of highway when conditions are so favorable that the design features of the highway govern."

Free-Flow Speed The average travel speed at which a single car could traverse a segment of road if no other vehicles are present on that segment (there might be vehicles on the side streets). It is defined as the length of the segment divided by the time to traverse the segment.

Level of Service The quality of traffic flow conditions as defined by the facility performance measures specified by the *Highway Capacity Manual.*

Node A point at the end of a segment where demand or capacity changes.

Practical Capacity 80 percent of the capacity of the facility.

Space Mean Speed The average speed of all vehicles present on a given segment (hence the term "space") of road at a particular point in time. Aerial photos and floating cars measure space mean speed.

Study Section The single direction portion of the facility to be evaluated.

Segment A portion of the study section of a facility where demand and capacity are comparatively constant.

Time Mean Speed The average speed of all vehicles passing a given point over a certain period. Loops and radar guns measure time mean speed.

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